SECTION 5: CONCRETE STRUCTURES WITH CFRP REINFORCEMENT
TABLE OF CONTENETS

5.1—SCOPE

The provisions in this Section apply to the design of bridge and retaining wall components constructed of normal weight or lightweight concrete and prestressed with carbon fiber reinforced polymer (CFRP) bars, strands, or tendons. The provisions are based on concrete strengths varying from 4.0 ksi to 12.0 ksi, except where higher strengths are allowed for normal and lightweight weight concrete. Segmental bridge construction, and components with partial prestressing are not covered under the provisions of this Section.

5.2—DEFINITIONS

Unless specified herein, refer to current version of AASHTO LRFD

Compression-Controlled Section—A cross section in which the net tensile strain in the extreme tension CFRP at nominal resistance is less than the net guaranteed strain limit just as the concrete in compression reaches its assumed maximum usable concrete strain.

Compression-Controlled Strain Limit—Compression control strain limit is not applicable for sections with CFRP material.

Creep—Time-dependent deformation of concrete or CFRP under permanent load

Creep-Rupture—Tensile fracture of CFRP when subjected to sustained high stress levels over a period of time

Creep-Rupture Strength—The maximum sustained stress that can be applied for a period of one-million hour without causing creep rupture in CFRP

Design CFRP Guaranteed Strength—The design CFRP guaranteed strength is the Suggested CFRP Guaranteed Strength reduced by the appropriate reduction factor to account for any loss of strength during the service life of the structure.

Extreme Tension CFRP—The reinforcement (prestressed or nonprestressed) that is farthest from the extreme compression fiber

Maximum Usable Concrete Strain— If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003. If the concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized if verified.

Net Guaranteed Strain Limit—The net guaranteed strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For all prestressed CFRP reinforcement, the net guaranteed strain limit may be taken as the specified guaranteed ultimate strain exclusive of the strain due to pretress, creep, shrinkage, and temperature.

Suggested CFRP Guaranteed Strength—Guaranteed rupture strength of CFRP defined as the mean tensile strength at failure of sample of test specimens minus three times standard deviation

Tendon—A high strength CFRP element used to prestress the concrete

Tension-Controlled Section—A cross section in which the strain of concrete in compression at nominal resistance is less than maximum usable concrete strain just as the net tensile strain in the extreme tension CFRP reaches its net guaranteed strain limit.

Tension-Controlled Strain Limit—Tension controlled strain limit is not applicable for sections with CFRP material.

5.3—NOTATION

A_c = area of concrete on the flexural tension side of the member (in.^{2}) (5.8.3.4.2)
A_f = total area of longitudinal non-prestressed CFRP reinforcement (in.^{2}) (5.6.3.4.1); area of nonprestressed CFRP on the flexural tension side of the member (in.^{2}) (5.8.3.4.2)
A_{fe} = equivalent area of reinforcement equal to a discrete area of CFRP reinforcement positioned at the extreme CFRP layer that results in the same flexural capacity of n layers of reinforcement (in.^{2}) (5.7.2.1)
$A_{fe(i)}$ equivalent area for the area of CFRP reinforcement at layer i (in.^{2}) (5.7.2.1)
A_{pf} = total area of longitudinal prestressed CFRP strands (in.^{2}) (5.6.3.4.1); area of prestressing CFRP on the flexural tension side of the member (in.^{2}) (5.8.3.4.2)
A_v = area of a transverse reinforcement within a distance S (in.^{2}) (5.8.2.5) (5.8.3.3)
a = $\beta_1 c$; depth of the equivalent stress block (in.) (5.7.3.2.2)
a_f = area of single CFRP strand in the *ith* layer (in.^{2}) (5.7.2.1) (5.7.3.2.2) (C5.7.2.1)
a_g = maximum aggregate size (in.) (5.8.3.4.2)
b = width of compression face of the member; for a flanged section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.) (5.7.3.1.1) (5.7.3.2.2)
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.) (5.8.2.9); width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in.) (5.8.2.5); effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.) (5.8.3.3)
b_w = width of web (in.) (5.7.3.1.1) (5.7.3.2.2)
c = depth of neutral axis from extreme compression fiber (in.) (5.7.2.1) (C5.7.2.1); Depth of neutral axis from extreme compression fiber as determined from Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, whichever is applicable (in.) (5.7.3.2.2)
d_i = depth of the *ith* CFRP layer from the extreme compression fiber (in.) (C5.7.2.1) (5.7.2.1)
d_1 = depth of the extreme CFRP layer from the extreme compression fiber (in.) (5.7.2.1) (C5.7.2.1) (5.7.3.2.2)
d_b = diameter of CFRP bar or strand (in.) (5.8.2.3)
d_p = distance from the extreme compression fiber to the centroid of prestressing strands (in.) (5.7.3.2.2)
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 $0.9d_e$or $0.72h$ (in.) (5.8.2.7) (5.8.2.9) (5.8.3.3)
E_c = elastic modulus of concrete (ksi) (5.7.1)
E_f = elastic modulus of CFRP reinforcing bars (ksi) (5.7.1); elastic modulus of CFRP (ksi) (C5.7.2.1) (5.6.3.4.1) (5.7.3.1.1) (5.7.3.2.2)
E_p elastic modulus of prestressing CFRP strands (ksi) (5.7.1) (5.9.5.4)
f_b = average ultimate bond strength between CFRP and uncracked concrete evaluated experimentally according to test method described in ACI 440.3R-12- B.3 (ksi) (C5.8.2.3)
f_{bN} = allowable bond strength after N load cycles (ksi) (C5.8.2.3)
f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi) (5.6.3.3.3) (5.7.3.2.2)
f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi) (5.8.2.3)
f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi) (5.7.3.3.2) (5.8.3.4.3)
f_f = stress in the transverse CFRP reinforcement (ksi) corresponding to a strain of 0.0035(5.8.2.5)
f_{gu} Design guaranteed strength of CFRP (ksi) (5.7.2.1) (5.8.2.3)
f_{pc} = compressive stress in concrete (after allowance for all prestresss losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (ksi). In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at junction of web and flange, due to both prestresss and moments resisted by precast member acting alone (5.8.3.4.3)
f_{pe} = stress in prestressing CFRP due to prestress after losses (ksi) (5.6.3.4.1) (5.8.2.3)

f_{pi}	=	prestressing CFRP stress immediately prior to transfer (ksi) (5.9.5.3)
f_{po}	=	a parameter taken as modulus of elasticity of the prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value equal to initial prestressing stress immediately prior to transfer will be appropriate for pretensioned members(5.8.3.4.2)
f_r	=	modulus of rupture of concrete specified in Article 5.4.2.6 (ksi) (5.4.2.6)(5.7.3.3.2)
H	=	The average annual ambient relative humidity (%)(5.9.5.3)
h_f	=	depth of compression flange (in.) (5.7.3.1.1) (5.7.3.2.2)
L_d	=	Development length of CFRP (in.) (5.8.2.3)
L_t	=	Transfer length of CFRP (in.) (5.8.2.3)
M_a	=	maximum moment in a component at the section for which deformation is computed (kip-in) (5.7.3.6.2)
M_{cre}	=	moment causing flexural cracking at section due to externally applied loads (kip-in.) (5.8.3.4.3)
M_{dnc}	=	total unfactored dead load moment acting on the monolithic or non-composite section (kip-in) (5.7.3.3.2) (5.8.3.4.3)
M_{max}	=	maximum factored moment at section due to externally applied loads (kip-in.) (5.8.3.4.3)
M_n	=	nominal resistance (kip-in.) (5.7.3.2.1)
$\lvert M_u \rvert$	=	absolute value of the factored moment, not to be taken less than $\lvert V_u - V_p \rvert d_v$ (kip-in.) (5.8.3.4.2)
N	=	number of load cycles (C5.8.2.3)
n_i	=	number of CFRP strands in the *ith* layer (5.7.2.1) (C5.7.2.1) (5.7.3.2.2)
N_u	=	factored axial force, taken as positive if tensile and negative if compressive (kip) (5.8.3.4.2)
P_e		effective prestressing force in the section (kip) (5.7.3.1.1) (5.7.3.2.2)
S	=	spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.) (5.8.2.5) (5.8.3.3)
S_c	=	section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3}) (5.7.3.3.2) (5.8.3.4.3)
S_{nc}	=	section modulus for the extreme fiber of the monolithic or non-composite section where tensile stress is caused by externally applied loads (in.^{3}) (5.7.3.3.2) (5.8.3.4.3)
s_x	=	the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than 0.003 b_v s_x (in.) (5.8.3.4.2)
s_{xe}		crack spacing parameter (5.8.3.4.2)
T_{Min}	=	Minimum temperatures specified in Article 3.12 using either Procedure A (3.12.2.1) or Procedure B (3.12.2.2) (°F). (5.9.5.4)
T_p	=	average air temperature at the time of prestressing, taken as 68 °F unless more accurate data are available. (5.9.5.4)
V_c	=	nominal shear resistance of the concrete (kip) (5.8.2.4)
V_{ci}	=	nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment (kip) (5.8.3.4.3)
V_{cw}	=	nominal shear resistance provided by concrete when inclined cracking results from excessive principal tensions in web (kip) (5.8.3.4.3)
V_d	=	shear force at section due to unfactored dead load and includes both DC and DW (kip) (5.8.3.4.3)
V_i	=	factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max}$ (kip) (5.8.3.4.3)
V_p	=	component of effective prestressing force in direction of the shear force, positive if resisting the applied shear; $V_p = 0$ when the simplified method of Article 5.8.3.4.3 is used (kip) (5.8.2.4) (5.8.3.3)
V_u	=	factored shear force (kip) (5.8.2.4) (5.8.3.4.2)
v_u	=	the shear stress calculated in accordance with 5.8.2.9 (ksi) (5.8.2.7)
y_s	=	distance from the neutral axis to the point, where the strain is calculated (in.) (5.7.3.6.2)
α	=	angle of inclination of transverse reinforcement to longitudinal axis (degrees) (5.8.3.3)
α_c	=	concrete coefficient of thermal expansion of concrete as given in Article 5.4.2.2 (/°F) (5.9.5.4)
α_d	=	development length factor, equal to 1.5 (5.8.2.3)
α_t	=	Transfer length factor, equal to 0.875 for CFCC strands and 0.96 for Leadline (5.8.2.3)
α_{CFRP}	=	CFRP coefficient of thermal expansion of CFRP, taken as 0 /°F unless more accurate data are available. (5.9.5.4)
α_s	=	the smallest angle between the compressive strut and adjoining ties (degrees) (5.6.3.3.3)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4 (5.8.3.3)
β_1 = stress block factor specified in Article 5.7.2.2(5.7.3.2.2)
θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, $cot\ \theta$ is defined therein (5.8.3.3)
Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi) (5.9.5.1)
Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of CFRP (ksi) (5.9.5.1)
Δf_{pR} = an estimate of relaxation loss taken as recommended by the CFRP manufacturer or as verified by testing (5.9.5.3)
Δf_{pT} = total loss (ksi) (5.9.5.1)
Δf_{pTE} = loss in effective prestressing force due to seasonal temperature change (ksi), taken as loss in effective prestressing when positive and gain in effective prestressing when negative. (ksi) (5.9.5.1) (5.9.5.4)
γ_1 = flexural cracking variability factor, may be taken as 1.6 (5.7.3.3.2)
γ_2 = Prestress variability factor, may be taken as 1.1(5.7.3.3.2)
γ_h = correction factor for relative humidity of the ambient air (5.9.5.3)
γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member (5.9.5.3)
ε_1 = net tensile strain at the extreme CFRP layer (C5.7.2.1) (5.7.3.2.2)
ε_{cu} = average concrete crushing strain, 0.003 (5.7.3.1.1)
ε_f the net longitudinal tensile strain in the section at the centroid of the tension reinforcement (5.8.3.4.2)
ε_i = net tensile strain at the ith layer of CFRP reinforcement determined from strain compatibility as: $\varepsilon_i = \varepsilon_1 \left(\frac{d_i - c}{d_1 - c}\right)$ (C5.7.2.1) (C5.7.2.1) (5.7.3.2.2)
ε_{pe} = effective prestressing strain in CFRP after subtracting applicable prestress losses (5.7.3.1.1)
ε_{gu} = design guaranteed strain of CFRP including environmental and durability effects (5.7.3.1.1)
ε_s = tensile strain in the concrete in the direction of the tension tie (in./in.) (5.6.3.3.3); strain at any point in through the depth of the section (5.7.3.6.2)
$\emptyset$ = resistance factor as specified in Article 5.5.4.2 (5.7.3.2.1) (5.8.2.4); resistance factor for shear specified in Article 5.5.4.2 (5.8.2.9)

5.4—MATERIAL PROPERTIES

5.4.1—General

Refer to current edition of AASHTO LRFD.

