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SECTION 5

CONCRETE STRUCTURES

5.1—SCOPE

The provisions in this section apply to the design of bridge and retaining wall components constructed of normal weight or lightweight concrete and reinforced with steel bars, welded wire reinforcement, and/or prestressing strands, bars, or wires. The provisions are based on concrete strengths varying from 2.4 ksi to 10.0 ksi, except where higher strengths are allowed for normal weight concrete.

The provisions of this section combine and unify the requirements for reinforced, prestressed, and partially prestressed concrete. Provisions for seismic design, analysis by the strut-and-tie model, and design of segmentally constructed concrete bridges and bridges made from precast concrete elements have been added.

A brief outline for the design of some routine concrete components is contained in Appendix A.

5.2—DEFINITIONS

Anchorage—In post-tensioning, a mechanical device used to anchor the tendon to the concrete; in pretensioning, a device used to anchor the tendon until the concrete has reached a predetermined strength, and the prestressing force has been transferred to the concrete; for reinforcing bars, a length of reinforcement, or a mechanical anchor or hook, or combination thereof at the end of a bar needed to transfer the force carried by the bar into the concrete.

Anchorage Blister—A build-out area in the web, flange, or flange-web junction for the incorporation of tendon anchorage fittings.

Anchorage Zone—The portion of the structure in which the prestressing force is transferred from the anchorage device onto the local zone of the concrete, and then distributed more widely into the general zone of the structure.

At Jacking—At the time of tensioning, the prestressing tendons.

At Loading—The maturity of the concrete when loads are applied. Such loads include prestressing forces and permanent loads but generally not live loads.

At Transfer—Immediately after the transfer of prestressing force to the concrete.

Blanketed Strand—See *Partially Debonded Strand*.

Bonded Tendon—A tendon that is bonded to the concrete, either directly or by means of grouting.

Bursting Force—Tensile forces in the concrete in the vicinity of the transfer or anchorage of prestressing forces.

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

Closely Spaced Anchorages—Anchorage devices are defined as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage devices in the direction considered.

Closure—A placement of cast-in-place concrete used to connect two or more previously cast portions of a structure.

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

Compression-Controlled Section—A cross-section in which the net tensile strain in the extreme tension steel at nominal resistance is less than or equal to the compression-controlled strain limit.

Compression-Controlled Strain Limit—The net tensile strain in the extreme tension steel at balanced strain conditions. See Article 5.7.2.1.

Concrete Cover—The specified minimum distance between the surface of the reinforcing bars, strands, post-tensioning ducts, anchorages, or other embedded items, and the surface of the concrete.

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

Confinement Anchorage—Anchorage for a post-tensioning tendon that functions on the basis of containment of the concrete in the local anchorage zone by special reinforcement.

Creep—Time-dependent deformation of concrete under permanent load.

Curvature Friction—Friction resulting from the tendon moving against the duct when tensioned due to the curvature of the duct.

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

Decompression—The stage at which the compressive stresses, induced by prestress, are overcome by the tensile stresses.

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

Deviation Saddle—A concrete block build-out in a web, flange, or web-flange junction used to control the geometry of, or to provide a means for changing direction of, external tendons.

Development Length—The distance required to develop the specified strength of a reinforcing bar or prestressing strand.

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

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

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

Effective Prestress—The stress or force remaining in the prestressing steel after all losses have occurred.

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

External Tendon—A post-tensioning tendon placed outside of the body of concrete, usually inside a box girder.

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

Fully Prestressed Component—Prestressed concrete component in which stresses satisfy the tensile stress limits at Service Limit State specified herein. Such components are assumed to remain uncracked at the Service Limit State.

General Zone—Region adjacent to a post-tensioned anchorage within which the prestressing force spreads out to an essentially linear stress distribution over the cross-section of the component.

Intermediate Anchorage—Anchorage not located at the end surface of a member or segment for tendons that do not extend over the entire length of the member or segment; usually in the form of embedded anchors, blisters, ribs, or recess pockets.

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

Internal Tendon—A post-tensioning tendon placed within the body of concrete.

Isotropic Reinforcement—An arrangement of reinforcement in which the bars are orthogonal, and the reinforcement ratios in the two directions are equal.

Jacking Force—The force exerted by the device that introduces tension into the tendons.

Launching Bearing—Temporary bearings with low friction characteristics used for construction of bridges by the incremental launching method.

Launching Nose—Temporary steel assembly attached to the front of an incrementally launched bridge to reduce superstructure force effects during launching.

Lightweight Concrete—Concrete containing lightweight aggregate and having an air-dry unit weight not exceeding 0.120 kcf, as determined by ASTM C567. Lightweight concrete without natural sand is termed “all-lightweight concrete” and lightweight concrete in which all of the fine aggregate consists of normal weight sand is termed “sand-lightweight concrete.”

Local Zone—The volume of concrete that surrounds and is immediately ahead of the anchorage device and that is subjected to high compressive stresses.

Low Relaxation Steel—Prestressing strand in which the steel relaxation losses have been substantially reduced by stretching at an elevated temperature.

Net Tensile Strain—The tensile strain at nominal resistance exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

Normal Weight Concrete—Concrete having a weight between 0.135 and 0.155 kcf.

Partially Debonded Strand—A pretensioned prestressing strand that is bonded for a portion of its length and intentionally debonded elsewhere through the use of mechanical or chemical means. Also called shielded or blanketed strand.

Partially Prestressed Component—See *Partially Prestressed Concrete*.

Partially Prestressed Concrete—Concrete with a combination of prestressing strands and reinforcing bars.

Post-Tensioning—A method of prestressing in which the tendons are tensioned after the concrete has reached a predetermined strength.

Post-Tensioning Duct—A form device used to provide a path for post-tensioning tendons or bars in hardened concrete. The following types are in general use:

Rigid Duct—Seamless tubing stiff enough to limit the deflection of a 20.0-ft length supported at its ends to not more than 1.0 in.

Semirigid Duct—A corrugated duct of metal or plastic sufficiently stiff to be regarded as not coilable into conventional shipping coils without damage.

Flexible Duct—A loosely interlocked duct that can be coiled into a 4.0-ft diameter without damage.

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

Precompressed Tensile Zone—Any region of a prestressed component in which prestressing causes compressive stresses and service load effects cause tensile stresses.

Prestressed Concrete—Concrete components in which stresses and deformations are introduced by application of prestressing forces.

Pretensioning—A method of prestressing in which the strands are tensioned before the concrete is placed.

Reinforced Concrete—Structural concrete containing no less than the minimum amounts of prestressing tendons or nonprestressed reinforcement specified herein.

Reinforcement—Reinforcing bars and/or prestressing steel.

Relaxation—The time-dependent reduction of stress in prestressing tendons.

Segmental Construction—The fabrication and erection of a structural element (superstructure and/or substructure) using individual elements, which may be either precast or cast-in-place. The completed structural element acts as a monolithic unit under some or all design loads. Post-tensioning is typically used to connect the individual elements. For superstructures, the individual elements are typically short (with respect to the span length), box-shaped segments with monolithic flanges that comprise the full width of the structure. (See Article 5.14.2.)

Seismic Hoop—A cylindrical noncontinuously wound tie with closure made using a butt weld or a mechanical coupler.

Shielded Strand—See *Partially Debonded Strand*.

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

Special Anchorage Device—Anchorage device whose adequacy should be proven in a standardized acceptance test. Most multiplane anchorages and all bond anchorages are special anchorage devices.

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

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

Spliced Precast Girder—A type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using post-tensioning, to form the completed girder. The bridge cross-section is typically a conventional structure consisting of multiple precast girders. This type of construction is not considered to be segmental construction for the purposes of these Specifications. (See Article 5.14.1.3.)

Splitting Tensile Strength—The tensile strength of concrete that is determined by a splitting test made in accordance with AASHTO T 198 (ASTM C496).

Stress Range—The algebraic difference between the maximum and minimum stresses due to transient loads.

Structural Concrete—All concrete used for structural purposes.

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

Strut-and-Tie Model—A model used principally in regions of concentrated forces and geometric discontinuities to determine concrete proportions and reinforcement quantities and patterns based on assumed compression struts in the concrete, tensile ties in the reinforcement, and the geometry of nodes at their points of intersection.

Temperature Gradient—Variation of temperature of the concrete over the cross-section.

Tendon—A high-strength steel element used to prestress the concrete.

Tension-Controlled Section—A cross-section in which the net tensile strain in the extreme tension steel at nominal resistance is greater than or equal to 0.005.

Transfer—The operation of imparting the force in a pretensioning anchoring device to the concrete.

Transfer Length—The length over which the pretensioning force is transferred to the concrete by bond and friction in a pretensioned member.

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

Wobble Friction—The friction caused by the deviation of a tendon duct or sheath from its specified profile.

Yield Strength—The specified yield strength of reinforcement.

5.3—NOTATION 2010 Revision

A = the maximum area of the portion of the supporting surface that is similar to the loaded area and concentric with it and that does not overlap similar areas for adjacent anchorage devices (in.^{2}); for segmental construction: static weight of precast segment being handled (kip) (5.10.9.7.2) (5.14.2.3.2)
A_b = area of an individual bar (in.^{2}); effective bearing area (in.^{2}); net area of a bearing plate (in.^{2}) (5.10.9.6.2) (5.10.9.7.2)
A_c = area of core of spirally reinforced compression member measured to the outside diameter of the spiral (in.^{2}); gross area of concrete deck slab (in.^{2}) (5.7.4.6) (C5.14.1.4.3)
A_{cb} = the area of the continuing cross-section within the extensions of the sides of the anchor plate or blister, i.e., the area of the blister or rib shall not be taken as part of the cross-section (in.^{2}) (5.10.9.3.4b)
A_{cp} = area enclosed by outside perimeter of concrete cross-section, including area of holes, if any (in.^{2}) (5.8.2.1) (5.8.6.3)
A_{cs} = cross-sectional area of a concrete strut in strut-and-tie model (in.^{2}) (5.6.3.3.1)
A_{cv} = area of concrete section resisting shear transfer (in.^{2}) (5.8.4.1)
A_d = area of deck concrete (in.^{2}) (5.9.5.4.3d)
A_g = gross area of section (in.^{2}); gross area of bearing plate (in.^{2}) (5.5.4.2.1) (5.10.9.7.2)
A_h = area of shear reinforcement parallel to flexural tension reinforcement (in.^{2}) (5.13.2.4.1)
A_{hr} = area of one leg of hanger reinforcement in beam ledges and inverted T-beams (in.^{2}) (5.13.2.5.5)
AI = for segmental construction: dynamic response due to accidental release or application of a precast segment (kip) (5.14.2.3.2)
A_ℓ = area of longitudinal torsion reinforcement in the exterior web of the box girder (in.^{2}) (5.8.3.6.3)
A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc} (in.^{2}) (5.13.2.4.2)
A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2}) (5.8.2.1)
A_{oh} = area enclosed by centerline of exterior closed transverse torsion reinforcement, including area of holes, if any (in.^{2}) (5.8.2.1)
A_{ps} = area of prestressing steel (in.^{2}); area of prestressing steel (in.^{2}) (5.5.4.2.1) (5.7.4.4)
A_{psb} = area of bonded prestressing steel (in.^{2}) (5.7.3.1.3b)
A_{psu} = area of unbonded prestressing steel (in.^{2}) (5.7.3.1.3b)
A_s = area of nonprestressed tension reinforcement (in.^{2}); total area of longitudinal deck reinforcement (in.^{2}) (5.5.4.2.1) (C5.14.1.4.3)
A'_s = area of compression reinforcement (in.^{2}) (5.7.3.1.1)
A_{sh} = cross-sectional area of column tie reinforcements (in.^{2}) (5.10.11.4.1d)
A_{sk} = area of skin reinforcement per unit height in one side face (in.^{2}) (5.7.3.4)
A_{sp1} = cross-sectional area of a tendon in the larger group (in.^{2}) (C5.9.5.2.3b)
A_{sp2} = cross-sectional area of a tendon in the smaller group (in.^{2}) (C5.9.5.2.3b)
A_{ss} = area of reinforcement in an assumed strut of a strut-and-tie model (in.^{2}) (5.6.3.3.4)
A_{st} = total area of longitudinal mild steel reinforcement (in.^{2}) (5.6.3.4.1)
$A_{s\text{-}BW}$ = area of steel in the footing band width (in.^{2}) (5.13.3.5)

$A_{s\text{-}SD}$ = total area of steel in short direction of a footing (in.^{2}) (5.13.3.5)
A_t = area of one leg of closed transverse torsion reinforcement (in.^{2}) (5.8.3.6.2)
A_{tr} = area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2}) (C5.14.1.4.3)
A_v = area of a transverse reinforcement within distance s (in.^{2}) (5.8.2.5)
A_{vf} = area of shear-friction reinforcement (in.^{2}); area of reinforcement for interface shear between concretes of slab and beam (in.^{2}/in.); total area of reinforcement, including flexural reinforcement (in.^{2}) (5.8.4.1) (5.10.11.4.4)
A_w = area of an individual wire to be developed or spliced (in.^{2}) (5.11.2.5.1)
A_1 = loaded area (in.^{2}) (5.7.5)
A_2 = area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area and having side slopes of 1 vertical to 2 horizontal (in.^{2}) (5.7.5)
a = depth of equivalent rectangular stress block (in.); the anchor plate width (in.); the lateral dimension of the anchorage device measured parallel to the larger dimension of the cross-section (in.) (5.7.2.2) (5.10.9.3.6) (5.10.9.6.1)
a_{eff} = lateral dimension of the effective bearing area measured parallel to the larger dimension of the cross-section (in.) (5.10.9.6.2)
a_f = distance between concentrated load and reinforcement parallel to load (in.) (5.13.2.5.1)
a_v = shear span: distance between concentrated load and face of support (in.) (5.13.2.4.1)
b = for rectangular sections, the width of the compression face of the member; for a flange section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.); least width of component section (in.); the lateral dimension of the anchorage device measured parallel to the smaller dimension of the cross-section (in.) (5.7.3) (5.10.8) (5.10.9.6.2)
b_e = effective width of the shear flow path (in.) (5.8.6.3)
b_{eff} = lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross-section (in.) (5.10.9.6.2)
b_o = perimeter of critical section for slabs and footings (in.) (5.13.3.6.1)
b_v = width of web adjusted for the presence of ducts (in.); width of the interface (in.) (5.8.2.9) (5.8.4.1)
b_w = width of member's web (in.); web width or diameter of a circular section (in.) (5.6.3.6) (5.7.3.1.1)
CEQ = for segmental construction: specialized construction equipment (kip) (5.14.2.3.2)
CLE = for segmental construction: longitudinal construction equipment load (kip) (5.14.2.3.2)
CLL = for segmental construction: distributed construction live load (ksf) (5.14.2.3.2)
CR = loss of prestress due to creep of concrete (ksi) (5.14.2.3.2)
c = distance from the extreme compression fiber to the neutral axis (in.); cohesion factor (ksi); required concrete cover over the reinforcing steel (in.); spacing from centerline of bearing to end of beam (in.) (5.5.4.2.1) (5.7.2.2) (5.8.4.1) (C5.10.9.7.1) (5.13.2.5.2)
D = external diameter of the circular member (in.) (C5.8.2.9)
DC = weight of supported structure (kip) (5.14.2.3.2)
$DIFF$ = for segmental construction: differential load (kip) (5.14.2.3.2)
D_r = diameter of the circle passing through the centers of the longitudinal reinforcement (in.) (C5.8.2.9)
DW = superimposed dead load (kip) or (klf) (5.14.2.3.2)
d = distance from compression face to centroid of tension reinforcement (in.) (5.7.3.4)
d_b = nominal diameter of a reinforcing bar, wire, or prestressing strand (in.) (5.10.2.1)
d_{burst} = distance from anchorage device to the centroid of the bursting force, T_{burst} (in.) (5.10.9.3.2)
d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto (in.); minimum concrete cover over the tendon duct, plus one-half of the duct diameter (in.) (5.7.3.4) (5.10.4.3.1)
d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) (5.8.2.9)
d_f = distance from top of ledge to compression reinforcement (in.) (5.13.2.5.5)
d_ℓ = distance from the extreme compression fiber to the centroid of extreme tension steel element (in.) (5.7.3.4)
d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.) (5.7.3.1.1)
d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement (in.) (5.7.3.2.2)
d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.) (5.7.3.2.2)
d_t = distance from extreme compression fiber to centroid of extreme tension steel (in.) (5.5.4.2.1)
d_v = effective shear depth (in.) (5.8.2.9)
E_b = modulus of elasticity of the bearing plate material (ksi) (5.10.9.7.2)

E_c = modulus of elasticity of concrete (ksi) (5.4.2.4)
E_{cd} = modulus of elasticity of deck concrete (ksi) (5.9.5.4.3d)
$E_{c\,deck}$ = modulus of elasticity of deck concrete (ksi) (C5.14.1.4.3)
E_{ci} = modulus of elasticity of concrete at transfer (ksi) (C5.9.5.2.3a)
E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi) (5.9.5.2.3a)
E_{eff} = effective modulus of elasticity (ksi) (C5.14.2.3.6)
EI = flexural stiffness (kip-in.^{2}) (5.7.4.3)
E_p = modulus of elasticity of prestressing tendons (ksi) (5.4.4.2) (5.7.4.4)
E_s = modulus of elasticity of reinforcing bars (ksi) (5.4.3.2)
e = base of Napierian logarithms; eccentricity of the anchorage device or group of devices with respect to the centroid of the cross-section; always taken as positive (in.); minimum edge distance for anchorage devices as specified by the supplier (in.) (5.9.2) (5.10.9.6.3) (C5.10.9.7.1)
e_d = eccentricity of deck with respect to the transformed composite section, taken as negative in common construction (in.) (5.9.5.4.3d)
e_m = average eccentricity at midspan (in.) (C5.9.5.2.3a)
e_{pc} = eccentricity of strands with respect to centroid of composite section (in.) (5.9.5.4.3a)
e_{pg} = eccentricity of strands with respect to centroid of girder (in.) (5.9.5.4.2a)
F = force effect calculated using instantaneous modulus of elasticity at time loading is applied (kip) (5.9.2)
F' = reduced force resultant accounting for creep in time corresponding to the ϕ used (kip) (5.9.2)
F_ε = reduction factor (5.8.3.4.2)
$F_{u\text{-}in}$ = in-plane deviation force effect per unit length of tendon (kips/ft) (5.10.4.3.1)
$F_{u\text{-}out}$ = out-of-plane force effect per unit length of tendon (kips/ft) (5.10.4.3.2)
f_b = stress in anchor plate at a section taken at the edge of the wedge hole or holes (ksi) (5.10.9.7.2)
f'_c = specified compressive strength of concrete for use in design (ksi) (5.4.2.1)
f_{ca} = concrete compressive stress ahead of the anchorage devices (ksi) (5.10.9.6.2)
f_{cb} = unfactored dead load compressive stress in the region behind the anchor (ksi) (5.10.9.3.4b)
f_{cgp} = concrete stress at the center of gravity of prestressing tendons, that results from the prestressing force at either transfer or jacking and the self-weight of the member at sections of maximum moment (ksi) (5.9.5.2.3a)
f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi); nominal concrete strength at time of application of tendon force (ksi) (5.4.2.3.2) (5.10.9.7.2)
f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi) (5.7.3.3.2)
f_{ct} = average splitting tensile strength of lightweight aggregate concrete (ksi) (5.8.2.2)
f_{cu} = limiting concrete compressive stress for design by strut-and-tie model (ksi) (5.6.3.3.1)
f_{min} = algebraic minimum stress level (ksi) (5.5.3.2)
f_n = nominal concrete bearing stress (ksi) (5.10.9.7.2)
f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi) (C5.9.5.2.3a)
f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange (ksi); in a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section or at the junction of the web and flange when the centroid lies within the flange, that results from both prestress and the bending moments resisted by the precast member acting alone (ksi) (C5.6.3.5)
f_{pe} = effective stress in the prestressing steel after losses (ksi) (5.6.3.4.1) (5.7.4.4)
f_{pj} = stress in the prestressing steel at jacking (ksi) (5.9.3)
f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi) (5.8.3.4.2)
f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of member is required (ksi) (C5.6.3.3.3)
f_{psl} = stress in the strand at the Service limit state. Cracked section shall be assumed (ksi) (C5.14.1.4.9)
f_{pt} = stress in prestressing steel immediately after transfer (ksi) (5.9.3)
f_{pu} = specified tensile strength of prestressing steel (ksi) (5.4.4.1)
f_{pul} = stress in the strand at the Strength limit state (ksi) (C5.14.1.4.9)
f_{px} = design stress in pretensioned strand at nominal flexural strength at section of member under consideration (ksi) (C5.11.4.2)
f_{py} = yield strength of prestressing steel (ksi) (5.4.4.1)
f_r = modulus of rupture of concrete (ksi) (5.4.2.6)

f_s = stress in the mild tension reinforcement at nominal flexural resistance (ksi) (5.7.3.1) (5.7.3.2)
f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi) (5.7.3.1) (5.7.3.2)
f_{ss} = tensile stress in mild steel reinforcement at the service limit state (ksi) (5.7.3.4)
f_y = specified minimum yield strength of reinforcing bars (ksi); specified yield strength of reinforcing bars ≤75 ksi (5.5.4.2.1) (5.10.8)
f'_y = specified minimum yield strength of compression reinforcement (ksi) (5.7.3.1.1)
f_{yh} = specified yield strength of transverse reinforcement (ksi) (5.7.4.6)
H = average annual ambient mean relative humidity (percent) (5.4.2.3.2)
h = overall thickness or depth of a member (in.); least thickness of component section (in.); lateral dimension of the cross-section in the direction considered (in.) (5.7.3.4) (5.10.8) (5.10.9.6.3)
h_c = core dimension of tied column in direction under consideration (in.) (5.10.11.4.1d)
h_f = compression flange depth (in.) (5.7.3.1.1)
h_1 = largest lateral dimension of member (in.) (C5.10.9.3.2)
h_2 = least lateral dimension of member (in.) (C5.10.9.3.2)
I_c = moment of inertia of section calculated using the net concrete section properties of the girder and the deck and the deck-to-girder modular ratio at service (in.^{4}) (5.9.5.4.3a)
I_{cr} = moment of inertia of the cracked section, transformed to concrete (in.^{4}) (5.7.3.6.2)
IE = for segmental construction: dynamic load from equipment (kip) (5.14.2.3.2)
I_e = effective moment of inertia (in.^{4}) (5.7.3.6.2)
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4}) (5.7.3.6.2)
I_s = moment of inertia of the reinforcing taken about the centroid of the column (in.^{4}) (5.7.4.3)
K = effective length factor for compression members; stress variable used in calculating torsional cracking moment; wobble friction coefficient (per ft of tendon) (5.7.4.1) (5.8.6.3) (5.9.5.2.2b)
K_{df} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between deck placement and final time (5.9.5.4.3a)
K_{id} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement (5.9.5.4.2a)
K_L = factor accounting for type of steel taken as 30 for low relaxation strands and 7 for other prestressing steel, unless more accurate manufacturer's data are available (5.9.5.4.2c)
K'_L = factor accounting for type of steel (C5.9.5.4.2c)
K_1 = correction factor for source of aggregate (5.4.2.4)
k_c = factor for the effect of the volume-to-surface ratio (C5.4.2.3.2)
k_f = factor for the effect of concrete strength (5.4.2.3.2)
k_{hc} = humidity factor for creep (5.4.2.3.2)
k_{hs} = humidity factor for shrinkage (5.4.2.3.3)
k_s = factor for the effect of the volume-to-surface ratio (C5.4.2.3.2)
k_{td} = time development factor (5.4.2.3.2)
k_{vs} = factor for the effect of the volume-to-surface ratio of the component (5.4.2.3.2)
L = span length (ft or in.); length of bearing plate or pad (in.) (5.7.3.1.2) (5.13.2.5.4)
ℓ_a = additional embedment length at support or at point of inflection (in.) (C5.11.1.2.2)
ℓ_c = longitudinal extent of confining reinforcement of the local zone but not more than the larger of 1.15 a_{eff} or 1.15 b_{eff} (in.); length of lap for compression lap splices (in.) (5.10.9.6.2) (5.11.5.5.1)
ℓ_d = development length (in.) (5.11.1.2.1)
ℓ_{db} = basic development length for straight reinforcement to which modification factors are applied to determine ℓ_d (in.) (5.11.2.1.1)
ℓ_{dh} = development length of standard hook in tension as measured from critical section to outside end of hook (in.) (5.11.2.4.1)
ℓ_{dsh} = total length of extended strand (in.) (C5.14.1.4.9)
ℓ_e = effective tendon length (in.); embedment length beyond standard stirrup hook (in.) (5.7.3.1.2) (5.11.2.6.2)
ℓ_{hb} = basic development length of standard hook in tension (in.) (5.11.2.4.1)
ℓ_{hd} = development length for deformed wire fabric (in.) (5.11.2.5.1)
ℓ_i = length of tendon between anchorages (in.) (5.7.3.1.2)
ℓ_{px} = distance from free end of pretensioned strand to section of member under consideration (in.) (C5.11.4.2)
ℓ_u = unsupported length of a compression member (in.) (5.7.4.1)
M_a = maximum moment in a member at the stage for which deformation is computed (kip-in.) (5.7.3.6.2)

