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^*_s = area of prestressing steel (Article 9.17)

A_{sf} = steel area required to develop the compressive strength of the overhanging portions of the flange (Article 9.17)

A_{sr} = steel area required to develop the compressive strength of the web of a flanged section (Articles 9.17-9.19)

A_v = area of web reinforcement (Article 9.20)

b = width of flange of flanged member or width of rectangular member

b_v = 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 9.28) +

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_t = 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_{cds} = average concrete compressive stress at the c.g. of the prestressing steel under full dead load (Article 9.16)

f_{cir} = 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'_{ci} = compressive strength of concrete at time of initial prestress (Article 9.15)

f_{ct} = 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, $A^*_s f'_s$

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

Δf_s = total prestress loss, excluding friction (Article 9.16)

f_{se} = effective steel prestress after losses

f^*_{su} = average stress in prestressing steel at ultimate load

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

f_{sy} = yield stress of non-prestressed conventional reinforcement in tension (Article 9.19 and 9.20)

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

f^*_y = yield stress of prestressing steel (Article 9.15)

= 0.90 f'_s for low-relaxation wire or strand

= 0.85 f'_s for stress-relieved wire or strand

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

= 0.80 f'_s 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)

$M_{d/c}$ = composite dead load moment at the section (Commentary to Article 9.18)

$M_{d/nc}$ = 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 $\leq$ M_n (Articles 9.17 and 9.18)

p = A_s/bd_t, ratio of non-prestressed tension reinforcement (Articles 9.7 and 9.17-9.19)

p^* = A^*_s/bd, ratio of prestressing steel (Articles 9.17 and 9.19)

p' = A'_s/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_b = non-composite 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)

v = 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_d = shear force at section due to unfactored dead load (Article 9.20)

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

V_{nh} = 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_u = factored shear force at section (Article 9.20)

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

μ = friction curvature coefficient (Article 9.16)

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

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

γ^* = factor for type of prestressing steel (Article 9.17)
= 0.28 for low-relaxation steel
= 0.40 for stress-relieved steel
= 0.55 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.

β_1 *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—Non-uniform 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 surrrounding 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.

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 prestressing (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 the 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 location (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 for non-segmental box girders shall be $^1/_{30}{}^{th}$ 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 $5^1/_2$ inches.
The top flange thickness for segmental box girders shall be determined in accordance with Article 9.7.3.2.

9.9.2 Bottom Flange

+ The minimum bottom flange thickness for non-segmental and segmental box girders shall be determined by maximum allowable unit stresses as specified in Article 9.15, but in no case shall be less than $^1/_{30}$th of the clear distance between fillets or webs or $5^1/_2$ 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 Article 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, Precast I and Bulb-tee Girders

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 $^1/_{800}$ of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall not exceed $^1/_{1000}$.

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 $^1/_{375}$.

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 accordance 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 the prestressing is first applied.

The prestressing force and required concrete strength shall be determined by allowable stress design using elastic theory for loads at the service level considering HS loads.

The ultimate moment capacity and the shear design shall be based on load factor design with factored HS or P loads.

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.

9.13.2.2 Before cracking, stress is linearly proportional to strain.

9.13.2.3 After cracking, tension in the concrete is neglected.

9.13.3 Composite Flexural Members

Composite flexural members consisting of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to superimposed loads as a unit shall conform to the provisions of Articles 8.14.2.1 through 8.14.2.4, 8.14.2.6, and the following.

9.13.3.1 Where an entire member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Articles 9.20.1 through 9.20.3.

9.13.3.2 The design shall provide for full transfer of horizontal shear forces at contact surfaces of interconnected elements. Design for horizontal shear shall be in accordance with the requirements of Article 9.20.4.

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 jacking force.

The following strength capactity reduction factors shall be used:

For factory produced precast prestressed concrete members ϕ = 1.0

For post-tensioned cast-in-place concrete members = 0.95

For shear = 0.90

For anchorage zones = 0.85 for normal weight concrete and = 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.

In Environmental Area III use f'_c = 5,000 psi maximum because of required air entrainment.

9.15.1 Prestressing Steel

Pretensioned members:
- Stress immediately prior to transfer—
- Low-relaxation strands .... 0.75 f'_s
- Stress-relieved strands .... 0.70 f'_s

Post-tensioned members:
- Stress immediately after seating—
  - At anchorage .... 0.70 f'_s
  - At the end of the seating loss zone .... 0.83 f^*_y
- Maximum jacking stress .... 0.75 f'_s
  - For longer frame structures, tensioning to 0.90 f^*_y 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^*_y
  - Service load consists of all loads contained in Article 3.2 but does not include overload provisions.

9.15.2 Concrete

9.15.2.1 Temporary Stresses Before Losses Due to Creep and Shrinkage

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

Tension:
- Precompressed tensile zone .... 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

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 .... $7.5\sqrt{f'_{ci}}$

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 $0.60f'_c$.

(b) The compressive stresses due to effective prestress plus permanent (dead) loads shall not exceed $0.40f'_c$.

(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 $0.40f'_c$.

Tension in the precompressed tensile zone:

Service Load Condition:

(a) For members with bonded reinforcement, including bonded prestressed strands .. $6\sqrt{f'_c}$

(b) For Environmental Area III and Marine Environment .... $3\sqrt{f'_c}$

(c) For members without bonded reinforcement 0

Dead and Additional Dead Load Condition: 0

Tension in other areas is limited by allowable temporary stresses specified in Article 9.15.2.1.

9.15.2.3 Cracking Stress (Refer to Article 9.18)

Modulus of rupture from tests or if not available.

For normal weight concrete .... $7.5\sqrt{f'_c}$

For sand-lightweight concrete .... $6.3\sqrt{f'_c}$

For all other lightweight concrete .... $5.5\sqrt{f'_c}$

9.15.2.4 Anchorage Bearing Stress

Post-tensioned anchorage at service load .... 3,000 psi
(but not to exceed $0.9 f'_{ci}$)

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:

$$T_o = T_x e^{(KL+\mu\alpha)} \qquad (9\text{-}1)$$

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

$$\Delta f_s = SH + ES + CR_c + CR_s$$

$$T_o = T_x (1 + KL + \mu\alpha) \qquad (9\text{-}2)$$

The following values for K and μ may be used when experimental data for the materials used are not available:

Type of Steel	Type of Duct	K/ft.	**
Wire or strand	Rigid and semi-rigid galvanized metal sheathing Tendon Length:		
	0 - 600 feet	0.0002	0.15
	600 - 900 feet	0.0002	0.20
	900 - 1200 feet	0.0002	0.25
	>1200 feet	0.0002	0.25*
	Polyethylene	0.0002	0.23
	Rigid steel pipe	0.0002	0.25*
High-strength bars	Galvanized metal sheathing	0.0002	0.15

*Lubrication will probably be required.
**Add effect of horizontal curvature if any.

