Section 9
PRESTRESSED CONCRETE

Part A
GENERAL REQUIREMENTS AND MATERIALS

9.1 APPLICATION

9.1.1 General

The specifications of this section are intended for design of prestressed concrete bridge members. Members designed as reinforced concrete, except for a percentage of tensile steel stressed to improve service behavior, shall conform to the applicable specifications of Section 8.

Exceptionally long span or unusual structures require detailed consideration of effects which under this Section may have been assigned arbitrary values.

9.1.2 Notations

- \( A_s \) = area of non-prestressed tension reinforcement (Articles 9.7 and 9.19)
- \( A'_s \) = area of compression reinforcement (Article 9.19)
- \( A_t \) = area of prestressing steel (Article 9.17)
- \( A_{of} \) = steel area required to develop the compressive strength of the overhanging portions of the flange (Article 9.17)
- \( A_{wr} \) = steel area required to develop the compressive strength of the web of a flanged section (Articles 9.17-9.19)
- \( A_w \) = area of web reinforcement (Article 9.20)
- \( b \) = width of flange of flanged member or width of rectangular member
- \( b_c \) = width of cross section at the contact surface being investigated for horizontal shear (Article 9.20).
- \( b' \) = width of a web of a flanged member
- \( CR_c \) = loss of prestress due to creep of concrete (Article 9.16)
- \( CR_s \) = loss of prestress due to relaxation of prestressing steel (Article 9.16)
- \( D \) = nominal diameter of prestressing steel (Articles 9.17 and 9.27)
- \( d \) = distance from extreme compressive fiber to centroid of the prestressing force, or to centroid of negative moment reinforcing for precast girder bridges made continuous
- \( d_s \) = distance from the extreme compressive fiber to the centroid of the non-prestressed tension reinforcement (Articles 9.7 and 9.17-9.19)
- \( ES \) = loss of prestress due to elastic shortening (Article 9.16)
- \( e \) = base of Naperian logarithms (Article 9.16)
- \( f_{c0} \) = average concrete compressive stress at the c.g. of the prestressing steel under full dead load (Article 9.16)
- \( f_{cr} \) = average concrete stress at the c.g. of the prestressing steel at time of release (Article 9.16)
- \( f'_c \) = compressive strength of concrete at 28 days
- \( f_{ct} \) = compressive strength of concrete at time of initial prestress (Article 9.15)
- \( f_s \) = average splitting tensile strength of lightweight aggregate concrete, psi
- \( f_d \) = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20)
- \( f_{pc} \) = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (In a composite member, \( f_{pc} \) is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.) (Article 9.20)
- \( f_{pe} \) = 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 (Article 9.20)
\( f_{ps} \) = guaranteed ultimate tensile strength of the prestressing steel, \( A_f f'_{ps} \)

\( f_r \) = the modulus of rupture of concrete, as defined in Article 9.15.2.3 (Article 9.18)

\( \Delta f_s \) = total prestress loss, excluding friction (Article 9.16)

\( f_{ec} \) = effective steel prestress after losses

\( f_{su} \) = average stress in prestressing steel at ultimate load

\( f'_u \) = ultimate stress of prestressing steel (Articles 9.15 and 9.17)

\( f_{cy} \) = yield stress of non-prestressed conventional reinforcement in tension (Articles 9.19 and 9.20)

\( f'_c \) = yield stress of non-prestressed conventional reinforcement in compression (Article 9.19)

\( f'_{ys} \) = yield stress of prestressing steel (Article 9.15)

\( = 0.90 f'_c \) for low-relaxation wire or strand

\( = 0.85 f'_c \) for stress-relieved wire or strand

\( = 0.85 f'_c \) for Type I (smooth) high-strength bar

\( = 0.80 f'_c \) for Type II (deformed) high-strength bar

\( h \) = overall depth of member (Article 9.20)

\( I \) = moment of inertia about the centroid of the cross section (Article 9.20)

\( K \) = friction wobble coefficient per foot of prestressing steel (Article 9.16)

\( L \) = length of prestressing steel element from jack end to point x (Article 9.16)

\( M_{cr} \) = moment causing flexural cracking at section due to externally applied loads (Article 9.20)

\( M'_{cr} \) = cracking moment (Article 9.18)

\( M_{dc} \) = composite dead load moment at the section (Commentary to Article 9.18)

\( M_{dnc} \) = non-composite dead load moment at the section (Article 9.18)

\( M_{max} \) = maximum factored moment at section due to externally applied loads (Article 9.20)

\( M_n \) = nominal moment strength of a section

\( M_u \) = factored moment at section \( \leq \phi M_n \), (Articles 9.17 and 9.18)

\( p \) = \( A_f f'_{bd} \), ratio of non-prestressed tension reinforcement (Articles 9.7 and 9.17-9.19)

\( p' \) = \( A_f f'_{bd} \), ratio of prestressing steel (Articles 9.17 and 9.19)

\( p' \) = \( A_f f'_{bd} \), ratio of compression reinforcement (Article 9.19)