M_c = magnified moment used for proportioning slender compression members (kip-in.) (5.7.4.3)
M_{cr} = cracking moment (kip-in.) (5.7.3.3.2) (5.7.3.6.2)
M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.) (5.7.3.3.2)
M_g = midspan moment due to member self-weight (kip-in.) (C5.9.5.2.3a)
M_n = nominal flexural resistance (kip-in.) (5.7.3.2.1)
M_r = factored flexural resistance of a section in bending (kip-in.) (5.7.3.2.1)
M_{rx} = uniaxial factored flexural resistance of a section in the direction of the x-axis (kip-in.) (5.7.4.5)
M_{ry} = uniaxial factored flexural resistance of a section in the direction of the y-axis (kip-in.) (5.7.4.5)
M_u = factored moment at the section (kip-in.) (C5.6.3.1)
M_{ux} = component of moment due to factored load in the direction of the x-axis (kip-in.) (5.7.4.5)
M_{uy} = component of moment due to factored load in the direction of the y-axis (kip-in.) (5.7.4.5)
M_1 = smaller end moment at the strength limit state due to factored load acting on a compression member; positive if the member is bent in single curvature and negative if bent in double curvature (kip-in.) (5.7.4.3)
M_2 = larger end moment at the strength limit state due to factored load acting on a compression member; always positive (kip-in.) (5.7.4.3)
m = modification factor (5.7.5)
N = the number of cycles of stress range; the number of identical prestressing tendons (5.5.3.4) (5.9.5.2.3b)
N_R = factored tensile resistance of transverse pair of reinforcing bars (kip) (5.13.2.3)
N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points (5.7.3.1.2)
N_u = applied factored axial force taken as positive if tensile (kip) (5.8.3.4.2)
N_{uc} = factored axial force normal to the cross-section, occurring simultaneously with V_u; taken to be positive for tension and negative for compression; includes effects of tension due to creep and shrinkage (kip) (5.13.2.4.1)
N_1 = number of tendons in the larger group (C5.9.5.2.3b)
N_2 = number of tendons is the smaller group (C5.9.5.2.3b)
n = modular ratio = E_s/E_c or E_p/E_c; number of anchorages in a row; projection of base plate beyond the wedge hole or wedge plate, as appropriate (in.); modular ratio between deck concrete and reinforcement (5.7.1) (5.10.9.6.2) (5.10.9.7.2) (C5.14.1.4.3)
P_c = permanent net compressive force (kip) (5.8.4.1)
P_n = nominal axial resistance of a section (kip); nominal axial resistance of strut or tie (kip); nominal bearing resistance (kip) (5.5.4.2.1) (5.6.3.2) (5.7.5)
P_o = nominal axial resistance of a section at 0.0 eccentricity (kip) (5.7.4.5)
PPR = partial prestressing ratio (5.5.4.2.1)
P_r = factored axial resistance of strut or tie (kip); factored bearing resistance of anchorages (kip); factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement (kip) (5.6.3.2) (5.10.9.7.2) (5.10.10.1)
P_{rx} = factored axial resistance corresponding to M_{rx} (kip) (5.7.4.5)
P_{rxy} = factored axial resistance with biaxial loading (kip) (5.7.4.5)
P_{ry} = factored axial resistance corresponding to M_{ry} (kip) (5.7.4.5)
P_s = maximum unfactored anchorage stressing force (kip) (5.10.9.3.4b)
P_u = factored axial force effect or factored tendon force (kip); factored tendon load on an individual anchor (kip) (5.7.4.3) (5.10.9.3.6)
p_c = length of outside perimeter of the concrete section (in.) (5.8.2.1) (5.8.6.3)
p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.); perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion (in.) (5.8.2.1) (5.8.6.4)
Q = force effect in associated units (5.14.2.3.4)
R = radius of curvature of the tendon at the considered location (ft) (5.10.4.3.1)
r = radius of gyration of gross cross-section (in.) (5.7.4.1)
r/h = ratio of base radius to height of rolled-on transverse deformations (5.5.3.2)
S = center-to-center spacing of bearing along a beam ledge (in.) (5.13.2.5.2)
S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3}) (5.7.3.3.2)
SH = shrinkage (5.14.2.3.2)
S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3}) (5.7.3.3.2)

s = average spacing of mild steel reinforcement in layer closest to tension face (in.); spacing of reinforcing bars (in.); spacing of rows of ties (in.); anchorage spacing (in.); center-to-center spacing of anchorages (in.); spacing of hanger reinforcing bars (in.) (5.7.3.4) (5.8.2.5) (5.8.4.1) (5.10.9.3.6) (5.10.9.6.2) (5.13.2.5.5)

s_{max} = maximum permitted spacing of transverse reinforcement (in.) (5.8.2.7)

s_w = spacing of wires to be developed or spliced (in.) (5.11.2.5.1)

s_x = crack spacing parameter (in.) (C5.8.3.4.2)

s_{xe} = equivalent value of s_x which allows for influence of aggregate size (in.) (5.8.3.4.2)

T_{burst} = tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis (kip) (5.10.9.6.3)

T_{cr} = torsional cracking resistance (kip-in.) (5.8.2.1)

T_{ia} = tie-back tension force at the intermediate anchorage (kip) (5.10.9.3.4b)

T_n = nominal torsion resistance (kip-in.) (5.8.2.1)

T_r = factored torsional resistance provided by circulatory shear flow (kip-in.) (5.8.2.1)

T_u = factored torsional moment (kip-in.) (C5.6.3.1)

T_1 = edge tension force (kip) (5.10.9.3.6)

T_2 = bursting force (kip) (5.10.9.3.6)

t = time (day); thickness of wall (in.); thickness of the section (in.); average thickness of bearing plate (in.) (5.4.2.3.2) (5.7.4.7.1) (5.10.9.6.2) (5.10.9.7.2)

t_d = age at deck placement (day) (5.9.5.4.2b)

t_f = final age (day) (5.9.5.4.2a)

t_i = age of concrete when load is initially applied (day) (5.4.2.3.2)

U = for segmental construction: segment unbalance (kip) (5.14.2.3.2)

V_c = nominal shear resistance provided by tensile stresses in the concrete (kip) (5.8.2.4)

V_n = nominal shear resistance of the section considered (kip) (5.8.2.1)

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip) (C5.8.2.3)

V_r = factored shear resistance (kip) (5.8.2.1)

V/S = volume-to-surface ratio (5.4.2.3.2)

V_s = shear resistance provided by shear reinforcement (kip) (5.8.3.3)

V_u = factored shear force at section (kip) (C5.6.3.1)

v_u = average factored shear stress on the concrete (ksi) (5.8.2.7) (5.8.2.9)

W = width of bearing plate measured along the length of a corbel, bracket, or beam ledge (in.) (C5.13.2.5.1)

W/C = water–cement ratio (5.12.3)

WE = for segmental construction: horizontal wind load on equipment (kip) (5.14.2.3.2)

WUP = for segmental construction: wind uplift on cantilever (ksf) (5.14.2.3.2)

w_c = unit weight of concrete (kcf) (5.4.2.4)

X_u = clear length of the constant thickness portion of a wall between other walls or fillers between walls (in.) (5.7.4.7.1)

x = length of a prestressing tendon from the jacking end to any point under consideration (ft) (5.9.5.2.2b)

y_t = distance from the neutral axis to the extreme tension fiber (in.) (5.7.3.6.2)

α = angle of inclination of transverse reinforcement to longitudinal axis (degrees); total angular change of prestressing steel path from jacking end to a point under investigation (rad.); the angle of inclination of a tendon force with respect to the centerline of the member (degrees) (5.8.3.3) (5.9.5.2.2b) (5.10.9.6.3)

α_h = total horizontal angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.5.2.2b)

α_s = angle between compressive strut and adjoining tension tie (degrees) (5.6.3.3.3)

α_v = total vertical angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.5.2.2b)

β = factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension; ratio of long side to short side of footing (5.8.3.3) (5.13.3.5)

β_b = ratio of the area of reinforcement cut off to the total area of tension reinforcement at the section (5.11.1.2.1)

β_c = ratio of the long side to the short side of the concentrated load or reaction area (5.13.3.6.3)

β_d = ratio of maximum factored dead load moments to maximum factored total load moment; always positive (5.7.4.3)

β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone (5.7.2.2)

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face (5.7.3.4)
γ = load factor
γ_e = crack control exposure condition factor (5.7.3.4)
Δf = live load stress range due to fatigue load (ksi) (5.5.3.1)
$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold (ksi) (5.5.3.1)
Δf_{cd} = change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement, combined with deck weight and superimposed loads (ksi) (5.9.5.4.3b)
Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi) (5.9.5.4.3d)
Δf_{cdp} = change in concrete stress at c.g. of prestressing steel due to all dead loads, except dead load acting at the time the prestressing force is applied (ksi) (5.9.5.4.3)
Δf_{pA} = loss in prestressing steel stress due to anchorage set (ksi) (5.9.5.1)
Δf_{pCD} = prestress loss due to creep of girder concrete between time of deck placement and final time (ksi) (5.9.5.4.1)
Δf_{pCR} = prestress loss due to creep of girder concrete between transfer and deck placement (ksi) (5.9.5.4.1)
Δf_{pES} = loss in prestressing steel stress due to elastic shortening (ksi) (5.9.5.1)
Δf_{pF} = loss in prestressing steel stress due to friction (ksi) (5.9.5.1)
Δf_{pR1} = prestress loss due to relaxation of prestressing strands between transfer and deck placement (ksi) (5.9.5.4.1)
Δf_{pR2} = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi) (5.9.5.4.1)
Δf_{pSD} = prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi) (5.9.5.4.1)
Δf_{pSR} = prestress loss due to shrinkage of girder concrete between transfer and deck placement (ksi) (5.9.5.4.1)
Δf_{pSS} = prestress loss due to shrinkage of deck composite section (ksi) (5.9.5.4.1)
Δf_{pT} = total loss in prestressing steel stress (ksi) (5.9.5.1)
ε_{bdf} = shrinkage strain of girder between time of deck placement and final time (in./in.) (5.9.5.4.3a)
ε_{bid} = concrete shrinkage strain of girder between transfer and deck placement (in./in.) (5.9.5.4.2a)
ε_{cu} = failure strain of concrete in compression (in./in.) (5.7.3.1.2) (5.7.4.4)
ε_{ddf} = shrinkage strain of deck concrete between placement and final time (in./in.) (5.9.5.4.3d)
$\varepsilon_{effective}$ = effective concrete shrinkage strain (in./in.) (C5.14.1.4.3)
ε_s = tensile strain in cracked concrete in direction of tension tie (in./in.); strain in nonprestressed longitudinal tension reinforcement (in./in.) (5.6.3.3.3) (5.8.3.4.2)
ε_{sh} = concrete shrinkage strain at a given time (in./in.); net longitudinal tensile strain in the section at the centroid of the tension reinforcement (in./in.) (5.4.2.3.3) (C5.14.1.4.3)
ε_t = net tensile strain in extreme tension steel at nominal resistance (C5.5.4.2.1)
ε_x = longitudinal strain in the web of the member (in./in.) (Appendix B5)
ε_1 = principal tensile strain in cracked concrete due to factored loads (in./in.) (5.6.3.3.3)
θ = angle of inclination of diagonal compressive stresses (degrees) (5.8.3.3)
θ_s = angle between compression strut and longitudinal axis of the member in a shear truss model of a beam (degrees) (5.6.3.3.2)
κ = correction factor for closely spaced anchorages; multiplier for strand development length (5.10.9.6.2) (5.11.4.2)
λ = parameter used to determine friction coefficient μ (5.8.4.2)
λ_w = wall slenderness ratio for hollow columns (5.7.4.7.1)
μ = coefficient of friction (5.8.4.1)
ρ_h = ratio of area of horizontal shear reinforcement to area of gross concrete area of a vertical section (5.10.11.4.2)
ρ_{min} = minimum ratio of tension reinforcement to effective concrete area (5.7.3.3.2)
ρ_s = ratio of spiral reinforcement to total volume of column core (5.7.4.6)
ρ_v = ratio of area of vertical shear reinforcement to area of gross concrete area of a horizontal section (5.10.11.4.2)
ϕ = resistance factor (5.5.4.2.1)
ϕ_w = hollow column reduction factor (5.7.4.7.2)
$\Psi(t, t_i)$ = creep coefficient—the ratio of the creep strain that exists t days after casting to the elastic strain caused when load p_i is applied t_i days after casting (5.4.2.3.2)
$\Psi_b(t_d, t_i)$ = girder creep coefficient at time of deck placement due to loading introduced at transfer (5.9.5.4.2b)

$\Psi_b(t_f, t_d)$ = girder creep coefficient at final time due to loading at deck placement; creep coefficient of deck concrete at final time due to loading introduced shortly after deck placement (i.e., overlays, barriers, etc.) (5.9.5.4.3b) (5.9.5.4.3d)

$\Psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer (5.9.5.4.2a)

5.4—MATERIAL PROPERTIES

5.4.1—General

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

When other grades or types of materials are used, their properties, including statistical variability, shall be established prior to design. The minimum acceptable properties and test procedures for such materials shall be specified in the contract documents.

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

C5.4.1

According to *AASHTO LRFD Bridge Construction Specifications*, all materials and tests must conform to the appropriate standards included in the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing* and/or the standards of the American Society for Testing and Materials.

Occasionally, it may be appropriate to use materials other than those included in the *AASHTO LRFD Bridge Construction Specifications*; for example, when concretes are modified to obtain very high-strengths through the introduction of special materials, such as:

- Silica fume,
- Cements other than Portland or blended hydraulic cements,
- Proprietary high early strength cements,
- Ground granulated blast-furnace slag, and
- Other types of cementitious and/or Pozzolanic materials.

In these cases, the specified properties of such materials should be measured using the testing procedures defined in the contract documents.

5.4.2—Normal Weight and Structural Lightweight Concrete

5.4.2.1—Compressive Strength

For each component, the specified compressive strength, f'_c, or the class of concrete shall be shown in the contract documents.

Design concrete strengths above 10.0 ksi for normal weight concrete shall be used only when allowed by specific Articles or when physical tests are made to establish the relationships between the concrete strength and other properties. Specified concrete with strengths below 2.4 ksi should not be used in structural applications.

The specified compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

For lightweight structural concrete, air dry unit weight, strength and any other properties required for the application shall be specified in the contract documents.

C5.4.2.1

The evaluation of the strength of the concrete used in the work should be based on test cylinders produced, tested, and evaluated in accordance with Section 8 of the *AASHTO LRFD Bridge Construction Specifications*.

This Section was originally developed based on an upper limit of 10.0 ksi for the design concrete compressive strength. As research information for concrete compressive strengths greater than 10.0 ksi becomes available, individual Articles are being revised or extended to allow their use with higher strength concretes. Appendix C5 contains a listing of the Articles affected by concrete compressive strength and their current upper limit.

It is common practice that the specified strength be attained 28 days after placement. Other maturity ages may be assumed for design and specified for components that will receive loads at times appreciably different than 28 days after placement.

For concrete Classes A, A(AE), and P used in or over saltwater, the *W*/*C* ratio shall be specified not to exceed 0.45.

The sum of Portland cement and other cementitious materials shall be specified not to exceed 800 pcy, except for Class P (HPC) concrete where the sum of Portland cement and other cementitious materials shall be specified not to exceed 1000 pcy.

Air-entrained concrete, designated "AE" in Table C5.4.2.1-1, shall be specified where the concrete will be subject to alternate freezing and thawing and exposure to deicing salts, saltwater, or other potentially damaging environments.

It is recommended that the classes of concrete shown in Table C5.4.2.1-1 and their corresponding specified strengths be used whenever appropriate. The classes of concrete indicated in Table C5.4.2.1-1 have been developed for general use and are included in *AASHTO LRFD Bridge Construction Specifications*, Section 8, "Concrete Structures," from which Table C5.4.2.1-1 was taken.

These classes are intended for use as follows:

Class A concrete is generally used for all elements of structures, except when another class is more appropriate, and specifically for concrete exposed to saltwater.

Class B concrete is used in footings, pedestals, massive pier shafts, and gravity walls.

Class C concrete is used in thin sections, such as reinforced railings less than 4.0 in. thick, for filler in steel grid floors, etc.

Class P concrete is used when strengths in excess of 4.0 ksi are required. For prestressed concrete, consideration should be given to limiting the nominal aggregate size to 0.75 in.

Class S concrete is used for concrete deposited underwater in cofferdams to seal out water.

Strengths above 5.0 ksi should be used only when the availability of materials for such concrete in the locale is verified.

Lightweight concrete is generally used only under conditions where weight is critical.

In the evaluation of existing structures, it may be appropriate to modify the f'_c and other attendant structural properties specified for the original construction to recognize the strength gain or any strength loss due to age or deterioration after 28 days. Such modified f'_c should be determined by core samples of sufficient number and size to represent the concrete in the work, tested in accordance with AASHTO T 24M/T 24 (ASTM C42/C42M).

There is considerable evidence that the durability of reinforced concrete exposed to saltwater, deicing salts, or sulfates is appreciably improved if, as recommended by ACI 318, either or both the cover over the reinforcing steel is increased or the *W*/*C* ratio is limited to 0.40. If materials, with reasonable use of admixtures, will produce a workable concrete at *W*/*C* ratios lower than those listed in Table C5.4.2.1-1, the contract documents should alter the recommendations in Table C5.4.2.1-1 appropriately.

The specified strengths shown in Table C5.4.2.1-1 are generally consistent with the *W*/*C* ratios shown. However, it is possible to satisfy one without the other.

Both are specified because W/C ratio is a dominant factor contributing to both durability and strength; simply obtaining the strength needed to satisfy the design assumptions may not ensure adequate durability.

Table C5.4.2.1-1—Concrete Mix Characteristics by Class

Class of Concrete	Minimum Cement Content	Maximum W/C Ratio	Air Content Range	Coarse Aggregate Per AASHTO M 43 (ASTM D448)	28-Day Compressive Strength
	pcy	lbs. Per lbs.	%	Square Size of Openings (in.)	ksi
A A(AE)	611 611	0.49 0.45	— 6.0 ± 1.5	1.0 to No. 4 1.0 to No. 4	4.0 4.0
B	517	0.58	—	2.0 to No. 3 and No. 3 to No. 4	2.4
B(AE)	517	0.55	5.0 ± 1.5	2.0 to No. 3 and No. 3 to No. 4	2.4
C C(AE)	658 658	0.49 0.45	— 7.0 ± 1.5	0.5 to No. 4 0.5 to No. 4	4.0 4.0
P P(HPC)	564	0.49	As specified elsewhere	1.0 to No. 4 or 0.75 to No. 4	As specified elsewhere
S	658	0.58	—	1.0 to No. 4	—
Lightweight	564	As specified in the contract documents			

5.4.2.2—Coefficient of Thermal Expansion

The coefficient of thermal expansion should be determined by the laboratory tests on the specific mix to be used.

In the absence of more precise data, the thermal coefficient of expansion may be taken as:

- For normal weight concrete: 6.0×10^{-6}/°F, and
- For lightweight concrete: 5.0×10^{-6}/°F

C5.4.2.2

The thermal coefficient depends primarily on the types and proportions of aggregates used and on the degree of saturation of the concrete.

The thermal coefficient of normal weight concrete can vary between 3.0 to 8.0×10.0^{-6}/°F, with limestone and marble aggregates producing the lower values, and chert and quartzite the higher. Only limited determinations of these coefficients have been made for lightweight concretes. They are in the range of 4.0 to 6.0×10^{-6}/°F and depend on the amount of natural sand used.

Additional information may be found in ACI 209, ACI 343 and ACI 213.

5.4.2.3—Shrinkage and Creep

5.4.2.3.1—General

Values of shrinkage and creep, specified herein and in Articles 5.9.5.3 and 5.9.5.4, shall be used to determine the effects of shrinkage and creep on the loss of prestressing force in bridges other than segmentally constructed ones. These values in conjunction with the moment of inertia, as specified in Article 5.7.3.6.2, may be used to determine the effects of shrinkage and creep on deflections.

C5.4.2.3.1

Creep and shrinkage of concrete are variable properties that depend on a number of factors, some of which may not be known at the time of design.

Without specific physical tests or prior experience with the materials, the use of the empirical methods referenced in these Specifications cannot be expected to yield results with errors less than ±50 percent.

These provisions shall be applicable for specified concrete strengths up to 15.0 ksi. In the absence of more accurate data, the shrinkage coefficients may be assumed to be 0.0002 after 28 days and 0.0005 after one year of drying.

When mix-specific data are not available, estimates of shrinkage and creep may be made using the provisions of:

- Articles 5.4.2.3.2 and 5.4.2.3.3,
- The CEB-FIP model code, or
- ACI 209.

For segmentally constructed bridges, a more precise estimate shall be made, including the effect of:

- Specific materials,
- Structural dimensions,
- Site conditions, and
- Construction methods, and
- Concrete age at various stages of erection.

5.4.2.3.2—Creep

The creep coefficient may be taken as:

$$\psi(t,t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118} \quad (5.4.2.3.2\text{-}1)$$

in which:

$$k_s = 1.45 - 0.13(V/S) \geq 1.0 \quad (5.4.2.3.2\text{-}2)$$

$$k_{hc} = 1.56 - 0.008H \quad (5.4.2.3.2\text{-}3)$$

$$k_f = \frac{5}{1 + f'_{ci}} \quad (5.4.2.3.2\text{-}4)$$

$$k_{td} = \left(\frac{t}{61 - 4f'_{ci} + t}\right) \quad (5.4.2.3.2\text{-}5)$$

where:

H = relative humidity (%). In the absence of better information, H may be taken from Figure 5.4.2.3.3-1.

k_s = factor for the effect of the volume-to-surface ratio of the component

k_f = factor for the effect of concrete strength

k_{hc} = humidity factor for creep

k_{td} = time development factor

C5.4.2.3.2

The methods of determining creep and shrinkage, as specified herein and in Article 5.4.2.3.3, are based on Huo et al. (2001), Al-Omaishi (2001), Tadros (2003), and Collins and Mitchell (1991). These methods are based on the recommendation of ACI Committee 209 as modified by additional recently published data. Other applicable references include Rusch et al. (1983), Bazant and Wittman (1982), and Ghali and Favre (1986).

The creep coefficient is applied to the compressive strain caused by permanent loads in order to obtain the strain due to creep.

Creep is influenced by the same factors as shrinkage, and also by:

- Magnitude and duration of the stress,
- Maturity of the concrete at the time of loading, and
- Temperature of concrete.

Creep shortening of concrete under permanent loads is generally in the range of 0.5 to 4.0 times the initial elastic shortening, depending primarily on concrete maturity at the time of loading.