Friction losses occur prior to anchoring but should be estimated for design and checked during stressing operations. 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.

Should more exact prestressed 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.*

TOTAL LOSS

(9-3)

where:

- Δf_s = total loss excluding friction in pounds per square inch;
- SH = loss due to concrete shrinkage in pounds per square inch;
- ES = loss due to elastic shortening in pounds per square inch;
- CR_c = loss due to creep of concrete in pounds per square inch;
- CR_s = loss due to relaxation of prestressing steel in pounds per square inch.

9.16.2.1.1 Shrinkage

Pretensioned Members:

$$SH = 17{,}000 - 150RH \tag{9-4}$$

Post-tensioned Members:

$$SH = 0.80(17{,}000 - 150RH) \tag{9-5}$$

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

$$ES = \frac{E_s}{E_{ci}} f_{cir} \tag{9-6}$$

Post-tensioned Members (certain tensioning procedures may alter the elastic shortening losses).

$$ES = 0.5\frac{E_s}{E_{ci}} f_{cir} \tag{9-7}$$

where:

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

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

$$E_{ci} = 33w^{3/2}\sqrt{f'_{ci}} \tag{9-8}$$

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

f_{cir} = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer; f_{cir} 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 $0.63 f'_s$ for stress relieved strand or $0.69 f'_s$ for low relaxation strand in typical pretensioned members.)

9.16.2.1.3 Creep of Concrete

Pretensioned and post-tensioned members

$$CR_c = 12f_{cir} - 7f_{cds} \tag{9-9}$$

where:

f_{cds} = 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

The relaxation losses are based on an initial stress equal to the stress at anchorages allowed by Article 9.15.1.

Pretensioned Members

250 to 270 ksi Strand

$CR_s = 20{,}000 - 0.4\,ES - 0.2\,(SH + CR_c)$
for stress relieved strand (9-10)

$CR_s = 5{,}000 - 0.10\,ES - 0.05\,(SH + CR_c)$
for low relaxation strand (9-10A)

Post-tensioned Members

250 to 270 ksi Strand

$CR_s = 20{,}000 - 0.3\,FR - 0.4\,ES - 0.2\,(SH + CR_c)$
for stress relieved strand (9-11)

$CR_s = 5{,}000 - 0.07FR - 0.1\,ES - 0.05\,(SH + CR_c)$
for low relaxation strand (9-11A)

240 ksi Wire

$$CR_s = 18{,}000 - 0.3\,FR - 0.4\,ES - 0.2\,(SH + CR_c) \tag{9-12}$$

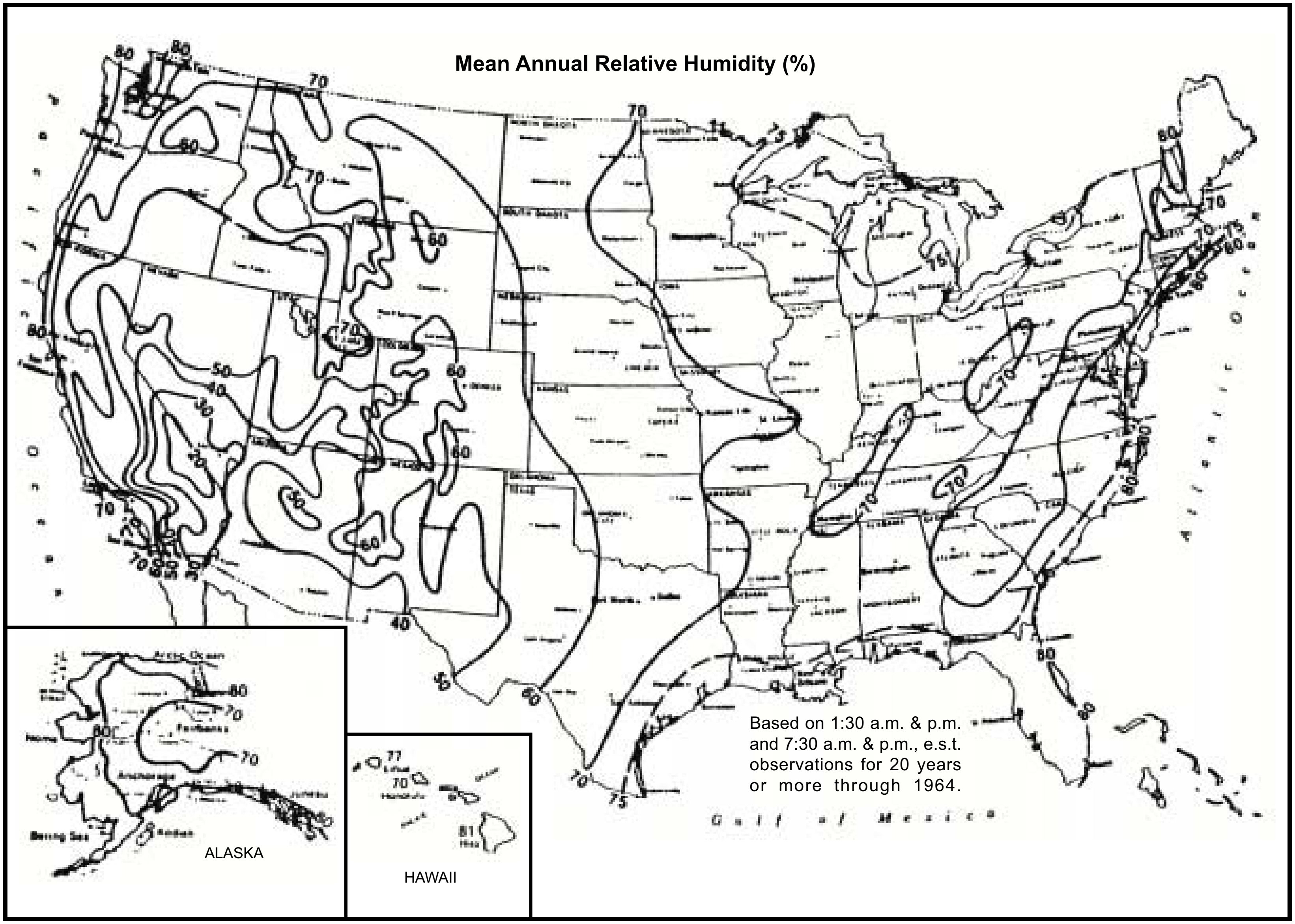

Figure 9.16.2.1.1 Mean Annual Relative Humidity

145 to 160 ksi Bars

CR_s = 3,000

where:

FR = friction loss stress reduction in psi below the level of $0.70f'_s$ at the point under consideration, computed according to Article 9.16.1;