\( P_u \) = factored tendon force

\( Q \) = statical moment of cross-sectional area, above or below the level being investigated for shear, about the centroid (Article 9.20)

\( SH \) = loss of prestress due to concrete shrinkage (Article 9.16)

\( s \) = longitudinal spacing of the web reinforcement (Article 9.20)

\( S_o \) = noncomposite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads (Article 9.18)

\( S_c \) = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads (Article 9.18)

\( t \) = average thickness of the flange of a flanged member (Articles 9.17 and 9.18)

\( T_o \) = steel stress at jacking end (Article 9.16)

\( T_x \) = steel stress at any point x (Article 9.16)

\( \nu \) = permissible horizontal shear stress (Article 9.20)

\( V_c \) = nominal shear strength provided by concrete (Article 9.20)

\( V_{ci} \) = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment (Article 9.20)

\( V_{cw} \) = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web (Article 9.20)

\( V_i \) = shear force at section due to unfactored dead load (Article 9.20)

\( V_f \) = factored shear force at section due to externally applied loads occurring simultaneously with \( M_{max} \) (Article 9.20)

\( V_{sh} \) = nominal horizontal shear strength (Article 9.20)

\( V_p \) = vertical component of effective prestress force at section (Article 9.20)

\( V_s \) = nominal shear strength provided by shear reinforcement (Article 9.20)

\( V_{a} \) = factored shear force at section (Article 9.20)

\( Y_i \) = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension (Article 9.20)

\( \mu \) = friction curvature coefficient (Article 9.16)

\( \alpha \) = total angular change of prestressing steel profile in radians from jacking end to point x (Article 9.16)

\( \beta_i \) = factor for concrete strength, as defined in Article 8.16.2.7 (Articles 9.17-9.19)

\( \gamma^* \) = factor for type of prestressing steel (Article 9.17)

\[ \gamma^* = 0.28 \text{ for low-relaxation steel} \]

\[ \gamma^* = 0.40 \text{ for stress-relieved steel} \]

\[ \gamma^* = 0.55 \text{ for bars} \]
9.1.3 Definitions

The following terms are defined for general use. Specialized definitions appear in individual articles.

Anchorage Device—The hardware assembly used for transferring a post-tensioning force from the tendon wires, strands or bars to the concrete.

Anchorage Seating—Deformation of anchorage or seating of tendons in anchorage device when prestressing force is transferred from jack to anchorage device.

Anchorage Spacing—Center-to-center spacing of anchorage devices.

Anchorage Zone—The portion of the structure in which the concentrated prestressing force is transferred from the anchorage device into the concrete (Local Zone), and then distributed more widely into the structure (General Zone) (Article 9.21.1).

Basic Anchorage Device—Anchorage device meeting the restricted bearing stress and minimum plate stiffness requirements of Articles 9.21.7.2.2 through 9.21.7.2.4; no acceptance test is required for Basic Anchorage Devices.

Bonded Tendon—Prestressing tendon that is bonded to concrete either directly or through grouting.

Coating—Material used to protect prestressing tendons against corrosion, to reduce friction between tendon and duct, or to debond prestressing tendons.

Couplers (Couplings)—Means by which prestressing force is transmitted from one partial-length prestressing tendon to another.

Creep of Concrete—Time-dependent deformation of concrete under sustained load.

Curvature Friction—Friction resulting from bends or curves in the specified prestressing tendon profile.

Debonding (blanketing)—Wrapping, sheathing, or coating prestressing strand to prevent bond between strand and surrounding concrete.

Diaphragm—Transverse stiffener in girders to maintain section geometry.

Duct—Hole or void formed in prestressed member to accommodate tendon for post-tensioning.

Edge Distance—Distance from the center of the anchorage device to the edge of the concrete member.

Effective Prestress—Stress remaining in concrete due to prestressing after all calculated losses have been deducted, excluding effects of superimposed loads and weight of member; stress remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load.

Elastic Shortening of Concrete—Shortening of member caused by application of forces induced by prestressing.

End Anchorage—Length of reinforcement, or mechanical anchor, or hook, or combination thereof, beyond point of zero stress in reinforcement.

End Block—Enlarged end section of member designed to reduce anchorage stresses.

Friction (post-tensioning)—Surface resistance between tendon and duct in contact during stressing.

General Zone—Region within which the concentrated prestressing force spreads out to a more linear stress distribution over the cross section of the member (Saint Venant Region) (Article 9.21.2.1)

Grout Opening or Vent—Inlet, outlet, vent, or drain in post-tensioning duct for grout, water, or air

Intermediate Anchorage—Anchorage not located at the end surface of a member or segment; usually in the form of embedded anchors, blisters, ribs, or recess pockets

Jacking Force—Temporary force exerted by device that introduces tension into prestressing tendons.

Local Zone—The volume of concrete surrounding and immediately ahead of the anchorage device, subjected to high local bearing stresses (Article 9.21.2.2)

Loss of Prestress—Reduction in prestressing force resulting from combined effects of strains in concrete and steel, including effects of elastic shortening, creep and shrinkage of concrete, relaxation of steel stress, and for post-tensioned members, friction and anchorage seating.