C5.4.1

Refer to current edition of AASHTO LRFD.

5.4.2—Normal Weight and Structural Lightweight Concrete

5.4.2.1—Compressive Strength

Refer to current edition of AASHTO LRFD.

C5.4.2.1

Refer to current edition of AASHTO LRFD.

5.4.2.2—Coefficient of Thermal Expansion

Refer to current edition of AASHTO LRFD.

C5.4.2.2

Refer to current edition of AASHTO LRFD.

5.4.2.3—Shrinkage and Creep

5.4.2.3.1—General

Refer to current edition of AASHTO LRFD.

C5.4.2.3.1

Refer to current edition of AASHTO LRFD.

5.4.2.3.2—Creep

Refer to current edition of AASHTO LRFD.

C5.4.2.3.2

Refer to current edition of AASHTO LRFD.

5.4.2.3.3—Shrinkage

Refer to current edition of AASHTO LRFD.

C5.4.2.3.3

Refer to current edition of AASHTO LRFD.

5.4.2.4—Modulus of Elasticity

Refer to current edition of AASHTO LRFD.

C5.4.2.4

Refer to current edition of AASHTO LRFD.

5.4.2.5—Poisson's Ratio

Refer to current edition of AASHTO LRFD.

C5.4.2.5

Refer to current edition of AASHTO LRFD.

5.4.2.6—Modulus of Rupture

Unless determined by physical tests, the modulus of rupture, f_r, for specified concrete strengths up to 15.0 ksi may be taken as:

- For normal-weight concrete:
  - Except as specified below .... $0.24\sqrt{f_c'}$

C5.4.2.6

Refer to current edition of AASHTO LRFD.

- When used to calculate the cracking moment of a member in Article 5.8.3.4.3............. $0.24\sqrt{f_c'}$

- For lightweight concrete:
  - For sand—lightweight concrete……...$0.20\sqrt{f_c'}$
  - For all—lightweight concrete……..….$0.17\sqrt{f_c'}$

When physical tests are used to determine modulus of rupture, the tests shall be performed in accordance with AASHTO T 97 and shall be performed on concrete using the same proportions and materials as specified for the structure.

Recent shear testing by Grace et al. (2014) showed that underestimating the modulus of rupture in calculating the cracking moment and the contribution of the concrete in the shear capacity in Article 5.8.3.4.3 can lead to overestimating the overall shear capacity of the member.

5.4.2.7—Tensile Strength

Refer to current edition of AASHTO LRFD.

C5.4.2.7

Refer to current edition of AASHTO LRFD.

5.4.3—Reinforcing CFRP

5.4.3.1—General

General properties of reinforcing CFRP shall be determined by using testing procedures specified in ACI-440-3R-12.

Acceptable forms of CFRP reinforcement include plain bars, deformed bars, twisted strands, grids, and pre-fabricated CFRP products such as stirrups, hoops, and spirals.

CFRP reinforcement has no yield strength and therefore the material is identified with the guaranteed tensile strength. The guaranteed tensile strength is determined for a group of test specimens by the manufacturer, or through testing, as the mean tensile strength minus three times the standard deviation. The guaranteed tensile strength of CFRP shall be shown in the contract documents.

C5.4.3.1

Unlike steel, the tensile strength of CFRP bars and strands is not directly proportional to the diameter. Tensile strength shall be established for each CFRP strand or bar diameter. The CFRP bar manufacturer should be contacted for strength values of differently sized CFRP bars. Determination of CFRP bar and strand strength by testing may also be recognized provided that special precautions are taken to avoid stress concentrations in and around anchorage points in the test specimens. An adequate testing grip should allow failure to occur in the test specimen and not in the anchorage.

Unlike steel, failure of a CFRP test specimen is not expected to occur at the middle third of the specimen and failure near the anchorage shall be acceptable provided that the failure does not stem from a slippage or a defect of the anchorage device. Test methods for determining the tensile strength and stiffness of CFRP bars are available in ACI 440.3R-12. Usually, a normal (Gaussian) distribution is assumed to represent the strength of a population of bar specimens (Kocaoz et al. 2005).

5.4.3.2—Modulus of Elasticity

The modulus of elasticity shall be obtained from tests conducted on sample specimens according to ASTM D7205 Standard Test Method.

C5.4.3.2

Modulus of elasticity of CFRP shall be obtained experimentally based on the effective cross sectional area of CFRP bar or strand.

5.4.3.3—Special Applications

Refer to current edition of AASHTO LRFD.

5.4.4—Prestressing CFRP

5.4.4.1—General

Prestressing CFRP materials include twisted strands, plain, and deformed bars. CFRP strands with a diameter larger than 0.6 in. are currently available but their use is limited to post-tensioning applications.

C5.4.4.1

CFRP strands and bars can be used as prestressed or non-prestressed reinforcement as required by design. Different types of CFRP materials from multiple vendors can be used simultaneously, to satisfy design requirements, as internal prestressed or non-prestressed reinforcement or as external post-tensioning strands. Internal prestressing strands with a diameter of 0.5 and 0.6 in. have been successfully deployed in the design and construction of several bridge projects. Larger-diameter CFRP strands are also available but have not been evaluated for the use as internal prestressing.

The size and grade or type of CFRP shall be specified on the contract documents.

Due to the wide variety of CFRP materials, the mechanical and physical properties of a certain CFRP material are considered unique. Therefore, the type, grade, or size of CFRP material cannot be altered without prior approval from the designer.

5.4.4.2—Modulus of Elasticity

The modulus of elasticity shall be obtained from tests conducted on sample specimens according to ASTM D7205 Standard Test Method.

C5.4.4.2

Other available test standards may be used to if approved by the designer. However, the effect of temperature change on the modulus of elasticity shall always be investigated. CFRP materials may experience a slight softening in their epoxy matrix with a moderate increase in the temperature. This softening can be represented by a temporary loss in the elastic modulus. Therefore, care shall be taken during construction to avoid excessive prestress loss due to heat generated by concrete curing. Test results on 0.6-in.-diameter prestressed CFCC specimens subjected to a temperature increase from 76 to 150 °F showed a temporary reduction in the elastic modulus of approximately 2.3 %, represented by a corresponding loss in prestressing force (Grace et. al. 2017). The specimens gained back their original elastic modulus when allowed to cool down. However, the loss in the prestressing force was not recoverable. The test specimens did not experience any further reduction in elastic modulus or prestressing force when heated to 150 °F for the second time.

5.4.5—Post-Tensioning Anchorages and Couplers

Different prestressing CFRP products shall be anchored with their approved type of anchors. Using anchors approved for a different CFRP product or a different diameter shall not be allowed. Corrosion protection shall be provided for anchorages, end fittings, and couplers.

C5.4.5

Anchor types such as clamp, plug and cone, resin sleeve, resin potted, resin overlay, and split wedge anchorages have been used successfully. Exact details of the anchor shall be verified and provided by the CFRP manufacturer. Anchors made for steel strands shall not be used with CFRP strands because they tend to damage the surface and lead to premature failure.

Some CFRP manufacturers have developed coupler systems to pair CFRP strands with steel strands and facilitate pre-tensioning. To date, no coupler system has been used in post-tensioning application.

The long-term performance of CFRP anchors and couplers can vary significantly from their short-term performance and shall be verified with the manufacturer before use.

5.4.6—Ducts

5.4.6.1—General

Ducts for strands and tendons shall be rigid or semirigid either galvanized ferrous metal or polyethylene, or they shall be formed in the concrete with removable cores. When friction is inevitable between the strand and the surrounding duct, only polyethylene ducts shall be used and CFRP strand shall be protected with buffer material along the entire length to avoid damaging the surface of the strand.

The minimum radius of curvature of tendon ducts shall be specified based on the type and diameter of the tendon and as recommended by the manufacturer.

In case of grouted ducts, the bonding characteristics of the ducts to the concrete and the grout should be investigated.

The effects of grouting pressure on the ducts and the surrounding concrete shall be investigated.

The maximum support interval for the ducts during construction shall be indicated in the contract documents and shall conform to Article 10.4.1.1 of the *AASHTO LRFD Bridge Construction Specifications.*

C5.4.6.1

The use of polyethylene duct is generally recommended in corrosive environments. Pertinent requirements for ducts can be found in Article 10.8.2 in *AASHTO LRFD Bridge Construction Specifications.*

Special care should be taken to avoid damaging the surface of CFRP strands and tendons. Friction with the ducts during tendon pulling-through and stressing can lead to a significant damage in the surface of CFRP tendon and consequently a reduction in the tensile capacity. Buffer material shall be used to minimize the friction and protect the tendon.

The contract documents should indicate the specific type of duct material to be used when only one type is to be allowed.

5.4.6.2—Size of Ducts

The inside diameter of ducts is constrained by the type of the CFRP tendon and the attached anchorage device in case of tendons with pre-attached anchors.

CFRP strands and tendons shall be placed by the pull-through method. Friction between CFRP surface and the duct shall be avoided during service conditions.

The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct.

C5.4.6.2

Only the pull-through method of placement is allowed in case of CFRP tendons and strands.

5.4.6.3—Ducts at Deviation Saddles

Ducts at deviation saddles shall be made of a material and dimensions approved by the CFRP manufacturer. The loss of CFRP strength at the saddle point shall be calculated and taken in to account in design.

C5.4.6.3

Galvanized steel roller-and-pin deviators have been used successfully with CFRP strands. Friction can be reduced at the deviator by using a buffer material and or lubricant. Minimum roller/deviator radius should be maintained to avoid damaging the CFRP strand/tendon.

5.5—LIMIT STATES

5.5.1—General

Refer to current edition of AASHTO LRFD.

5.5.2—Service Limit State

Refer to current edition of AASHTO LRFD.

5.5.3—Fatigue Limit State

5.5.3.1—General

Refer to current edition of AASHTO LRFD.

C5.5.3.1

Refer to current edition of AASHTO LRFD.

Several research studies showed that CFRP materials are not susceptible to fatigue (Grace et al. 2002). Nevertheless, CFRP materials vary greatly in properties and strength. Therefore, when fatigue is a concern, designer shall select CFRP materials with an established fatigue resistance that has been verified experimentally.

5.5.3.2—Reinforcing bars

Refer to current edition of AASHTO LRFD.

C5.5.3.2

Generally, members with CFRP reinforcement are less susceptible to fatigue than those with steel reinforcement. ACI 440.1R-15 recommends a limit of 0.55 f_u for fatigue stress calculated due to all sustained loads plus the maximum moment induced in a fatigue loading cycle., where f_u is the guaranteed tensile strength of CFRP.