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

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 9.16.2.2 Estimate of Prestress Losses

Type of Prestressing Steel	Total Loss	
	Normal Weight Aggregate Concrete	Light Weight Aggregate Concrete
Pretensioning:		
Normal Relaxation Strand	45,000 psi	50,000 psi
Low Relaxation Strand	35,000 psi	40,000 psi
Post-Tensioning*:		
Normal Relaxation Strand or wires	32,000 psi	40,000 psi
Low Relaxation Strand	20,000 psi	30,000 psi
Bars	22,000 psi	

* Losses 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 $(A^*_s f^*_{su})/(0.85f'_c b)$, is not greater than the compression flange thickness "t", and which satisfy Eq. (9-20), the design flexural strength shall be assumed as:

$$\phi M_n = \phi\left[A_s^* f_{su}^* d\left(1 - 0.6\,\frac{p^* f_{su}^*}{f_c'}\right)\right] \qquad \text{(9-13)}$$

For rectangular or flanged sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as $(A^*_s f^*_{su} + A_s f_{sy})/(0.85\, f'_c b)$, is not greater than the compression flange thickness "t", and which satisfy Eq. (9-24), the design flexural strength shall be assumed as:

$$\phi M_n = \phi\left\{A_s^* f_{su}^* d\left[1 - 0.6\left(\frac{p^* f_{su}^*}{f_c'} + \frac{d_t}{d}\,\frac{pf_{sy}}{f_c'}\right)\right] + A_s f_{sy} d_t\left[1 - 0.6\left(\frac{d}{d_t}\,\frac{p^* f_{su}^*}{f_c'} + \frac{pf_{sy}}{f_c'}\right)\right]\right\} \qquad \text{(9-13a)}$$

9.17.3 Flanged Sections

For sections having prestressing steel only, in which the depth of the equivalent rectangular stress block, defined as $(A_{sr} f^*_{su})/(0.85 f'_c b')$ is greater than the compression flange thickness "t", and which satisfy Eq. (9-21), the design flexural strength shall be assumed as:

$$\phi M_n = \phi \left\{ A_{sr} f^*_{su} d \left[1 - 0.6 \left(\frac{A_{sr} f^*_{su}}{b' d f'_c} \right) \right] + 0.85 f'_c (b - b')(t)(d - 0.5t) \right\} \quad (9\text{-}14)$$

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

$$\phi M_n = \phi \left\{ A_{sr} f^*_{su} d \left[1 - 0.6 \left(\frac{A_{sr} f^*_{su}}{b' d f'_c} \right) \right] + A_s f_{sy} (d_t - d) + 0.85 f'_c (b - b')(t)(d - 0.5t) \right\} \quad (9\text{-}14a)$$

where:

A_{sr} = $A^*_s - A_{sf}$ in Eq. (9-14); (9-15)

A_{sr} = $A^*_s + (A_s f_{sy}/f^*_{su}) - A_{sf}$ in Eq. (9-14a) (9-15a)

+ A_{sr} = the steel area required to develop the ultimate
+ compressive strength of the web of a flanged
+ section.

A_{sf} = $0.85 f'_c(b - b')t/f^*_{su}$; (9-16)

A_{sf} = 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);

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

with non-prestressed tension reinforcement included;

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

Unbonded members .....

$$f^*_{su} = f_{se} + 900((d - y_u)/I_e) \quad (9\text{-}18)$$

but shall not exceed f^*_y.

where:

y_u = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded.
l_e = $l_i/(1 + 0.5N_s)$; effective tendon length.
l_i = 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:

$$f_{su}^{*} = \frac{\ell_x}{D} + \frac{2}{3} f_{se} \qquad (9\text{-}19)$$

but shall not be greater than f^*_{su} 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_x = 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:

$$p^{*} \frac{f_{su}^{*}}{f'} \text{ (for rectangular sections)} \qquad (9\text{-}20)$$

and

$$\frac{A_{sr} f_{su}^{*}}{b' d f_c'} \text{ (for flanged sections)} \qquad (9\text{-}21)$$

does not exceed 0.36 β_1. (See Article 9.19 for reinforcement indices of sections with non-prestressed reinforcement.).

For members with reinforcement indices greater than 0.36 β_1, the design flexural strength shall be assumed not greater than:

For rectangular sections:

$$\phi M_n = \phi \left(0.36\,\beta_1 - 0.08\,\beta_1^2\right) f_c' b d^2 \qquad (9\text{-}22)$$

For flanged sections:

$$\phi M_n = \phi \left[\left(0.36\,\beta_1 - 0.08\,\beta_1^2\right) f_c' b' d^2 + 0.85\, f_c' \left(b - b'\right) t \left(d - 0.5t\right)\right] \qquad (9\text{-}23)$$

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}^{*}.

$$\phi M_n \geq 1.2 M_{cr}^{*}$$

where:

$$M_{cr}^{*} = \left(f_r + f_{pe}\right) S_c - M_{d/nc}\left(S_c / S_b - 1\right)$$

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

9.18.2.2 The requirements of Article 9.18.2.1 may be waived if the area of prestressed and non-prestressed reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22.

9.18.2.3 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:

$$\left(\frac{pf_{sy}}{f'_c}\right)\frac{d_t}{d}+\left(\frac{p^* f^*_{su}}{f'_c}\right)-\left(\frac{p'f'_y}{f'_c}\right)\leq 0.36\beta_1 \qquad (9\text{-}24)$$

For flanged sections:

$$\left(\frac{A_s f_{sy}}{b'df'_c}\right)+\left(\frac{A_{sr} f^*_{su}}{b'df'_c}\right)-\left(\frac{A'_s f'_y}{b'df'_c}\right)\leq 0.36\beta_1 \qquad (9\text{-}25)$$

Design flexural strength shall be calculated based on Eq. (9-13a) of 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

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

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_u, is less than half the shear strength provided by the concrete ϕV_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 to 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

$$V_u \leq \phi\left(V_c + V_s\right) \qquad (9\text{-}26)$$

where V_u 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_u 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_{ci} or V_{cw}.

9.20.2.2 The shear strength, V_{ci}, shall be computed by:

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (9\text{-}27)$$

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:

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right) \qquad (9\text{-}28)$$

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

9.20.2.3 The shear strength, V_{cw}, shall be computed by:

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right)b'd + V_p \qquad (9\text{-}29)$$

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 is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

9.20.2.5 The provisions for computing the shear strength provided by concrete, V_{ci} and V_{cw}, apply to normal weight concrete. When lightweight aggregate concretes are used (see definition, concrete, structural lightweight, Article 8.1.3), one of the following modifications shall apply:

(a) When f_{ct} is specified, the shear strength, V_{ci} and V_{cw}, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_{ci}}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.