Post-Tensioning—Method of prestressing in which tendons are tensioned after concrete has hardened.

Precompressed Zone—Portion of flexural member cross section compressed by prestressing force.

Prestressed Concrete—Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

Pretensioning—Method of prestressing in which tendons are tensioned before concrete is placed.

Relaxation of Tendon Stress—Time-dependent reduction of stress in prestressing tendon at constant strain.

Shear Lag—Nonuniform distribution of bending stress over the cross section.

Shrinkage of Concrete—Time-dependent deformation of concrete caused by drying and chemical changes (hydration process).

Special Anchorage Device—Anchorage device whose adequacy must be proven experimentally in the standardized acceptance tests of Division II, Section 10.3.2.3.
Tendon—Wire, strand, or bar, or bundle of such elements, used to impart prestress to concrete.
Transfer—Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.
Transfer Length—Length over which prestressing force is transferred to concrete by bond in pretensioned members.
Wobble Friction—Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment.
Wrapping or Sheathing—Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.

9.2 CONCRETE

The specified compressive strength, f’c, of the concrete for each part of the structure shall be shown on the plans. The requirements for f’c shall be based on tests of cylinders made and tested in accordance with Division II, Section 8, “Concrete Structures.”

9.3 REINFORCEMENT

9.3.1 Prestressing Steel

Wire, strands, or bars shall conform to one of the following specifications.

“Uncoated Stress-Relieved Wire for Prestressed Concrete,” AASHTO M 204.
“Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete,” AASHTO M 203.
“Uncoated High-Strength Steel Bar for Prestressing Concrete,” ASTM A 722.

Wire, strands, and bars not specifically listed in AASHTO M 204, AASHTO M 203, or ASTM A 722 may be used provided they conform to the minimum requirements of these specifications.

9.3.2 Non-Prestressed Reinforcement

Non-prestressed reinforcement shall conform to the requirements in Article 8.3.

Part B
ANALYSIS

9.4 GENERAL

Members shall be proportioned for adequate strength using these specifications as minimum guidelines. Continuous beams and other statically indeterminate structures shall be designed for adequate strength and satisfactory behavior. Behavior shall be determined by elastic analysis, taking into account the reactions, moments, shear, and axial forces produced by prestressing, the effects of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlement.

9.5 EXPANSION AND CONTRACTION

9.5.1 In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes.

9.5.2 Movements not otherwise provided for, including shortening during stressing, shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads, or other devices.

9.6 SPAN LENGTH

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam. The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth.

9.7 FRAMES AND CONTINUOUS CONSTRUCTION

9.7.1 Cast-in-Place Post-Tensioned Bridges

The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by pre-
stressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

9.7.2 Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous

9.7.2.1 General

When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans.

9.7.2.2 Positive Moment Connection at Piers

9.7.2.2.1 Provision shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

9.7.2.2.2 Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi.

9.7.2.3 Negative Moments

9.7.2.3.1 Negative moment reinforcement shall be proportioned by strength design with load factors in accordance with Article 9.14.

9.7.2.3.2 The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete.

9.7.3 Segmental Box Girders

9.7.3.1 General

9.7.3.1.1 Elastic analysis and beam theory may be used in the design of segmental box girder structures.

9.7.3.1.2 In the analysis of precast segmental box girder bridges, no tension shall be permitted across any joint between segments during any stage of erection or service loading.

9.7.3.1.3 In addition to the usual substructure design considerations, unbalanced cantilever moments due to segment weights and erection loads shall be accommodated in pier design or with auxiliary struts. Erection equipment which can eliminate these unbalanced moments may be used.

9.7.3.2 Flexure

The transverse design of segmental box girders for flexure shall consider the segments as rigid box frames. Top slabs shall be analyzed as variable depth sections considering the fillets between top slab and webs. Wheel loads shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load condition. (See Article 3.11.) Transverse prestressing of top slabs is generally recommended.

9.7.3.3 Torsion

In the design of the cross section, consideration shall be given to the increase in web shear resulting from eccentric loading or geometry of structure.

9.8 EFFECTIVE FLANGE WIDTH

9.8.1 T-Beams

9.8.1.1 For composite prestressed construction where slabs or flanges are assumed to act integrally with the beam, the effective flange width shall conform to the provisions for T-girder flanges in Article 8.10.1.

9.8.1.2 For monolithic prestressed construction, with normal slab span and girder spacing, the effective flange width shall be the distance center-to-center of beams. For very short spans, or where girder spacing is excessive, analytical investigations shall be made to determine the anticipated width of flange acting with the beam.

9.8.1.3 For monolithic prestressed design of isolated beams, the flange width shall not exceed 15 times the web width and shall be adequate for all design loads.

9.8.2 Box Girders

9.8.2.1 For cast-in-place box girders with normal slab span and girder spacing, where the slabs are considered an integral part of the girder, the entire slab width shall be assumed to be effective in compression.