The time development of shrinkage, given by Eq. 5.4.2.3.2-5, is proposed to be used for both precast concrete and cast-in-place concrete components of a bridge member, and for both accelerated curing and moist curing conditions. This simplification is based on a parametric study documented in Tadros (2003), on prestress losses in high strength concrete. It was found that various time development prediction methods have virtually no impact on the final creep and shrinkage coefficients, prestress losses, or member deflections.

t = maturity of concrete (day), defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects

t_i = age of concrete at time of load application (day)

V/S = volume-to-surface ratio (in.)

f'_{ci} = specified compressive strength of concrete at time of prestressing for pretensioned members and at time of initial loading for nonprestressed members. If concrete age at time of initial loading is unknown at design time, f'_{ci} may be taken as $0.80 f'_c$ (ksi).

The surface area used in determining the volume-to-surface ratio should include only the area that is exposed to atmospheric drying. For poorly ventilated enclosed cells, only 50 percent of the interior perimeter should be used in calculating the surface area. For precast members with cast-in-place topping, the total precast surface should be used. For pretensioned stemmed members (I-beams, T-beams, and box beams), with an average web thickness of 6.0 to 8.0 in., the value of k_{vs} may be taken as 1.00.

It was also observed in that study that use of modern concrete mixtures with relatively low water/cement ratios and with high range water reducing admixtures, has caused time development of both creep and shrinkage to have similar patterns. They have a relatively rapid initial development in the first several weeks after concrete placement and a slow further growth thereafter. For calculation of intermediate values of prestress losses and deflections in cast-in-place segmental bridges constructed with the balanced cantilever method, it may be warranted to use actual test results for creep and shrinkage time development using local conditions. Final losses and deflections would be substantially unaffected whether Eq. 5.4.2.3.2-5 or another time-development formula is used.

The factors for the effects of volume-to-surface ratio are an approximation of the following formulas:

For creep:

$$k_c = \left[\frac{\dfrac{t}{26\,e^{0.36(V/S)} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1.80 + 1.77\,e^{-0.54(V/S)}}{2.587}\right] \tag{C5.4.2.3.2-1}$$

For shrinkage:

$$k_s = \left[\frac{\dfrac{t}{26e^{0.36(V/S)} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1064 - 94(V/S)}{923}\right] \tag{C5.4.2.3.2-2}$$

The maximum V/S ratio considered in the development of Eqs. C5.4.2.3.2-1 and C5.4.2.3.2-2 was 6.0 in.

Ultimate creep and shrinkage are less sensitive to surface exposure than intermediate values at an early age of concrete. For accurately estimating intermediate deformations of such specialized structures as segmentally constructed balanced cantilever box girders, it may be necessary to resort to experimental data or use the more detailed Eqs. C5.4.2.3.2-1 and C5.4.2.3.2-2.

5.4.2.3.3—Shrinkage

For concretes devoid of shrinkage-prone aggregates, the strain due to shrinkage, ε_{sh}, at time, t, may be taken as:

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td}\, 0.48 \times 10^{-3} \tag{5.4.2.3.3-1}$$

in which:

C5.4.2.3.3

Shrinkage of concrete can vary over a wide range from nearly nil if continually immersed in water to in excess of 0.0008 for thin sections made with high shrinkage aggregates and sections that are not properly cured.

Shrinkage is affected by:

- Aggregate characteristics and proportions,

$$k_{hs} = (2.00 - 0.014\,H) \qquad (5.4.2.3.3\text{-}2)$$

where:

k_{hs} = humidity factor for shrinkage

If the concrete is exposed to drying before 5 days of curing have elapsed, the shrinkage as determined in Eq. 5.4.2.3.3-1 should be increased by 20 percent.

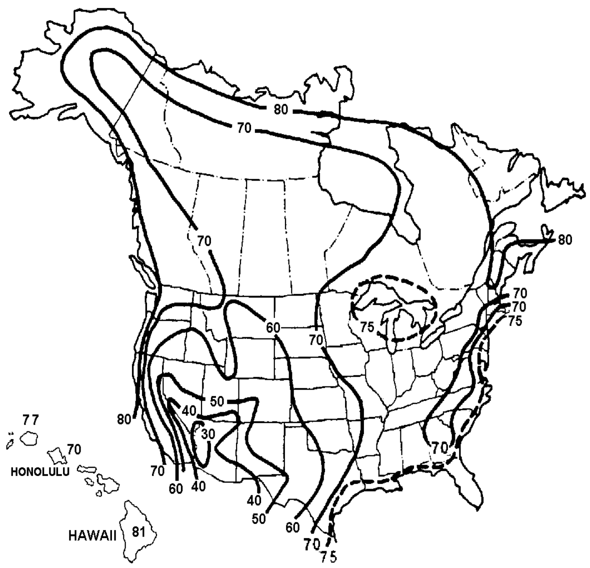


Figure 5.4.2.3.3-1—Annual Average Ambient Relative Humidity in Percent

- Average humidity at the bridge site,
- W/C ratio,
- Type of cure,
- Volume to surface area ratio of member, and
- Duration of drying period.

Large concrete members may undergo substantially less shrinkage than that measured by laboratory testing of small specimens of the same concrete. The constraining effects of reinforcement and composite actions with other elements of the bridge tend to reduce the dimensional changes in some components.

5.4.2.4—Modulus of Elasticity

In the absence of measured data, the modulus of elasticity, E_c, for concretes with unit weights between 0.090 and 0.155 kcf and specified compressive strengths up to 15.0 ksi may be taken as:

$$E_c = 33{,}000\, K_1\, w_c^{1.5} \sqrt{f'_c} \qquad (5.4.2.4\text{-}1)$$

where:

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = unit weight of concrete (kcf); refer to Table 3.5.1-1 or Article C5.4.2.4

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

C5.4.2.4

See commentary for specified strength in Article 5.4.2.1.

For normal weight concrete with w_c = 0.145 kcf, E_c may be taken as:

$$E_c = 1{,}820\sqrt{f'_c} \qquad (C5.4.2.4\text{-}1)$$

Test data show that the modulus of elasticity of concrete is influenced by the stiffness of the aggregate. The factor K_1 is included to allow the calculated modulus to be adjusted for different types of aggregate and local materials. Unless a value has been determined by physical tests, K_1 should be taken as 1.0. Use of a measured K_1 factor permits a more accurate prediction of modulus of elasticity and other values that utilize it.

5.4.2.5—Poisson's Ratio

Unless determined by physical tests, Poisson's ratio may be assumed as 0.2. For components expected to be subject to cracking, the effect of Poisson's ratio may be neglected.

C5.4.2.5

This is a ratio between the lateral and axial strains of an axially and/or flexurally loaded structural element.

5.4.2.6—Modulus of Rupture

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

- For normal-weight concrete:
  - When used to calculate the cracking moment of a member in Articles 5.7.3.4, 5.7.3.6.2, and 6.10.4.2.1................ $0.24\sqrt{f'_c}$
  - When used to calculate the cracking moment of a member in Article 5.7.3.3.2 .... $0.37\sqrt{f'_c}$
  - When used to calculate the cracking moment of a member in Article 5.8.3.4.3 .... $0.20\sqrt{f'_c}$
- For lightweight concrete:
  - For sand-lightweight concrete ...... $0.20\sqrt{f'_c}$
  - For all-lightweight concrete .... $0.17\sqrt{f'_c}$

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

C5.4.2.6

Data show that most modulus of rupture values are between $0.24\sqrt{f'_c}$ and $0.37\sqrt{f'_c}$ (ACI 1992; Walker and Bloem 1960; Khan, Cook, and Mitchell 1996). It is appropriate to use the lower bound value when considering service load cracking. The purpose of the minimum reinforcement in Article 5.7.3.3.2 is to assure that the nominal moment capacity of the member is at least 20 percent greater than the cracking moment. Since the actual modulus of rupture could be as much as 50 percent greater than $0.24\sqrt{f'_c}$ the 20 percent margin of safety could be lost. Using an upper bound is more appropriate in this situation.

The properties of higher strength concretes are particularly sensitive to the constitutive materials. If test results are to be used in design, it is imperative that tests be made using concrete with not only the same mix proportions, but also the same materials as the concrete used in the structure.

The given values may be unconservative for tensile cracking caused by restrained shrinkage, anchor zone splitting, and other such tensile forces caused by effects other than flexure. The direct tensile strength stress should be used for these cases.

5.4.2.7—Tensile Strength

Direct tensile strength may be determined by either using ASTM C900, or the split tensile strength method in accordance with AASHTO T 198 (ASTM C496).

C5.4.2.7

For normal-weight concrete with specified compressive strengths up to 10 ksi, the direct tensile strength may be estimated as $f_r = 0.23\sqrt{f'_c}$.

5.4.3—Reinforcing Steel

5.4.3.1—General

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in Article 9.2 of the *AASHTO LRFD Bridge Construction Specifications*.

C5.4.3.1

Reinforcement shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric.

The nominal yield strength shall be the minimum as specified for the grade of steel selected, except that yield strengths in excess of 75.0 ksi shall not be used for design purposes. The yield strength or grade of the bars or wires shall be shown in the contract documents. Bars with yield strengths less than 60.0 ksi shall be used only with the approval of the Owner.

Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement," should be specified.

ASTM A706 reinforcement should be considered for seismic design because of the greater quality control by which unanticipated overstrength is limited.

5.4.3.2—Modulus of Elasticity

The modulus of elasticity, E_s, of steel reinforcing shall be assumed as 29,000 ksi.

5.4.3.3—Special Applications

Reinforcement to be welded shall be indicated in the contract documents, and the welding procedure to be used shall be specified.

Reinforcement conforming to ASTM A1035/A1035M may only be used as top and bottom flexural reinforcement in the longitudinal and transverse directions of bridge decks in Seismic Zones 1 and 2.

C5.4.3.3

In 2004, ASTM published A1035/A1035M, *Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement*. This reinforcement offers the potential for corrosion resistance.

Epoxy-coated reinforcing steel provides a physical barrier to inhibit corrosion of the steel in the presence of chlorides. The handling, placement, and repair of epoxy-coated reinforcing steel requires significant care and attention.

Reinforcement conforming to ASTM A1035/A1035M has a specified minimum yield strength of 100 ksi determined by the 0.2 percent offset method, a specified minimum tensile strength of 150 ksi, and a specified minimum elongation of six or seven percent depending on bar size. There is also a requirement that the stress corresponding to a tensile strain of 0.0035 shall be a minimum of 80 ksi. The reinforcement has a non-linear stress-strain relationship. Article 5.4.3.1 of the Design Specifications states that yield strengths in excess of 75.0 ksi shall not be used for design purposes. Consequently, design is based on a stress of 75.0 ksi, but the actual strength is at least twice that value. This has lead to concerns about the applicability of the existing specifications with ASTM A1035 reinforcement. Consequently, it is proposed that initial usage of the reinforcement be restricted to top and bottom flexural reinforcement in the transverse and longitudinal directions of bridge decks in Seismic Zones 1 and 2.

5.4.4—Prestressing Steel

5.4.4.1—General

Uncoated, stress-relieved or low-relaxation, seven-wire strand, or uncoated plain or deformed, high-strength bars, shall conform to the following materials standards, as specified for use in *AASHTO LRFD Bridge Construction Specifications*:

- AASHTO M 203/M 203M (ASTM A416/A416M), or
- AASHTO M 275/M 275M (ASTM A722/A722M).

Tensile and yield strengths for these steels may be taken as specified in Table 5.4.4.1-1.

C5.4.4.1

Low relaxation strand shall be regarded as the standard type. Stress-relieved (normal relaxation) strand will not be furnished unless specifically ordered, or by arrangement between purchaser and supplier.

Table 5.4.4.1-1—Properties of Prestressing Strand and Bar

Material	Grade or Type	Diameter (in.)	Tensile Strength, f_{pu} (ksi)	Yield Strength, f_{py} (ksi)
Strand	250 ksi 270 ksi	1/4 to 0.6 3/8 to 0.6	250 270	85% of f_{pu}, except 90% of f_{pu} for low-relaxation strand
Bar	Type 1, Plain Type 2, Deformed	3/4 to 1-3/8 5/8 to 1-3/8	150 150	85% of f_{pu} 80% of f_{pu}

Where complete prestressing details are included in the contract documents, the size and grade or type of steel shall be shown. If the plans indicate only the prestressing forces and locations of application, the choice of size and type of steel shall be left to the Contractor, subject to the Engineer's approval.

5.4.4.2—Modulus of Elasticity

If more precise data are not available, the modulus of elasticity for prestressing steels, based on nominal cross-sectional area, may be taken as:

for strand: E_p = 28,500 ksi, and
for bar: E_p = 30,000 ksi

C5.4.4.2

The suggested modulus of elasticity of 28,500 ksi for strands is based on recent statistical data. This value is higher than that previously assumed because of the slightly different characteristics and the near universal use of low-relaxation strands.

As shown in Figure C5.4.4.2-1, there is no sharp break in the curves to indicate a distinct elastic limit or yield point. Arbitrary methods of establishing yield strength, based on a specific set or measured strain, are generally used. The 0.2 percent offset and the one percent extension methods are the most common.

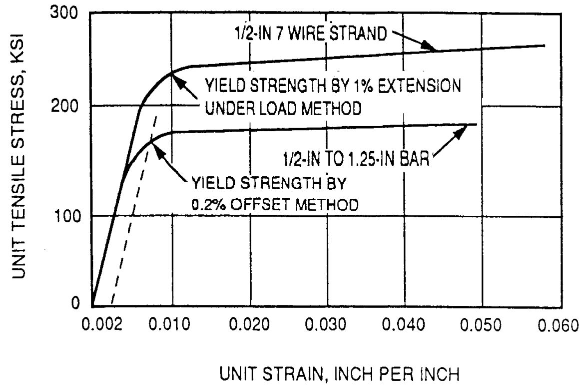


Figure C5.4.4.2-1—Typical Stress-Strain Curve for Prestressing Steels

5.4.5—Post-Tensioning Anchorages and Couplers

Anchorages and tendon couplers shall conform to the requirements of Article 10.3.2 of *AASHTO LRFD Bridge Construction Specifications*.

Corrosion protection shall be provided for tendons, anchorages, end fittings, and couplers.

C5.4.5

Complete details for qualification testing of anchorages and couplers are included in Article 10.3.2 of *AASHTO LRFD Bridge Construction Specifications*.

Characteristics of anchorages and couplers related to design and detailing are summarized below from *AASHTO LRFD Bridge Construction Specifications*:

- Anchorages and couplers are to develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding the anchorage set movement assumed for the design. Unbonded systems are to also pass a dynamic loading test.
- Couplers are not to be used at points of sharp tendon curvature.
- Couplers are to be used only at locations shown on the contract documents or approved by the Engineer.
- Couplers are to be enclosed in housings long enough to permit the necessary movements.
- Where bonded anchorages or couplers are located at sections that are critical at strength limit state, the strength required of the bonded tendons is not to exceed the resistance of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.
- Bearing stresses on concrete under anchorage distribution plates are not to exceed specified limits.
- Unless waived by the Engineer because of suitable previous tests and/or experience, qualification of anchorages and couplers are to be verified by testing.

5.4.6—Ducts

5.4.6.1—General

Ducts for tendons shall be rigid or semirigid either galvanized ferrous metal or polyethylene, or they shall be formed in the concrete with removable cores.

The radius of curvature of tendon ducts shall not be less than 20.0 ft, except in the anchorage areas where 12.0 ft may be permitted.

Polyethylene ducts shall not be used when the radius of curvature of the tendon is less than 30.0 ft.

Where polyethylene ducts are used and the tendons are to be bonded, the bonding characteristics of polyethylene ducts to the concrete and the grout should be investigated.

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

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

C5.4.6.1

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

Polyethylene duct should not be used on radii under 30.0 ft because of its lower resistance to abrasion during pulling-through and stressing tendons.

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

5.4.6.2—Size of Ducts

The inside diameter of ducts shall be at least 0.25 in. larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct shall be at least 2.0 times the net area of the prestressing steel with one exception where tendons are to be placed by the pull-through method, the duct area shall be at least 2.5 times the net area of the prestressing steel.

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

C5.4.6.2

The pull-through method of tendon placement is usually employed by contractors where tendons exceed 400 ft in length.

5.4.6.3—Ducts at Deviation Saddles

Ducts at deviation saddles shall be galvanized steel pipe conforming to the requirements of ASTM A53, Type E, Grade B. The nominal wall thickness of the pipe shall be not less than 0.125 in.

5.5—LIMIT STATES

5.5.1—General

Structural components shall be proportioned to satisfy the requirements at all appropriate service, fatigue, strength, and extreme event limit states.

Prestressed and partially prestressed concrete structural components shall be investigated for stresses and deformations for each stage that may be critical during construction, stressing, handling, transportation, and erection as well as during the service life of the structure of which they are part.

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

5.5.2—Service Limit State

Actions to be considered at the service limit state shall be cracking, deformations, and concrete stresses, as specified in Articles 5.7.3.4, 5.7.3.6, and 5.9.4, respectively.

The cracking stress shall be taken as the modulus of rupture specified in Article 5.4.2.6.

5.5.3—Fatigue Limit State

5.5.3.1—General

Fatigue need not be investigated for concrete deck slabs in multigirder applications or reinforced-concrete box culverts.

In regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.

Fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in Table 5.9.4.2.2-1.

For fatigue considerations, concrete members shall satisfy:

$$\gamma\left(\Delta f\right) \leq \left(\Delta F\right)_{TH} \quad (5.5.3.1\text{-}1)$$

where:

γ = load factor specified in Table 3.4.1-1 for the Fatigue I load combination

Δf = force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold, as specified in Article 5.5.3.2, 5.5.3.3, or 5.5.3.4, as appropriate (ksi)

C5.5.3.1

Stresses measured in concrete deck slabs of bridges in service are far below infinite fatigue life, most probably due to internal arching action; see Article C9.7.2.

Fatigue evaluation for reinforced-concrete box culverts showed that the live load stresses in the reinforcement due to Fatigue I load combination did not reduce the member resistance at the strength limit state.

In determining the need to investigate fatigue, Table 3.4.1-1 specifies a load factor of 1.50 on the live load force effect resulting from the fatigue truck for the Fatigue I load combination. This factored live load force effect represents the greatest fatigue stress that the bridge will experience during its life.

Fatigue limit state load factor, girder distribution factors, and dynamic allowance cause fatigue limit state stress to be considerably less than the corresponding value determined from Service Limit State III. For fully prestressed components, the net concrete stress is usually significantly less than the concrete tensile stress limit specified in Table 5.9.4.2.2-1. Therefore, the calculated flexural stresses are significantly reduced. For this situation, the calculated steel stress range, which is equal to the modular ratio times the concrete stress range, is almost always less than the steel fatigue stress range limit specified in Article 5.5.3.3.

For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses.

The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads and prestress, and the Fatigue I load combination is tensile and exceeds $0.095\sqrt{f'_c}$.

5.5.3.2—Reinforcing Bars

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight reinforcement and welded wire reinforcement without a cross weld in the high-stress region shall be taken as:

$$\left(\Delta F\right)_{TH} = 24 - 0.33 f_{\min} \qquad (5.5.3.2\text{-}1)$$

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight welded wire reinforcement with a cross weld in the high-stress region shall be taken as:

$$\left(\Delta F\right)_{TH} = 16 - 0.33 f_{\min} \qquad (5.5.3.2\text{-}2)$$

where:

f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

The definition of the high-stress region for application of Eqs. 5.5.3.2-1 and 5.5.3.2-2 for flexural reinforcement shall be taken as one-third of the span on each side of the section of maximum moment.

C5.5.3.2

Bends in primary reinforcement should be avoided in regions of high stress range.

Structural welded wire reinforcement has been increasingly used in bridge applications in recent years, especially as auxiliary reinforcement in bridge I- and box beams and as primary reinforcement in slabs. Design for shear has traditionally not included a fatigue check of the reinforcement as the member is expected to be uncracked under service conditions and the stress range in steel minimal. The stress range for steel bars has existed in previous editions. It is based on Hansen et al. (1976). The simplified form in this edition replaces the (r/h) parameter with the default value 0.3 recommended by Hansen et al. Inclusion of limits for WWR is based on recent studies by Hawkins et al. (1971, 1987) and Tadros et al. (2004).

Since the fatigue provisions were developed based primarily on ASTM A615 steel reinforcement, their applicability to other types of reinforcement is largely unknown. Consequently, a cautionary note is added to the Commentary.

5.5.3.3—Prestressing Tendons

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for prestressing tendons shall be taken as:

- 18.0 ksi for radii of curvature in excess of 30.0 ft, and
- 10.0 ksi for radii of curvature not exceeding 12.0 ft.

A linear interpolation may be used for radii between 12.0 and 30.0 ft.

C5.5.3.3

Where the radius of curvature is less than shown, or metal-to-metal fretting caused by prestressing tendons rubbing on hold-downs or deviations is apt to be a consideration, it will be necessary to consult the literature for more complete presentations that will allow the increased bending stress in the case of sharp curvature, or fretting, to be accounted for in the development of permissible fatigue stress ranges. Metal-to-metal fretting is not normally expected to be a concern in conventional pretensioned beams.

5.5.3.4—Welded or Mechanical Splices of Reinforcement

For welded or mechanical connections that are subject to repetitive loads, the constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, shall be as given in Table 5.5.3.4-1.

Table 5.5.3.4-1—Constant-Amplitude Fatigue Threshold of Splices

Type of Splice	$(\Delta F)_{TH}$ for greater than 1,000,000 cycles
Grout-filled sleeve, with or without epoxy coated bar	18 ksi
Cold-swaged coupling sleeves without threaded ends and with or without epoxy-coated bar; Integrally-forged coupler with upset NC threads; Steel sleeve with a wedge; One-piece taper-threaded coupler; and Single V-groove direct butt weld	12 ksi
All other types of splices	4 ksi

Where the total cycles of loading, N, as specified in Eq. 6.6.1.2.5-2, are less than one million, $(\Delta F)_{TH}$ in Table 5.5.3.4-1 may be increased by the quantity 24 $(6-\log N)$ ksi to a total not greater than the value given by Eq. 5.5.3.2-1 in Article 5.5.3.2. Higher values of $(\Delta F)_{TH}$, up to the value given by Eq. 5.5.3.2-1, may be used if justified by fatigue test data on splices that are the same as those that will be placed in service.

Welded or mechanical splices shall not be used with ASTM A1035/A1035M reinforcement.

C5.5.3.4

Review of the available fatigue and static test data indicates that any splice, that develops 125 percent of the yield strength of the bar will sustain one million cycles of a 4 ksi constant amplitude stress range. This lower limit is a close lower bound for the splice fatigue data obtained in NCHRP Project 10-35, and it also agrees well with the limit of 4.5 ksi for Category E from the provisions for fatigue of structural steel weldments. The strength requirements of Articles 5.11.5.2.2 and 5.11.5.2.3 also will generally ensure that a welded splice or mechanical connector will also meet certain minimum requirements for fabrication and installation, such as sound welding and proper dimensional tolerances. Splices that do not meet these requirements for fabrication and installation may have reduced fatigue performance. Further, splices designed to the lesser force requirements of Article 5.11.5.3.2 may not have the same fatigue performance as splices designed for the greater force requirement. Consequently, the minimum strength requirement indirectly provides for a minimum fatigue performance.

It was found in NCHRP Project 10-35 that there is substantial variation in the fatigue performance of different types of welds and connectors. However, all types of splices appeared to exhibit a constant amplitude fatigue limit for repetitive loading exceeding about one million cycles. The stress ranges for over one million cycles of loading given in Table 5.5.3.4-1 are based on statistical tolerance limits to constant amplitude staircase test data, such that there is a 95 percent level of confidence that 95 percent of the data would exceed the given values for five million cycles of loading. These values may, therefore, be regarded as a fatigue limit below which fatigue damage is unlikely to occur during the design lifetime of the structure. This is the same basis used to establish the fatigue design provisions for unspliced reinforcing bars in Article 5.5.3.2, which is based on fatigue tests reported in NCHRP Report 164, *Fatigue Strength of High-Yield Reinforcing Bars*.