(b) When f_{ct} is not specified, V_{ci} and V_{cw} shall be modified by multiplying each term containing $\sqrt{f'_c}$ by 0.75 for "all lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

9.20.3 Shear Strength Provided by Web Reinforcement

9.20.3.1 The shear strength provided by web reinforcement shall be taken as:

$$V_s = \frac{A_v f_{sy} d}{s} \qquad (9\text{-}30)$$

where A_v is the area of web reinforcement within a distance s. V_s shall not be taken greater than $8\sqrt{f'_c}\,b'd$ and d need not be taken less than $0.8h$.

9.20.3.2 The spacing of web reinforcing shall not exceed $0.75h$ or 24 inches. When V_s exceeds $4\sqrt{f'_{ci}}\,b'd$, this maximum spacing shall be reduced by one-half.

9.20.3.3 The minimum area of web reinforcement shall be:

$$A_v = \frac{50b's}{f_{sy}} \qquad (9\text{-}31)$$

where b' and s are in inches and f_{sy} is in psi.

9.20.3.4 The design yield strength of web reinforcement, f_{sy}, shall not exceed 60,000 psi.

9.20.4 Horizontal Shear Design—Composite Flexural Members

9.20.4.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

9.20.4.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Article 9.20.4.3 or 9.20.4.4, or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

9.20.4.3 Design of cross sections subject to horizontal shear may be based on:

$$V_u \le \phi V_{nh} \qquad (9\text{-}31a)$$

where V_u is factored shear force at a section considered, V_{nh} is nominal horizontal shear strength in accordance with the following, and where d is for the entire composite section.

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$, in pounds.

(b) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$, in pounds.

(c) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately $^1/_4$ in., shear strength V_{nh} shall not be taken greater than $350b_vd$, in pounds.

(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by article 9.20.4.5, shear strength V_{nh} may be increased by $(160f_y/40{,}000)b_v d$, in pounds.

9.20.4.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} in accordance with Article 9.20.4.3, except that the length of segment considered shall be substituted for *d*.

9.20.4.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing "*s*" shall not exceed four times the least web width of support element, nor 24 inches.
(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks.

9.21 POST-TENSIONED ANCHORAGE ZONES

9.21.1 Geometry of the Anchorage Zone

9.21.1.1 The anchorage zone is geometrically defined as the volume of concrete through which the concentrated prestressing force at the anchorage device spreads transversely to a linear stress distribution across the entire cross section.

9.21.1.2 For anchorage zones at the end of a member or segment, the transverse dimensions may be taken as the depth and width of the section. The longitudinal extent of the anchorage zone in the direction of the tendon (ahead of the anchorage) shall be taken as not less than the larger transverse dimension but not more than $1^1/_2$ times that dimension.

9.21.1.3 For intermediate anchorages in addition to the length of Article 9.21.1.2 the anchorage zone shall be considered to also extend in the opposite direction for a distance not less than the larger transverse dimension.

9.21.1.4 For multiple slab anchorages, both width and length of the anchorage zone shall be taken as equal to the center-to-center spacing between stressed tendons, but not more than the length of the slab in the direction of the tendon axis. The thickness of the anchorage zone shall be taken equal to the thickness of the slab.

9.21.1.5 For design purposes, the anchorage zone shall be considered as comprised of two regions; the *general zone* as defined in Article 9.21.2.1 and the *local zone* as defined in Article 9.21.2.2.

9.21.2 General Zone and Local Zone

9.21.2.1 General Zone

9.21.2.1.1 The geometric extent of the general zone is identical to that of the overall anchorage zone as defined in Article 9.21.1 and includes the local zone.

9.21.2.1.2 Design of general zones shall meet the requirements of Articles 9.14 and 9.21.3.

9.21.2.2 Local Zone

9.21.2.2.1 The local zone is defined as the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete surrounding and immediately ahead of the anchorage device and any integral confining reinforcement. The dimensions of the local zone are defined in Article 9.21.7.

9.21.2.2.2 Design of local zones shall meet the requirements of Articles 9.14 and 9.21.7 or shall be based on the results of experimental tests required in Article 9.21.7.3 and described in Article 10.3.2.3 of Division II. Anchorage devices based on the acceptance test of Division II, Article 10.3.2.3, are referred to as *special anchorage devices*.

9.21.2.3 Responsibilities

9.21.2.3.1 The engineer of record is responsible for the overall design and approval of working drawings for the general zone, including the specific location of the tendons and anchorage devices, general zone reinforcement, and the specific stressing sequence. The engineer of record is also responsible for the design of local zones based on Article 9.21.7.2 and for the approval of special anchorage devices used under the provisions of Section

9.21.7.3. All working drawings for the local zone must be approved by the engineer of record.

9.21.2.3.2 Anchorage device suppliers are responsible for furnishing anchorage devices which satisfy the anchor efficiency requirements of Division II, Article 10.3.2. In addition, if special anchorage devices are used, the anchorage device supplier is responsible for furnishing anchorage devices that satisfy the acceptance test requirements of Article 9.21.7.3 and of Division II, Article 10.3.2.3. This acceptance test and the anchor efficiency test shall be conducted by an independent testing agency acceptable to the engineer of record. The anchorage device supplier shall provide records of the acceptance test in conformance with Division II, Article 10.3.2.3.12 to the engineer of record and to the constructor and shall specify auxiliary and confining reinforcement, minimum edge distance, minimum anchor spacing, and minimum concrete strength at time of stressing required for proper performance of the local zone.

9.21.2.3.3 The responsibilities of the constructor are specified in Division II, Article 10.4.

9.21.3 General Zone and Local Zone

9.21.3.1 Design Methods

The following methods may be used for the design of general zones:

(1) Equilibrium based plasticity models (strut-and-tie models) (see Article 9.21.4)
(2) Elastic stress analysis (finite element analysis or equivalent) (see Article 9.21.5)
(3) Approximate methods for determining the compression and tension forces, where applicable. (See Article 9.21.6)

Regardless of the design method used, all designs shall conform to the requirements of Article 9.21.3.4.

The effects of stressing sequence and three-dimensional effects shall be considered in the design. When these three dimensional effects appear significant, they may be analyzed using three-dimensional analysis procedures or may be approximated by considering two or more planes. However, in these approximations the interaction of the planes' models must be considered, and the model loadings and results must be consistent.

9.21.3.2 Nominal Material Strengths

9.21.3.2.1 The nominal tensile strength of bonded reinforcement is limited to f_{sy} for non-prestressed reinforcement and to f_y for prestressed reinforcement. The nominal tensile strength of unbonded prestressed reinforcement is limited to f_{se} + 15,000 psi.