9.8.2.2 For box girders of unusual proportions, including segmental box girders, methods of analysis which
consider shear lag shall be used to determine stresses in the cross section due to longitudinal bending.

9.8.2.3 Adequate fillets shall be provided at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required.

9.8.3 Precast/Prestressed Concrete Beams with Wide Top Flanges

9.8.3.1 For composite prestressed concrete where slabs or flanges are assumed to act integrally with the precast beam, the effective web width of the precast beam shall be the lesser of (1) six times the maximum thickness of the flange (excluding fillets) on either side of the web plus the web and fillets, and (2) the total width of the top flange.

9.8.3.2 The effective flange width of the composite section shall be the lesser of (1) one-fourth of the span length of the girder, (2) six (6) times the thickness of the slab on each side of the effective web width as determined by Article 9.8.3.1 plus the effective web width, and (3) one-half the clear distance on each side of the effective web width plus the effective web width.

9.9 FLANGE AND WEB THICKNESS—BOX GIRDERS

9.9.1 Top Flange

The minimum top flange thickness shall be \(\frac{1}{5}\)th of the clear distance between fillets or webs but not less than 6 inches, except the minimum thickness may be reduced for factory produced precast, pretensioned elements to \(\frac{5}{8}\) inches.

9.9.2 Bottom Flange

The minimum bottom flange thickness shall be \(\frac{1}{5}\)th of the clear distance between fillets or webs but not less than \(\frac{5}{8}\) inches, except the minimum thickness may be reduced for factory produced precast, pretensioned elements to 5 inches.

9.9.3 Web

Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

9.10 DIAPHRAGMS

9.10.1 General

Diaphragms shall be provided in accordance with Articles 9.10.2 and 9.10.3 except that diaphragms may be omitted where tests or structural analysis show adequate strength.

9.10.2 T-Beams

Diaphragms or other means shall be used at span ends to strengthen the free edge of the slab and to transmit lateral forces to the substructure. Intermediate diaphragms shall be placed between the beams at the points of maximum moment for spans over 40 feet.

9.10.3 Box Girders

9.10.3.1 For spread box beams, diaphragms shall be placed within the box and between boxes at span ends and at the points of maximum moment for spans over 80 feet.

9.10.3.2 For precast box multi-beam bridges, diaphragms are required only if necessary for slab-end support or to contain or resist transverse tension ties.

9.10.3.3 For cast-in-place box girders, diaphragms or other means shall be used at span ends to resist lateral forces and maintain section geometry. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater.

9.10.3.4 For segmental box girders, diaphragms shall be placed within the box at span ends. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater.

9.10.3.5 For all types of prestressed boxes in bridges with inside radius of curvature less than 800 feet, intermediate diaphragms may be required and the spacing and strength of diaphragms shall be given special consideration in the design of the structure.

9.11 DEFLECTIONS

9.11.1 General

Deflection calculations shall consider dead load, live load, prestressing, erection loads, concrete creep and shrinkage, and steel relaxation.
9.11.2 Segmental Box Girders

Deflections shall be calculated prior to casting of segments and they shall be based on the anticipated casting and erection schedules. Calculated deflections shall be used as a guide against which actual deflection measurements are checked.

9.11.3 Superstructure Deflection Limitations

When making deflection computations, the following criteria are recommended.

9.11.3.1 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load plus impact shall not exceed $V_{SW}$ of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall not exceed $V_{SW}$.

9.11.3.2 The deflection of cantilever arms due to service live load plus impact preferably should be limited to 1/300 of the cantilever arm except for the case including pedestrian use, where the ratio preferably should be $V_{SW}$.

9.12 DECK PANELS

9.12.1 General

9.12.1.1 Precast prestressed deck panels used as permanent forms spanning between stringers may be designed compositely with the cast-in-place portion of the slabs to support additional dead loads and live loads.

9.12.1.2 The panels shall be analyzed assuming they support their self-weight, any construction loads, and the weight of the cast-in-place concrete, and shall be analyzed assuming they act compositely with the cast-in-place concrete to support moments due to additional dead loads and live loads.

9.12.2 Bending Moment

9.12.2.1 Live load moments shall be computed in according with Article 3.24.3.

9.12.2.2 In calculating stresses in the deck panel due to negative moment near the stringer, no compression due to prestressing shall be assumed to exist.

Part C
DESIGN

9.13 GENERAL

9.13.1 Design Theory and General Considerations

9.13.1.1 Members shall meet the strength requirements specified herein.

9.13.1.2 Design shall be based on strength (Load Factor Design) and on behavior at service conditions (Allowable Stress Design) at all load stages that may be critical during the life of the structure from the time prestressing is first applied.

9.13.1.3 Stress concentrations due to the prestressing shall be considered in the design.

9.13.1.4 The effects of temperature and shrinkage shall be considered.

9.13.2 Basic Assumptions

The following assumptions are made for design purposes for monolithic members.