5.5.4—Strength Limit State

5.5.4.1—General

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

Factored resistance shall be the product of nominal resistance as determined in accordance with the applicable provisions of Articles 5.6, 5.7, 5.8, 5.9, 5.10, 5.13, and 5.14, unless another limit state is specifically identified, and the resistance factor is as specified in Article 5.5.4.2.

C5.5.4.1

Additional resistance factors are specified in Article 12.5.5 for buried pipes and box structures made of concrete.

5.5.4.2—Resistance Factors

5.5.4.2.1—Conventional Construction

Resistance factor ϕ shall be taken as:

- For tension-controlled reinforced concrete sections as defined in Article 5.7.2.1 .... 0.90
- For tension-controlled prestressed concrete sections as defined in Article 5.7.2.1 .... 1.00
- For shear and torsion:
  - normal weight concrete........................ 0.90
  - lightweight concrete............................ 0.70
  - For compression-controlled sections with spirals or ties, as defined in Article 5.7.2.1, except as specified in Articles 5.10.11.3 and 5.10.11.4.1b for Seismic Zones 2, 3, and 4 at the extreme event limit state... 0.75
- For bearing on concrete...................................... 0.70
- For compression in strut-and-tie models............ 0.70

C5.5.4.2.1

In applying the resistance factors for tension-controlled and compression-controlled sections, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

In editions of and interims to the LRFD Specifications prior to 2005, the provisions specified the magnitude of the resistance factor for cases of axial load or flexure, or both, it terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross-section, at nominal strength. The background and basis for these provisions are given in Mast (1992) and ACI 318-02.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

For sections subjected to axial load with flexure, factored resistances are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in Article 5.7.2.1 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Figure C5.5.4.2.1-1. The concept of net tensile strain ε_t is discussed in Article C5.7.2.1. Classifying sections as tension-controlled, transition or compression-controlled, and linearly varying the resistance factor in the transition zone between reasonable values for the two extremes, provides a rational approach for determining ϕ and limiting the capacity of over-reinforced sections.

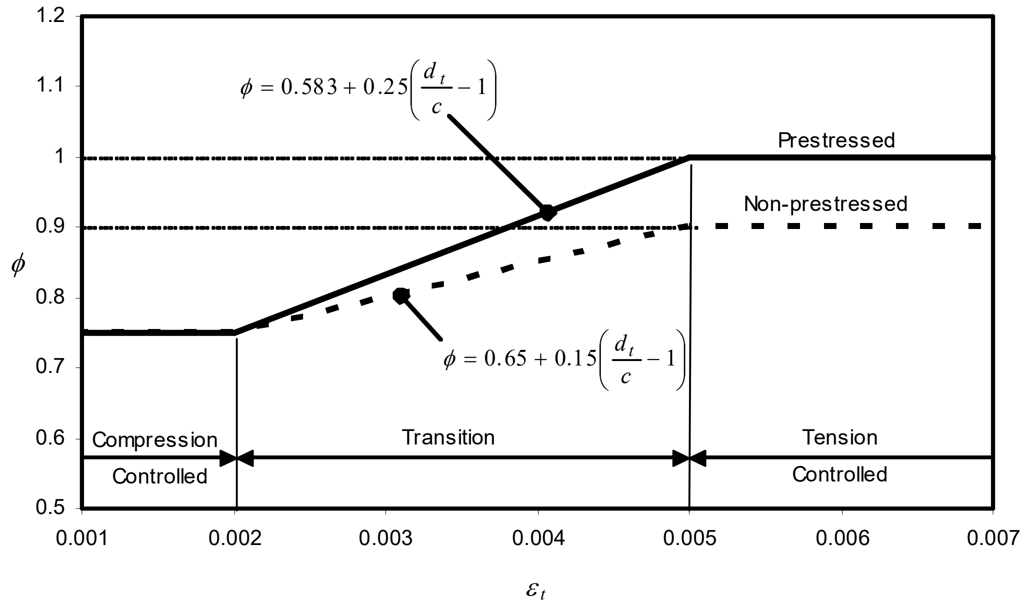


Figure C5.5.4.2.1-1—Variation of ϕ with Net Tensile Strain ε_t and d_t/c for Grade 60 Reinforcement and for Prestressing Steel

- For compression in anchorage zones:
  - normal weight concrete 0.80
  - lightweight concrete 0.65
- For tension in steel in anchorage zones 1.00
- For resistance during pile driving 1.00

For sections in which the net tensile strain in the extreme tension steel at nominal resistance is between the limits for compression-controlled and tension-controlled sections, ϕ may be linearly increased from 0.75 to that for tension-controlled sections as the net tensile strain in the extreme tension steel increases from the compression-controlled strain limit to 0.005.

This variation ϕ may be computed for prestressed members such that:

$$0.75 \le \phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \le 1.0 \qquad (5.5.4.2.1\text{-}1)$$

and for nonprestressed members such that:

$$0.75 \le \phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \le 0.9 \qquad (5.5.4.2.1\text{-}2)$$

where:

c = distance from the extreme compression fiber to the neutral axis (in.)

d_t = distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.)

The ϕ-factor of 0.8 for normal weight concrete reflects the importance of the anchorage zone, the brittle failure mode for compression struts in the anchorage zone, and the relatively wide scatter of results of experimental anchorage zone studies. The ϕ-factor of 0.65 for lightweight concrete reflects its often lower tensile strength and is based on the multipliers used in ACI 318-89, Section 11.2.1.2.

The design of intermediate anchorages, anchorages, diaphragms, and multiple slab anchorages are addressed in Breen et al. (1994).

For tension-controlled partially prestressed components in flexure, the values of ϕ may be taken as:

$$\phi = 0.90 + 0.10(PPR) \qquad (5.5.4.2.1\text{-}3)$$

in which:

$$PPR = \frac{A_{ps} f_{py}}{A_{ps} f_{py} + A_s f_y} \qquad (5.5.4.2.1\text{-}4)$$

where:

PPR = partial prestress ratio

A_s = area of nonprestressed tension reinforcement (in.^{2})

A_{ps} = area of prestressing steel (in.^{2})

f_y = specified yield strength of reinforcing bars (ksi)

f_{py} = yield strength of prestressing steel (ksi)

Resistance factors shall not be applied to the development and splice lengths of reinforcement as specified in Article 5.11.

5.5.4.2.2—Segmental Construction

Resistance factors for the strength limit state shall be taken as provided in Table 5.5.4.2.2-1 for the conditions indicated and in Article 5.5.4.2.1 for conditions not covered in Table 5.5.4.2.2-1.

In selecting resistance factors for flexure, ϕ_f, and shear and torsion, ϕ_v, the degree of bonding of the post-tensioning system shall be considered. In order for a tendon to be considered as fully bonded at a section, it should be fully developed at that section for a development length not less than that required by Article 5.11.4. Shorter embedment lengths may be permitted if demonstrated by full-size tests and approved by the Engineer.

Where the post-tensioning is a combination of fully bonded tendons and unbonded or partially bonded tendons, the resistance factor at any section shall be based upon the bonding conditions for the tendons providing the majority of the prestressing force at the section.

Joints between precast units shall be either cast-in-place closures or match cast and epoxied joints.

C5.5.4.2.2

Comprehensive tests of a large continuous three-span model of a twin-cell box girder bridge built from precast segments with fully bonded internal tendons and epoxy joints indicated that cracking was well distributed through the segment lengths. No epoxy joint opened at failure, and the load deflection curve was identical to that calculated for a monolithic specimen. The complete ultimate strength of the tendons was developed at failure. The model had substantial ductility and full development of calculated deflection at failure. Flexural cracking concentrated at joints and final failure came when a central joint opened widely and crushing occurred at the top of the joint. Based on the observation of this limited test data, a maximum ϕ of 0.95 was selected.

Table 5.5.4.2.2-1—Resistance Factor for Joints in Segmental Construction

	ϕ_f Flexure	ϕ_v Shear
Normal Weight Concrete		
Fully Bonded Tendons	0.95	0.90
Unbonded or Partially Bonded Tendons	0.90	0.85
Sand-Lightweight Concrete		
Fully Bonded Tendons	0.90	0.70
Unbonded or Partially Bonded Tendons	0.85	0.65

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

A modified resistance factor for columns in Seismic Zones 2, 3, and 4 shall be taken as specified in Articles 5.10.11.3 and 5.10.11.4.1b.

5.5.4.3—Stability

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

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

5.5.5—Extreme Event Limit State

The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1, as may be appropriate to its site and use.

5.6—DESIGN CONSIDERATIONS

5.6.1—General

Components and connections shall be designed to resist load combinations, as specified in Section 3, at all stages during the life of the structure, including those during construction. Load factors shall be as specified in Section 3.

As specified in Section 4, equilibrium and strain compatibility shall be maintained in the analysis.

C5.6.1

This Article reflects the AASHTO *Standard Specifications for Highway Bridges* (1996), the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (1989) and the *Ontario Highway Bridge Design Code* (1991).

5.6.2—Effects of Imposed Deformation

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

C5.6.2

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

5.6.3—Strut-and-Tie Model

5.6.3.1—General

Strut-and-tie models may be used to determine internal force effects near supports and the points of application of concentrated loads at strength and extreme event limit states.

The strut-and-tie model should be considered for the design of deep footings and pile caps or other situations in which the distance between the centers of applied load and the supporting reactions is less than about twice the member thickness.

If the strut-and-tie model is selected for structural analysis, Articles 5.6.3.2 through 5.6.3.6 shall apply.

C5.6.3.1

Where the conventional methods of strength of materials are not applicable because of nonlinear strain distribution, the strut-and-tie modeling may provide a convenient way of approximating load paths and force effects in the structure. In fact, the load paths may be visualized and the geometry of concrete and steel selected to implement the load path.

The strut-and-tie model is new to these Specifications. More detailed information on this method is given by Schlaich et al. (1987) and Collins and Mitchell (1991).

Traditional section-by-section design is based on the assumption that the reinforcement required at a particular section depends only on the separated values of the factored section force effects V_u, M_u, and T_u and does not consider the mechanical interaction among these force effects as the strut-and-tie model does. The traditional method further assumes that shear distribution remains uniform and that the longitudinal strains will vary linearly over the depth of the beam.

For members such as the deep beam shown in Figure C5.6.3.2-1, these assumptions are not valid. The shear stresses on a section just to the right of a support will be concentrated near the bottom face. The behavior of a component, such as the deep beam, can be predicted more accurately if the flow of forces through the complete structure is studied. Instead of determining V_u and M_u at different sections along the span, the flow of compressive stresses going from the loads P to the supports and the required tension force to be developed between the supports should be established.

For additional applications of the strut-and-tie model see Articles 5.10.9.4, 5.13.2.3, and 5.13.2.4.1.

5.6.3.2—Structural Modeling

The structure and a component or region, thereof, may be modeled as an assembly of steel tension ties and concrete compressive struts interconnected at nodes to form a truss capable of carrying all the applied loads to the supports. The required widths of compression struts and tension ties shall be considered in determining the geometry of the truss.

The factored resistance, P_r, of struts and ties shall be taken as that of axially loaded components:

$$P_r = \phi P_n \qquad (5.6.3.2\text{-}1)$$

where:

C5.6.3.2

Cracked reinforced concrete carries load principally by compressive stresses in the concrete and tensile stresses in the reinforcement. After significant cracking has occurred, the principal compressive stress trajectories in the concrete tend toward straight lines and hence can be approximated by straight compressive struts. Tension ties are used to model the principal reinforcement.

A strut-and-tie truss model is shown in Figures C5.6.3.2-1 and C5.6.3.2-2. The zones of high unidirectional compressive stress in the concrete are represented by compressive struts. The regions of the concrete subjected to multidirectional stresses, where the struts and ties meet the joints of the truss, are represented by nodal zones.

P_n = nominal resistance of strut or tie (kip)

ϕ = resistance factor for tension or compression specified in Article 5.5.4.2, as appropriate

Because of the significant transverse dimensions of the struts and ties, a "truss joint" becomes a "nodal zone" with finite dimensions. Establishing the geometry of the truss usually involves trial and error in which member sizes are assumed, the truss geometry is established, member forces are determined, and the assumed member sizes are verified.

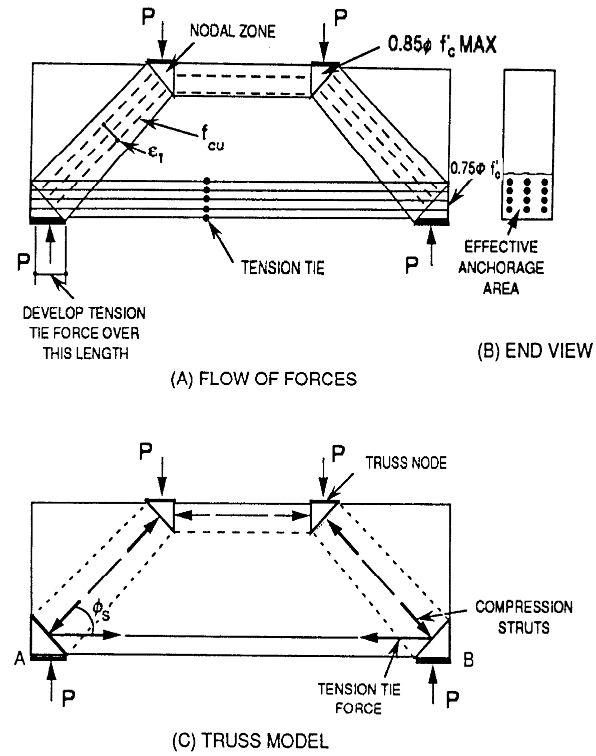


Figure C5.6.3.2-1—Strut-and-Tie Model for a Deep Beam

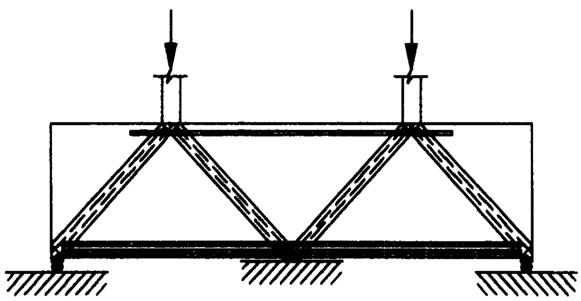


Figure C5.6.3.2-2—Strut-and-Tie Model for Continuous Deep Beam

5.6.3.3—Proportioning of Compressive Struts

5.6.3.3.1—Strength of Unreinforced Strut

The nominal resistance of an unreinforced compressive strut shall be taken as:

$$P_n = f_{cu} A_{cs} \quad (5.6.3.3.1\text{-}1)$$

where:

P_n = nominal resistance of a compressive strut (kip)

f_{cu} = limiting compressive stress as specified in Article 5.6.3.3.3 (ksi)

A_{cs} = effective cross-sectional area of strut as specified in Article 5.6.3.3.2 (in.^{2})

5.6.3.3.2—Effective Cross-Sectional Area of Strut

The value of A_{cs} shall be determined by considering both the available concrete area and the anchorage conditions at the ends of the strut, as shown in Figure 5.6.3.3.2-1.

When a strut is anchored by reinforcement, the effective concrete area may be considered to extend a distance of up to six bar diameters from the anchored bar, as shown in Figure 5.6.3.3.2-1 (a).

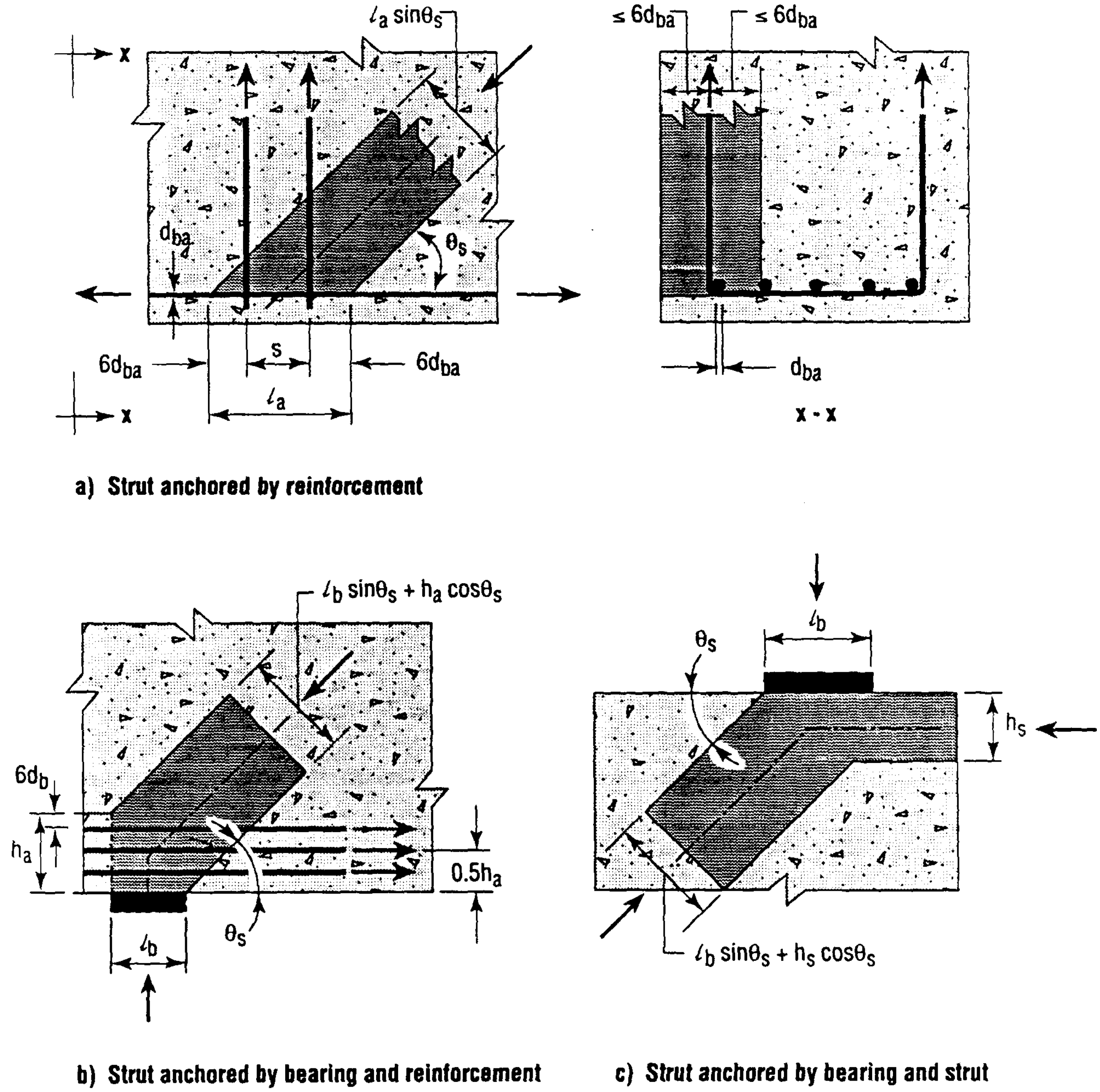


Figure 5.6.3.3.2-1—Influence of Anchorage Conditions on Effective Cross-Sectional Area of Strut

5.6.3.3.3—Limiting Compressive Stress in Strut

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

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

in which:

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

C5.6.3.3.3

If the concrete is not subjected to principal tensile strains greater than about 0.002, it can resist a compressive stress of 0.85 f'_c. This will be the limit for regions of the struts not crossed by or joined to tension ties. The reinforcing bars of a tension tie are bonded to the surrounding concrete. If the reinforcing bars are to yield in tension, there should be significant tensile strains imposed on the concrete. As these tensile strains increase, f_{cu} decreases.

where:

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

ε_s = the tensile strain in the concrete in the direction of the tension tie (in./in.)

f'_c = specified compressive strength (ksi)

The expression for ε_1 is based on the assumption that the principal compressive strain ε_2 in the direction of the strut equals 0.002 and that the tensile strain in the direction of the tension tie equals ε_s. As the angle between the strut-and-tie decreases, ε_1 increases and hence f_{cu} decreases. In the limit, no compressive stresses would be permitted in a strut that is superimposed on a tension tie, i.e., $\alpha_s = 0$, a situation that violates compatibility.

For a tension tie consisting of reinforcing bars, ε_s can be taken as the tensile strain due to factored loads in the reinforcing bars. For a tension tie consisting of prestressing, ε_s can be taken as 0.0 until the precompression of the concrete is overcome. For higher stresses, ε_s would equal $(f_{ps} - f_{pe})/E_p$.

If the strain ε_s varies over the width of the strut, it is appropriate to use the value at the centerline of the strut.

5.6.3.3.4—Reinforced Strut

If the compressive strut contains reinforcement that is parallel to the strut and detailed to develop its yield stress in compression, the nominal resistance of the strut shall be taken as:

$$P_n = f_{cu} A_{cs} + f_y A_{ss} \tag{5.6.3.3.4-1}$$

where:

A_{ss} = area of reinforcement in the strut (in.^{2})

5.6.3.4—Proportioning of Tension Ties

5.6.3.4.1—Strength of Tie

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

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

$$P_n = f_y A_{st} + A_{ps} \left[f_{pe} + f_y \right] \tag{5.6.3.4.1-1}$$

where:

A_{st} = total area of longitudinal mild steel reinforcement in the tie (in.^{2})

A_{ps} = area of prestressing steel (in.^{2})

f_y = yield strength of mild steel longitudinal reinforcement (ksi)

f_{pe} = stress in prestressing steel due to prestress after losses (ksi)

C5.6.3.4.1

The second term of the equation for P_n is intended to ensure that the prestressing steel does not reach its yield point, thus a measure of control over unlimited cracking is maintained. It does, however, acknowledge that the stress in the prestressing elements will be increased due to the strain that will cause the concrete to crack. The increase in stress corresponding to this action is arbitrarily limited to the same increase in stress that the mild steel will undergo. If there is no mild steel, f_y may be taken as 60.0 ksi for the second term of the equation.

5.6.3.4.2—Anchorage of Tie

The tension tie reinforcement shall be anchored to transfer the tension force therein to the node regions of the truss in accordance with the requirements for development of reinforcement as specified in Article 5.11.

5.6.3.5—Proportioning of Node Regions

Unless confining reinforcement is provided and its effect is supported by analysis or experimentation, the concrete compressive stress in the node regions of the strut shall not exceed:

- For node regions bounded by compressive struts and bearing areas: $0.85\phi f'_c$
- For node regions anchoring a one-direction tension tie: $0.75\phi f'_c$
- For node regions anchoring tension ties in more than one direction: $0.65\phi f'_c$

where:

ϕ = the resistance factor for bearing on concrete as specified in Article 5.5.4.2.

The tension tie reinforcement shall be uniformly distributed over an effective area of concrete at least equal to the tension tie force divided by the stress limits specified herein.

In addition to satisfying strength criteria for compression struts and tension ties, the node regions shall be designed to comply with the stress and anchorage limits specified in Articles 5.6.3.4.1 and 5.6.3.4.2.

The bearing stress on the node region produced by concentrated loads or reaction forces shall satisfy the requirements specified in Article 5.7.5.

C5.6.3.5

The limits in concrete compressive stresses in nodal zones are related to the degree of expected confinement in these zones provided by the concrete in compression.

The stresses in the nodal zones can be reduced by increasing the:

- Size of the bearing plates,
- Dimensions of the compressive struts, and
- Dimensions of the tension ties.

The reduced stress limits on nodes anchoring tension ties are based on the detrimental effect of the tensile straining caused by these ties. If the ties consist of post-tensioned tendons and the stress in the concrete does not need to be above f_{pc}, no tensile straining of the nodal zone will be required. For this case, the $0.85\phi f'_c$ limit is appropriate.

5.6.3.6—Crack Control Reinforcement

Structures and components or regions thereof, except for slabs and footings, which have been designed in accordance with the provisions of Article 5.6.3, shall contain orthogonal grids of reinforcing bars. The spacing of the bars in these grids shall not exceed the smaller of $d/4$ and 12.0 in.