5.5.3.3—Prestressing Tendons

In uncracked members, fatigue is unlikely to cause a problem and therefore it may be ignored in design.

C5.5.3.3

Based on recent research studies (Grace et al. 2000 and 2002), it was determined that the stress range in the tendons under repeated loading will be small and will not affect the strength of CFRP or lead to premature failure.

5.5.3.4—Mechanical Splices of Reinforcement

Mechanical splices are not available for CFRP reinforcement. Only lap splices are allowed in CFRP reinforcement.

C5.5.3.4

5.5.4—Strength Limit State

5.5.4.1—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 5.6, 5.7, 5.8, 5.9, 5.10, 5.13, and 5.14, unless another limit state is specifically identified, and the resistance factor is as specified in article 5.5.4.2.

5.5.4.2—Resistance Factors

5.5.4.2.1—Conventional Construction

Resistance factor Ø shall be taken as:

- For shear and torsion:

  normal weight concrete 0.75
  lightweight concrete 0.75

- For sections in which the net tensile strain in the extreme CFRP at nominal resistance is equal to or more than 0.005……………………….….….....0.85

- For sections in which the net tensile strain in the extreme CFRP at nominal resistance is equal to or less than 0.002…………………………..…..…..0.75

- For sections in which the net tensile strain in the extreme CFRP at nominal resistance is between 0.002 and 0.005, Ø may be linearly increased from 0.75 to 0.85 as the net tensile strain in the extreme tension CFRP increases from 0.002 to 0.005.

- For bearing on concrete………………..……..…0.70

- For compression in strut-and-tie models……..…0.70

- For compression in anchorage zones:

  Normal weight concrete…………………....0.80
  Lightweight concrete………………………0.65

- For resistance during pile driving……………......1.00

C5.5.4.2.1

The net tensile strain of CFRP is that caused by external forces. Effects of primary prestressing forces are not included.

The definition of tension-controlled and compression-controlled for sections with CFRP reinforcement is different from that for sections with steel reinforcement. Therefore, the resistance factor Ø shall not be confused with the mode of failure discussed in Article 5.7.

To ensure proper ductility before the failure of the section, the Ø-factor is chosen based on the net tensile strain in the extreme CFRP reinforcement. The limits of 0.002 and 0.005 are selected to ensure ductility similar to that acknowledged in members with steel reinforcement.

For sections in which the net tensile strain in the extreme CFRP at nominal resistance is equal to or larger than 0.005, large deflection and extensive cracking pattern serve as visual warning signs before failure. At that stain level, the failure may be triggered by the crushing of concrete or rupture of extreme CFRP strands.

To evaluate the effect of the failure mode on strength reduction factor for sections in which the net tensile strain in the extreme CFRP at nominal resistance is equal to or more than 0.005, Grace et al (2017) compared the experimental ultimate moment capacity for a series of bridge beam specimens with their analytical nominal moment capacity, calculated using the suggested guaranteed strength of CFRP (mean of ultimate strength minus three times standard deviation) and a concrete crushing strain of 0.003. In all beam specimens under consideration, the net tensile strain in the extreme CFRP exceeded the limit of 0.005. Test results showed that beam specimens belonged to one of two groups:

- Beam specimens that experienced rupture of CFRP strands at the time of failure. Those beams consistently had the ratio between the experimental and analytical nominal moment capacities larger than 1.10. This ratio was similar to the ratio between the average ultimate strength and the guaranteed strength of CFRP.

- Beams specimens that experienced crushing of the concrete at the time of failure, including beams with balanced or near-balanced reinforcement ratio. Those beams had the ratio between the experimental and analytical nominal moment capacities within the range from 0.95 to

1.10 because a concrete crushing strain of 0.003 was not consistently achievable and some beams failed at a concrete crushing strain lower than 0.003, which corresponded to a lower experimental to analytical moment ratio.

Therefore, a $\emptyset$ factor of 0.85 shall provide an adequate margin of safety for sections experiencing compression failure while the net tensile strain in the extreme CFRP at nominal resistance is equal to or more than 0.005. Meanwhile, the $\emptyset$ factor of 0.85 shall also provide a conservative margin of safety for sections experiencing tension failure while the net tensile strain in the extreme CFRP at nominal resistance is equal to or more than 0.005.

The strength reduction factor for sections with net tensile strain less than 0.002 is taken as 0.65 in the ACI 440-4R-04. This is different from the strength reduction factor of 0.75 that is currently recommended for similar sections in AASHTO LRFD 2014 Section 5.5.4.2 because AASHTO LRFD considers different load combinations than those considered by ACI. Current recommendations for CFRP prestressed members are based on loads and load combinations specified in AASHTO LRFD (2014) Chapter 3.

5.5.4.2.2—Segmental Construction

Not Applicable

C5.5.4.2.2

There is no available data regarding the construction of segmental bridges with bonded or unbonded CFRP tendons.

5.5.4.2.3—Special Requirements for Seismic Zones 2, 3, and 4

Refer to current edition of AASHTO LRFD.

5.5.4.3—Stability

Refer to current edition of AASHTO LRFD.

5.5.5—Extreme Event Limit State

Refer to current edition of AASHTO LRFD.

C5.5.5

Test results on beams prestressed with CFRP strands and subjected to fire event according the ASTM E119 time-temperature curve while supporting a service load level showed fire endurance of approximately 47, 53, 69 minutes for initial prestressing stresses of 54, 41, and 30 % of the suggested guaranteed CFRP strength, respectively. All test beams were supplied with a clear concrete cover of 0.75 in.

Test results of unbonded CFRP specimens at elevated temperatures showed that the average ultimate strength of CFRP strands is adversely affected by the increase in temperature. When loaded to failure at 660 °F, CFRP test specimens achieved approximately 50% of their ambient mean tensile strength. In addition, it was observed that the decrease in strength was linearly proportional to the increase in temperature of CFRP to a temperature limit of 660 °F.

Based on the test results and to account for fire events, a suitable concrete model can be used to predict the temperature of CFRP and estimate the fire endurance based on the CFRP stress-temperature relationship.

5.5.6—Environmental Reduction Factor

The strength of CFRP prestressed and reinforced members shall be calculated based on the strength of the material at the end of the anticipated service life of the component as follows:

- Suggested guaranteed strength of CFRP strands shall be reduced with the appropriate reduction factor to account for the loss in strength by the end of the service life of the structure. In lieu of more accurate estimate, the design guaranteed strength of CFRP material at the end of the service life may be taken as 90 % of the suggested guaranteed strength.

- In flexural members with or without composite deck, the concrete strength in regions resisting compression stresses shall be reduced to account for the loss in concrete strength due to environmental conditions. In lieu of a more accurate estimate, the concrete strength shall be taken as minimum allowable compressive strength before rehabilitation is required.

C5.5.6

Environmental effect and degradation in material strength over time is not typically considered in the design of steel prestressed members. However, research community tends to agree that flexural members reinforced or prestressed with CFRP materials shall be designed with the effect of different environmental conditions in mind.

Test results (Grace 2017) on bridge beam specimens prestressed with bonded CFRP strands and exposed to 300 cycles of freezing and thawing according to ASTM 666 showed no reduction in the CFRP strength due to freeze-and-thaw cycles. However, the test results showed a reduction in concrete strength from 11 ksi to 4 ksi. The reduction in concrete strength resulted in changing the mode of failure from tension-controlled to compression-controlled and a reduction in the nominal moment capacity of approximately 12%

In addition, test results on unbonded prestressed CFRP strands exposed to 300 cycles of freezing and thawing showed an increase of approximately 15 % in the mean tensile strength after exposure to freeze-and-thaw cycles.

Analytical analysis (Grace 2017) showed that the nominal moment capacity of a CFRP tension-controlled section is linearly reduced with the reduction in the strength of CFRP material. Nevertheless, the reduction in the nominal moment capacity is only marginal with the reduction in concrete compressive strength unless the reduction in concrete strength resulted in the change of mode of failure from tension-controlled to compression-controlled.

On the other hand, that nominal moment capacity of a CFRP compression-controlled section is not reduced with the reduction in the strength of CFRP materials. Nevertheless, the reduction in the nominal moment capacity was significant with the reduction in concrete strength. For instance, a reduction of concrete compressive strength from 7 ksi to 3 ksi resulted in 30 % reduction in the nominal moment capacity of compression-controlled sections.

The analytical analysis combined with the test results of freeze-and-thaw cycles indicate that compression-controlled sections are more susceptible to the effect of environmental conditions than tension-controlled sections and they shall be carefully investigated to account for the degradation of concrete strength over time. Besides, applying an environmental reduction factor to the suggested guaranteed strength of CFRP is not expected to provide any margin of safety in compression-controlled sections.

5.6—DESIGN CONSIDERATIONS

5.6.1—General

Refer to current edition of AASHTO LRFD.

C5.6.1

Refer to current edition of AASHTO LRFD.

5.6.2—Effects of Imposed Deformation

Refer to current edition of AASHTO LRFD.

C5.6.2

Refer to current edition of AASHTO LRFD.

5.6.3—Strut-and-Tie Model

5.6.3.1—General

Refer to current edition of AASHTO LRFD.

C5.6.3.1

Refer to current edition of AASHTO LRFD.

5.6.3.2—Structural Modeling

Refer to current edition of AASHTO LRFD.

C5.6.3.2

Refer to current edition of AASHTO LRFD.

5.6.3.3—Proportioning of Compressive Struts

5.6.3.3.1—Strength of Unreinforced Strut

Refer to current edition of AASHTO LRFD.

5.6.3.3.2—Effective Cross-Sectional Area of Strut

Refer to current edition of AASHTO LRFD.

C5.6.3.3.2

Replacing steel reinforcement with CFRP reinforcement shall not affect the calculations of the cross-sectional area of the strut provided that the proper anchorage requirements are met.

5.6.3.3.3—Limiting Compressive Stress in Strut

The limiting compressive stress, f_{cu} , shall be taken as:

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \leq 0.85 f_c' \qquad (5.6.3.3.3\text{-}1)$$

in which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\alpha_s \qquad (5.6.3.3.3\text{-}2)$$

where:

α_s = the smallest angle between the compressive strut and adjoining ties (degrees)

C5.6.3.3.3

The equation for the limiting compressive stress in the concrete assumes that concrete can resist a compressive stress of $0.85f_c'$ if the concrete is not subjected to principal tensile strains greater than about 0.002. This principal tensile strain is selected to represent the yield of steel reinforcement of the tie. In the case of CFRP reinforcement, the limit of 0.002 can be relaxed. However, there is no enough research to support using a limit higher than 0.002.

ε_s = the tensile strain in the concrete in the direction of the tension tie (in./in.)
f'_c = specified compressive strength (ksi)

5.6.3.3.4—Reinforced Strut

The strength of CFRP reinforcement in compression shall be ignored and the nominal resistance of the strut shall be taken as given in Eq. 5.6.3.3.1.1

C5.6.3.3.4

Generally, the compression capacity of CFRP reinforcement is far less than the tensile capacity and therefore it shall be ignored. If the compressive capacity of CFRP reinforcement is significant, it can be included in the calculations but the strain in the CFRP reinforcement shall be limited to 0.002 to match the assumption from Article 5.6.3.3.3 for the compressive strain in the strut.

5.6.3.4—Proportioning of Tension Ties

5.6.3.4.1—Strength of Tie

Tension tie reinforcement shall be anchored to the nodal zones by specified embedment lengths, hooks, or mechanical anchorages. The tension force shall be developed at the inner face of the nodal zone.