9.13.2.1 Strains vary linearly over the depth of the member throughout the entire load range.
9.13.3.3 In structures with a cast-in-place slab on precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the beams. Because the tensile shrinkage develops over an extended time period, the effect on the beams is reduced by creep. Differential shrinkage may influence the cracking load and the beam deflection profile. When these factors are particularly significant, the effect of differential shrinkage should be added to the effect of loads.

9.14 LOAD FACTORS

The computed strength capacity shall not be less than the largest value from load factor design in Article 3.22. For the design of post-tensioned anchorage zones a load factor of 1.2 shall be applied to the maximum tendon jack force.

The following strength capacity reduction factors shall be used:

- For factory produced precast prestressed concrete members $\phi = 1.0$
- For post-tensioned cast-in-place concrete members $\phi = 0.95$
- For shear $\phi = 0.90$
- For anchorage zones $\phi = 0.85$ for normal weight concrete and $\phi = 0.70$ for lightweight concrete.

9.15 ALLOWABLE STRESSES

The design of precast prestressed members ordinarily shall be based on $f'_{c} = 5,000$ psi. An increase to 6,000 psi is permissible where, in the Engineer’s judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the Engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths. The provisions of this Section are equally applicable to prestressed concrete structures and components designed with lower concrete strengths.

9.15.1 Prestressing Steel

Pretensioned members:
- Stress immediately prior to transfer—
  - Low-relaxation strands $0.75 f'_{c}$
  - Stress-relieved strands $0.70 f'_{c}$

Post-tensioned members:
- Stress immediately after seating—
  - At anchorage $0.70 f'_{c}$

At the end of the seating loss zone $0.83 f'_{c}$
Tensioning to $0.90 f'_{c}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value.
Stress at service load† after losses $0.80 f'_{c}$

9.15.2 Concrete

9.15.2.1 Temporary Stresses Before Losses Due to Creep and Shrinkage

Compression:
- Pretensioned members $0.60 f'_{c}$
- Post-tensioned members $0.55 f'_{c}$

Tension:
- Precompressed tensile zone $0.00$ No temporary allowable stresses are specified. See Article 9.15.2.2 for allowable stresses after losses.

Other Areas
- In tension areas with no bonded reinforcement $200$ psi or $3 \sqrt{f'_{c}}$
- Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed $7.5 \sqrt{f'_{c}}$

9.15.2.2 Stress at Service Load After Losses Have Occurred

Compression:
(a) The compressive stresses under all load combinations, except as stated in (b) and (c), shall not exceed $0.60 f'_{c}$.
(b) The compressive stresses due to effective prestress plus permanent (dead) loads shall not exceed $0.40 f'_{c}$.
(c) The compressive stress due to live loads plus one-half of the sum of compressive stresses due to prestress and permanent (dead) loads shall not exceed $0.40 f'_{c}$.

Tension in the precompressed tensile zone:
(a) For members with bonded reinforcement$^{*}$
  - $6 \sqrt{f'_{c}}$
  - For severe corrosive exposure conditions, such as coastal areas $3 \sqrt{f'_{c}}$

$^{*}$Includes bonded prestressed strands.
†Service load consists of all loads contained in Article 3.2 but does not include overload provisions.
(b) For members without bonded reinforce-
ment ..................................................... 0
Tension in other areas is limited by allowable temporary stresses specified in Article 9.15.2.1.

9.15.2.3 Cracking Stress*

Modulus of rupture from tests or if not available.
For normal weight concrete ........................ 7.5 \sqrt{T_e}
For sand-lightweight concrete .................. .63 \sqrt{T_e}
For all other lightweight concrete ............ .5 \sqrt{T_e}

9.15.2.4 Anchorage Bearing Stress

Post-tensioned anchorage at service load ... 3,000 psi
(but not to exceed 0.9 f'_c)

9.16 LOSS OF PRESTRESS

9.16.1 Friction Losses

Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations shall be shown on the plans. These friction losses shall be calculated as follows:

\[ T_o = T_x e^{(KL + \mu \alpha)} \]  \hspace{1cm} (9-1)

When \( (KL + \mu \alpha) \) is not greater than 0.3, the following equation may be used:

\[ T_o = T_x (1 + KL + \mu \alpha) \]  \hspace{1cm} (9-2)

The following values for K and \( \mu \) may be used when experimental data for the materials used are not available:

<table>
<thead>
<tr>
<th>Type of Steel</th>
<th>Type of Duct</th>
<th>K/ft</th>
<th>( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wire or strand</td>
<td>Rigid and semi-rigid galvanized metal sheathing</td>
<td>0.0002</td>
<td>0.15-0.25**</td>
</tr>
<tr>
<td></td>
<td>Polyethylene</td>
<td>0.0002</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>Rigid steel pipe</td>
<td>0.0002</td>
<td>0.25***</td>
</tr>
<tr>
<td>High Strength bars</td>
<td>Galvanized metal sheathing</td>
<td>0.0002</td>
<td>0.15</td>
</tr>
</tbody>
</table>

**A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes.
***Lubrication will probably be required.

Friction losses occur prior to anchoring but should be estimated for design and checked during stressing opera-
tions. Rigid ducts shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete. Rigid ducts may be fabricated with either welded or interlocked seams. Galvanizing of the welded seam will not be required.