The reinforcement in the vertical and horizontal direction shall satisfy the following:

$$\frac{A_v}{b_w s_v} \geq 0.003 \qquad (5.6.3.6\text{-}1)$$

$$\frac{A_h}{b_w s_h} \geq 0.003 \qquad (5.6.3.6\text{-}2)$$

C5.6.3.6

This reinforcement is intended to control the width of cracks and to ensure a minimum ductility for the member so that, if required, significant redistribution of internal stresses is possible.

The total horizontal reinforcement can be calculated as 0.003 times the effective area of the strut denoted by the shaded portion of the cross-section in Figure C5.6.3.6-1. For thinner members, this crack control reinforcement will consist of two grids of reinforcing bars, one near each face. For thicker members, multiple grids of reinforcement through the thickness may be required in order to achieve a practical layout.

where:

A_h = total area of horizontal crack control reinforcement within spacing s_h, respectively (in.^{2})

A_v = total area of vertical crack control reinforcement within spacing s_v, respectively (in.^{2})

b_w = width of member's web (in.)

s_v, s_h = spacing of vertical and horizontal crack control reinforcement, respectively (in.)

Crack control reinforcement shall be distributed evenly within the strut area.

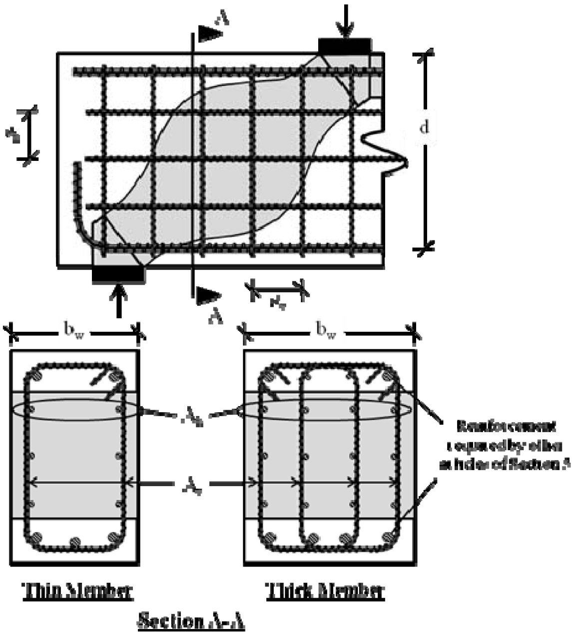


Figure C5.6.3.6-1—Distribution of Crack Control Reinforcement in Compression Strut

5.7—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.1—Assumptions for Service and Fatigue Limit States

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

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

C5.7.1

Prestressing is treated as part of resistance, except for anchorages and similar details, where the design is totally a function of the tendon force and for which a load factor is specified in Article 3.4.3. External reactions caused by prestressing induce force effects that normally are taken to be part of the loads side of Eq. 1.3.2.1-1. This represents a philosophical dichotomy. In lieu of more precise information, in these Specifications the load factor for these induced force effects should be taken as that for the permanent loads.

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

5.7.2—Assumptions for Strength and Extreme Event Limit States

5.7.2.1—General

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

- In components with fully bonded reinforcement or prestressing, or in the bonded length of locally debonded or shielded strands, strain is directly proportional to the distance from the neutral axis, except for deep members that shall satisfy the requirements of Article 5.13.2, and for other disturbed regions.
- In components with fully unbonded or partially unbonded prestressing tendons, i.e., not locally debonded or shielded strands, the difference in strain between the tendons and the concrete section and the effect of deflections on tendon geometry are included in the determination of the stress in the tendons.
- If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003.
- If the concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized if verified. Calculation of the factored resistance shall consider that the concrete cover may be lost at strains compatible with those in the confined concrete core.
- Except for the strut-and-tie model, the stress in the reinforcement is based on a stress-strain curve representative of the steel or on an approved mathematical representation, including development of reinforcing and prestressing elements and transfer of pretensioning.
- The tensile strength of the concrete is neglected.
- The concrete compressive stress-strain distribution is assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the test results.
- The development of reinforcing and prestressing elements and transfer of pretensioning are considered.

C5.7.2.1

The first paragraph of C5.7.1 applies.

Research by Bae and Bayrak (2003) has shown that, for well-confined High Strength Concrete (HSC) columns, the concrete cover may be lost at maximum useable strains at the extreme concrete compression fiber as low as 0.0022. The heavy confinement steel causes a weak plane between the concrete core and cover, causing high shear stresses and the resulting early loss of concrete cover.

- Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.
- Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.

The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure C5.7.2.1-1, using similar triangles.

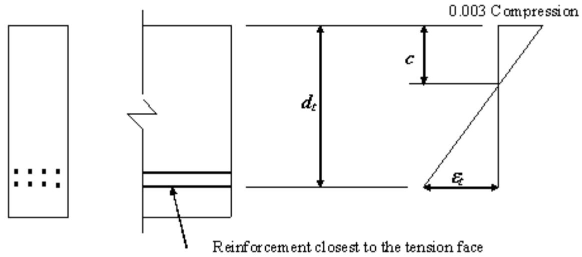


Figure C5.7.2.1-1—Strain Distribution and Net Tensile Strain

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.
- The use of compression reinforcement in conjunction with additional tension reinforcement is permitted to increase the strength of flexural members.

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum reinforcement limit that was given as $c/d_e \leq 0.42$, which corresponded to a net tensile strain at the centroid of the tension reinforcement of 0.00414. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this Specification.

- In the approximate flexural resistance equations of Articles 5.7.3.1 and 5.7.3.2, f_y and f'_y may replace f_s and f'_s, respectively, subject to the following conditions:
  - f_y may replace f_s when, using f_y in the calculation, the resulting ratio c/d_s does not exceed 0.6. If c/d_s exceeds 0.6, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.
  - f'_y may replace f'_s when, using f'_y in the calculation, $c \geq 3d'_s$. If $c < 3d'_s$, strain compatibility shall be used to determine the stress in the mild steel compression reinforcement. The compression reinforcement shall be conservatively ignored, i.e., $A'_s = 0$.

Additional limitations on the maximum usable extreme concrete compressive strain in hollow rectangular compression members shall be investigated as specified in Article 5.7.4.7.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Article 5.7.3.5 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

When using the approximate flexural resistance equations in Articles 5.7.3.1 and 5.7.3.2, it is important to assure that both the tension and compression mild steel reinforcement are yielding to obtain accurate results. In previous editions of the *AASHTO LRFD Bridge Design Specifications*, the maximum reinforcement limit of $c/d_e \leq 0.42$ assured that the mild tension steel would yield at nominal flexural resistance, but this limit was eliminated in the 2006 interim revisions. The current limit of $c/d_s \leq 0.6$ assures that the mild tension steel will be at or near yield, while $c \geq 3d'_s$ assures that the mild compression steel will yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

The mild steel tension reinforcement limitation does not apply to prestressing steel used as tension reinforcement. The equations used to determine the stress in the prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis.

5.7.2.2—Rectangular Stress Distribution

The natural relationship between concrete stress and strain may be considered satisfied by an equivalent rectangular concrete compressive stress block of $0.85f'_c$ over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$ from the extreme compression fiber. The distance c shall be measured perpendicular to the neutral axis. The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65.

C5.7.2.2

For practical design, the rectangular compressive stress distribution defined in this Article may be used in lieu of a more exact concrete stress distribution. This rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but in many practical cases it does provide essentially the same results as those obtained in tests. All strength equations presented in Article 5.7.3 are based on the rectangular stress block.

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

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

Additional limitations on the use of the rectangular stress block when applied to hollow rectangular compression members shall be investigated as specified in Article 5.7.4.7.

5.7.3—Flexural Members

5.7.3.1—Stress in Prestressing Steel at Nominal Flexural Resistance

5.7.3.1.1—Components with Bonded Tendons

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in Article 5.7.2.2 is used and for which f_{pe} is not less than 0.5 f_{pu}, the average stress in prestressing steel, f_{ps}, may be taken as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad (5.7.3.1.1\text{-}1)$$

in which:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad (5.7.3.1.1\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps}\dfrac{f_{pu}}{d_p}} \qquad (5.7.3.1.1\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{0.85 f'_c \beta_1 b + k A_{ps}\dfrac{f_{pu}}{d_p}} \qquad (5.7.3.1.1\text{-}4)$$

where:

A_{ps} = area of prestressing steel (in.^{2})

f_{pu} = specified tensile strength of prestressing steel (ksi)

f_{py} = yield strength of prestressing steel (ksi)

C5.7.3.1.1

Equations in this Article and subsequent equations for flexural resistance are based on the assumption that the distribution of steel is such that it is reasonable to consider all of the tensile reinforcement to be lumped at the location defined by d_s and all of the prestressing steel can be considered to be lumped at the location defined by d_p. Therefore, in the case where a significant number of prestressing elements are on the compression side of the neutral axis, it is more appropriate to use a method based on the conditions of equilibrium and strain compatibility as indicated in Article 5.7.2.1.

The background and basis for Eqs. 5.7.3.1.1-1 and 5.7.3.1.2-1 can be found in Naaman (1985), Loov (1988), Naaman (1989), and Naaman (1990–1992).

Values of f_{py}/f_{pu} are defined in Table C5.7.3.1.1-1. Therefore, the values of k from Eq. 5.7.3.1.1-2 depend only on the type of tendon used.

Table C5.7.3.1.1-1—Values of *k*

Type of Tendon	f_{py}/f_{pu}	Value of k
Low relaxation strand	0.90	0.28
Stress-relieved strand and Type 1 high-strength bar	0.85	0.38
Type 2 high-strength bar	0.80	0.48

A_s = area of mild steel tension reinforcement (in.^{2})

A'_s = area of compression reinforcement (in.^{2})

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

b = width of the compression face of the member; for a flange section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.)

b_w = width of web (in.)

h_f = depth of compression flange (in.)

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)

c = distance between the neutral axis and the compressive face (in.)

β_1 = stress block factor specified in Article 5.7.2.2

5.7.3.1.2—Components with Unbonded Tendons

For rectangular or flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used, the average stress in unbonded prestressing steel may be taken as:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py} \quad (5.7.3.1.2\text{-}1)$$

in which:

$$\ell_e = \left(\frac{2\,\ell_i}{2 + N_s}\right) \quad (5.7.3.1.2\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps}\,f_{ps} + A_s\,f_s - A'_s\,f'_s - 0.85\,f'_c\,(b - b_w)\,h_f}{0.85\,f'_c\,\beta_1\,b_w} \quad (5.7.3.1.2\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps}f_{ps} + A_sf_s - A'_s\,f'_s}{0.85f'_c\,\beta_1\,b} \quad (5.7.3.1.2\text{-}4)$$

C5.7.3.1.2

A first estimate of the average stress in unbonded prestressing steel may be made as:

$$f_{ps} = f_{pe} + 15.0 \text{ (ksi)} \quad (C5.7.3.1.2\text{-}1)$$

In order to solve for the value of f_{ps} in Eq. 5.7.3.1.2-1, the equation of force equilibrium at ultimate is needed. Thus, two equations with two unknowns (f_{ps} and c) need to be solved simultaneously to achieve a closed-form solution.

where:

c = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded, given by Eqs. 5.7.3.1.2-3 and 5.7.3.1.2-4 for T-section behavior and rectangular section behavior, respectively (in.)

ℓ_e = effective tendon length (in.)

ℓ_i = length of tendon between anchorages (in.)

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

f_{py} = yield strength of prestressing steel (ksi)

f_{pe} = effective stress in prestressing steel at section under consideration after all losses (ksi)

5.7.3.1.3—Components with Both Bonded and Unbonded Tendons

5.7.3.1.3a—Detailed Analysis

Except as specified in Article 5.7.3.1.3b, for components with both bonded and unbonded tendons, the stress in the prestressing steel shall be computed by detailed analysis. This analysis shall take into account the strain compatibility between the section and the bonded prestressing steel. The stress in the unbonded prestressing steel shall take into account the global displacement compatibility between bonded sections of tendons located within the span. Bonded sections of unbonded tendons may be anchorage points and any bonded section, such as deviators. Consideration of the possible slip at deviators shall be taken into consideration. The nominal flexural strength should be computed directly from the stresses resulting from this analysis.

5.7.3.1.3b—Simplified Analysis

In lieu of the detailed analysis described in Article 5.7.3.1.3a, the stress in the unbonded tendons may be conservatively taken as the effective stress in the prestressing steel after losses, f_{pe}. In this case, the stress in the bonded prestressing steel shall be computed using Eqs. 5.7.3.1.1-1 through 5.7.3.1.1-4, with the term $A_{ps}f_{pu}$ in Eqs. 5.7.3.1.1-3 and 5.7.3.1.1-4 replaced with the term $A_{psb}f_{pu} + A_{psu}f_{pe}$.

where:

A_{psb} = area of bonded prestressing steel (in.^{2})

A_{psu} = area of unbonded prestressing steel (in.^{2})

When computing the nominal flexural resistance using Eq. 5.7.3.2.2-1, the average stress in the prestressing steel shall be taken as the weighted average of the stress in the bonded and unbonded prestressing steel, and the total area of bonded and unbonded prestressing shall be used.

5.7.3.2—Flexural Resistance

5.7.3.2.1—Factored Flexural Resistance

The factored resistance M_r shall be taken as:

$$M_r = \phi M_n \quad (5.7.3.2.1\text{-}1)$$

where:

M_n = nominal resistance (kip-in.)

ϕ = resistance factor as specified in Article 5.5.4.2

C5.7.3.2.1

Moment at the face of the support may be used for design. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained member are built monolithic with the member and support, the face of support should be considered at a section where the combined depth of the member and fillet is at least one and one-half times the thickness of the member. No portion of a fillet should be considered as adding to the effective depth when determining the nominal resistance.

5.7.3.2.2–Flanged Sections

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used and where the compression flange depth is less than $a = \beta_1 c$, as determined in accordance with Eqs. 5.7.3.1.1-3, 5.7.3.1.1-4, 5.7.3.1.2-3, or 5.7.3.1.2-4, the nominal flexural resistance may be taken as:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + 0.85 f'_c\left(b - b_w\right) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \quad (5.7.3.2.2\text{-}1)$$

where:

A_{ps} = area of prestressing steel (in.^{2})

f_{ps} = average stress in prestressing steel at nominal bending resistance specified in Eq. 5.7.3.1.1-1 (ksi)

d_p = distance from extreme compression fiber to the centroid of prestressing tendons (in.)

A_s = area of nonprestressed tension reinforcement (in.^{2})

f_s = stress in the mild steel tension reinforcement at nominal flexural resistence (ksi), as specified in Article 5.7.2.1

C5.7.3.2.2

In previous editions and interims of the LRFD Specifications, the factor β_1 was applied to the flange overhang term of Eqs. 5.7.3.2.2-1, 5.7.3.1.1-3, and 5.7.3.1.2-3. This was not consistent with the original derivation of the equivalent rectangular stress block as it applies to flanged sections (Mattock, Kriz, and Hognestad. 1961). For the current LRFD Specifications, the β_1 factor has been removed from the flange overhang term of these equations. See also Seguirant (2002), Girgis, Sun, and Tadros (2002), Naaman (2002), Weigel, Seguirant, Brice, and Khaleghi (2003), Baran, Schultz, and French (2004), and Seguirant, Brice, and Khaleghi (2004).

d_s = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (in.)

A'_s = area of compression reinforcement (in.^{2})

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.)

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

b = width of the compression face of the member; for a flange section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.)

b_w = web width or diameter of a circular section (in.)

β_1 = stress block factor specified in Article 5.7.2.2

h_f = compression flange depth of an I or T member (in.)

a = $c\beta_1$; depth of the equivalent stress block (in.)

5.7.3.2.3—Rectangular Sections

For rectangular sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used and where the compression flange depth is not less than $a = \beta_1 c$ as determined in accordance with Eqs. 5.7.3.1.1-4 or 5.7.3.1.2-4, the nominal flexural resistance M_n may be determined by using Eqs. 5.7.3.1.1-1 through 5.7.3.2.2-1, in which case b_w shall be taken as b.

5.7.3.2.4—Other Cross-Sections

For cross-sections other than flanged or essentially rectangular sections with vertical axis of symmetry or for sections subjected to biaxial flexure without axial load, the nominal flexural resistance, M_n, shall be determined by an analysis based on the assumptions specified in Article 5.7.2. The requirements of Article 5.7.3.3 shall apply.

5.7.3.2.5—Strain Compatibility Approach

Alternatively, the strain compatibility approach may be used if more precise calculations are required. The appropriate provisions of Article 5.7.2.1 shall apply.

The stress and corresponding strain in any given layer of reinforcement may be taken from any representative stress-strain formula or graph for mild reinforcement and prestressing strands.

5.7.3.3—Limits for Reinforcement

5.7.3.3.1—Maximum Reinforcement

[PROVISION DELETED IN 2005]

C5.7.3.3.1

In editions of and interims to the LRFD Specifications prior to 2005, Article 5.7.3.3.1 limited the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if “it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved.” No guidance was given for what “sufficient ductility” should be, and it was not clear what value of ϕ should be used for such over-reinforced members.

The current provisions of LRFD eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in Mast (1992). Below a net tensile strain in the extreme tension steel of 0.005, as the tension reinforcement quantity increases, the factored resistance of prestressed and nonprestressed sections is reduced in accordance with Article 5.5.4.2.1. This reduction compensates for decreasing ductility with increasing overstrength. Only the addition of compression reinforcement in conjunction with additional tension reinforcement can result in an increase in the factored flexural resistance of the section.

5.7.3.3.2—Minimum Reinforcement

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

- 1.2 times the cracking moment, M_{cr}, determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of the concrete as specified in Article 5.4.2.6, where M_{cr} may be taken as:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r \tag{5.7.3.3.2-1}$$

where:

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

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

Appropriate values for M_{dnc} and S_{nc} shall be used for any intermediate composite sections. Where the beams are designed for the monolithic or noncomposite section to resist all loads, substitute S_{nc} for S_c in the above equation for the calculation of M_{cr}.

- 1.33 times the factored moment required by the applicable strength load combinations specified in Table 3.4.1-1.

The provisions of Article 5.10.8 shall apply.

5.7.3.4—Control of Cracking by Distribution of Reinforcement

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in Article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad (5.7.3.4\text{-}1)$$

C5.7.3.4

All reinforced concrete members are subject to cracking under any load condition, including thermal effects and restraint of deformations, which produces tension in the gross section in excess of the cracking strength of the concrete. Locations particularly vulnerable to cracking include those where there is an abrupt change in section and intermediate post-tensioning anchorage zones.

Provisions specified, herein, are used for the distribution of tension reinforcement to control flexural cracking.

Crack width is inherently subject to wide scatter, even in careful laboratory work, and is influenced by shrinkage and other time-dependent effects. Steps should be taken in detailing of the reinforcement to control cracking. From the standpoint of appearance, many fine cracks are preferable to a few wide cracks. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are more effective in controlling cracking than one or two larger bars of equivalent area.

Extensive laboratory work involving deformed reinforcing bars has confirmed that the crack width at the service limit state is proportional to steel stress. However, the significant variables reflecting steel detailing were found to be the thickness of concrete cover and spacing of the reinforcement.

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

γ_e = exposure factor
= 1.00 for Class 1 exposure condition
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness or depth of the component (in.)

d_ℓ = distance from the extreme compression fiber to the centroid of extreme tension steel element (in.)

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and/or corrosion.

In the computation of d_c, the actual concrete cover thickness is to be used.

When computing the actual stress in the steel reinforcement, axial tension effects shall be considered, while axial compression effects may be considered.

The minimum and maximum spacing of reinforcement shall also comply with the provisions of Articles 5.10.3.1 and 5.10.3.2, respectively.

The effects of bonded prestressing steel may be considered, in which case the value of f_s used in Eq. 5.7.3.4-1, for the bonded prestressing steel, shall be the stress that develops beyond the decompression state calculated on the basis of a cracked section or strain compatibility analysis.

Where flanges of reinforced concrete T-girders and box girders are in tension at the service limit state, the flexural tension reinforcement shall be distributed over the lesser of:

- The effective flange width, specified in Article 4.6.2.6, or
- A width equal to 1/10 of the average of adjacent spans between bearings.

Eq. 5.7.3.4-1 is expected to provide a distribution of reinforcement that will control flexural cracking. The equation is based on a physical crack model (Frosch, 2001) rather than the statistically-based model used in previous editions of the specifications. It is written in a form emphasizing reinforcement details, i.e., limiting bar spacing, rather than crack width. Furthermore, the physical crack model has been shown to provide a more realistic estimate of crack widths for larger concrete covers compared to the previous equation (Destefano 2003).

Eq. 5.7.3.4-1 with Class 1 exposure condition is based on an assumed crack width of 0.017 in. Previous research indicates that there appears to be little or no correlation between crack width and corrosion, however, the different classes of exposure conditions have been so defined in order to provide flexibility in the application of these provisions to meet the needs of the Authority having jurisdiction. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Authority having jurisdiction may consider for Class 2 exposure condition would include decks and substructures exposed to water. The crack width is directly proportional to the γ_e exposure factor, therefore, if the individual Authority with jurisdiction desires an alternate crack width, the γ_e factor can be adjusted directly. For example a γ_e factor of 0.5 will result in an approximate crack width of 0.0085 in.

Where members are exposed to aggressive exposure or corrosive environments, additional protection beyond that provided by satisfying Eq. 5.7.3.4-1 may be provided by decreasing the permeability of the concrete and/or waterproofing the exposed surface.

Cracks in segmental concrete box girders may result from stresses due to handling and storing segments for precast construction and to stripping forms and supports from cast-in-place construction before attainment of the nominal f'_c.

The β_s factor, which is a geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level, has been incorporated into the basic crack control equation in order to provide uniformity of application for flexural member depths ranging from thin slabs in box culverts to deep pier caps and thick footings. The theoretical definition of β_s may be used in lieu of the approximate expression provided.

Distribution of the negative reinforcement for control of cracking in T-girders should be made in the context of the following considerations:

- Wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web.
- Close spacing near the web leaves the outer regions of the flange unprotected.

If the effective flange width exceeds 1/10 the span, additional longitudinal reinforcement, with area not less than 0.4 percent of the excess slab area, shall be provided in the outer portions of the flange.

If d_ℓ of nonprestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance $d_\ell/2$ nearest the flexural tension reinforcement. The area of skin reinforcement A_{sk} in in.^{2}/ft of height on each side face shall satisfy:

$$A_{sk} \geq 0.012\ (d_\ell - 30) \leq \frac{A_s + A_{ps}}{4} \tag{5.7.3.4-2}$$

where:

A_{ps} = area of prestressing steel (in.^{2})

A_s = area of tensile reinforcement (in.^{2})

However, the total area of longitudinal skin reinforcement (per face) need not exceed one-fourth of the required flexural tensile reinforcement $A_s + A_{ps}$.

The maximum spacing of the skin reinforcement shall not exceed either $d_e/6$ or 12.0 in.

Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires.

The 1/10 of the span limitation is to guard against an excessive spacing of bars, with additional reinforcement required to protect the outer portions of the flange.

The requirements for skin reinforcement are based upon ACI 318-95. For relatively deep flexural members, some reinforcement should be placed near the vertical faces in the tension zone to control cracking in the web. Without such auxiliary steel, the width of the cracks in the web may greatly exceed the crack widths at the level of the flexural tension reinforcement.

5.7.3.5—Moment Redistribution

In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.11 is provided at the internal supports of continuous reinforced concrete beams, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

Positive moments shall be adjusted to account for the changes in negative moments to maintain equilibrium of loads and force effects.

C5.7.3.5

In editions and interims to the LRFD Specifications prior to 2005, Article 5.7.3.5 specified the permissible redistribution percentage in terms of the c/d_e ratio. The current specification specifies the permissible redistribution percentage in terms of net tensile strain ε_t. The background and basis for these provisions are given in Mast (1992).

5.7.3.6—Deformations

5.7.3.6.1—General

The provisions of Article 2.5.2.6 shall be considered.

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

C5.7.3.6.1

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

5.7.3.6.2—Deflection and Camber

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

For determining deflection and camber, the provisions of Articles 4.5.2.1, 4.5.2.2, and 5.9.5.5 shall apply.