The nominal resistance of a tension tie in kips shall be taken as:

$$P_n = 0.002\, E_f\, A_f + A_{pf}[f_{pe} + 0.002E_f] \quad (5.6.3.4.1\text{-}1)$$

where:

E_f = elastic modulus of CFRP reinforcement (ksi)
A_f = Total area of longitudinal non-prestressed CFRP reinforcement (in.^{2})
A_{pf} = Total area of longitudinal prestressed CFRP strands (in.^{2})
f_{pe} = stress in prestressing CFRP due to prestress after losses (ksi)

C5.6.3.4.1

The strain limit of 0.002 in Eq. 5.6.3.4.1-1 is selected to avoid excessive concrete cracking and to ensure adequate integrity of the section, similar to that in sections with steel reinforcement. This strain limit may be relaxed if test results show that the integrity of the structural member and the accuracy of the strut-and-tie model are not compromised.

5.6.3.4.2—Anchorage of Tie

The tension tie reinforcement shall be anchored to transfer the tension force therein to the node regions of the truss in accordance with the requirements for development of reinforcement.

5.6.3.5—Proportioning of Node Regions

Refer to current edition of AASHTO LRFD.

C5.6.3.5

Refer to current edition of AASHTO LRFD.

5.6.3.6—Crack Control Reinforcement

Refer to current edition of AASHTO LRFD.

C5.6.3.6

The limits of crack control CFRP reinforcement is similar to those specified for members with steel reinforcement. Nevertheless, CFRP in general has a lower elastic modulus than that of steel. Therefore, larger amount of CFRP reinforcement may be required to satisfy the limits of crack control.

5.7—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.1—Assumptions for Service and Fatigue Limit states

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

- Prestressed concrete resists tension at sections that are uncracked, except as specified in Article 5.7.6.
- The strains in the concrete vary linearly, except in components or regions of components for which conventional strength of materials is inappropriate.
- The modular ratio, *n*, is rounded to the nearest integer number.
- The modular ratio is calculated as follows:
  - E_f / E_c for CFRP reinforcing bars
  - E_p / E_c for CFRP prestressing tendons
- An effective modular ratio of *2n* is applicable to permanent loads and prestress.

C5.7.1

Refer to current edition of AASHTO LRFD.

5.7.2—Assumptions for Strength and Extreme Event Limit States

5.7.2.1—General

Factored resistance of concrete components shall be based on the conditions of equilibrium, strain compatibility, the resistance factors as specified in Article 5.5.4.2, and the following assumptions:

- In components with fully bonded reinforcement or prestressing, or in the bonded length of locally debonded or shielded strands, strain is directly proportional to the distance from the neutral axis, except for deep members that shall satisfy the requirements of Article 5.13.2 (refer to current edition of AASHTO LRFD), and for other disturbed regions.

C5.7.2.1

The first paragraph of C5.7.1 applies

- In components with fully unbonded or partially unbonded prestressing tendons, i.e., not locally debonded or shielded strands, the difference in strain between the tendons and the concrete section and the effect of deflections on tendon geometry are included in the determination of the stress in the tendons.

- If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003.

- If the concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized if verified. Calculation of the factored resistance shall consider that the concrete cover may be lost at strains compatible with those in the confined concrete core.

- The stress in the CFRP reinforcement is based on a linear stress-strain relationship representative of the CFRP or on an approved mathematical representation, including development of reinforcing and prestressing elements and transfer of pretensioning.

- The tensile strength of 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 the test results.

- The development of reinforcing and prestressing elements and transfer of pretensioning are considered.

- Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its design guaranteed strength f_{gu} just as the concrete in compression reaches its assumed ultimate strain of 0.003.

- The CFRP design guaranteed strength f_{gu} is the Suggested CFRP Guaranteed Strength reduced by the appropriate reduction factor to account for any loss of strength during the service life of the structure.

- Sections are compression-controlled when the net tensile strain in the extreme tension CFRP is less than the net guaranteed strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The net guaranteed strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For all prestressed CFRP reinforcement, the net guaranteed strain limit may be taken as the specified guaranteed ultimate strain exclusive of the

Steel reinforced/prestressed sections are classified as tension-controlled or compression-controlled according to the net tensile steel strain at the time the concrete in compression reaches its assumed strain limit of 0.003.

strain due to pretress, creep, shrinkage, and temperature.

- Sections are tension-controlled when the strain of concrete in compression is less than 0.003 just as the net tensile strain in the extreme tension CFRP reaches its net guaranteed strain limit.

- The use of compression CFRP reinforcement in conjunction with additional tension reinforcement is not permitted to increase the strength of flexural members.

- In flexural members with n layers of tensile CFRP reinforcement, the flexural capacity is calculated based on an equivalent area of CFRP reinforcement A_{fe}. The equivalent area of CFRP reinforcement is a discrete area of CFRP reinforcement positioned at the extreme CFRP layer that results in the same flexural capacity of n layers of reinforcement. A_{fe} is calculated as:

$$A_{fe} = \sum_{i=1}^{n} A_{fe(i)} \qquad (5.7.2.1\text{-}1)$$

- where $A_{fe(i)}$ is the equivalent area for the area of CFRP reinforcement at layer i and can be calculated as:

$$A_{fe(i)} = \frac{d_i - c}{d_1 - c}(n_i\, a_f) \qquad (5.7.2.1\text{-}2)$$

where:

d_i = depth of the *ith* CFRP layer from the extreme compression fiber (in.)
d_1 = depth of the extreme CFRP layer from the extreme compression fiber (in.)
c = Depth of neutral axis from extreme compression fiber (in.)
n_i = number of CFRP strands in the *ith* layer
a_f = area of single CFRP strand in the *ith* layer (in.^{2})

- The depth of the neutral axis from the extreme compression fiber, c, can be initially set equal to $0.1d_1$ and Eq. 5.7.2.1-2 can be rewritten as:

$$A_{fe(i)} = \left(1 - \frac{s_i}{0.9d_1}\right)(n_i\, a_f) \qquad (5.7.2.1\text{-}3)$$

s_i = Distance between ith CFRP layer and extreme CFRP layer (in.) = $d_1 - d_i$

However, CFRP reinforced/prestressed sections are classified as tension or compression-controlled based on the actual failure mode whether it is crushing of the concrete or rupture of the CFRP reinforcement. If the concrete reaches a crushing strain of 0.003 while the net strain in the extreme CFRP remains less than net guaranteed tensile strain, the section is regarded as compression-controlled section. If the net extreme CFRP strain reaches its guaranteed net tensile strain, while the concrete strain remains less than 0.003, the section is regarded as tension-controlled section.

CFRP reinforcement has a significantly lower compressive strength than its tensile strength, and is subject to significant variation (Kobayashi and Fujisaki 1995; JSCE 1997b). Therefore, the strength of any CFRP bar in compression should be ignored in design calculations (Almusallam et al. 1997).

Due to the elastic nature of CFRP material, when the tension CFRP reinforcement is distributed over multiple layers, the failure of tension-controlled sections is usually governed by the failure of CFRP reinforcement at the extreme layer, which is the layer farthest from the compression fiber. CFRP reinforcements at layers closer to the compression fiber are likely to fail progressively once CFRP reinforcement at the extreme layer fails. It is therefore not recommended to sum the layers of CFRP reinforcements through their center of gravity. Rather, the areas of CFRP reinforcement at different layers are converted to equivalent areas at the level of the extreme layer with appropriate area reduction factors. The sum of equivalent reinforcement areas at the extreme layer is regarded as the equivalent area of reinforcement and is used to calculate the depth of the neutral axis and the nominal moment capacity of the section.

This area reduction factor is obtained by assuming linear strain distribution through the depth of the section. Thereby, the area of CFRP reinforcement at the ith layer is reduced with a factor depending on the distance from the ith layer to the extreme layer.

The net tensile strain at any layer (i) is related to the net tensile strain at the extreme CFRP layer by:

$$\varepsilon_i = \varepsilon_1 \left(\frac{d_i - c}{d_1 - c}\right) \qquad (C5.7.2.1\text{-}1)$$

where:

ε_i = net tensile strain at the ith CFRP reinforcement layer
ε_1 = net tensile strain at the extreme CFRP layer
d_i = depth of the *ith* CFRP layer from the extreme compression fiber (in.)
d_1 = depth of the extreme CFRP layer from the extreme compression fiber (in.)

c = depth of neutral axis from the extreme compression fiber (in.)

The tensile force, T_i, in any CFRP layer (i) may be calculated as:

$$T_i = \varepsilon_i \, n_i \, a_f \, E_f \quad \text{(C5.7.2.1-2)}$$

Where:

ε_i = net tensile strain at the ith CFRP reinforcement layer
n_i = number of CFRP strands in the *ith* layer
a_f = area of single CFRP strand in the *ith* layer (in.^{2})
E_f = elastic modulus of CFRP (ksi)

Therefore, the force in ith layer of CFRP can be expressed as:

$$T_i = \varepsilon_1 \left(\frac{d_i - c}{d_1 - c}\right) n_i \, a_f \, E_f \quad \text{(C5.7.2.1-3)}$$

$$T_i = \varepsilon_1 \left(\frac{d_i - c}{d_1 - c} \, n_i \, a_f\right) E_f \quad \text{(C5.7.2.1-4)}$$

$$T_i = \varepsilon_1 \, A_{fe(i)} \, E_f \quad \text{(C5.7.2.1-5)}$$

The initial assumption of $c = 0.1d_1$ is based on observations from multiple experimental flexural tests of CFRP prestressed beams. This assumption usually yields accurate estimate for the depth of the neutral axis and the flexural capacity of the section. It needs not to be adjusted unless more refined calculations are required.

5.7.2.2—Rectangular Stress Distribution

The natural relationship between concrete stress and strain may be considered satisfied by an equivalent rectangular concrete compressive stress block of $0.85f_c'$ 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 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65.

C5.7.2.2

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 5.7.3 are based on the rectangular stress block.

The factor β_1 is basically 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.

For sections that consist of a beam with a composite slab of different concrete strength, and the compression block includes both types of concrete, it is conservative to assume the composite beam to be of uniform strength at the lower of the concrete strengths in the flange and web. If a more refined estimate of flexural capacity is warranted, a more rigorous analysis method should be used. Examples of such analytical techniques are presented in Weigel, Seguirant, Brice, and Khaleghi (2003) and Seguirant, Brice, and Khaleghi (2004).