9.21.3.2.2 The effective nominal compressive strength of the concrete of the general zone, exclusive of confined concrete, is limited to $0.7f'_c$. The tensile strength of the concrete shall be neglected.

9.21.3.2.3 The compressive strength of concrete at transfer of prestressing shall be specified on the construction drawings. If not otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the member, is at least 4,000 psi.

9.21.3.3 Use of Special Anchorage Devices

Whenever special anchorage devices which do not meet the requirements of Article 9.21.7.2 are to be used, reinforcement similar in configuration and at least equivalent in volumetric ratio to the supplementary skin reinforcement permitted under the provisions of Division II, Article 10.3.2.3.4 shall be furnished in the corresponding regions of the anchorage zone.

9.21.3.4 General Design Principles and Detailing Requirements

Good detailing and quality workmanship are essential for the satisfactory performance of anchorage zones. Sizes and details for anchorage zones should respect the need for tolerances on the bending, fabrication and placement of reinforcement, the size of aggregate and the need for placement and sound consolidation of the concrete.

9.21.3.4.1 Compressive stresses in the concrete ahead of basic anchorage devices shall meet the requiremens of Article 9.21.7.2.

9.21.3.4.2 Compressive stresses in the concrete ahead of special anchorage devices shall be checked at a distance measured from the concrete-bearing surface equal to the smaller of:

(1) The depth to the end of the local confinement reinforcement.
(2) The smaller lateral dimension of the anchorage device.

These compressive stresses may be determined according to the strut-and-tie model procedures of Article 9.21.4, from an elastic stress analysis according to Article 9.21.5.2, or by the approximate method outlined in Article 9.21.6.2. These compressive stresses shall not exceed $0.7f'_{ci}$.

9.21.3.4.3 Compressive stresses shall also be checked where geometry or loading discontinuities within or ahead of the anchorage zone may cause stress concentrations.

9.21.3.4.4 The bursting force is the tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis. The magnitude of the bursting force, T_{burst}, and it's corresponding distance from the loaded surface, d_{burst}, can be determined using the strut-and-tie model procedures of Article 9.21.4, from an elastic stress analysis according to Article 9.21.5.3, or by the approximate method outlined in Article 9.21.6.3. Three-dimensional effects shall be considered for the determination of the bursting reinforcement requirements.

9.21.3.4.5 Resistance to bursting forces, $\phi A_s f_{sy}$ and/or $\phi A_s^* f_y^*$, shall be provided by non-prestressed or prestressed reinforcement, in the form of spirals, closed hoops, or well-anchored transverse ties. This reinforcement is to be proportioned to resist the total factored bursting force. Arrangement and anchorage of bursting reinforcement shall satisfy the following:

(1) Bursting reinforcement shall extend over the full width of the member and must be anchored as close to the outer faces of the member as cover permits.
(2) Bursting reinforcement shall be distributed ahead of the loaded surface along both sides of the tendon throughout a distance $2.5d_{burst}$ for the plane considered, but not to exceed 1.5 times the corresponding lateral dimension of the section. The centroid of the bursting reinforcement shall coincide with the distance d_{burst} used for the design.
(3) Spacing of bursting reinforcement shall exceed neither 24 bar diameters nor 12 inches.

9.21.3.4.6 Edge tension forces are tensile forces in the anchorage zone acting parallel and close to the transverse edge and longitudinal edges of the member. The transverse edge is the surface loaded by the anchors. The tensile force along the transverse edge is referred to as *spalling force*. The tensile force along the longitudinal edge is referred to as *longitudinal edge tension force*.

9.21.3.4.7 Spalling forces are induced in concentrically loaded anchorage zones, eccentrically loaded anchorage zones, and anchorage zones for multiple anchors. Longitudinal edge tension forces are induced when the resultant of the anchorage forces considered causes eccentric loading of the anchorage zone. The edge tension forces can be determined from an elastic stress analysis, strut-and-tie models, or in accordance with the approximate methods of Article 9.21.6.4.

9.21.3.4.8 In no case shall the spalling force be taken as less than 2 percent of the total factored tendon force.

9.21.3.4.9 Resistance to edge tension forces, $\phi A_s f_{sy}$ and/or $\phi A_s^* f_y^*$, shall be provided in the form of non-prestressed or prestressed reinforcement located close to the longitudinal and transverse edge of the concrete. Arrangement and anchorage of the edge tension reinforcement shall satisfy the following:

(1) Minimum spalling reinforcement satisfying Article 9.21.3.4.8 shall extend over the full width of the member.
(2) Spalling reinforcement between multiple anchorage devices shall effectively tie these anchorage devices together.
(3) Longitudinal edge tension reinforcement and spalling reinforcement for eccentric anchorage devices shall be continuous. The reinforcement shall extend along the tension face over the full length of the anchorage zone and shall extend along the loaded face from the longitudinal edge to the other side of the eccentric anchorage device or group of anchorage devices.

9.21.3.5 Intermediate Anchorages

9.21.3.5.1 Intermediate anchorages shall not be used in regions where significant tension is generated behind the anchor from other loads. Whenever practical, blisters shall be located in the corner between flange and webs, or shall be extended over the full flange width or web height to form a continuous rib. If isolated blisters must be used on a flange or web, local shear, bending and direct force effects shall be considered in the design.

9.21.3.5.2 Bonded reinforcement shall be provided to tie back at least 25 percent of the intermediate anchorage unfactored stressing force into the concrete section behind the anchor. Stresses in this bonded reinforcement are limited to a maximum of $0.6f_{sy}$ or 36 ksi. The amount of tie back reinforcement may be reduced using Equation (9-32), if permanent compressive stresses are generated behind the anchor from other loads.

$$T_{ia} = 0.25P_s - f_{cb}A_{cb} \quad (9\text{-}32)$$

where:

T_{ia} = the tie back tension force at the intermediate anchorage;
P_s = the maximum unfactored anchorage stressing force;
f_{cb} = the compressive stress in the region behind the anchor;
A_{cb} = the area of the continuing cross section within the extensions of the sides of the anchor plate or blister. The area of the blister or rib shall not be taken as part of the cross section.

9.21.3.5.3 Tie back reinforcement satisfying Article 9.21.3.5.2 shall be placed no further than one plate width from the tendon axis. It shall be fully anchored so that the yield strength can be developed at a distance of one plate width or half the length of the blister or rib ahead of the anchor as well as at the same distance behind the anchor. The centroid of this reinforcement shall coincide with the tendon axis, where possible. For blisters and ribs, the reinforcement shall be placed in the continuing section near that face of the flange or web from which the blister or rib is projecting.