In the absence of a more comprehensive analysis, instantaneous deflections may be computed using the modulus of elasticity for concrete as specified in Article 5.4.2.4 and taking the moment of inertia as either the gross moment of inertia, I_g, or an effective moment of inertia, I_e, given by Eq. 5.7.3.6.2-1:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (5.7.3.6.2\text{-}1)$$

in which:

$$M_{cr} = f_r \frac{I_g}{y_t} \qquad (5.7.3.6.2\text{-}2)$$

where:

M_{cr} = cracking moment (kip-in.)

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

y_t = distance from the neutral axis to the extreme tension fiber (in.)

M_a = maximum moment in a component at the stage for which deformation is computed (kip-in.)

For prismatic members, effective moment of inertia may be taken as the value obtained from Eq. 5.7.3.6.2-1 at midspan for simple or continuous spans, and at support for cantilevers. For continuous nonprismatic members, the effective moment of inertia may be taken as the average of the values obtained from Eq. 5.7.3.6.2-1 for the critical positive and negative moment sections.

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

- If the instantaneous deflection is based on I_g: 4.0
- If the instantaneous deflection is based on I_e: 3.0–1.2(A'_s/A_s) ≥1.6

where:

A'_s = area of compression reinforcement (in.^{2})

C5.7.3.6.2

For structures such as segmentally constructed bridges, camber calculations should be based on the modulus of elasticity and the maturity of the concrete when each increment of load is added or removed, as specified in Articles 5.4.2.3 and 5.14.2.3.6.

In prestressed concrete, the long-term deflection is usually based on mix-specific data, possibly in combination with the calculation procedures in Article 5.4.2.3. Other methods of calculating deflections which consider the different types of loads and the sections to which they are applied, such as that found in (PCI, 1992), may also be used.

A_s = area of nonprestressed tension reinforcement (in.^{2})

The contract documents shall require that deflections of segmentally constructed bridges shall be calculated prior to casting of segments based on the anticipated casting and erection schedules and that they shall be used as a guide against which actual deflection measurements are checked.

5.7.3.6.3—Axial Deformation

Instantaneous shortening or expansion due to loads shall be determined using the modulus of elasticity of the materials at the time of loading.

Instantaneous shortening or expansion due to temperature shall be determined in accordance with Articles 3.12.2, 3.12.3, and 5.4.2.2.

Long-term shortening due to shrinkage and creep shall be determined as specified in Article 5.4.2.3.

5.7.4—Compression Members

5.7.4.1—General

Unless otherwise permitted, compression members shall be analyzed with consideration of the effects of:

- Eccentricity,
- Axial loads,
- Variable moments of inertia,
- Degree of end fixity,
- Deflections,
- Duration of loads, and
- Prestressing.

In lieu of a refined procedure, nonprestressed columns with the slenderness ratio, $K\ell_u/r < 100$, may be designed by the approximate procedure specified in Article 5.7.4.3.

where:

K = effective length factor specified in Article 4.6.2.5

ℓ_u = unbraced length (in.)

r = radius of gyration (in.)

The requirements of this Article shall be supplemented and modified for structures in Seismic Zones 2, 3, and 4, as specified in Article 5.10.11.

C5.7.4.1

Compression members are usually prestressed only where they are subjected to a high level of flexure or when they are subjected to driving stresses, as is the case with prestressed concrete piles.

Provisions shall be made to transfer all force effects from compression components, adjusted for second-order moment magnification, to adjacent components.

Where the connection to an adjacent component is by a concrete hinge, longitudinal reinforcement shall be centralized within the hinge to minimize flexural resistance and shall be developed on both sides of the hinge.

5.7.4.2—Limits for Reinforcement

Additional limits on reinforcement for compression members in Seismic Zones 2, 3, and 4 shall be considered as specified in Articles 5.10.11.3 and 5.10.11.4.1a.

The maximum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \qquad (5.7.4.2\text{-}1)$$

and:

$$\frac{A_{ps} f_{pe}}{A_g f'_c} \leq 0.30 \qquad (5.7.4.2\text{-}2)$$

The minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s f_y}{A_g f'_c} + \frac{A_{ps} f_{pu}}{A_g f'_c} \geq 0.135 \qquad (5.7.4.2\text{-}3)$$

where:

A_s = area of nonprestressed tension steel (in.^{2})

A_g = gross area of section (in.^{2})

A_{ps} = area of prestressing steel (in.^{2})

f_{pu} = specified tensile strength of prestressing steel (ksi)

f_y = specified yield strength of reinforcing bars (ksi)

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

f_{pe} = effective prestress (ksi)

The minimum number of longitudinal reinforcing bars in the body of a column shall be six in a circular arrangement and four in a rectangular arrangement. The minimum size of bar shall be No. 5.

C5.7.4.2

According to current ACI codes, the area of longitudinal reinforcement for nonprestressed noncomposite compression components should be not less than 0.01 A_g. Because the dimensioning of columns is primarily controlled by bending, this limitation does not account for the influence of the concrete compressive strength. To account for the compressive strength of concrete, the minimum reinforcement in flexural members is shown to be proportional to f'_c/f_y in Article 5.7.3.3.2. This approach is also reflected in the first term of Eq. 5.7.4.2-3. For fully prestressed members, current codes specify a minimum average prestress of 0.225 ksi. Here also the influence of compressive strength is not accounted for. A compressive strength of 5.0 ksi has been used as a basis for these provisions, and a weighted averaging procedure was used to arrive at the equation.

Where columns are pinned to their foundations, a small number of central bars have sometimes been used as a connection between footing and column.

For bridges in Seismic Zone 1, a reduced effective area may be used when the cross-section is larger than that required to resist the applied loading. The minimum percentage of total (prestressed and nonprestressed) longitudinal reinforcement of the reduced effective area is to be the greater of one percent or the value obtained from Eq. 5.7.4.2-3. Both the reduced effective area and the gross area must be capable of resisting all applicable load combinations from Table 3.4.1-1.

For low risk seismic zones, the one percent reduced effective area rule, which has been used successfully since 1957 in the Standard Specifications, is implemented, but modified to account for the dependency of the minimum reinforcement on the ratio of f'_c/f_y.

For columns subjected to high, permanent axial compressive stresses where significant concrete creep is likely, using an amount of longitudinal reinforcement less than that given by Eq. 5.7.4.2-3 is not recommended because of the potential for significant transfer of load from the concrete to the reinforcement as discussed in the report of ACI Committee 105.

5.7.4.3—Approximate Evaluation of Slenderness Effects

For members not braced against sidesway, the effects of slenderness may be neglected where the slenderness ratio, $K\ell_u/r$, is less than 22.

For members braced against sidesway, the effects of slenderness may be neglected where $K\ell_u/r$ is less than $34-12(M_1/M_2)$, in which M_1 and M_2 are the smaller and larger end moments, respectively, and the term (M_1/M_2) is positive for single curvature flexure.

The following approximate procedure may be used for the design of nonprestressed compression members with $K\ell_u/r$ less than 100:

- The design is based on a factored axial load, P_u, determined by elastic analysis and a magnified factored moment, M_c, as specified in Article 4.5.3.2.2b.
- The unsupported length, ℓ_u, of a compression member is taken as the clear distance between components capable of providing lateral support for the compression components. Where haunches are present, the unsupported length is taken to the extremity of any haunches in the plane considered.
- The radius of gyration, r, is computed for the gross concrete section.
- For members braced against sidesway, the effective length factor, K, is taken as 1.0, unless it is shown by analysis that a lower value may be used.
- For members not braced against sidesway, K is determined with due consideration for the effects of cracking and reinforcement on relative stiffness and is taken as not less than 1.0.

In lieu of a more precise calculation, EI for use in determining P_e, as specified in Eq. 4.5.3.2.2b-5, shall be taken as the greater of:

C5.7.4.3

These procedures were developed for reinforced concrete columns but are currently used for prestressed concrete columns as well.

For members in structures, which undergo appreciable lateral deflections resulting from combinations of vertical load or combinations of vertical and lateral loads, force effects should be determined using a second-order analysis.

For a rectangular compression member, r may be taken as 0.30 times the overall dimension in the direction in which stability is being considered. For a circular compression member, r may be taken as 0.25 times the diameter.

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \quad (5.7.4.3\text{-}1)$$

$$EI = \frac{\dfrac{E_c I_g}{2.5}}{1+\beta_d} \quad (5.7.4.3\text{-}2)$$

where:

E_c = modulus of elasticity of concrete (ksi)

I_g = moment of inertia of the gross concrete section about the centroidal axis (in.^{4})

E_s = modulus of elasticity of longitudinal steel (ksi)

I_s = moment of inertia of longitudinal steel about the centroidal axis (in.^{4})

β_d = ratio of maximum factored permanent load moments to maximum factored total load moment; always positive

For eccentrically prestressed members, consideration shall be given to the effect of lateral deflection due to prestressing in determining the magnified moment.

5.7.4.4—Factored Axial Resistance

The factored axial resistance of concrete compressive components, symmetrical about both principal axes, shall be taken as:

$$P_r = \phi P_n \quad (5.7.4.4\text{-}1)$$

in which:

- For members with spiral reinforcement:

$$P_n = 0.85\begin{bmatrix} 0.85 f'_c\left(A_g - A_{st} - A_{ps}\right) \\ +f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \end{bmatrix} \quad (5.7.4.4\text{-}2)$$

- For members with tie reinforcement:

$$P_n = 0.80\begin{bmatrix} 0.85 f'_c\left(A_g - A_{st} - A_{ps}\right) \\ +f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \end{bmatrix} \quad (5.7.4.4\text{-}3)$$

where:

P_r = factored axial resistance, with or without flexure (kip)

P_n = nominal axial resistance, with or without flexure (kip)

C5.7.4.4

The values of 0.85 and 0.80 in Eqs. 5.7.4.4-2 and 5.7.4.4-3 place upper limits on the usable resistance of compression members to allow for unintended eccentricity.

In the absence of concurrent bending due to external loads or eccentric application of prestress, the ultimate strain on a compression member is constant across the entire cross-section. Prestressing causes compressive stresses in the concrete, which reduces the resistance of compression members to externally applied axial loads. The term, $E_p\varepsilon_{cu}$, accounts for the fact that a column or pile also shortens under externally applied loads, which serves to reduce the level of compression due to prestress. Assuming a concrete compressive strain at ultimate, ε_{cu} = 0.003, and a prestressing steel modulus, E_p = 28,500 ksi, gives a relatively constant value of 85.0 ksi for the amount of this reduction. Therefore, it is acceptable to reduce the effective prestressing by this amount. Conservatively, this reduction can be ignored.

f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)

A_g = gross area of section (in.^{2})

A_{st} = total area of longitudinal reinforcement (in.^{2})

f_y = specified yield strength of reinforcement (ksi)

ϕ = resistance factor specified in Article 5.5.4.2

A_{ps} = area of prestressing steel (in.^{2})

E_p = modulus of elasticity of prestressing tendons (ksi)

f_{pe} = effective stress in prestressing steel after losses (ksi)

ε_{cu} = failure strain of concrete in compression (in./in.)

5.7.4.5—Biaxial Flexure

In lieu of an analysis based on equilibrium and strain compatibility for biaxial flexure, noncircular members subjected to biaxial flexure and compression may be proportioned using the following approximate expressions:

- If the factored axial load is not less than $0.10\,\phi f'_c A_g$:

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{\phi P_o} \qquad (5.7.4.5\text{-}1)$$

in which:

$$P_o = 0.85 f'_c\left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \qquad (5.7.4.5\text{-}2)$$

- If the factored axial load is less than $0.10\,\phi f'_c A_g$:

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \qquad (5.7.4.5\text{-}3)$$

where:

ϕ = resistance factor for members in axial compression

P_{rxy} = factored axial resistance in biaxial flexure (kip)

P_{rx} = factored axial resistance determined on the basis that only eccentricity e_y is present (kip)

C5.7.4.5

Eqs. 5.7.3.2.1-1 and 5.7.4.4-1 relate factored resistances, given in Eqs. 5.7.4.5-1 and 5.7.4.5-2 by the subscript r, e.g., M_{rx}, to the nominal resistances and the resistance factors. Thus, although previous editions of the Standard Specifications included the resistance factor explicitly in equations corresponding to Eqs. 5.7.4.5-1 and 5.7.4.5-2, these Specifications implicitly include the resistance factor by using factored resistances in the denominators.

The procedure for calculating corresponding values of M_{rx} and P_{rx} or M_{ry} and P_{ry} can be found in most texts on reinforced concrete design.

P_{ry} = factored axial resistance determined on the basis that only eccentricity e_x is present (kip)

P_u = factored applied axial force (kip)

M_{ux} = factored applied moment about the x-axis (kip-in.)

M_{uy} = factored applied moment about the y-axis (kip-in.)

e_x = eccentricity of the applied factored axial force in the x direction, i.e., $= M_{uy}/P_u$ (in.)

e_y = eccentricity of the applied factored axial force in the y direction, i.e., $= M_{ux}/P_u$ (in.)

P_o = nominal axial resistance of a section at 0.0 eccentricity

The factored axial resistance P_{rx} and P_{ry} shall not be taken to be greater than the product of the resistance factor, ϕ, and the maximum nominal compressive resistance given by either Eqs. 5.7.4.4-2 or 5.7.4.4-3, as appropriate.

5.7.4.6—Spirals and Ties

The area of steel for spirals and ties in bridges in Seismic Zones 2, 3, or 4 shall comply with the requirements specified in Article 5.10.11.

Where the area of spiral and tie reinforcement is not controlled by:

- Seismic requirements,
- Shear or torsion as specified in Article 5.8, or
- Minimum requirements as specified in Article 5.10.6,

the ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals, shall satisfy:

$$\rho_s \geq 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_{yh}} \qquad (5.7.4.6\text{-}1)$$

where:

A_g = gross area of concrete section (in.^{2})

A_c = area of core measured to the outside diameter of the spiral (in.^{2})

f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)

f_{yh} = specified yield strength of spiral reinforcement (ksi)

Other details of spiral and tie reinforcement shall conform to the provisions of Articles 5.10.6 and 5.10.11.

5.7.4.7—Hollow Rectangular Compression Members

5.7.4.7.1—Wall Slenderness Ratio

The wall slenderness ratio of a hollow rectangular cross-section shall be taken as:

$$\lambda_w = \frac{X_u}{t} \qquad (5.7.4.7.1\text{-}1)$$

where:

X_u = the clear length of the constant thickness portion of a wall between other walls or fillets between walls (in.)

t = thickness of wall (in.)

λ_w = wall slenderness ratio for hollow columns

Wall slenderness greater than 35 may be used only when the behavior and resistance of the wall is documented by analytic and experimental evidence acceptable to the Owner.

C5.7.4.7.1

The definition of the parameter X_u is illustrated in Figure C5.7.4.7.1-1, taken from Taylor et al. (1990).

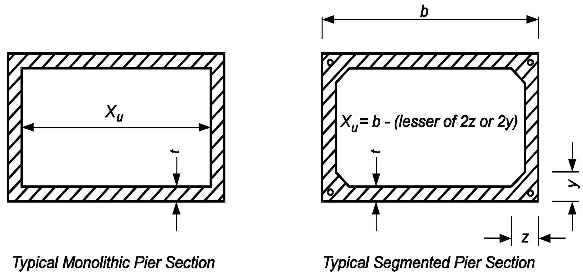


Figure C5.7.4.7.1-1—Illustration of X_u

The test program, reported in Taylor et al. (1990), was limited to the case of loading under simultaneous axial and uniaxial bending about the weak axis of the section. The results of the study have not been confirmed for the case of biaxial bending. Until such a study is completed, the Designer should investigate the effects of biaxial loading on hollow sections.

5.7.4.7.2—Limitations on the Use of the Rectangular Stress Block Method

5.7.4.7.2a—General

Except as specified in Article 5.7.4.7.2c, the equivalent rectangular stress block method shall not be employed in the design of hollow rectangular compression members with a wall slenderness ratio ≥15.

Where the wall slenderness ratio is less than 15, the rectangular stress block method may be used based on a compressive strain of 0.003.

5.7.4.7.2b—Refined Method for Adjusting Maximum Usable Strain Limit

Where the wall slenderness ratio is 15 or greater, the maximum usable strain at the extreme concrete compression fiber is equal to the lesser of the computed local buckling strain of the widest flange of the cross-section, or 0.003.

The local buckling strain of the widest flange of the cross-section may be computed assuming simply supported boundary conditions on all four edges of the flange. Nonlinear material behavior shall be considered by incorporating the tangent material moduli of the concrete and reinforcing steel in computations of the local buckling strain.

Discontinuous, nonpost-tensioned reinforcement in segmentally constructed hollow rectangular compression members shall be neglected in computations of member strength.

Flexural resistance shall be calculated using the principles of Article 5.7.3 applied with anticipated stress-strain curves for the types of material to be used.

5.7.4.7.2c—Approximate Method for Adjusting Factored Resistance

The provisions of this Article and the rectangular stress block method may be used in lieu of the provisions of Articles 5.7.4.7.2a and 5.7.4.7.2b where the wall slenderness is ≤ 35.

The factored resistance of a hollow column, determined using a maximum usable strain of 0.003, and the resistance factors specified in Article 5.5.4.2 shall be further reduced by a factor ϕ_w taken as:

- If $\lambda_w \leq 15$, then $\phi_w = 1.0$ (5.7.4.7.2c-1)

- If $15 < \lambda_w \leq 25$, then $\phi_w = 1 - 0.025(\lambda_w - 15)$ (5.7.4.7.2c-2)

- If $25 < \lambda_w \leq 35$, then $\phi_w = 0.75$ (5.7.4.7.2c-3)

5.7.5—Bearing

C5.7.5

In the absence of confinement reinforcement in the concrete supporting the bearing device, the factored bearing resistance shall be taken as:

$$P_r = \phi P_n \tag{5.7.5-1}$$

in which:

$$P_n = 0.85 f_c' A_1 m \tag{5.7.5-2}$$

where:

P_n = nominal bearing resistance (kip)

A_1 = area under bearing device (in.^{2})

m = modification factor

A_2 = a notional area defined herein (in.^{2})

The modification factor may be determined as follows:

- Where the supporting surface is wider on all sides than the loaded area:

$$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0 \qquad (5.7.5\text{-}3)$$

- Where the loaded area is subjected to nonuniformly distributed bearing stresses:

$$m = 0.75\sqrt{\frac{A_2}{A_1}} \leq 1.50 \qquad (5.7.5\text{-}4)$$

Where the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, as well as side slopes of 1.0 vertical to 2.0 horizontal.

Where the factored applied load exceeds the factored resistance, as specified herein, provisions shall be made to resist the bursting and spalling forces in accordance with Article 5.10.9.

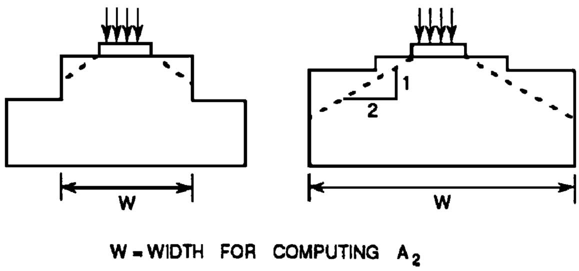


Figure C5.7.5-1—Determination of A_2 for a Stepped Support

5.7.6—Tension Members

5.7.6.1—Factored Tension Resistance

Members in which the factored loads induce tensile stresses throughout the cross-section shall be regarded as tension members, and the axial force shall be assumed to be resisted only by the steel elements. The provisions of Article 5.11.5.4 shall apply.

The factored resistance to uniform tension shall be taken as:

$$P_r = \phi P_n \qquad (5.7.6.1\text{-}1)$$

where:

P_n = nominal tension resistance specified in Article 5.6.3.4

ϕ = resistance factor specified in Article 5.5.4.2

5.7.6.2—Resistance to Combinations of Tension and Flexure

Members subjected to eccentric tension loading, which induces both tensile and compressive stresses in the cross-section, shall be proportioned in accordance with the provisions of Article 5.7.2.

5.8—SHEAR AND TORSION

5.8.1—Design Procedures

5.8.1.1—Flexural Regions

Where it is reasonable to assume that plane sections remain plane after loading, regions of components shall be designed for shear and torsion using either the sectional model as specified in Article 5.8.3 or the strut-and-tie model as specified in Article 5.6.3. The requirements of Article 5.8.2 shall apply.

In lieu of the provisions of Article 5.8.3, segmental post-tensioned concrete box girder bridges may be designed for shear and torsion using the provisions of Article 5.8.6.

Components in which the distance from the point of zero shear to the face of the support is less than $2d$, or components in which a load causing more than 1/2 (1/3 in case of segmental box girders) of the shear at a support is closer than $2d$ from the face of the support, may be considered to be deep components for which the provisions of Article 5.6.3 and the detailing requirements of Article 5.13.2.3 apply.

C5.8.1.1

The sectional model is appropriate for the design of typical bridge girders, slabs, and other regions of components where the assumptions of traditional engineering beam theory are valid. This theory assumes that the response at a particular section depends only on the calculated values of the sectional force effects, i.e., moment, shear, axial load, and torsion, and does not consider the specific details of how the force effects were introduced into the member. Although the strut-and-tie model can be applied to flexural regions, it is more appropriate and generally yields less conservative designs for regions near discontinuities where the actual flow of forces should be considered in more detail.

5.8.1.2—Regions Near Discontinuities

Where the plane sections assumption of flexural theory is not valid, regions of members shall be designed for shear and torsion using the strut-and-tie model as specified in Article 5.6.3. The provisions of Article 5.13.2 shall apply.

C5.8.1.2

The response of regions adjacent to abrupt changes in cross-section, openings, dapped ends, deep beams, and corbels is influenced significantly by the details of how the loads are introduced into the region and how the region is supported.

5.8.1.3—Interface Regions

Interfaces between elements shall be designed for shear transfer in accordance with the provisions of Article 5.8.4.

5.8.1.4—Slabs and Footings

Slab-type regions shall be designed for shear in accordance with the provisions of Article 5.13.3.6 or Article 5.6.3.

5.8.2—General Requirements

5.8.2.1—General

The factored torsional resistance, T_r, shall be taken as:

$$T_r = \phi T_n \qquad (5.8.2.1\text{-}1)$$

C5.8.2.1

where:

T_n = nominal torsional resistance specified in Article 5.8.3.6 (kip-in.)

ϕ = resistance factor specified in Article 5.5.4.2

The factored shear resistance, V_r, shall be taken as:

$$V_r = \phi V_n \qquad (5.8.2.1\text{-}2)$$

V_n = nominal shear resistance specified in Article 5.8.3.3 (kip)

ϕ = resistance factor as specified in Article 5.5.4.2

For normal weight concrete, torsional effects shall be investigated where:

$$T_u > 0.25\phi T_{cr} \qquad (5.8.2.1\text{-}3)$$

in which:

$$T_{cr} = 0.125\sqrt{f'_c}\,\frac{A_{cp}^{\ 2}}{p_c}\sqrt{1+\frac{f_{pc}}{0.125\sqrt{f'_c}}} \qquad (5.8.2.1\text{-}4)$$

where:

T_u = factored torsional moment (kip-in.)

T_{cr} = torsional cracking moment (kip-in.)

A_{cp} = total area enclosed by outside perimeter of concrete cross-section (in.^{2})

p_c = the length of the outside perimeter of the concrete section (in.)

f_{pc} = compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi)

ϕ = resistance factor specified in Article 5.5.4.2

For cellular structures:

$$\frac{A_{cp}^{\ 2}}{p_c} \le 2A_o b_v \qquad (5.8.2.1\text{-}5)$$

where:

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

The equivalent factored shear force, V_u, shall be taken equal to:

If the factored torsional moment is less than one-quarter of the factored pure torsional cracking moment, it will cause only a very small reduction in shear capacity or flexural capacity and, hence, can be neglected.

Sections that are designed for live loads using approximate methods of analysis in Article 4.6.2.2 need not be investigated for torsion.