5.7.3—Flexural Members

5.7.3.1—Stress in Prestressing CFRP at Nominal Flexural Resistance

5.7.3.1.1—Components with Bonded Tendons

For sections subjected to flexure about one axis where the approximate stress distribution specified in Article 5.7.2.2 is used, the depth of neutral axis shall be calculated from Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, whichever is applicable:

For tension-controlled flanged sections:

$$c = \frac{E_f\, A_{fe}\left(\varepsilon_{gu} - \varepsilon_{pe}\right) + P_e - 0.85\, f_c'\, h_f\,(b - b_w)}{0.85\, f_c'\, \beta_1\, b_w} \quad (5.7.3.1.1\text{-}1)$$

For tension-controlled rectangular sections:

$$c = \frac{E_f\, A_{fe}\left(\varepsilon_{gu} - \varepsilon_{pe}\right) + P_e}{0.85\, f_c'\, \beta_1\, b} \quad (5.7.3.1.1\text{-}2)$$

For compression controlled flanged sections:

$$0.85\, f_c'\, \beta_1\, b_w\, c + 0.85\, f_c'\, h_f\,(b - b_w) = E_f\, A_{fe}\, \varepsilon_{cu}\left(\frac{d_1}{c} - 1\right) + P_e \quad (5.7.3.1.1\text{-}3)$$

For compression-controlled rectangular sections:

$$0.85\, f_c'\, \beta_1\, b\, c = E_f\, A_{fe}\, \varepsilon_{cu}\left(\frac{d_1}{c} - 1\right) + P_e \quad (5.7.3.1.1\text{-}4)$$

where:

b = width of compression face of the member; for a flanged section in compression, the effective width of the flange as specified in Article 4.6.2.6 of current edition of AASHTO LRFD (in.)
P_e = effective prestressing force in the section (kip)
E_f = elastic modulus of CFRP (ksi)
ε_{cu} = Average concrete crushing strain, 0.003
ε_{gu} = Design guaranteed strain of CFRP including environmental and durability effects
ε_{pe} = effective prestressing strain in CFRP after subtracting applicable prestress losses

C5.7.3.1.1

To calculate the depth of the neutral axis, the section may be initially assumed as tension-controlled rectangular section and the depth of the neutral axis is calculated using Eq. 5.7.3.1.1.-2. The initial assumptions are then verified by calculating the depth of the stress block $a = \beta_1 c$ and calculating the compressive strain at the extreme compression fiber and the net tensile strain at the extreme CFRP layer. If any of the assumptions is not correct, the depth of the neutral axis is recalculated using the appropriate Eq.

h_f = depth of compression flange (in.)
b_w = width of web (in.)

5.7.3.1.2—Components with Unbonded Tendons

Not Applicable

5.7.3.1.3—Components with both Bonded and Unbonded Tendons

5.7.3.1.3a——Detailed Analysis
Not Applicable
5.7.3.1.3b——Simplified Analysis
Not Applicable

5.7.3.2—Flexural Resistance

5.7.3.2.1—Factored Flexural Resistance

The factored resistance M_r shall be taken as:

$$M_r = \emptyset\, M_n \qquad (5.7.3.2.1\text{-}1)$$

where:

M_n = nominal resistance (kip-in.)
$\emptyset$ = resistance factor as specified in Article 5.5.4.2

C5.7.3.2.1

Refer to current edition of AASHTO LRFD

5.7.3.2.2—Flanged Sections

For flanged sections subjected to flexure about one axis, where the approximate stress distribution specified in Article 5.7.2.2 is used and where the compression flange depth is less than $a = \beta_1 c$, as determined in accordance with Eqs. 5.7.3.1.1-1, 5.7.3.1.1-2, 5.7.3.1.1-3, or 5.7.3.1.1-4, whichever is applicable, the nominal flexural resistance may be taken as:

$$M_n = \sum_{i=1}^{n}\left[a_f\, n_i\, \varepsilon_i\, E_f\left(d_i - \frac{a}{2}\right)\right] + P_e\left(d_p - \frac{a}{2}\right) + 0.85\, f_c'\, h_f\,(b - b_w)\left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad (5.7.3.2.2\text{-}1)$$

where:

a_f = area of single CFRP strand in the *ith* layer (in.^{2})
n_i = number of CFRP strands in the *ith* layer
ε_i = net tensile strain at the ith layer of CFRP reinforcement determined from strain compatibility, taken equal to $\varepsilon_1\left(\frac{d_i - c}{d_1 - c}\right)$

ε_1 = net tensile strain at the extreme CFRP layer
d_i depth of the *ith* CFRP layer from the extreme compression fiber (in.)
d_1 depth of the extreme CFRP layer from the extreme compression fiber (in.)
f_c' = Specified compressive strength of concrete at 28 days, unless another age is specified (ksi)
P_e = effective prestressing force in the section (kip)
d_p = Distance from the extreme compression fiber to the centroid of prestressing strands (in.)
E_f = elastic modulus of CFRP (ksi)
h_f = depth of compression flange (in.)
b width of compression face of the member; for a flanged section in compression, the effective width of the flange as specified in Article 4.6.2.6 of current edition of AASHTO LRFD (in.)
b_w = width of web (in.)
a = $\beta_1 c$; depth of the equivalent stress block (in.)
β_1 = stress block factor specified in Article 5.7.2.2
c = depth of neutral axis from extreme compression fiber as determined from Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, whichever is applicable (in.)

5.7.3.2.3—Rectangular Sections

For rectangular sections subjected to flexure about one axis, where the approximate stress distribution specified in Article 5.7.2.2 is used and where the compression flange depth is not less than $a = \beta_1 c$ as determined in accordance with Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, whichever is applicable (in.), the nominal flexural resistance M_n may be determined by using Eq. 5.7.3.2.2-1, in which case b_w shall be taken as b.

5.7.3.2.4—Other Cross-Sections

For cross-sections other than flanged or essentially 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 5.7.2. The requirements of Article 5.7.3.3 shall apply.

5.7.3.2.5—Strain compatibility Approach

The equivalent area method in Article 5.7.2 and subsequent Eqs in Article 5.7.3 are based on strain compatibility approach with an initial assumption for the depth of the neutral axis as 0.1 times the depth of the extreme CFRP reinforcement from the extreme compression fiber. If more refined calculations are required, a revised assumption for the depth of the neutral axis may be used based on the calculated neutral axis depth as determined in accordance with Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, whichever is applicable. Alternatively, the strain computability approach may be used. The stress and corresponding strain at any layer of reinforcement may be taken from any representative stress-strain formula or graph for CFRP bars or strands.

5.7.3.3—Limits for Reinforcement

5.7.3.3.1—Maximum Reinforcement

There is no maximum CFRP reinforcement.

C5.7.3.3.1

Although there is no maximum CFRP reinforcement limit, compression-controlled sections should be avoided whenever possible in bridge beam design. The nominal moment capacity of compression-controlled sections will be directly related to the conditions and strength of the concrete in the bridge deck, which is the most susceptible element to environmental conditions and deterioration.

5.7.3.3.2—Minimum Reinforcement

Unless otherwise specified, at any section of a non-compression controlled flexural component, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored resistance, M_r , at least equal to the lesser of:

- 1.15 times the factored moment required by applicable strength load combination specified in Table 3.4.1-1
- $M_{cr} = (\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)$

(5.7.3.3.2-1)

where:

f_r = modulus of rupture of concrete specified in Article 5.4.2.6 (ksi)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc} = total unfactored dead load moment acting on the monolithic or non-composite 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 non-composite section where

C5.7.3.3.2

Minimum reinforcement provisions are intended to reduce the probability of brittle failure by providing flexural capacity greater than the cracking moment. In CFRP prestressed elements, ACI 440 committee recommends that the factored resistance M_r be at least 150 % the cracking moment. However, no specifications are given for the calculations of the cracking moment. Since Eq. 5.7.3.3.2-1 is refined to account for the variability of modulus of rupture and prestressing force through the factors γ_1 and γ_2, respectively, it is no longer mandatory to impose the requirements of ACI 440 committee.

AASHTO LRFD (2014) recommends that for steel reinforced sections, the total amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored resistance, M_r , at least equal to the lesser of 1.33 times the factored moment or M_{cr}. In other words, when M_r is at least equal to 1.33 times the factored moment, the provision of minimum reinforcement is waived. The factor of 1.33 is related to the resistance factors specified in AASHTO LRFD Article 5.5.4.2. For steel prestressed members, the $\emptyset$-factor is taken equal to 1.0 for sections with net tensile strain larger than 0.005 (ductile failure) and 0.75 for sections with net tensile strain of 0.002 or less (brittle failure) at the time of failure. When the factored resistance, M_r , is at least equal to 1.33 times the factored moment, the provision of minimum reinforcement is waived because the $\emptyset$-factor is now equal to 1.0/1.33 = 0.75, which is equal to that assigned for brittle sections.

In a similar manner, CFRP prestressed sections with net tensile strain exceeding 0.005 at the time for failure 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 non-composite section to resist all loads, S_{nc} shall be substituted for S_c in the above equation for the calculation of M_{cr}.

The following factors shall be used to account for variability in the flexural cracking strength of concrete, and variability of prestress.

γ_1 = flexural cracking variability factor, may be taken as 1.6
γ_2 = Prestress variability factor, may be taken as 1.1

The provisions for minimum spacing of reinforcement shall apply.

are assigned a Ø-factor of 0.85, while brittle sections with net tensile strain less than 0.002 at the time of failure are assigned a Ø-factor of 0.75. Therefore, the provision of minimum reinforcement may be waived when the resistance moment is at least 0.85/0.75 ≈ 1.15 times the factored moment.

Similar to steel prestressed/reinforced elements, the sources of variability in computing the cracking moment and resistance are appropriately factored (Holombo and Tadros, 2009). The factor applied to the modulus of rupture (γ_1) is greater than the factor applied to the amount of prestress (γ_2) to account for greater variability.

5.7.3.4—Control of Cracking by Distribution of Reinforcement

Not Applicable

5.7.3.5—Moment Redistribution

Not Applicable

C5.7.3.5

CFRP materials exhibit a linear-elastic behavior to failure. Moment redistribution in continuous beams or other statically indeterminate structures should not be considered in CFRP reinforced or prestressed concrete design.

5.7.3.6—Deformations

5.7.3.6.1—General

Refer to current edition of AASHTO LRFD

C5.7.3.6.1

Refer to current edition of AASHTO LRFD

5.7.3.6.2—Deflection and Camber

Deflection and camber calculations shall consider dead load, live load, prestressing, erection loads, concrete creep and shrinkage, and CFRP relaxation. For determining deflection and camber, the provisions for structural material behavior and deflection losses shall apply.

The instantaneous deflection, due to moment, at any point before or after cracking in a member can be evaluated using the direct elastic approach, taking the flexural rigidity EI as:

$$EI = \frac{M_a}{\varepsilon_s} y_s \qquad (5.7.3.6.2\text{-}1)$$

where:

C7.3.6.2

An experimental study (Grace et al. 2013) on bridge beam specimens with CFRP reinforcement has shown that the deflection can be calculated with high degree of accuracy using the depth of the neutral axis and the strain through the depth of the section with no need to calculate the cracked or the effective moment of inertia of the section. Elastic deflection equations under different loads can still be used after the section is cracked but with calculating effective EI using Eq. 5.7.3.6.2-1

M_a = maximum moment in a component at the section for which deformation is computed (kip-in)
ε_s = strain at any point in through the depth of the section
y_s = distance from the neutral axis to the point, where the strain is calculated (in.)

The strain in the concrete and reinforcement at different load levels in beams can be directly calculated with high level of accuracy using strain compatibility analysis and assuming either linear stress distribution at low concrete strain levels or equivalent stress block at high concrete strain levels. The deflection of the beam can also be accurately estimated whether the section is cracked or not by calculating the depth of the neutral axis and calculating the curvature of the section ε_s/y_s .

Unless a more exact determination is made, the long-time deflection may be taken as the instantaneous deflection multiplied by the following factor:

- 4.0: If the instantaneous deflection is based on an uncracked section
- 3.0: If the instantaneous deflection is based on a cracked section

5.7.3.6.3—Axial Deformation

Refer to current edition of AASHTO LRFD.

5.7.4—Compression Members

Not Applicable

5.7.5—Bearing

Refer to current edition of AASHTO LRFD.

5.7.6—Tension Members

Refer to current edition of AASHTO LRFD.

5.8—SHEAR AND TORSION

5.8.1—Design Procedures

5.8.1.1—Flexural Regions

Refer to current edition of AASHTO LRFD.

C.5.8.1.1

Refer to current edition of AASHTO LRFD.

5.8.1.2—Regions near Discontinuities

Refer to current edition of AASHTO LRFD.

C5.8.1.2

Refer to current edition of AASHTO LRFD.

5.8.1.3—Interface Regions

Refer to current edition of AASHTO LRFD.