9.16.2 Prestress Losses

9.16.2.1 General

Loss of prestress due to all causes, excluding friction, may be determined by the following method.* The method is based on normal weight concrete and one of the following types of prestressing steel: 250 or 270 ksi, seven-wire, stress-relieved or low-relaxation strand; 240 ksi stress-relieved wires; or 145 to 160 ksi smooth or deformed bars. Refer to documented tests for data regarding the properties and the effects of lightweight aggregate concrete on prestress losses.

TOTAL LOSS

\[ \Delta f_c = SH + ES + CR_c + CR_s \]  \hspace{1cm} (9-3)

where:

\[ \Delta f_c = \text{total loss excluding friction in pounds per square inch}; \]
\[ SH = \text{loss due to concrete shrinkage in pounds per square inch}; \]
\[ ES = \text{loss due to elastic shortening in pounds per square inch}; \]
\[ CR_c = \text{loss due to creep of concrete in pounds per square inch}; \]
\[ CR_s = \text{loss due to relaxation of prestressing steel in pounds per square inch}. \]

9.16.2.1.1 Shrinkage

Pretensioned Members:

\[ SH = 17,000 - 150 RH \]  \hspace{1cm} (9-4)

Post-tensioned Members:

\[ SH = 0.80 (17,000 - 150 RH) \]  \hspace{1cm} (9-5)

*Should more exact prestress losses be desired, data representing the materials to be used, the methods of curing, the ambient service condition and any pertinent structural details should be determined for use in accordance with a method of calculating prestress losses that is supported by appropriate research data. See also FHWA Report FHWA/RD 85/045, *Criteria for Designing Lightweight Concrete Bridges.*
where RH = mean annual ambient relative humidity in percent. (See Figure 9.16.2.1.1.)

**9.16.2.1.2 Elastic Shortening**

Pretensioned Members

\[ ES = \frac{E_s}{E_{ci}} f_{cirk} \]  \hspace{1cm} (9 - 6)

Post-tensioned Members*

\[ ES = 0.5 \frac{E_s}{E_{ci}} f_{cirk} \]  \hspace{1cm} (9 - 7)

where

\( E_s \) = modulus of elasticity of prestressing steel strand, which can be assumed to be 28 \times 10^6 psi;

\( E_{ci} \) = modulus of elasticity of concrete in psi at transfer of stress, which can be calculated from:

\[ E_{ci} = 33w^{3/2} \sqrt{f'_{ci}} \]  \hspace{1cm} (9 - 8)

in which \( w \) is the concrete unit weight in pounds per cubic foot and \( f'_{ci} \) is in pounds per square inch;

\( f_{cirk} \) = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer; \( f_{cirk} \) shall be computed at the section or sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing the concrete for pretensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 \( f_t \) or 0.69 \( f_t \) for stress relieved strand or low relaxation strand in typical pretensioned members.)

**9.16.2.1.3 Creep of Concrete**

Pretensioned and post-tensioned members

\[ CR_c = 12 f_{cirk} - 7 f_{cirs} \]  \hspace{1cm} (9 - 9)

where

\( f_{cirs} \) = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied.

**9.16.2.1.4 Relaxation of Prestressing Steel**

Pretensioned Members

250 to 270 ksi Strand

\[ CR_c = 20,000 - 0.4 ES - 0.2 (SH + CR_c) \]

for stress relieved strand

(9-10)

Post-tensioned Members

250 to 270 ksi Strand

\[ CR_c = 20,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CR_c) \]

for stress relieved strand

(9-11)

240 ksi Wire

\[ CR_c = 18,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CR_c) \]

(9-12)

where

\( FR \) = friction loss stress reduction in psi below the level of 0.70 \( f_t \) at the point under consideration, computed according to Article 9.16.1.

\( ES, SH, CR_c \) = appropriate values as determined for and \( CR_c \) either pre-tensioned or post-tensioned members.

---

*Certain tensioning procedures may alter the elastic shortening losses.
FIGURE 9.16.2.1.1 Mean Annual Relative Humidity.
9.16.2.2 Estimated Losses

In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of usual design. These loss values are based on use of normal weight concrete, normal prestress levels, and average exposure conditions. For exceptionally long spans, or for unusual designs, the method in Article 9.16.2.1 or a more exact method shall be used.

<table>
<thead>
<tr>
<th>Type of Prestressing Steel</th>
<th>Total Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pretensioning Strand</td>
<td>45,000 psi</td>
</tr>
<tr>
<td>Post-Tensioning*</td>
<td>32,000 psi</td>
</tr>
<tr>
<td>Wire or Strand</td>
<td>22,000 psi</td>
</tr>
<tr>
<td>Bars</td>
<td>23,000 psi</td>
</tr>
</tbody>
</table>

*Loses due to friction are excluded. Friction losses should be computed according to Article 9.16.1.