9.21.3.5.4 Reinforcement shall be provided throughout blisters or ribs are required for shear friction, corbel action, bursting forces, and deviation forces due to tendon curvature. This reinforcement shall be in the form of ties or U-stirrups which encase the anchorage and tie it effectively into the adjacent web and flange. This reinforcement shall extend as far as possible into the flange or web and be developed by standard hooks bent around transverse bars or equivalent. Spacing shall not exceed the smallest of blister or rib height at anchor, blister width, or 6 inches.

9.21.3.5.5 Reinforcement shall be provided to resist local bending in blisters and ribs due to eccentricity of the tendon force and to resist lateral bending in ribs due to tendon deviation forces.

9.21.3.5.6 Reinforcement required by Article 9.21.3.4.4 through 9.21.3.4.9 shall be provided to resist tensile forces due to transfer of the anchorage force from the blister or rib into the overall structure.

9.21.3.6 Diaphragms

9.21.3.6.1 For tendons anchored in diaphragms, concrete compressive stresses shall be limited within the diaphragm in accordance with Articles 9.21.3.4.1 through 9.21.3.4.3. Compressive stresses shall also be checked at the transition from the diaphragm to webs and flanges of the member.

9.21.3.6.2 Reinforcement shall be provided to ensure full transfer of diaphragm anchor loads into the flanges and webs of the girder. The more general methods of Article 9.21.4 or 9.21.5 shall be used to determine this reinforcement. Reinforcement shall also be provided to tie back deviation forces due to tendon curvature.

9.21.3.7 Multiple Slab Anchorages

9.21.3.7.1 Minimum reinforcement meeting the requirements of Articles 9.21.3.7.2 through 9.21.3.7.4 shall be provided unless a more detailed analysis is made.

9.21.3.7.2 Reinforcement shall be provided for the bursting force in the direction of the thickness of the slab and normal to the tendon axis in accordance with Articles 9.21.3.4.4 and 9.21.3.4.5. This reinforcement shall be anchored close to the faces of the slab with standard hooks bent around horizontal bars, or equivalent. Minimum reinforcement is two #3 bars per anchor located at a distance equal to one-half the slab thickness ahead of the anchor.

9.21.3.7.3 Reinforcement in the plane of the slab and normal to the tendon axis shall be provided to resist edge tension forces, T_1, between anchorages (Equation

(9-33)) and bursting forces, T_2, ahead of the anchorages (Equation (9-34)). Edge tension reinforcement shall be placed immediately ahead of the anchors and shall effectively tie adjacent anchors together. Bursting reinforcement shall be distributed over the length of the anchorage zones. (See Article 9.21.1.4.)

$$T_1 = 0.10P_u\left(1-\frac{a}{s}\right) \qquad (9\text{-}33)$$

$$T_2 = 0.20P_u\left(1-\frac{a}{s}\right) \qquad (9\text{-}34)$$

where:

T_1 = the edge tension force;
T_2 = the bursting force;
P_u = the factored tendon load on an individual anchor;
a = the anchor plate width;
s = the anchorage spacing.

9.21.3.7.4 For slab anchors with an edge distance of less than two plate widths or one slab thickness, the edge tension reinforcement shall be proportioned to resist 25 percent of the factored tendon load. This reinforcement shall preferably be in the form of hairpins and shall be distributed within one plate width ahead of the anchor. The legs of the hairpin bars shall extend from the edge of the slab past the adjacent anchor but not less than a distance equal to five plate widths plus development length.

9.21.4 Application of Strut-and-Tie Models to the Design of Anchorage Zones

9.21.4.1 General

9.21.4.1.1 The flow of forces in the anchorage zone may be approximated by a series of straight compression members (struts) and straight-tension members (ties) that are connected at discrete points (nodes). Compression forces are carried by concrete compression struts and tension forces are carried by non-prestressed or prestressed reinforcement.

9.21.4.1.2 The selected strut-and-tie model shall follow a load path from the anchorages to the end of the anchorage zone. Other forces acting on the anchorage zone, such as reaction forces, tendon deviation forces, and applied loads, shall be considered in the selection of the strut-and-tie model. The forces at the end of the anchorage zone can be obtained from an axial-flexural beam analysis.

9.21.4.2 Nodes

Local zones which meet the provisions of Article 9.21.7 or Division II, Article 10.3.2.3 are considered as properly detailed, adequate nodes. The other nodes in the anchorage zone are adequate if the effective concrete stresses in the struts meet the requirements of Article 9.21.4.3 and the tension ties are properly detailed to develop the full-yield strength of the reinforcement.

9.21.4.3 Struts

9.21.4.3.1 The effective concrete compressive strength for the general zone shall usually be limited to $0.7\phi f'_{ci}$. In areas where the concrete may be extensively cracked at ultimate due to other load effects, or if large plastic rotations are required, the effective compressive strength shall be limited to $0.6\phi f'_{ci}$.

9.21.4.3.2 In anchorage zones the critical section for compression struts is ordinarily located at the interface with the local zone node. If special anchorage devices are used, the critical section of the strut can be taken as that section whose extension intersects the axis of the tendon at a depth equal to the smaller of the depth of the local confinement reinforcement or the lateral dimension of the anchorage device.

9.21.4.3.3 For thin members with a ratio of member thickness to anchorage width of no more than three, the dimension of the strut in the direction of the thickness of the member can be approximated by assuming that the thickness of the compression strut varies linearly from the transverse lateral dimension of the anchor at the surface of the concrete to the total thickness of the section at a depth equal to the thickness of the section.

9.21.4.3.4 The compression stresses can be assumed as acting parallel to the axis of the strut and as uniformly distributed over its cross section.

9.21.4.4 Ties

9.21.4.4.1 Tension forces in the strut-and-tie model shall be assumed to be carried completely by non-prestressed or prestressed reinforcement. Tensile strength of the concrete shall be neglected.

9.21.4.4.2 Tension ties shall be properly detailed and shall extend beyond the nodes to develop the full tension tie force at the node. The reinforcement layout must closely follow the directions of the ties in the strut-and-tie model.

9.21.5 Elastic Stress Analysis

9.21.5.1 Analyses based on assumed elastic material properties, equilibrium, and compatibility of strains are acceptable for analysis and design of anchorage zones.

9.21.5.2 If the compressive stresses in the concrete ahead of the anchorage device are determined from a linear-elastic stress analysis, local stress maxima may be averaged over an area equal to the bearing area of the anchorage device.

9.21.5.3 Location and magnitude of the bursting force may be obtained by integration of the corresponding tensile bursting stresses along the tendon path.