5.8.1.4—Slabs and Footings

Not Applicable.

5.8.1.5—Webs of Curved Post-Tensioned Box Girder Bridges

Not Applicable.

C5.8.1.5

Not Applicable.

5.8.2—General Requirements

5.8.2.1—General

Refer to current edition of AASHTO LRFD.

5.8.2.2—Modifications for Lightweight Concrete

Refer to current edition of AASHTO LRFD.

C.5.8.2.2

Refer to current edition of AASHTO LRFD.

5.8.2.3—Transfer and Development Lengths

The minimum length required to transfer the full prestressing force from CFRP strand to the concrete can be calculated as:

$$L_t = \frac{f_{pi} d_b}{\alpha_t f'^{\,0.67}_{ci}} \qquad (5.8.2.3\text{-}1)$$

where:

L_t = transfer length of CFRP (in.)
f_{pi} = prestressing CFRP stress immediately prior to transfer (ksi)
d_b = diameter of CFRP bar or strand (in.)
f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)
α_t = Transfer length factor, equal to 0.875 for CFCC strands and 0.96 for Leadline

The development length of a CFRP bar or strand can be calculated as:

$$L_d = \frac{(f_{gu} - f_{pe}) d_b}{\alpha_d f'^{0.67}_c} \qquad (5.8.2.3\text{-}2)$$

Where:

L_d = development length of CFRP (in.)
f_{gu} = design guaranteed strength of CFRP (ksi)
f_{pe} = stress in prestressing CFRP due to prestress after losses (ksi)

C5.8.2.3

A more general estimate for the transfer length of CFCC strands can be taken as 50 times the diameter of the strand.

The equation for development length is adopted from ACI-440.4R-04. Test results (Grace 2017) showed that the bond strength between CFCC strand with a diameter of 0.6 in. and uncrakced concrete block with a 28-day compressive strength of approximately 10 ksi averaged 4.0 kip/in. Considering a guaranteed strength of 60.7 kip, the development length is approximately 15 in.

Further Testing (Grace 2017) showed that the pull-out of a CFRP strand from uncracked concrete after 2 million load cycles is unlikely if the bond stress is limited to 64% of the average ultimate bond strength. For any

d_b = diameter of CFRP bar or strand (in.)
f_c' = specified compressive strength of concrete at 28 days (ksi)
α_d = development length factor, equal to 1.5

number of load cycles less than 2 million cycles, the bond stress shall be limited to:

$$f_{bN} = [0.87 - 0.016\ln(N)]f_b \qquad \text{(C5.8.2.3-1)}$$

where:

f_b = average ultimate bond strength between CFRP and uncracked concrete evaluated experimentally according to test method described in ACI 440.3R-12- B.3 (ksi)
N = number of load cycles
f_{bN} = allowable bond strength after N load cycles (ksi)

Therefore, development length of 0.6-in. CFCC strand in uncracked concrete shall be at least 15/0.64 ≈ 24 in. On the other hand, Eq. 5.8.2.3.2 estimates the development length as 29 in.

5.8.2.4—Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

- $V_u > 0.5\,\emptyset\,(V_c + V_p)$ (5.8.2.4-1)

Or

- Where consideration of torsion is required by Eq. 5.8.2.1-3 or Eq. 5.8.6.3-1

where:

V_u = factored shear force (kip)
V_c = nominal shear resistance of the concrete (kip)
V_p = component of prestressing force in direction of the shear force; $V_p = 0$ when the simplified method of 5.8.3.4.3 is used (kip)
$\emptyset$ = resistance factor specified in Article 5.5.4.2

C.5.8.2.4

Refer to current edition of AASHTO LRFD.

5.8.2.5—Minimum Transverse Reinforcement

Where transverse reinforcement is required as specified in Article 5.8.2.4, the area of CFRP shall satisfy:

$$A_v \geq 0.0316\sqrt{f_c'}\,\frac{b_v\,S}{f_f} \qquad \text{(5.8.2.5-1)}$$

where:

C5.8.2.5

Refer to current edition of AASHTO LRFD.

Members with CFRP transverse reinforcement may be provided with minimum transverse reinforcement calculated based on a maximum transverse strain of 0.0035. Research studies (Grace et al. 2014) showed that a section with CFRP stirrups can reach a transverse strain in excess of 0.0035 before the section losses its integrity and collapses under the shear loads.

A_v = area of a transverse reinforcement within a distance S (in.^{2})
b_v = width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in.)
S = spacing of transverse reinforcement (in.)
f_f = stress in the transverse CFRP reinforcement (ksi) corresponding to a strain of 0.0035

Segmental post-tensioned construction is excluded from current provision until further research data become available.

5.8.2.6—Types of Transverse Reinforcement

Transverse reinforcement to resist shear may consist of:

- Stirrups perpendicular to the longitudinal axis of the member;
- CFRP grids;
- Anchored prestressed tendons, detailed and constructed to minimize seating and time-dependent losses, which make an angle not less than 45 degrees with the longitudinal tension reinforcement;
- Combinations of stirrups and tendons;
- Spiral or hoops;
- Inclined stirrups making an angle of not less than 45 degrees with the longitudinal tension member.

Inclined stirrups shall be spaced so that every 45-degree line, extending towards the reaction from mid-depth of the member, h/2, to the longitudinal tension reinforcement shall be crossed by at least one line of transverse reinforcement.

Transverse reinforcement shall be detailed such that the shear force between different elements or zones of a member are effectively transferred.

Torsional reinforcement shall consist of both transverse and longitudinal reinforcement. Longitudinal reinforcement shall consist of bars and/or tendons. Transverse reinforcement may consist of:

- Closed stirrups or closed ties, perpendicular to the longitudinal axis of the member, as specified in Article 5.11.2.6.4
- Spiral or hoops

C5.8.2.6

Stirrups inclined at less than 45 degrees to the longitudinal reinforcement are difficult to anchor effectively against slip and, hence, are not permitted. Inclined stirrups and prestressed tendons should be oriented to intercept potential diagonal cracks at an angle as close to normal as practical.

To increase shear capacity, transverse reinforcement should be capable of undergoing a strain of 0.0035 prior to failure.

For some large bridge girders, prestressed tendons perpendicular to the member axis may be an efficient form of transverse reinforcement. Because the tendons are short, care must be taken to avoid excessive loss of prestress due to anchorage slip or seating losses. The requirements for transverse reinforcement assume it is perpendicular to the longitudinal axis of prismatic members or vertical for non-prismatic or tapered members. Requirements for bent bars were added to make the provisions consistent with those in AASHTO (2002).

5.8.2.7—Maximum Spacing of Transverse Reinforcement

C5.8.2.7

Refer to current edition of AASHTO LRFD.

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, S_{max}, determined as:

- If $v_u < 0.125\ f_c'$, then:

$$S_{max} = 0.8\ d_v \leq 24.0\ in. \qquad (5.8.2.7\text{-}1)$$

- If $v_u \geq 0.125\ f_c'$, then:

$$S_{max} = 0.4\ d_v \leq 12.0\ in. \qquad (5.8.2.7\text{-}2)$$

where:

v_u = the shear stress calculated in accordance with 5.8.2.9 (ksi)

d_v = effective shear depth as defined in Article 5.8.2.9 (in.)

Segmental bridge construction is not included in the article

5.8.2.8—Design and Detailing Requirements

Adequate end anchorage shall be provided for CFRP transverse reinforcement.

The design strength of CFRP nonprestressed transverse reinforcement shall be taken as the stress corresponding to a strain of 0.0035. The design strength of prestressed transverse reinforcement shall be taken as the effective stress, after allowance for all prestress losses, plus the stress corresponding to a strain of 0.0035.

C5.8.2.8

To be effective, the transverse reinforcement should be anchored at each end in a manner that minimizes slip.

The provisions of Article 5.8.3 are based on the assumption that the strain in the transverse reinforcement has to attain a value of 0.0035 before the section losses its integrity. For prestressed tendons, it is the additional strain required to increase the stress above the effective stress caused by the prestress that is of concern. Limiting the design strength of nonprestressed transverse reinforcement to a stress corresponding to a strain of 0.0035 provides control of crack widths at service limit state.

The components in the direction of the applied shear of inclined flexural compression and inclined flexural tension can be accounted for in the same manner as the component of the longitudinal prestressing force, V_p.

5.8.2.9—Shear Stress on Concrete

The shear stress on the concrete shall be determined as:

$$V_u = \frac{|V_u - \emptyset\ V_p|}{\emptyset\ b_v\ d_v} \qquad (5.8.2.9\text{-}1)$$

where:

$\emptyset$ = resistance factor for shear specified in Article 5.5.4.2

C5.8.2.9

For flexural members, the distance between the resultants of the tensile and compressive forces due to flexure can be determined as:

$$d_v = \frac{M_n}{A_{pf}\ f_{pf} + A_f\ f_f} \qquad (C5.8.2.9\text{-}1)$$

In continuous members near the point of inflection, if Eq. C5.8.2.9-1 is used, it should be evaluated in terms of both the top and the bottom reinforcement. Note that other limitations on the value of d_v to be used are specified and

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.)

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 $0.9d_e$ or $0.72h$ (in.)

in which:

$$d_e = \frac{A_{pf}\, f_{pf}\, d_p + A_f\, f_f\, d_f}{A_p\, f_{pf} + A_f\, f_f} \qquad (5.8.2.9\text{-}2)$$

In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width.

that d_v is the value at the section at which shear is being investigated.

Refer to current edition of AASHTO LRFD.

5.8.3—Sectional Design Model

5.8.3.1—General

Refer to current edition of AASHTO LRFD.

C5.8.3.1

Refer to current edition of AASHTO LRFD.

5.8.3.2—Sections near Supports

Refer to current edition of AASHTO LRFD.

C5.8.3.2

Refer to current edition of AASHTO LRFD.

5.8.3.3—Nominal Shear Resistance

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_f + V_p \qquad (5.8.3.3\text{-}1)$$

$$V_n = 0.20\, f_c'\, b_v\, d_v + V_p \qquad (5.8.3.3\text{-}2)$$

in which:

$V_c = 0.0316\, \beta \sqrt{f_c'}\, b_v d_v$, if the procedures of Articles 5.8.3.4.1 or 5.8.3.4.2 are used (5.8.3.3-3)

V_c = the lesser of V_{ci} and V_{cw} if the procedures of Article 5.8.3.4.3 are used

$$V_f = \frac{A_v\, f_f\, d_v\, (\cot\theta + \cot\alpha) + \sin\alpha}{S} \qquad (5.8.3.3\text{-}4)$$

where:

C5.8.3.3

The shear resistance of a concrete member may be separated into a component, V_c, that relies on tensile stresses in the concrete, a component, V_f, that relies on tensile stresses in the transverse reinforcement, and a component, V_p, that is the vertical component of the prestressing force.

The expressions for V_c and V_f apply to both prestressed and nonprestressed sections, with the terms β and θ depending on the applied loading and the properties of the section.

The upper limit of V_n, given by Eq. 5.8.3.3-2, is based on the maximum experimental shear strength obtained through the experimental investigation (Grace et al., 2014) and is intended to ensure that the concrete in the web of the beam will not crush prior to reaching a design strain of 0.0035 in transverse CFRP reinforcement

where α = 90 degrees, Eq. 5.8.3.3-4 reduces to:

$$V_f = \frac{A_v\, f_f\, d_v \cot\theta}{S} \qquad (C5.8.3.3\text{-}1)$$

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)
d_v = effective shear depth as determined in Article 5.8.2.9 (in.)
S = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)
β factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4
θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, $cot\ \theta$ is defined therein
α = angle of inclination of transverse reinforcement to longitudinal axis (degrees)
A_v = area of transverse reinforcement within a distance S (in.^{2})
V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear; $V_p = 0$ when Article 5.8.3.4.3 is applied (kip)

The angle θ is, therefore, also taken as the angle between a strut and the longitudinal axis of a member.