9.17 FLEXURAL STRENGTH

9.17.1 General

Prestressed concrete members may be assumed to act as uncracked members subjected to combined axial and bending stresses within specified service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included in pretensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted.

9.17.2 Rectangular Sections

For rectangular or flanged sections having prestressing steel only, which the depth of the equivalent rectangular stress block, defined as \((A_{sf} f_{sa})/(0.85 f'_c b')\), is not greater than the compression flange thickness “t,” and which satisfy Eq. (9-20), the design flexural strength shall be assumed as:

\[
\phi M_n = \phi \left( A_{sf} f_{sa} d \left[ 1 - 0.6 \left( \frac{p' f_{sa}}{f'_c} + \frac{d_t p_{sy}}{d f'_c} \right) \right] + A_t f_{sy} d_t \left[ 1 - 0.6 \left( \frac{d_t p' f_{sa}}{d f'_c} + \frac{p_{sy}}{f'_c} \right) \right] \right)
\]

(9-13a)

9.17.3 Flanged Sections

For sections having prestressing steel only, in which the depth of the equivalent rectangular stress block, defined as \((A_{sf} f_{sa})(0.85 f'_c b')\) is greater than the compression flange thickness “t,” and which satisfy Eq. (9-21), the design flexural strength shall be assumed as:

\[
\phi M_n = \phi \left( A_{sf} f_{sa} d \left[ 1 - 0.6 \left( \frac{A_{sf} f_{sa}}{b'd f'_c} \right) \right] + 0.85 f'_c (b - b')(t)(d - 0.5t) \right)
\]

(9-14)

For sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as \((A_{sf} f_{sa})/(0.85 f'_c b')\) is greater than the compression flange thickness “t,” and which satisfy Eq. (9-25), the design flexural strength shall be assumed as:

\[
\phi M_n = \phi \left( A_{sf} f_{sa} d \left[ 1 - 0.6 \left( \frac{A_{sf} f_{sa}}{b'd f'_c} \right) \right] + A_{sf} f_{sy} (d_t - d) \right)
\]

(9-14a)

where:

\[
A_{uf} = A'_{uf} - A_{sf} \quad \text{in Eq. (9-14)};
\]

(9-15)

\[
A_{uf} = A'_{uf} + (A_{sf}/f_{sa}) - A_{sf} \quad \text{in Eq. (9-14a)}
\]

(9-15a)

\[
A_{uf} = 0.85 f'_c (b - b')(t)d; \quad \text{(9-16)}
\]

\[
A_{uf} = \text{the steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.}
\]
9.17.4 Steel Stress

9.17.4.1 Unless the value of $f_{su}^*$ can be more accurately known from detailed analysis, the following values may be used:

Bonded Members . . .
with prestressing only (as defined):

$$f_{su}^* = f'_s \left[ 1 - \frac{\gamma^*}{\beta_1} \left( \frac{p^* f'_{t_c}}{f'_{c}} \right) \right] \quad (9-17)$$

with non-prestressed tension reinforcement included;

$$f_{su}^* = f'_s \left[ 1 - \frac{\gamma^*}{\beta_1} \left( \frac{p^* f'_{t_c}}{f'_{c}} + \frac{d_t}{d} \left( \frac{p f_{wy}}{f'_{c}} \right) \right) \right] \quad (9-17a)$$

Unbonded members . . . $f_{su}^* = f_{se} + 900(d - y_s)/t_s \quad (9-18)$

but shall not exceed $f'_s$.

Where

$y_s$ = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded.

$l_e = l/(1 + 0.5 N_s)$; effective tendon length.

$l_t$ = tendon length between anchorages (in.).

$N_s$ = number of support hinges crossed by the tendon between anchorages or discretely bonded points.

provided that

1. The stress-strain properties of the prestressing steel approximate those specified in Division II, Article 10.3.1.1.
2. The effective prestress after losses is not less than 0.5 $f'_s$.

9.17.4.2 At ultimate load, the stress in the prestressing steel of precast deck panels shall be limited to

$$f_{su}^* = f'_x + \frac{2}{3} f_{se} \quad (9-19)$$

but shall not be greater than $f_{se}$ as given by the equations in Article 9.17.4.1. In the above equation:

$D = $ nominal diameter of strand in inches;

$f_{se}$ = effective stress in prestressing strand after losses in kips per square inch;

$l_s$ = distance from end of prestressing strand to center of panel in inches.

9.18 DUCTILITY LIMITS

9.18.1 Maximum Prestressing Steel

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general, the reinforcement index shall be such that

$$(p^* f'_{su} / f'_{c}) \text{ for rectangular sections} \quad (9-20)$$

and

$$A_{st} f_{su}^* / (b'd f'_{c}) \text{ for flanged sections} \quad (9-21)$$

does not exceed 0.36$\beta_1$. (See Article 9.19 for reinforcement indices of sections with non-prestressed reinforcement.)