9.21.6 Approximate Methods

9.21.6.1 Limitations

In the absence of a more accurate analysis, concrete compressive stresses ahead of the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated by Equations (9-35) through (9-38), provided that:

(1) The member has a rectangular cross section and its longitudinal extent is at least equal to the largest transverse dimension of the cross section.
(2) The member has no discontinuities within or ahead of the anchorage zone.
(3) The minimum edge distance of the anchorage in the main plane of the member is at least $1^1/_2$ times the corresponding lateral dimension, *a*, of the anchorage device.
(4) Only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone. Anchorage devices can be treated as closely spaced if their center-to-center spacing does not exceed $1^1/_2$ times the width of the anchorage devices in the direction considered.
(5) The angle of inclination of the tendon with respect to the center line of the member is not larger than 20 degrees if the anchor force points toward the centroid of the section and for concentric anchors, and is not larger than 5 degrees if the anchor force points away from the centroid of the section.

9.21.6.2 Compressive Stresses

9.21.6.2.1 No additional check of concrete compressive stresses is necessary for basic anchorage devices satisfying Article 9.21.7.2.

9.21.6.2.2 The concrete compressive stresses ahead of special anchorage devices at the interface between local zone and general zone shall be approximated by Equations (9-35) and (9-36).

$$f_{ca} = \kappa \frac{0.6P_u}{A_b} \frac{1}{1+\ell_c\left(\frac{1}{b_{eff}}-\frac{1}{t}\right)} \qquad (9\text{-}35)$$

$$\kappa = 1+\left(2-\frac{s}{a_{eff}}\right)\left(0.3+\frac{n}{15}\right) \quad \text{for } s < 2a_{eff} \qquad (9\text{-}36)$$

$$\kappa = 1 \qquad \text{for } s \quad 2a_{eff}$$

where:

f_{ca} = the concrete compressive stress ahead of the anchorage device;
= a correction factor for closely spaced anchorages;
A_b = an effective bearing area as defined in Article 9.21.6.2.3;
a_{eff} = the lateral dimension of the effective bearing area measured parallel to the larger dimension of the cross section or in the direction of closely spaced anchors;
b_{eff} = the lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross section;
$_c$ = the longitudinal extent of confining reinforcement for the local zone, but not more

than the larger of 1.15 a_{eff} or 1.15 b_{eff};
P_u = the factored tendon load;
t = the thickness of the section;
s = the center-to-center spacing of multiple anchorages;
n = the number of anchorages in a row.

If a group of anchorages is closely spaced in two directions, the product of the correction factors, , for each direction is used in Equation (9-36).

9.21.6.2.3 Effective bearing area, A_b, in Equation (9-35) shall be taken as the larger of the anchor bearing plate area, A_{plate}, or the bearing area of the confined concrete in the local zone, A_{conf}, with the following limitations:

(1) If A_{plate} controls, A_{plate} shall not be taken larger than A_{conf}.
(2) If A_{conf} controls, the maximum dimension of A_{conf} shall not be more than twice the maximum dimension of A_{plate} or three times the minimum dimension of A_{plate}. If any of these limits is violated the effective bearing area, A_b, shall be based on A_{plate}.
(3) Deductions shall be made for the area of the duct in the determination of A_b.

9.21.6.3 Bursting Forces

Values for the magnitude of the bursting force, T_{burst}, and for its distance from the loaded surface, d_{burst}, shall be estimated by Equations (9-37) and (9-38), respectively. In the application of Equations (9-37) and (9-38) the specified stressing sequence shall be considered if more than one tendon is present.

$$T_{burst} = 0.25 \sum P_u \left(1 - \frac{a}{h}\right) + 0.5 P_u \sin\alpha \qquad (9\text{-}37)$$

$$d_{burst} = 0.5\left(h - 2e\right) + 5e \sin\alpha \qquad (9\text{-}38)$$

where:

ΣP_u = the sum of the total factored tendon loads for the stressing arrangement considered;
a = the lateral dimension of the anchorage device or group of devices in the direction considered;
e = The eccentricity (always taken as positive) of the anchorage device or group of devices with respect to the centroid of the cross section;
h = the lateral dimension of the cross section in the direction considered;
α = the angle of inclination of the resultant of the tendon forces with respect to the center line of the member.

9.21.6.4 Edge-Tension Forces

9.21.6.4.1 For multiple anchorages with a center-to-center spacing of less than 0.4 times the depth of the section, the spalling forces shall be given by Article 9.21.3.4.8. For larger spacings, the spalling forces shall be determined from a more detailed analysis, such as strut-and-tie models or other analytical procedures.

9.21.6.4.2 If the centroid of all tendons considered is located outside of the kern of the section both spalling forces and longitudinal edge tension forces are induced. The longitudinal edge-tension force shall be determined from an axial-flexural beam analysis at a section located at one-half the depth of the section away from the loaded surface. The spalling force shall be taken as equal to the longitudinal edge-tension force but not less than specified in Article 9.21.3.4.8.

9.21.7 Design of the Local Zone

9.21.7.1 Dimensions of the Local Zone

9.21.7.1.1 When no independently verified manufacturer's edge-distance recommendations for a particular anchorage device are available, the transverse dimensions of the local zone in each direction shall be taken as the larger of:

(1) The corresponding bearing plate size plus twice the minimum concrete cover required for the particular application and environment.
(2) The outer dimension of any required confining reinforcement plus the required concrete cover over the confining reinforcing steel for the particular application and environment.

9.21.7.1.2 When independently verified manufacturer's recommendations for minimum cover, spacing and edge distances for a particular anchorage device are available, the transverse dimensions of the local zone in each direction shall be taken as the smaller of:

(1) Twice the edge distance specified by the anchorage device supplier.
(2) The center-to-center spacing specified by the anchorage device supplier.

The manufacturer's recommendations for spacing and edge distance of anchorages shall be considered minimum values.

9.21.7.1.3 The length of the local zone along the tendon axis shall be taken as the greater of:

(1) The maximum width of the local zone.
(2) The length of the anchorage device confining reinforcement.
(3) For anchorage devices with multiple-bearing surfaces, the distance from the loaded concrete surface to the bottom of each bearing surface plus the maximum dimension of that bearing surface.

In no case shall the length of the local zone be taken as greater than $1^1/_2$ times the width of the local zone.

9.21.7.1.4 For closely spaced anchorages an enlarged local zone enclosing all individual anchorages shall also be considered.

9.21.7.2 Bearing Strength

9.21.7.2.1 Anchorage devices may be either basic anchorage devices meeting the bearing compressive strength limits of Articles 9.21.7.2.2 through 9.21.7.2.4 or special anchorage devices meeting the requirements of Section 9.21.7.3.