V_p is part of V_{cw} by the method in Article 5.8.3.4.3 and thus V_p need be taken as zero in Eq. 5.8.3.3-1.

Bent longitudinal CFRP reinforcement is currently unavailable and therefore, no provision is provided for shear resistance in members with bent reinforcement.

Where more than one type of transverse reinforcement is used to provide shear resistance in the same portion of a member, the shear resistance V_p shall be determined as the sum of V_p values computed from each type.

5.8.3.4—Procedures for Determining Shear Resistance

Design for shear may utilize any of the three methods identified herein provided that all requirements for usage of the chosen method are satisfied.

C5.8.3.4

Three complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.8.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in Article 5.8.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load. Two approaches are presented in Method 2: a direct calculation, specified in Article 5.8.3.4.2, and an evaluation using tabularized values presented in Appendix B5. The approaches to Method 2 may be considered statistically equivalent. Method 3, specified in Article 5.8.3.4.3, is applicable for both prestressed and nonprestressed sections in which there is no net axial tensile load and at least minimum shear reinforcement is provided. Axial load effects can otherwise be accounted for through adjustments to the level of effective precompression stress, f_{pc}. In regions of overlapping applicability between the latter two methods, Method 3 will generally lead to somewhat more shear reinforcement being required, particularly in areas of negative moment and near points of contraflexure. If Method 3 leads to an unsatisfactory rating, it is permissible to use Method 2.

5.8.3.4.1—Simplified Procedure for Nonprestressed Sections

For concrete footings in which the distance from point of zero shear to the face of the column, pier or wall is less than $3d_v$ with or without transverse reinforcement, and for other nonprestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, or having an overall depth of less than 16.0 in., the following values may be used:

$\beta = 2.0$
$\theta = 45°$

C5.8.3.4.1

With β taken as 2.0 and θ as 45 degrees, the expressions for shear strength become essentially identical to those traditionally used for evaluating shear resistance. Recent large-scale experiments (Shioya et al. 1989), however, have demonstrated that these traditional expressions can be seriously unconservative for large members not containing transverse reinforcement.

5.8.3.4.2—General Procedure

The general procedure is similar to that currently recommended for steel prestressed beams with steel stirrups. The parameters β and θ may be determined by the provisions herein.

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, the value of β may be determined by Eq. 5.8.3.4.2-1

$$\beta = \frac{4.8}{(1 + 750\,\varepsilon_f)} \quad (5.8.3.4.2\text{-}1)$$

When sections do not contain at least the minimum amount of shear reinforcement, the value of β may be as specified in Eq. 5.8.3.4.2-2:

$$\beta = \frac{4.8}{(1 + 750\,\varepsilon_f)}\frac{51}{(39 + s_{xe})} \quad (5.8.3.4.2\text{-}2)$$

The value of θ in both cases may be as specified in Eq 5.8.3.4.2-3:

$$\theta = 29 + 3500\,\varepsilon_f \quad (5.8.3.4.2\text{-}3)$$

In Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3, ε_f is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement. In lieu of more involved procedures, ε_f may be determined by Eq. 5.8.3.4.2-4:

$$\varepsilon_f = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{pf}f_{po}\right)}{E_fA_f + E_pA_{pf}} \quad (5.8.3.4.2\text{-}4)$$

The crack spacing parameter, s_{xe} shall be determined as:

C5.8.3.4.2

Due to the relatively low elastic modulus of CFRP compared to that of steel, beams prestressed with CFRP strands experience higher post-cracking bottom strains (ε_f) than those observed in beams prestressed with steel strands. This is especially true when comparing the strain at a same load level beyond cracking and before the yield of steel reinforcement.

Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-5 are similar to those recommended for steel prestressed beams in AASHTO LRFD. According to the equations, increasing ε_f results in decreasing both V_c and V_f and thereby, reducing the overall shear capacity of the section.

Consequently, due to the higher bottom strain in CFRP prestressed beams, the equations tend to yield remarkably low shear capacities when used in beams prestressed with CFPR strands.

Grace et. al (2014) conducted a shear testing study on beams prestressed with CFCC strands and transversely reinforced with CFCC stirrups on one side steel stirrups on the other side. The reported net tensile strain in the bottom reinforcement at the time of shear failure far exceeded the limit of 0.006 noted herein as an upper limit of ε_f. Therefore, the nominal shear capacities calculated based on Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-5 were always less than those observed experimentally. For instance, without enforcing ε_f upper limit of 0.006, and based on the shear testing of eighteen half-scale CFCC prestressed decked bulb T beam specimens (nine with steel stirrups and nine with CFCC stirrup), current AASHTO LRFD equations were conservative with an average experimental to theoretical shear capacity of 1.82 (maximum of 2.28 and minimum of 1.1) and a standard deviation of 0.35. With enforcing the ε_s upper limit of 0.006, current AASHTO equations were conservative with an average experimental to theoretical shear capacity

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \qquad (5.8.3.4.2\text{-}5)$$

where:

A_c = area of concrete on the flexural tension side of the member (in.^{2})

A_{pf} = area of prestressing CFRP on the flexural tension side of the member (in.^{2})

A_f = area of nonprestressed CFRP on the flexural tension side of the member (in.^{2})

a_g maximum aggregate size (in.)

f_{po} = a parameter taken as modulus of elasticity of the prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value equal to initial prestressing stress immediately prior to transfer will be appropriate for pretensioned members

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

$|M_u|$= absolute value of the factored moment, not to be taken less than $|V_u - V_p|d_v$ (kip-in.)

s_x = the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003\, b_v\, s_x$ (in.)

V_u = factored shear force (kip)

Within the transfer length, f_{po} shall be increased linearly from zero at the location where the bond between the strands and the concrete commences to its full value at the end of the transfer length.

The flexural tension side of the member shall be taken as the half-depth containing the flexural tension zone.

In the use of Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-5, the following should be considered:

- $|M_u|$ should not be taken less than $|V_u - V_p|d_v$.

- In calculating A_f and A_{pf} the area of bars or tendons terminated less than their development length from the section under consideration should be reduced in proportion to their lack of full development.

- If the value of ε_f calculated from Eq. 5.8.3.4.2-4 is negative, it should be taken as zero or the value should be recalculated with the denominator of Eq. 5.8.3.4.2-4 replaced by $(E_f A_f + E_p A_{pf} + E_c A_{ct})$.

of 1.65 (maximum of 2.14 and minimum of 1.04) and a standard deviation of 0.32.

For the purpose of comparison, the shear testing of one half-scale steel prestressed decked bulb T beam specimen provided with steel stirrups showed experimental to theoretical shear capacity of 1.29. In addition, the shear testing of one half-scale steel prestressed decked bulb T beam specimen provided with CFCC stirrups showed an experimental to theoretical shear capacity of 1.14. Therefore, it can be concluded that current AASHTO eqn. are conservative when directly used to estimate the shear capacity of beams prestressed with CFCC strands regardless of the type of transverse reinforcement whether it is steel stirrups or CFCC stirrups.

However, ε_f should not be taken as less than -4×10^{-3}.

- For sections closer than d_v to the face of the support, the value of ε_f calculated at d_v from the face of the support may be used in evaluating β and θ.
- If the axial tension is large enough to crack the flexural compression face of the section, the value calculated from Eq. 5.8.3.4.2-4 should be doubled.
- It is permissible to determine β and θ from Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3 using a value of ε_f, which is greater than that calculated from Eq. 5.8.3.4.2-4. However ε_f should not be taken greater than 6.0×10^{-3}.

5.8.3.4.3—Simplified Procedure for Prestressed and Non Prestressed sections

For concrete beams not subject to significant axial tension, prestressed and nonprestressed, and containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, V_n in Article 5.8.3.3 may be determined with V_p taken as zero and V_c taken as the lesser of V_{ci} and V_{cw}, where:

V_{ci} = nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment (kip)

V_{cw} = nominal shear resistance provided by concrete when inclined cracking results from excessive principal tensions in web (kip)

V_{ci} shall be determined as:

$$V_{ci} = V_d + \frac{V_i\,M_{cre}}{M_{max}} \qquad (5.8.3.4.3\text{-}1)$$

where:

C5.8.3.4.3

Similar to members with steel reinforcement, Article 5.8.3.4.3 is based on the recommendations of NCHRP Report 549 (Hawkins et al., 2005). The concepts of this Article are compatible with the concepts of ACI Code 318-05 and AASHTO Standard Specifications for Highway Bridges (2002) for evaluations of the shear resistance of prestressed concrete members. However, those concepts are modified so that this Article applies to both prestressed and nonprestressed sections.

The nominal shear resistance V_n is the sum of the shear resistances V_c and V_f provided by the concrete and shear reinforcement, respectively. Both V_c and V_f depend on the type of inclined cracking that occurs at the given section. There are two types of inclined cracking: flexure-shear cracking and web-shear cracking for which the associated resistances are V_{ci} and V_{cw}, respectively.

Based on recent research study on beams prestressed with CFRP strands and transversely reinforced with CFRP stirrups on one side and steel stirrups on the other side (Grace et. al 2014), it was found that Eq. 5.8.3.4.3-1 in AASHTO LRFD (2012) tends to overestimate nominal shear resistance provided by concrete when inclined

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kip)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max}$ (kip)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in.)

M_{max} = maximum factored moment at section due to externally applied loads (kip-in.)

M_{cre} shall be determined as:

$$M_{cre} = S_c\left(f_r + f_{cpe} - \frac{M_{dnc}}{S_{nc}}\right) \qquad (5.8.3.4.3\text{-}2)$$

where:

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc}= total unfactored dead load moment acting on the monolithic or non-composite 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 non-composite section where tensile stress is caused by externally applied loads (in.^{3})

In Eq. 5.8.3.4.3.-1, M_{max} and V_i shall be determined from the load combination causing maximum moment at the section.

V_{cw} shall be determined as:

$$V_{cw} = f_r\sqrt{1 + \frac{f_{pc}}{f_r}}\, b_v\, d_v + V_p \qquad (5.8.3.4.3\text{-}3)$$

where:

f_{pc} = compressive stress in concrete (after allowance for all prestresss losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (ksi). In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at junction of web and flange, due to both prestresss and moments resisted by precast member acting alone.

cracking results from combined shear and moment, V_{ci}, and at the same time Eq. 5.8.3.4.3-3 tends to underestimate the nominal shear resistance provided by concrete when inclined cracking results from excessive principal tensions in web, V_{cw}; a situations that causes V_{cw} to be always less than V_{ci} and consequently $\cot\theta$ is taken according to Eq. 5.8.3.4.3 as larger than 1.0. The nominal shear capacity of the section, V_n, with $\cot\theta$ larger than 1.0 was consistently larger than the experimental shear capacity. Therefore, Eqs. 5.8.3.4.3-1, 5.8.3.4.3-3, and 5.8.3.4.3-4 are adjusted to obtain a more reliable estimation for the shear capacity of section with CFRP stirrups. A constant value of f_r shall be assumed in Eqs. 5.8.3.4.3-2, 5.8.3.4.3-3, and 5.8.3.4.3-4.

V_f shall be determined using Eq. 5.8.3.3-4 with $\cot\theta$ taken as follows:

where $V_{ci} < V_{cw}$:

$\cot\theta = 1.0$

where$V_{ci} > V_{cw}$:

$$\cot\theta = \sqrt{1 + \frac{f_{pc}}{f_r}} \leq 1.8 \qquad (5.8.3.4.3\text{-}4)$$

5.8.3.5—Longitudinal Reinforcement

Refer to current edition of AASHTO LRFD.