The limit to Eq. 5.8.2.1-4 was added to avoid over-estimating T_{cr} in the case of cellular structures. Eq. 5.8.2.1-4 was derived from a solid section assuming an equivalent thin wall tube. When the actual b_v and $A_{cp}^{\ 2}$ is considered, torsional resistance can be much less. The resulting expression matches that in the current edition of AASHTO's *Guide Specifications for Design and Construction of Segmental Bridges*.

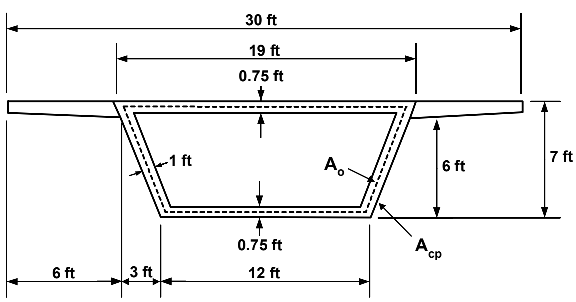


Figure C5.8.2.1-1—Sketch Showing Data Used in Sample Calculation for A_o Shown Below

For solid sections:

$$\sqrt{V_u^2+\left(\frac{0.9p_hT_u}{2A_o}\right)^2} \qquad (5.8.2.1\text{-}6)$$

For box sections:

$$V_u+\frac{T_ud_s}{2A_o} \qquad (5.8.2.1\text{-}7)$$

where:

p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.)

T_u = factored torsional moment (kip-in.)

$$A_o=\frac{1}{2}\left(11\text{ ft}+18\text{ ft}\right)\left(6.25\text{ ft}\right)=90.6\text{ ft}^2$$

Alternatively, the term A_o can usually be taken as 85 percent of the area enclosed by the centerline of the exterior closed transverse torsion reinforcement, including area of any holes. The justification for this generally conservative substitution is given in Collins and Mitchell (1991).

A stress limit for principal tension at the neutral axis in the web was added in 2004. This check requires shear demand, and not the resistance, to be modified for torsion. Eqs. 5.8.2.1-6 and 5.8.2.1-7 were added to clarify how demand is modified for torsion. Note that the V_u in Eqs. 5.8.3.4.2-1, 5.8.3.4.2-2, and 5.8.3.4.2-3 for ε_x, and in Eq. 5.8.2.9-1 for v_u, are not modified for torsion. In other words, the values used to select β, θ in Tables 5.8.3.4.2-1 and 5.8.3.4.2-2 have not been modified for torsion.

For solid cross-section shapes, such as a rectangle or an "I," there is the possibility of considerable redistribution of shear stresses. To make some allowance for this favorable redistribution it is safe to use a root-mean-square approach in calculating the nominal shear stress for these cross-sections, as indicated in Eq. 5.8.2.1-6. The $0.9p_h$ comes from 90 percent of the perimeter of the spalled concrete section. This is similar to multiplying 0.9 times the lever arm in flexural calculations.

For a box girder, the shear flow due to torsion is added to the shear flow due to flexure in one exterior web, and subtracted from the opposite exterior web. In the controlling web, the second term in Eq. 5.8.2.1-7 comes from integrating the distance from the centroid of the section, to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying by the web height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal.

5.8.2.2—Modifications for Lightweight Concrete

Where lightweight aggregate concretes are used, the following modifications shall apply in determining resistance to torsion and shear:

- Where the average splitting tensile strength of lightweight concrete, f_{ct}, is specified, the term $\sqrt{f'_c}$ in the expressions given in Articles 5.8.2 and 5.8.3 shall be replaced by:

  $$4.7f_{ct}\le\sqrt{f'_c}$$

- Where f_{ct} is not specified, the term $0.75\sqrt{f'_c}$ for all lightweight concrete, and $0.85\sqrt{f'_c}$ for sand-lightweight concrete shall be substituted for $\sqrt{f'_c}$ in the expressions given in Articles 5.8.2 and 5.8.3

C5.8.2.2

The tensile strength and shear capacity of lightweight concrete is typically somewhat less than that of normal weight concrete having the same compressive strength.

Linear interpolation may be employed when partial sand replacement is used.

5.8.2.3—Transfer and Development Lengths

The provisions of Article 5.11.4 shall be considered.

C5.8.2.3

The reduced prestress in the transfer length reduces V_p, f_{pc}, and f_{pe}. The transfer length influences the tensile force that can be resisted by the tendons at the inside edge of the bearing area, as described in Article 5.8.3.5.

5.8.2.4—Regions Requiring Transverse Reinforcement

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

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

  or

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

where:

V_u = factored shear force (kip)

V_c = nominal shear resistance of the concrete (kip)

V_p = component of prestressing force in direction of the shear force; V_p = 0 when the simplified method of 5.8.3.4.3 is used (kip)

ϕ = resistance factor specified in Article 5.5.4.2

C5.8.2.4

Transverse reinforcement, which usually consists of stirrups, is required in all regions where there is a significant chance of diagonal cracking.

5.8.2.5—Minimum Transverse Reinforcement

Except for segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.2.4, the area of steel shall satisfy:

$$A_v \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \tag{5.8.2.5-1}$$

where:

A_v = area of a transverse reinforcement within distance s (in.^{2})

b_v = width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in.)

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi)

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.6.5, the area of transverse reinforcement shall satisfy:

C5.8.2.5

A minimum amount of transverse reinforcement is required to restrain the growth of diagonal cracking and to increase the ductility of the section. A larger amount of transverse reinforcement is required to control cracking as the concrete strength is increased.

Additional transverse reinforcement may be required for transverse web bending.

$$A_v \geq 0.05 \frac{b_w s}{f_y} \quad (5.8.2.5\text{-}2)$$

where:

A_v = area of a transverse shear reinforcement per web within distance s (in.^{2})

b_w = width of web (in.)

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi)

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is not required, as specified in Article 5.8.6.5, the minimum area of transverse shear reinforcement per web shall not be less than the equivalent of two No. 4 Grade 60 reinforcement bars per foot of length.

5.8.2.6—Types of Transverse Reinforcement

Transverse reinforcement to resist shear may consist of:

- Stirrups perpendicular to the longitudinal axis of the member;
- Welded wire reinforcement, with wires located perpendicular to the longitudinal axis of the member, provided that the transverse wires are certified to undergo a minimum elongation of four percent, measured over a gage length of at least 4.0 in. including at least one cross wire;
- Anchored prestressed tendons, detailed and constructed to minimize seating and time-dependent losses, which make an angle not less than 45 degrees with the longitudinal tension reinforcement;
- Combinations of stirrups, tendons, and bent longitudinal bars;
- Spirals or hoops;
- Inclined stirrups making an angle of not less than 45 degrees with the longitudinal tension reinforcement; or
- Bent longitudinal bars in nonprestressed members with the bent portion making an angle of 30 degrees or more with the longitudinal tension reinforcement.

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

C5.8.2.6

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

To increase shear capacity, transverse reinforcement should be capable of undergoing substantial strain prior to failure. Welded wire fabric, particularly if fabricated from small wires and not stress-relieved after fabrication, may fail before the required strain is reached. Such failures may occur at or between the cross-wire intersections.

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

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

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

- Closed stirrups or closed ties, perpendicular to the longitudinal axis of the member, as specified in Article 5.11.2.6.4,
- A closed cage of welded wire reinforcement with transverse wires perpendicular to the longitudinal axis of the member, or
- Spirals or hoops.

5.8.2.7—Maximum Spacing of Transverse Reinforcement

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

- If $v_u < 0.125 f'_c$, then:

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

- If $v_u \geq 0.125 f'_c$, then:

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

where:

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

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

For segmental post-tensioned concrete box girder bridges, spacing of closed stirrups or closed ties required to resist shear effects due to torsional moments shall not exceed one-half of the shortest dimension of the cross-section, nor 12.0 in.

C5.8.2.7

Sections that are highly stressed in shear require more closely spaced reinforcement to provide crack control.

5.8.2.8—Design and Detailing Requirements

Transverse reinforcement shall be anchored at both ends in accordance with the provisions of Article 5.11.2.6. For composite flexural members, extension of beam shear reinforcement into the deck slab may be considered when determining if the development and anchorage provisions of Article 5.11.2.6 are satisfied.

C5.8.2.8

To be effective, the transverse reinforcement should be anchored at each end in a manner that minimizes slip. Fatigue of welded wire reinforcement is not a concern in prestressed members as long as the specially fabricated reinforcement is detailed to have welded joints only in the flanges where shear stress is low.

The design yield strength of nonprestressed transverse reinforcement shall be taken equal to the specified yield strength when the latter does not exceed 60.0 ksi. For nonprestressed transverse reinforcement with yield strength in excess of 60.0 ksi, the design yield strength shall be taken as the stress corresponding to a strain of 0.0035, but not to exceed 75.0 ksi. The design yield strength of prestressed transverse reinforcement shall be taken as the effective stress, after allowance for all prestress losses, plus 60.0 ksi, but not greater than f_{py}.

When welded wire reinforcement is used as transverse reinforcement, it shall be anchored at both ends in accordance with Article 5.11.2.6.3. No welded joints other than those required for anchorage shall be permitted.

Components of inclined flexural compression and/or flexural tension in variable depth members shall be considered when calculating shear resistance.

Some of the provisions of Article 5.8.3 are based on the assumption that the strain in the transverse reinforcement has to attain a value of 0.002 to develop its yield strength. For prestressed tendons, it is the additional strain required to increase the stress above the effective stress caused by the prestress that is of concern. Limiting the design yield strength of nonprestressed transverse reinforcement to 75.0 ksi or a stress corresponding to a strain of 0.0035 provides control of crack widths at service limit state. For reinforcement without a well-defined yield point, the yield strength is determined at a strain of 0.0035 at strength limit state. Research by Griezic (1994), Ma (2000), and Bruce (2003) has indicated that the performance of higher strength steels as shear reinforcement has been satisfactory. Use of relatively small diameter deformed welded wire reinforcement at relatively small spacing, compared to individually field tied reinforcing bars results in improved quality control and improved member performance in service.

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

5.8.2.9—Shear Stress on Concrete

The shear stress on the concrete shall be determined as:

$$v_u = \frac{|V_u - \phi V_p|}{\phi b_v d_v} \tag{5.8.2.9-1}$$

where:

ϕ = resistance factor for shear specified in Article 5.5.4.2

b_v = effective web width taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces due to flexure, or for circular sections, the diameter of the section, modified for the presence of ducts where applicable (in.)

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of 0.9 d_e or 0.72h (in.)

C5.8.2.9

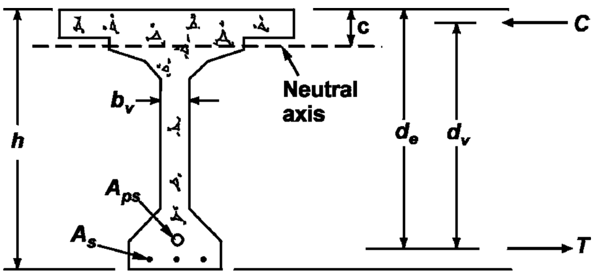


Figure C5.8.2.9-1—Illustration of the Terms b_v and d_v

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

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}} \tag{C5.8.2.9-1}$$

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

in which:

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \quad (5.8.2.9\text{-}2)$$

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

Previous editions of the Standard Specifications permitted d for prestressed members to be taken as $0.8h$. The $0.72h$ limit on d_v is $0.9 \times 0.8h$.

Post-tensioning ducts act as discontinuities and hence, can reduce the crushing strength of concrete webs. In determining which level over the effective depth of the beam has the minimum width, and hence controls b_v, levels which contain a post-tensioning duct or several ducts shall have their widths reduced. Thus, for the section shown in Figure C5.8.2.9-1, the post-tensioning duct in the position shown would not reduce b_v, because it is not at a level where the width of the section is close to the minimum value. If the location of the tendon was raised such that the tendon is located within the narrow portion of the web, the value of b_v would be reduced.

For circular members, such as reinforced concrete columns or prestressed concrete piles, d_v can be determined from Eq. C5.8.2.9-1 provided that M_n is calculated ignoring the effects of axial load and that the reinforcement areas, A_s and A_{ps}, are taken as the reinforcement in one-half of the section. Alternatively, d_v can be taken as $0.9d_e$, where:

$$d_e = \frac{D}{2} + \frac{D_r}{\pi} \quad (C5.8.2.9\text{-}2)$$

where:

D = external diameter of the circular member (in.)

D_r = diameter of the circle passing through the centers of the longitudinal reinforcement (in.)

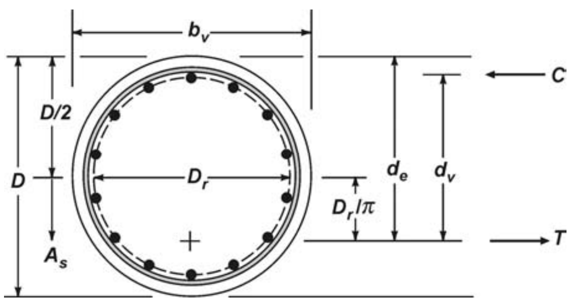


Figure C5.8.2.9-2—Illustration of Terms b_v, d_v, and d_e for Circular Sections

Circular members usually have the longitudinal reinforcement uniformly distributed around the perimeter of the section. When the member cracks, the highest shear stresses typically occur near the middepth of the section. This is also true when the section is not cracked. It is for this reason that the effective web width can be taken as the diameter of the section.

5.8.3—Sectional Design Model

5.8.3.1—General

The sectional design model may be used for shear design where permitted in accordance with the provisions of Article 5.8.1

In lieu of the methods specified herein, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the conditions of equilibrium and compatibility of strains and by using experimentally verified stress-strain relationships for reinforcement and for diagonally cracked concrete. Where consideration of simultaneous shear in a second direction is warranted, investigation shall be based either on the principles outlined above or on a three-dimensional strut-and-tie model.

C5.8.3.1

In the sectional design approach, the component is investigated by comparing the factored shear force and the factored shear resistance at a number of sections along its length. Usually this check is made at the tenth points of the span and at locations near the supports.

See Articles 5.10.11.3 and 5.10.11.4.1c for additional requirements for Seismic Zones 2, 3, and 4 and Articles 5.8.1.2 and 5.8.3.2 for additional requirements for member end regions.

An appropriate nonlinear finite element analysis or a detailed sectional analysis would satisfy the requirements of this Article. More information on appropriate procedures and a computer program that satisfies these requirements are given by Collins and Mitchell (1991). One possible approach to the analysis of biaxial shear and other complex loadings on concrete members is outlined in Rabbat and Collins (1978), and a corresponding computer-aided solution is presented in Rabbat and Collins (1976). A discussion of the effect of biaxial shear on the design of reinforced concrete beam-to-column joints can be found in Paulay and Priestley (1992).

5.8.3.2—Sections Near Supports

The provisions of Article 5.8.1.2 shall be considered.

Where the reaction force in the direction of the applied shear introduces compression into the end region of a member, the location of the critical section for shear shall be taken as d_v from the internal face of the support as illustrated in Figure 5.8.3.2-1.

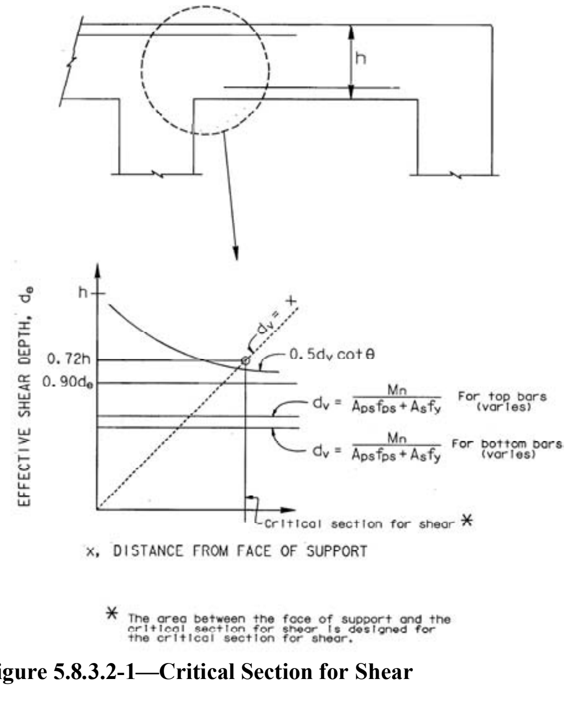


Figure 5.8.3.2-1—Critical Section for Shear

C5.8.3.2

Loads close to the support are transferred directly to the support by compressive arching action without causing additional stresses in the stirrups.

The traditional approach to proportioning transverse reinforcement involves the determination of the required stirrup spacing at discrete sections along the member. The stirrups are then detailed such that this spacing is not exceeded over a length of the beam extending from the design section to the next design section out into the span. In such an approach, the shear demand and resistance provided is assumed to be as shown in Figure C5.8.3.2-1.

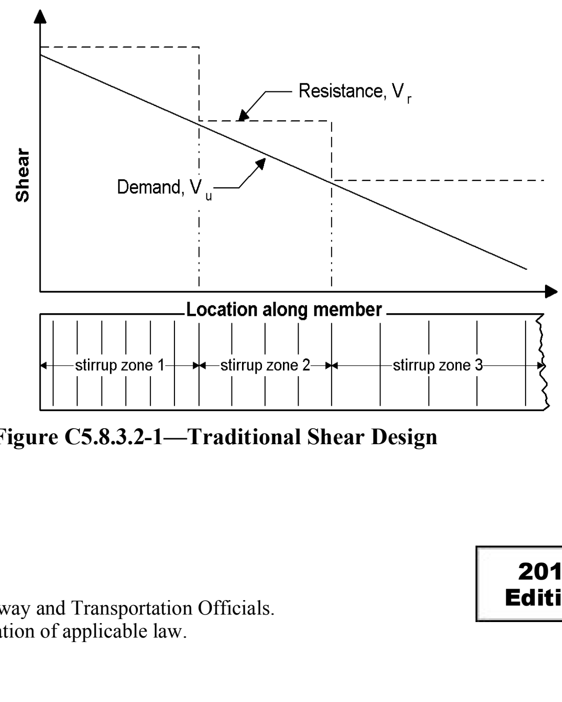


Figure C5.8.3.2-1—Traditional Shear Design

Otherwise, the design section shall be taken at the internal face of the support. Where the beam-type element extends on both sides of the reaction area, the design section on each side of the reaction shall be determined separately based upon the loads on each side of the reaction and whether their respective contribution to the total reaction introduces tension or compression into the end region.

For post-tensioned beams, anchorage zone reinforcement shall be provided as specified in Article 5.10.9. For pretensioned beams, a reinforcement cage confining the ends of strands shall be provided as specified in Article 5.10.10. For nonprestressed beams supported on bearings that introduce compression into the member, only minimal transverse reinforcement may be provided between the inside edge of the bearing plate or pad and the end of the beam.

For typical cases where the applied load acts at or above the middepth of the member, it is more practical to take the traditional approach as shown in Figure C5.8.3.2-1 or a more liberal yet conservative approach as shown in Figure C5.8.3.2-2. The approach taken in Figure C5.8.3.2-2 has the effect of extending the required stirrup spacing for a distance of $0.5d_v \cot \theta$ toward the bearing.

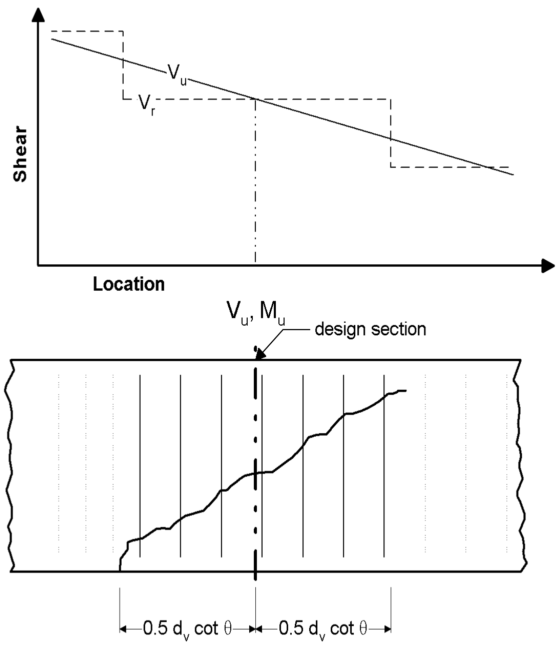


Figure C5.8.3.2-2—Simplified Design Section for Loads Applied at or above the Middepth of the Member

Figure C5.8.3.2-3 shows a case where an inverted T-beam acts as a pier cap and the longitudinal members are supported by the flange of the T. In this case, a significant amount of the load is applied below the middepth of the member, and it is more appropriate to use the traditional approach to shear design shown in Figure C5.8.3.2-1.

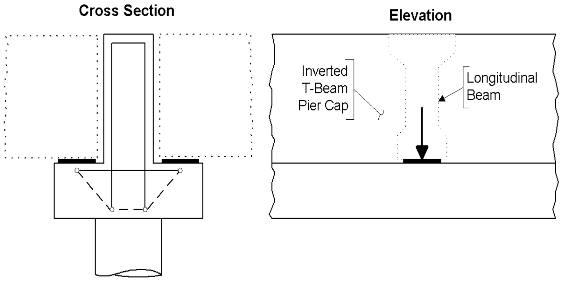


Figure C5.8.3.2-3—Inverted T-Beam Pier Cap

The T-beam pier cap shown in Figure C5.8.3.2-3 acts as a beam ledge and should be designed for the localized effects caused by the concentrated load applied to the T-beam flange. Provisions for beam ledge design are given in Article 5.13.2.5.

If the shear stress at the design section calculated in accordance with Article 5.8.2.9 exceeds $0.18f'_c$ and the beam-type element is not built integrally with the support, its end region shall be designed using the strut-and-tie model specified in Article 5.6.3.

Where a beam is loaded on top and its end is not built integrally into the support, all the shear funnels down into the end bearing. Where the beam has a thin web so that the shear stress in the beam exceeds $0.18\,f'_c$, there is the possibility of a local diagonal compression or horizontal shear failure along the interface between the web and the lower flange of the beam. Usually the inclusion of additional transverse reinforcement cannot prevent this type of failure and either the section size must be increased or the end of the beam designed using a strut-and-tie model.

5.8.3.3—Nominal Shear Resistance

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

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

$$V_n = 0.25\, f'_c b_v d_v + V_p \qquad (5.8.3.3\text{-}2)$$

in which:

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

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

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5.8.3.3\text{-}4)$$

Where transverse reinforcement consists of a single longitudinal bar or a single group of parallel longitudinal bars bent up at the same distance from the support, the shear resistance V_s provided by these bars shall be determined as:

$$V_s = A_v f_y \sin\alpha \le 0.095\sqrt{f'_c}\, b_v d_v \qquad (5.8.3.3\text{-}5)$$

C5.8.3.3

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

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

The upper limit of V_n, given by Eq. 5.8.3.3-2, is intended to ensure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement.

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

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad (C5.8.3.3\text{-}1)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)

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

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

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

α = angle of inclination of transverse reinforcement to longitudinal axis (degrees)

A_v = area of shear reinforcement within a distance s (in.^{2})

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear; V_p = 0 when Article 5.8.3.4.3 is applied (kip)

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

Where bent longitudinal reinforcement is used, only the center three-fourths of the inclined portion of the bent bar shall be considered effective for transverse reinforcement.

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

Where shear resistance is provided by bent longitudinal reinforcement or a combination of bent longitudinal reinforcement and stirrups, the nominal shear resistance shall be determined using the simplified procedure in accordance with Article 5.8.3.4.1.

Requirements for bent bars were added to make the provisions consistent with those in AASHTO (2002).

5.8.3.4—Procedures for Determining Shear Resistance

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

C5.8.3.4

Three complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.8.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in Article 5.8.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load. Two approaches are presented in Method 2: a direct calculation, specified in Article 5.8.3.4.2, and an evaluation using tabularized values presented in

Appendix B5. The approaches to Method 2 may be considered statistically equivalent. Method 3, specified in Article 5.8.3.4.3, is applicable for both prestressed and nonprestressed sections in which there is no net axial tensile load and at least minimum shear reinforcement is provided. Axial load effects can otherwise be accounted for through adjustments to the level of effective precompression stress, f_{pc}. In regions of overlapping applicability between the latter two methods, Method 3 will generally lead to somewhat more shear reinforcement being required, particularly in areas of negative moment and near points of contraflexure. If Method 3 leads to an unsatisfactory rating, it is permissible to use Method 2.