9.21.7.2.2 The effective concrete bearing compressive strength f_b used for design shall not exceed that of Equations (9-39) or (9-40).

$$f_b \leq 0.7\phi\, f'_{ci}\sqrt{A/A_g} \qquad (9\text{-}39)$$

but,

$$f_b \leq 2.25\phi\, f'_{ci} \qquad (9\text{-}40)$$

where:

f_b = the maximum factored tendon load, P_u, divided by the effective bearing area A_b;
f'_{ci} = the concrete compressive strength at stressing;
A = the maximum area of the portion of the supporting surface that is geometrically similar to the loaded area and concentric with it;
A_g = the gross area of the bearing plate if the requirements of Article 9.21.7.2.3 are met, or is the area calculated in accordance with Article 9.21.7.2.4;
A_b = the effective net area of the bearing plate calculated as the area A_g minus the area of openings in the bearing plate.

Equations (9-39) and (9-40) are only valid if general zone reinforcement satisfying Article 9.21.3.4 is provided and if the extent of the concrete along the tendon axis ahead of the anchorage device is at least twice the length of the local zone as defined in Article 9.21.7.1.3.

9.21.7.2.3 The full bearing plate area may be used for A_g and the calculation of A_b if the anchorage device is sufficiently rigid. To be considered sufficiently rigid, the slenderness of the bearing plate (n/t) must not exceed the value given in equation (9-41). The plate must also be checked to ensure that the plate material does not yield.

$$n/t \leq 0.08\sqrt[3]{E_b/f_b} \qquad (9\text{-}41)$$

where:

n = the largest distance from the outer edge of the wedge plate to the outer edge of the bearing plate. For rectangular bearing plates this distance is measured parallel to the edges of the bearing plate. If the anchorage has no separate wedge plate, the size of the wedge plate shall be taken as the distance between the extreme wedge holes in the corresponding direction.
t = the average thickness of the bearing plate.
E_b = the modulus of elasticity of the bearing plate material.

9.21.7.2.4 For bearing plates that do not meet the stiffness requirements of Article 9.21.7.2.3, the effective gross-bearing area, A_g, shall be taken as the area geometrically similar to the wedge plate (or to the outer perimeter of the wedge-hole pattern for plates without separate wedge plate) with dimensions increased by assuming load spreading at a 45-degree angle. A larger effective bearing area may be calculated by assuming an effective area and checking the new f_b and n/t values for conformance with Articles 9.21.7.2.2 and 9.21.7.2.3.

9.21.7.3 *Deleted*

9.22 PRETENSIONED ANCHORAGE ZONES

9.22.1 In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total prestressing force shall be placed within the distance of $d/4$ of the end of the beam.

9.22.2 For at least the distance d from the end of the beam, nominal reinforcement shall be placed to enclose the prestressing steel in the bottom flange.

9.22.3 For box girders, transverse reinforcement shall be provided and anchored by extending the leg into the web of the girder.

9.22.4 Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the member, is at least 4,000 psi.

9.23 CONCRETE STRENGTH AT STRESS TRANSFER

Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the members, is at least 4,000 psi for pretensioned members (other than piles) and 3,500 psi for post-tensioned members and pretensioned piles.

9.24 DECK PANELS

9.24.1 Deck panels shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the panels are placed on the supporting stringers. The top surface of the panels shall be roughened in such a manner as to ensure composite action between the precast and cast-in-place concrete.

9.24.2 Reinforcing bars, or equivalent mesh, shall be placed in the panel transverse to the strands to provide at least 0.11 square inches per foot of panel.

Part D
Detailing

9.25 FLANGE REINFORCEMENT

Bar reinforcement for cast-in-place T-beam and box girder flanges shall conform to the provisions in Articles 8.17.2.2 and 8.17.2.3 except that the minimum reinforcement in bottom flanges shall be 0.3 percent of the flange section.

9.26 COVER AND SPACING OF STEEL

9.26.1 Minimum Cover

The minimum cover for steel shall be in accordance with Article 8.22.

9.26.1.1 *Deleted*

9.26.1.3 *Deleted*

9.26.1.3 *Deleted*

9.26.1.4 When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.

9.26.2 Minimum Spacing

9.26.2.1 The minimum clear spacing of prestressing steel at the ends of beams shall be as follows:

Pretensioning steel: The clear distance between strands shall not be less than $1^1/_3$ times the maximum size of the concrete aggregate. Also, the minimum spacing center-to-center of strand shall be as follows:

Strand Size	Spacing
0.6 inch	2 inches
$^9/_{16}$ inch	$1^7/_8$ inches
$^1/_2$ inch	$1^3/_4$ inches
$^7/_{16}$ inch	$1^5/_8$ inches
$^3/_8$ inch	$1^1/_2$ inches

Post-tensioning steel: $1^1/_2$ inches or $1^1/_2$ times the maximum size of the concrete aggregate, whichever is greater.

9.26.2.2 Prestressing strands in deck panels shall be spaced symmetrically and uniformly across the width of the panel. They shall not be spaced farther apart than $1^1/_2$ times the total composite slab thickness or more than 18 inches.

9.26.3 Bundling

9.26.3.1 When post-tensioning steel is draped or deflected, post-tensioning ducts may be bundled in groups of three maximum, provided that the spacing specified in Article 9.26.2 is maintained in the end 3 feet of the member.

9.26.3.2 Where pretensioning steel is bundled, all bundling shall be done in the middle third of the beam length and the deflection points shall be investigated for secondary stresses.

9.26.4 Size of Ducts

9.26.4.1 For tendons made up of a number of wires, bars, or strands, duct area shall be at least twice the net area of the prestressing steel.

9.26.4.2 For tendons made up of a single wire, bar, or strand, the duct diameter shall be at least ¼ inch larger than the nominal diameter of the wire, bar, or strand.

9.27 POST-TENSIONING ANCHORAGES AND COUPLERS

9.27.1 Anchorages, couplers, and splices for bonded post-tensioned reinforcement shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. Couplers and splices shall be placed in areas approved by the Engineer and enclosed in a housing long enough to permit the necessary movements. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

9.27.2 The anchorages of unbonded tendons shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The total elongation under ultimate load of the tendon shall not be less than 2 percent measured in a minimum gauge length of 10 feet.

9.27.3 For unbonded tendons, a dynamic test shall be performed on a representative specimen and the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full size tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10 percent of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

9.27.4 Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

9.27.5 Anchorages, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion.

9.28 EMBEDMENT OF PRESTRESSED STRAND

9.28.1 Three or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than

$$\left(f_{su}^{*} - \frac{2}{3} f_{se}\right)D \qquad (9\text{-}32)$$

where D is the nominal diameter in inches, f^*_{su} and f_{se} are in kips per square inch, and the parenthetical expression is considered to be without units.

9.28.2 Investigations may be limited to those cross sections nearest each end of the member which are required to develop their full ultimate capacity.

9.28.3 Where strand is debonded at the end of a member and tension at service load is allowed in the precompressed tensile zone, the development length required above shall be doubled.

9.29 BEARINGS

Bearing devices for prestressed concrete structures shall be designed in accordance with Article 10.29 and Section 14.