C5.8.3.5

Refer to current edition of AASHTO LRFD

5.8.3.6—Sections Subjected to Combined Shear and Torsion

Refer to current edition of AASHTO LRFD. Combined action of shear and torsion in CFRP prestressed members has not been investigated.

5.8.4—Interface Shear Transfer—Shear Friction

Refer to current edition of AASHTO LRFD.

5.8.5—Principal Stresses in Webs of Segmental Concrete Bridges

Not Applicable

5.8.6—Shear and Torsion for Segmental Box Girder Bridges

Not Applicable

5.9—PRESTRESSING

C5.9

This Article has been experimentally verified for flexural members fully prestressed with CFRP prestressing strands (Grace et al. 2017). The Article also covers flexural members containing combinations of CFRP prestressing strands and CFRP reinforcing bars. Non-prestressed and partially CFRP prestressed concrete structures is not covered under this Article.

5.9.1—General Design Considerations

5.9.1.1—General

C5.9.1.1

Refer to current edition of AASHTO LRFD

5.9.1.2—Specified Concrete Strengths

Refer to current edition of AASHTO LRFD

5.9.1.3—Buckling

Refer to current edition of AASHTO LRFD

5.9.1.4—Section Properties

Refer to current edition of AASHTO LRFD.

C5.9.1.4

Refer to current edition of AASHTO LRFD.

5.9.1.5—Crack Control

Flexural cracking is not permitted in CFRP prestressed concrete members.

C5.9.1.5

Wide flexural cracks may expose prestressing CFRP strands to a shearing force. Since CFRP strands are weak in shear, flexural cracking shall be avoided.

5.9.1.6—Tendons with Angle Points or Curves

Refer to current edition of AASHTO LRFD.

C5.9.1.6

Refer to current edition of AASHTO LRFD.

5.9.2—Stresses Due to Imposed Deformation

Refer to current edition of AASHTO LRFD.

C5.9.2

There is not enough literature or test results on monolithic frames prestressed with CFRP. Nevertheless, general rules of stress due to imposed deformation shall still apply.

5.9.3—Stress Limitations for Prestressing Tendons

The tendon stress due to prestress or at the service limit state shall not exceed the smaller of (1) the values recommended by the manufacturer or (2) as given in Table 5.9.3-1 as a function of the one-million-hour creep rupture strength, f_{cr}.

Table 5.9.3-1—Stress Limits for CFRP Prestressing Tendons and Strands

Condition	Stress
Immediately prior to transfer (f_{pbt})	0.80 f_{cr}
At service limit state after all losses (f_{pe})	0.75 f_{cr}

In pretensioning CFRP strands, seating loss is expected and it varies according to the anchorage device. Therefore, the jacking stress may be taken as the maximum allowable prestress limit prior to transfer plus the anticipated seating loss. In no case shall the stress in the CFRP strand exceed the smaller of:

- 0.75 times the guaranteed strength, and
- The one-million-hour creep rupture strength.

C5.9.3

The CFRP strands can be stressed safely to its anticipated one-million-hour creep rupture strength, f_{cr}. However, due to the uncertainty associated with estimating the creep rupture strength and to account for any possible degradation due to environmental conditions, it is recommended to limit the stress in CFRP strands immediately prior to transfer and at service to 0.80 f_{cr} and 0.75 f_{cr}, respectively.

Testing of 0.5, 0.6, and 0.7 in. carbon fiber composite cable (CFCC) strands showed a minimum estimate for the one-million-hour creep rupture strength of approximately 83 % of the guaranteed strength. The minimum estimate is calculated based on creep rupture testing of CFCC strands of different diameters as well as continuous monitoring of forty CFCC strands specimens with a diameter of 0.6 in. prestressed up to 88 % of their guaranteed strength for over three years at the time of writing this Article with 20 strands are exposed to outdoor environmental conditions while 20 strands are exposed to controlled laboratory conditions. As the strand specimens continue to hold the applied load over time, the estimate for one-million-hour creep rupture strength is expected to increase.

The minimum one-million hour creep rupture strength of 0.6 CFCC strand is 50.3 kip. Therefore, a CFCC strand with a diameter of 0.6 in. can be stressed to approximately 40.25 kip prior to transfer, while the stress at service limit state after all losses shall not exceed 37.7 kip. Nevertheless, other factors such as the strength of the anchorage device or the strength of the coupler system may govern the initial jacking force and shall be considered in design.

5.9.4—Stress Limits for concrete

5.9.4.1—For Temporary Stresses before Losses

5.9.4.1.1—Compression Stresses

Refer to current edition of AASHTO LRFD.

5.9.4.1.2—Tension Stresses

Refer to current edition of AASHTO LRFD

5.9.4.2—For Stresses at Service Limit State after Losses

5.9.4.2.1—Compression Stresses

Refer to current edition of AASHTO LRFD.

C5.9.4.2.1

5.9.4.2.2—Tension Stresses

For service load combinations that involve traffic loading, tension stresses in members with bonded or unbonded prestressing tendons should be investigated using Load Combination Service III specified in Table 3.4.1-1.

The limits in Table 5.9.4.2.2-1 shall apply.

C5.9.4.2.2

Severe corrosive conditions include exposure to deicing salt, water, or airborne sea salt and airborne chemicals in heavy industrial areas.

See Figure C5.9.4.1.2-1 for calculation of required area of bonded reinforcement.

Table 5.9.4.2.2-1—Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	No Tension
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	No Tension
	• For components with unbonded prestressing tendons	Not Applicable
Segmentally Constructed Bridges	• Not Applicable	

5.9.4.3—Partially Prestressed Components

Not Applicable

5.9.5—Loss of Prestress

5.9.5.1—Total Loss of Prestress

Values of prestress losses specified herein shall be applicable to normal weight concrete only and for specified concrete strengths up to 15.0 ksi, unless stated otherwise.

In lieu of more detailed analysis, prestress losses in members constructed and prestressed in a single stage, relative to the stress immediately before transfer, may be taken as:

- In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \pm \Delta f_{pTE} \qquad (5.9.5.1\text{-}1)$$

- In post-tensioned members:

Not Applicable

where:

Δf_{pT} = total loss (ksi)
Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)
Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of CFRP (ksi)
Δf_{pTE} = change in prestressing stress due to seasonal change in temperature (ksi)

C5.9.5.1

Refer to current edition of AASHTO LRFD.

In addition to short and long-term losses, CFRP strands experience loss or gain in effective prestressing force due to seasonal change in temperature

5.9.5.2—Instantaneous Losses

5.9.5.2.1—Anchorage Set

Refer To current edition of AASHTO LRFD

5.9.5.2.2—Friction

Not Applicable

5.9.5.2.3—Elastic Shortening

5.9.5.2.3a—Pretensioned Members

Refer to current edition of AASHTO LRFD.

C5.9.5.2.3a

Refer to current edition of AASHTO LRFD.

5.9.5.2.3b—Post-Tensioned Members

Not Applicable

5.9.5.2.3c—Combined Pretensioning and Post-Tensioning

Not Applicable

5.9.5.3—Approximate Estimate of Time-Dependent Losses

For standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- members are made from normal-weight concrete,
- the concrete is either steam- or moist-cured, and
- average exposure conditions and temperatures characterize the site,

the long-term prestress loss Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of CFRP shall be estimated using the following formula:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR} \quad (5.9.5.3\text{-}1)$$

$$\gamma_h = 1.7 - 0.01H \quad (5.9.5.3\text{-}2)$$

$$\gamma_{st} = \frac{5}{1 + f'_{ci}} \quad (5.9.5.3\text{-}3)$$

where:

f_{pi}	=	prestressing CFRP stress immediately prior to transfer (ksi)
f'_{ci}	=	specified compressive strength of concrete at time of initial loading or prestressing (ksi)
H	=	The average annual ambient relative humidity (%)
γ_h	=	correction factor for relative humidity of the ambient air
γ_{st}	=	correction factor for specified concrete strength at time of prestress transfer to the concrete member
Δf_{pR}	=	an estimate of relaxation loss taken as recommended by the CFRP manufacturer or as verified by testing

C5.9.5.2.3b

Not Applicable

C5.9.5.2.3c

Not Applicable

C5.9.5.3

Refer to current edition of AASHTO LRFD

The approximate estimates of time-dependent prestress losses given in Eq. 5.9.5.3-1 has been verified using half-scale CFRP prestressed bridge beams prototypes with and without composite deck. The approximate method should not be used for members of uncommon shapes, i.e., having *V/S* ratios much different from 3.5 in., level of prestressing, or construction staging. The first term in Eq. 5.9.5.3-1 corresponds to creep losses, the second term to shrinkage losses, and the third to relaxation losses.

Relaxation losses shall be taken as recommended by the CFRP manufacturer or as verified by testing. Monitoring of prestressed CFCC test specimens with a diameter of 0.6 in. showed that one-million hour relaxation loss is approximately 1.75% of the initial prestressing force after subtracting seating losses.

Short CFRP specimens evaluated for relaxation losses tend to overestimate the losses due to relaxation because of the relaxation of the anchorage devices, especially at higher stress level. Therefore, care should be taken to separate the losses due to relaxation of CFRP from that due to relaxation of the anchorage device. Otherwise, long test specimens shall be used to minimize the relaxation effect of anchorage devices.

Based on experimental evaluation for prestress loss in non-composite beams prestressed with CFRP strands and exposed to different weather conditions, Refined Method presented in Article 5.9.5.4 did not provide more accurate estimate for the time dependent losses in CFRP prestrssed members. Both the Approximate and the Refined methods yielded approximately the same estimate for prestress loss, which matched the prestress loss observed experimentally.

For girders other than those made with composite slabs, the time-dependent prestress losses resulting from creep and shrinkage of concrete and relaxation of CFRP shall be determined using the refined method of Article 5.9.5.4.

5.9.5.4—Estimate of Loss in Effective Prestressing Force due to Seasonal Temperature Drop

For standard precast, pretensioned members subject to normal loading and environmental conditions, the loss in effective prestressing force due to the seasonal drop in temperature during winter shall be estimated using the following formula:

$$\Delta f_{pTE} = (\alpha_c - \alpha_{CFRP})(T_{prestress} - T_{MinDesign})E_p \quad (5.9.5.4\text{-}1)$$

where:

Δf_{pTE} = Loss in effective prestressing force due to seasonal temperature change (ksi), taken as loss in effective prestressing when positive and gain in effective prestressing when negative.

α_c = concrete coefficient of thermal expansion of concrete as given in Article 5.4.2.2 (/°F)

α_{CFRP} = CFRP coefficient of thermal expansion of CFRP, taken as 0 /°F unless more accurate data are available.

T_p = average air temperature at the time of prestressing, taken as 68 °F unless more accurate data are available.

T_{Min} = Minimum temperatures specified in Article 3.12 using either Procedure A (3.12.2.1) or Procedure B (3.12.2.2) (°F).

E_p = elastic modulus of prestressing CFRP strands (ksi)

C5.9.5.4

Due to the difference between concrete and CFRP coefficient of thermal expansions, the effective prestressing force in beams prestressed with CFRP strands fluctuates with the seasonal temperature change. Grace et al. (2017) showed that CFRP prestressed beams constructed at ambient temperature (68 °F) experienced an increase in the effective prestressing force when they were loaded at 176 °F and experienced loss in the effective prestressing force when they were loaded at -40 °F. The increase or decrease in effective prestressing force was only temporarily and diminished as soon as the temperature of the beams returned to ambient.