For members with reinforcement indices greater than 0.36$\beta_1$, the design flexural strength shall be assumed not greater than:

For rectangular sections

$$\phi M_n = \phi [(0.36 \beta_1 - 0.08 \beta^2_1) f'_{c} b d^2 + 0.85 f'_{c} (b - b') t (d - 0.5 t)] \quad (9-22)$$

For flanged sections

$$\phi M_n = \phi[(0.36 \beta_1 - 0.08 \beta^2_1) f'_{c} b'd^2 + 0.85 f'_{c} (b - b') t (d - 0.5 t)] \quad (9-23)$$

9.18.2 Minimum Steel

9.18.2.1 The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment $M_{cr}^*$.

$$\phi M_n = 1.2 M_{cr}^* \quad (9-24)$$

where

$$M_{cr}^* = (f_c + f_{pe}) S_c - M_{dcr} (S_o/S_o - 1)$$

Appropriate values for $M_{dcr}$ and $S_o$ shall be used for any intermediate composite sections. Where beams are designed to be noncomposite, substitute $S_o$ for $S_c$ in the above equation for the calculation of $M_{cr}^*$.

9.18.2.2 The minimum amount of non-prestressed longitudinal reinforcement provided in the cast-in-place portion of slabs utilizing precast prestressed deck panels shall be 0.25 square inch per foot of slab width.
9.19 NON-PRESTRESSED REINFORCEMENT

Non-prestressed reinforcement may be considered as contributing to the tensile strength of the beam at ultimate strength in an amount equal to its area times its yield point, provided that

For rectangular sections

$$ \left( \frac{f_y}{f'_c} \right) d + \left( \frac{f'_{yw}}{f''_c} \right) \left( \frac{p^{*}f'_{yw}}{f''_c} \right) \leq 0.36 \beta_1 \quad (9-24) $$

For flanged sections

$$ (A_{sy}f_{yw})/(b'dc') + (A_{sy}f_{yw})/(b'dc') \geq 0.36 \beta_1 \quad (9-25) $$

Design flexural strength shall be calculated based on Eq. (9-13a) or Eq. (9-14a) if these values are met, and on Eq. (9-22) or Eq. (9-23) if these values are exceeded.

9.20 SHEAR*

9.20.1 General

9.20.1.1 Prestressed concrete flexural members, except solid slabs and footings, shall be reinforced for shear and diagonal tension stresses. Voided slabs shall be investigated for shear, but shear reinforcement may be omitted if the factored shear force, $V_{es}$, is less than half the shear strength provided by the concrete $f_c$.

9.20.1.2 Web reinforcement shall consist of stirrups perpendicular to the axis of the member or welded wire fabric with wires located perpendicular to the axis of the member. Web reinforcement shall extend a distance $d$ from the extreme compression fiber and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Web reinforcement shall be anchored at both ends for its design yield strength in accordance with the provisions of Article 8.27.

9.20.1.3 Members subject to shear shall be designed so that

$$ V_{es} \leq \phi (V_c + V_s) \quad (9-26) $$

where $V_s$ is the factored shear force at the section considered, $V_c$ is the nominal shear strength provided by concrete and $V_s$ is the nominal shear strength provided by web reinforcement.

9.20.1.4 When the reaction to the applied loads introduces compression into the end regions of the member, sections located at a distance less than $h/2$ from the face of the support may be designed for the same shear $V_{es}$ as that computed at a distance $h/2$.

9.20.1.5 Reinforced keys shall be provided in the webs of precast segmental box girders to transfer erection shear. Possible reverse shearing stresses in the shear keys shall be investigated, particularly in segments near a pier. At time of erection, the shear stress carried by the shear key shall not exceed $2 \sqrt{f_c'}$.

9.20.2 Shear-Strength Provided by Concrete

9.20.2.1 The shear strength provided by concrete, $V_c$, shall be taken as the lesser of the values $V_{es}$ or $V_{ew}$.

9.20.2.2 The shear strength, $V_{es}$, shall be computed by

$$ V_{es} = 0.6 \frac{\sqrt{f_c'}}{b'd} + V_d + \frac{V_{M_{cr}}}{M_{max}} \quad (9-27) $$

but need not be less than $1.7 \sqrt{f_c'} b' d$ and $d$ need not be taken less than $0.8h$.

The moment causing flexural cracking at the section due to externally applied loads, $M_{cr}$, shall be computed by:

$$ M_{cr} = \frac{I}{Y_{c}} (6 \sqrt{f_c'} + f_{pc} - f_d) \quad (9-28) $$

The maximum factored moment and factored shear at the section due to externally applied loads, $M_{max}$ and $V_{es}$, shall be computed from the load combination causing maximum moment at the section.

9.20.2.3 The shear strength, $V_{ew}$, shall be computed by

$$ V_{ew} = (3.5 \sqrt{f_c'} + 0.3 f_{pc}) b'd + V_p \quad (9-29) $$

but $d$ need not be taken less than $0.8h$.

9.20.2.4 For a pretensioned member in which the section at a distance $h/2$ from the face of support

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*The method for design of web reinforcement presented in the 1979 Interim AASHTO Standard Specifications for Highway Bridges is an acceptable alternate.