5.8.3.4.1—Simplified Procedure for Nonprestressed Sections

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

$\beta = 2.0$
$\theta = 45°$

C5.8.3.4.1

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

5.8.3.4.2—General Procedure 2010 Revision

The parameters β and θ may be determined either by the provisions herein, or alternatively by the provisions of Appendix B5.

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

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

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

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

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

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

In Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3, ε_s is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement as shown in Figures 5.8.3.4.2-1 and 5.8.3.4.2-2. In lieu of more involved procedures, ε_s may be determined by Eq. 5.8.3.4.2-4:

C5.8.3.4.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.8.3.1. Such an analysis, see Figure C5.8.3.4.2-1, would show that the shear stresses are not uniform over the depth of the web and that the direction of the principal compressive stresses changes over the depth of the beam. The more direct procedure given herein assumes that the concrete shear stresses are uniformly distributed over an area b_v wide and d_v deep, that the direction of principal compressive stresses (defined by angle θ and shown as *D*) remains constant over d_v, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. See Figure C5.8.3.4.2-2.

This design procedure (Collins et al, 1994) was derived from the Modified Compression Field Theory (MCFT, Vecchio, and Collins, 1986) which is a comprehensive behavioral model for the response of diagonally cracked concrete subject to in-plane shear and normal stresses. Prior to the 2008 interim revisions, the General Procedure for shear design was iterative and required the use of tables for the evaluation of β and θ. With the 2008 revisions, this design procedure was modified to be non-iterative and algebraic equations were introduced for the evaluation of β and θ. These equations are functionally equivalent to those used in the Canadian design code (A23.2-M04, 2004), were also derived from the MCFT (Bentz et al. 2006), and were evaluated as

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

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

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

where:

$12.0\,in. \le s_{xe} \le 80.0\ in.$

where:

A_c = area of concrete on the flexural tension side of the member as shown in Figure 5.8.3.4.2-1 (in.^{2})

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure 5.8.3.4.2-1 (in.^{2})

A_s = area of nonprestressed steel on the flexural tension side of the member at the section under consideration, as shown in Figure 5.8.3.4.2-1 (in.^{2})

a_g = maximum aggregate size (in.)

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value of 0.7 f_{pu} will be appropriate for both pretensioned and post-tensioned members

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

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

s_x = the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003b_vs_x$, as shown in Figure 5.8.3.4.2-3 (in.)

V_u = factored shear force (kip)

appropriate for use in the *AASHTO LRFD Bridge Design Specifications* (Hawkins et al., 2006, 2007).

The longitudinal strain, ε_s, can be determined by the procedure illustrated in Figure C5.8.3.4.2-3. The actual section is represented by an idealized section consisting of a flexural tension flange, a flexural compression flange, and a web. The area of the compression flange is taken as the area on the flexure compression side of the member, i.e., the total area minus the area of the tension flange as defined by A_c. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u - V_p$, will primarily be carried by diagonal compressive stresses in the web concrete.
These diagonal compressive stresses will result in a longitudinal compressive force in the web concrete of $(V_u - V_p)$ cot θ. Equilibrium requires that this longitudinal compressive force in the web needs to be balanced by tensile forces in the two flanges, with half the force, that is 0.5 $(V_u - V_p)$ cot θ, being taken by each flange. For simplicity, 0.5 cot θ may be taken as = 2.0 and the longitudinal demand due to shear in the longitudinal tension reinforcement becomes $V_u - V_p$ without significant loss of accuracy. After the required axial forces in the two flanges are calculated, the resulting axial strains, ε_t and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure C5.8.3.4.2-4.

For pretensioned members, f_{po} can be taken as the stress in the strands when the concrete is cast around them, i.e., approximately equal to the jacking stress. For post-tensioned members, f_{po} can be conservatively taken as the average stress in the tendons when the post-tensioning is completed.

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

The flexural tension side of the member shall be taken as the half-depth containing the flexural tension zone, as illustrated in Figure 5.8.3.4.2-1.

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

- $|M_u|$ should not be taken less than $|V_u - V_p| d_v$.
- In calculating A_s and A_{ps} the area of bars or tendons terminated less than their development length from the section under consideration should be reduced in proportion to their lack of full development.
- If the value of ε_s calculated from Eq. 5.8.3.4.2-4 is negative, it should be taken as zero or the value should be recalculated with the denominator of Eq. 5.8.3.4.2-4 replaced by ($E_sA_s + E_pA_{ps} + E_cA_{ct}$). However, ε_s should not be taken as less than -0.40×10^{-3}.
- For sections closer than d_v to the face of the support, the value of ε_s calculated at d_v from the face of the support may be used in evaluating β and θ.
- If the axial tension is large enough to crack the flexural compression face of the section, the value calculated from Eq. 5.8.3.4.2-4 should be doubled.
- It is permissible to determine β and θ from Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3 using a value of ε_s which is greater than that calculated from Eq. 5.8.3.4.2-4 However ε_s should not be taken greater than 6.0×10^{-3}.

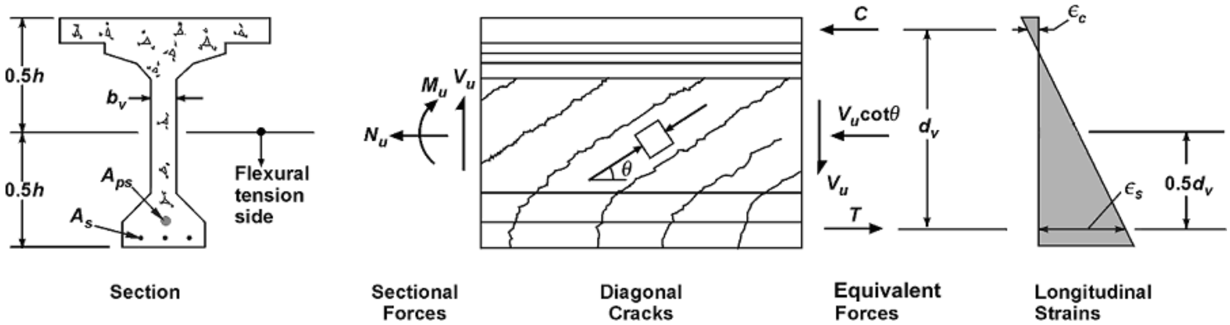


Figure 5.8.3.4.2-1—Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, $V_p = 0$

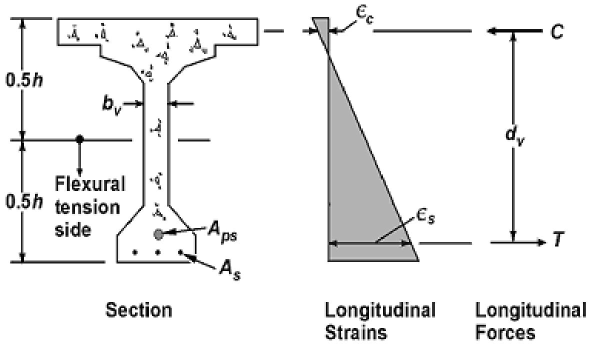


Figure 5.8.3.4.2-2—Longitudinal Strain, ε_s, for Sections Containing Less than the Minimum Amount of Transverse Reinforcement

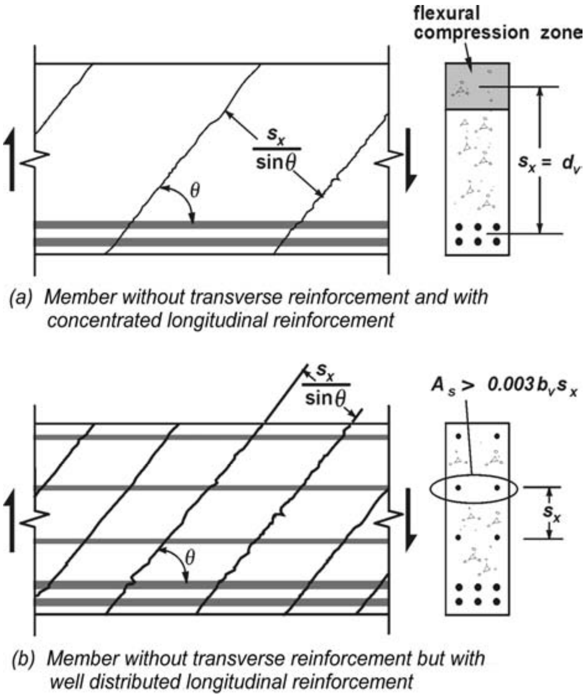


Figure 5.8.3.4.2-3—Definition of Crack Spacing Parameter, s_x

The relationships for evaluating β and θ in Eqs. 5.8.3.4.2-1 and 5.8.3.4.2-2 are based on calculating the stresses that can be transmitted across diagonally cracked concrete. As the cracks become wider, the stress that can be transmitted decreases. For members containing at least the minimum amount of transverse reinforcement, it is assumed that the diagonal cracks will be spaced about 12.0 in. apart. For members without transverse reinforcement, the spacing of diagonal cracks inclined at θ degrees to the longitudinal reinforcement is assumed to be $s_x/\sin\theta$, as shown in Figure 5.8.3.4.2-3. Hence, deeper members having larger values of s_x are calculated to have more widely spaced cracks and hence, cannot transmit such high shear stresses. The ability of the crack surfaces to transmit shear stresses is influenced by the aggregate size of the concrete. Members made from concretes that have a smaller maximum aggregate size will have a larger value of s_{xe} and hence, if there is no transverse reinforcement, will have a smaller shear strength.

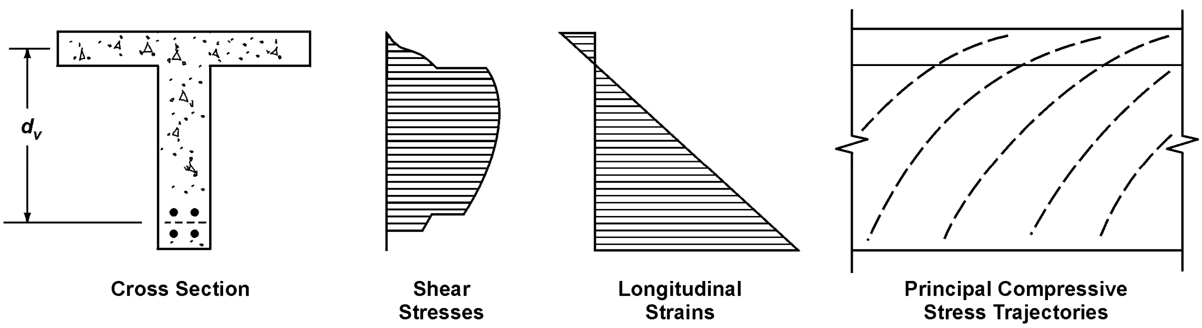


Figure C5.8.3.4.2-1—Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.8.3.1

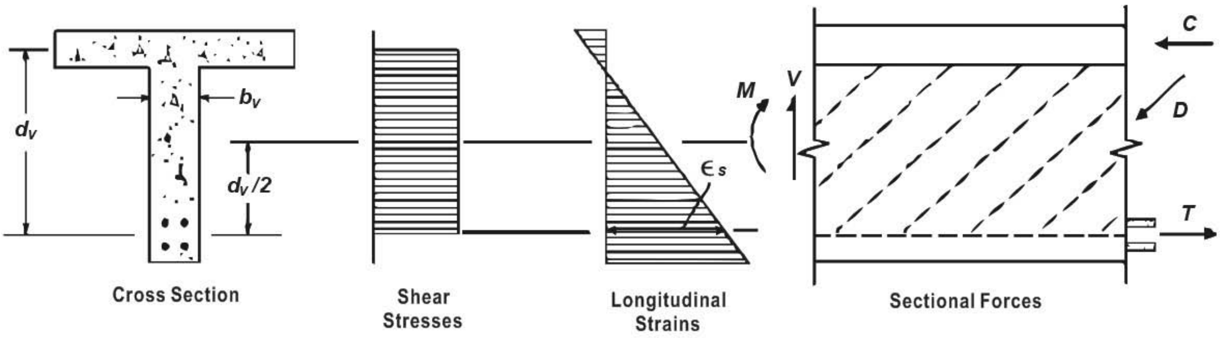


Figure C5.8.3.4.2-2—More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.8.3.4.2

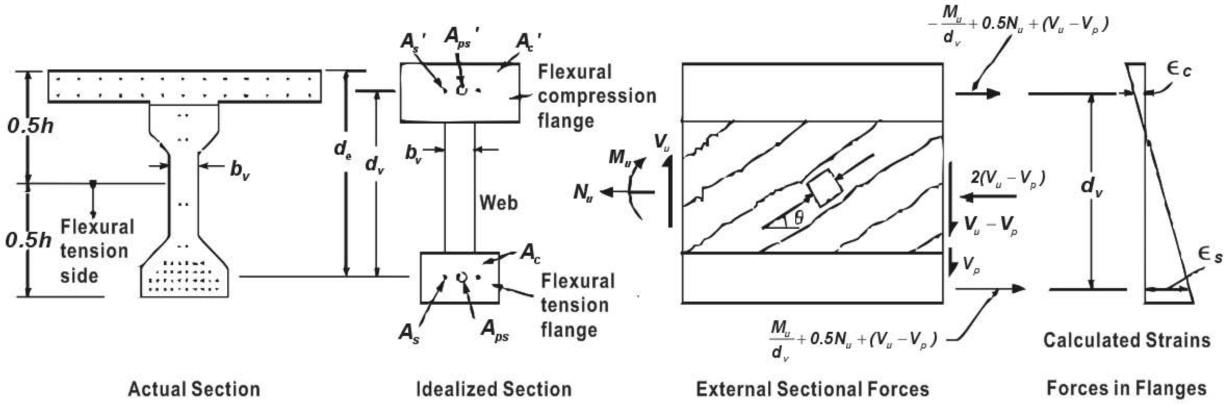


Figure C5.8.3.4.2-3—More Accurate Calculation Procedure for Determining ε_s

5.8.3.4.3—Simplified Procedure for Prestressed and Nonprestressed Sections

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

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

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

V_{ci} shall be determined as:

$$V_{ci} = 0.02\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}b_v d_v \qquad (5.8.3.4.3\text{-}1)$$

C5.8.3.4.3

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

The nominal shear resistance V_n is the sum of the shear resistances V_c and V_s provided by the concrete and shear reinforcement, respectively. Both V_c and V_s depend on the type of inclined cracking that occurs at the given section. There are two types of inclined cracking: flexure-shear cracking and web-shear cracking for which the associated resistances are V_{ci} and V_{cw}, respectively. Figure C5.8.3.4.3-1 shows the development of both types of cracking when increasing uniform load was applied to a 63-in. bulb-tee girder. NCHRP Report XX2 (Hawkins et al., 2005).

where:

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

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

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

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

M_{cre} shall be determined as:

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

where:

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

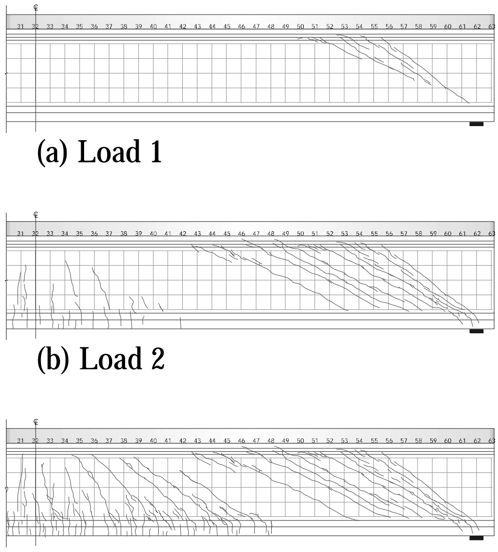
M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

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

V_{cw} shall be determined as:



(c) Load 3

Figure C5.8.3.4.3-1—Development of Shear Cracking with Increasing Loads for Uniformly Loaded Bulb Tee Beam; Load 1 < Load 2 < Load 3

Web-shear cracking begins from an interior point in the web of the member before either flange in that region cracks in flexure. In Figure C5.8.3.4.3-1, at load 1, web-shear cracking developed in the web of the member adjacent to the end support. Flexure-shear cracking is initiated by flexural cracking. Flexural cracking increases the shear stresses in the concrete above the flexural crack. In Figure C5.8.3.4.3-1, flexural cracking had developed in the central region of the beam by load 2 and by load 3, the flexural cracks had become inclined cracks as flexural cracking extended towards the end support with increasing load.

For sections with shear reinforcement equal to or greater than that required by Article 5.8.2.5, the shear carried by the concrete may drop below V_c shortly after inclined cracking, and the shear reinforcement may yield locally. However, sections continue to resist increasing shears until resistances provided by the concrete again reach V_c. Thus, V_{ci} and V_{cw} are measures of the resistance that can be provided by the concrete at the nominal shear resistance of the section and are not directly equal to the shears at inclined cracking.

The angle θ of the inclined crack, and therefore of the diagonal compressive stress, is less for a web-shear crack than a flexure-shear crack. Consequently, for a given section the value of V_s associated with web-shear cracking is greater than that associated with flexure-shear cracking.

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$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30 f_{pc}\right) b_v d_v + V_p \quad (5.8.3.4.3\text{-}3)$$

where:

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

V_s shall be determined using Eq. 5.8.3.3-4 with cot θ taken as follows:

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

$\cot\theta = 1.0$

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

$$\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8 \quad (5.8.3.4.3\text{-}4)$$

V_{ci} is the sum of the shear ($V_i M_{cr} / M_{max}$) required to cause flexural cracking at the given section plus the increment of shear necessary to develop the flexural crack into a shear crack, For a non-composite beam, the total cross section resists all applied shears, dead and live, I_c equals the moment of inertia of the gross section and V_d equals the unfactored dead load shear acting on the section. In this case Eq. 5.8.3.4.3-1 can be used directly.

For a composite beam, part of the dead load is resisted by only part of the final section. Where the final gross concrete section is achieved with only one addition to the initial concrete section (two-stage construction), Eq. 5.8.3.4.3-1 can be used directly. In Eq. 5.8.3.4.3-2 appropriate section properties are used to compute f_d and in Eq. 5.8.3.4.3-1 the shear due to dead load V_d and that due to other loads V_i are separated. V_d is the total shear force due to unfactored dead loads acting on the part of the section carrying the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The term V_i may be taken as $(V_u - V_d)$ and M_{max} as $M_u - M_d$ where V_u and M_u are the factored shear and moment at the given section due to the total factored loads M_d is the moment due to unfactored dead load at the same section.

Where the final gross section is developed with more than one concrete composite addition to the initial section (multiple-stage construction), it is necessary to trace the build up of the extreme fiber flexural stresses to compute M_{cr}. For each stage in the life history of the member, the increments in the extreme fiber flexural stress at the given section due to the unfactored loads acting on that section are calculated using the section properties existing at that stage. V_d, V_i, and M_{max} are calculated in the same manner as for two-stage construction.

A somewhat lower modulus of rupture is used in evaluating M_{cre} by Eq. 5.8.3.4.3-2 to account for the effects of differential shrinkage between the slab and the girder, and the effects of thermal gradients that can occur over the depth of the girder.

5.8.3.5—Longitudinal Reinforcement

At each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5 V_s\right)\cot\theta \quad (5.8.3.5\text{-}1)$$

where:

C5.8.3.5

Shear causes tension in the longitudinal reinforcement. For a given shear, this tension becomes larger as θ becomes smaller and as V_c becomes larger. The tension in the longitudinal reinforcement caused by the shear force can be visualized from a free-body diagram such as that shown in Figure C5.8.3.5-1.

Taking moments about Point 0 in Figure C5.8.3.5-1, assuming that the aggregate interlock force on the crack, which contributes to V_c, has a negligible moment about Point 0, and neglecting the small difference in location of V_u and V_p leads to the requirement for the tension force in the longitudinal reinforcement caused by shear.

V_s = shear resistance provided by the transverse reinforcement at the section under investigation as given by Eq. 5.8.3.3-4, except V_s shall not be taken as greater than V_u/ϕ (kip)

θ = angle of inclination of diagonal compressive stresses used in determining the nominal shear resistance of the section under investigation as determined by Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

ϕ_f, ϕ_v, ϕ_c = resistance factors taken from Article 5.5.4.2 as appropriate for moment, shear and axial resistance

The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or the load introduces direct compression into the flexural compression face of the member.

Eq. 5.8.3.5-1 shall be evaluated where simply-supported girders are made continuous for live loads. Where longitudinal reinforcement is discontinuous, Eq. 5.8.3.5-1 shall be re-evaluated.

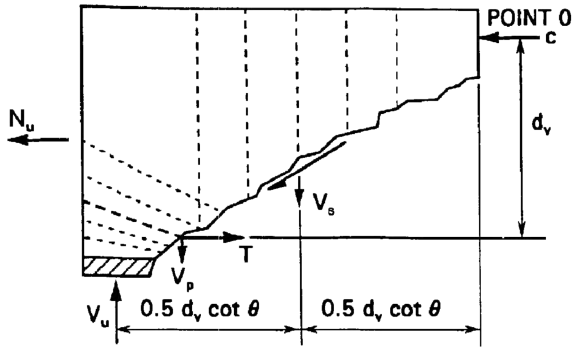


Figure C5.8.3.5-1—Forces Assumed in Resistance Model Caused by Moment and Shear

At maximum moment locations, the shear force changes sign, and hence the inclination of the diagonal compressive stresses changes. At direct supports including simply-supported girder ends and bent/pier caps pinned to columns, and at loads applied directly to the top or bottom face of the member, this change of inclination is associated with a fan-shaped pattern of compressive stresses radiating from the point load or the direct support as shown in Figure C5.8.3.5-2. This fanning of the diagonal stresses reduces the tension in the longitudinal reinforcement caused by the shear; i.e., angle θ becomes steeper. The tension in the reinforcement does not exceed that due to the maximum moment alone. Hence, the longitudinal reinforcement requirements can be met by extending the flexural reinforcement for a distance of $d_v$cotθ or as specified in Article 5.11, whichever is greater.

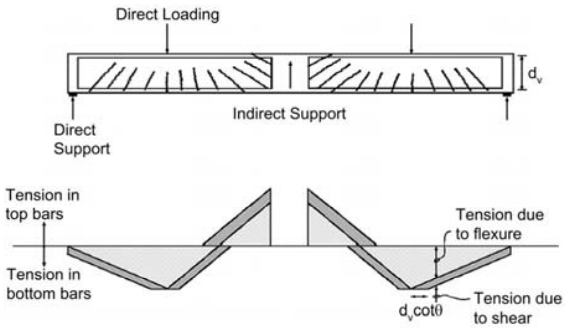


Figure C5.8.3.5-2—Force Variation in Longitudinal Reinforcement Near Maximum Moment Locations

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad (5.8.3.5\text{-}2)$$

Eqs. 5.8.3.5-1 and 5.8.3.5-2 shall be taken to apply to sections not subjected to torsion. Any lack of full development shall be accounted for.

In determining the tensile force that the reinforcement is expected to resist at the inside edge of the bearing area, the values of V_u, V_s, V_p, and θ, calculated for the section d_v from the face of the support may be used. In calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the development length of Article 5.11.2.1.1 or the bi-linear variation of resistance over the transfer and development length of Article 5.11.4.2 may be assumed.

5.8.3.6—Sections Subjected to Combined Shear and Torsion

5.8.3.6.1—Transverse Reinforcement

The transverse reinforcement shall not be less than the sum of that required for shear, as specified in Article 5.8.3.3, and for the concurrent torsion, as specified in Articles 5.8.2.1 and 5.8.3.6.2.

C5.8.3.6.1

The shear stresses due to torsion and shear will add on one side of the section and offset on the other side. The transverse reinforcement is designed for the side where the effects are additive.

Usually the loading that causes the highest torsion differs from the loading that causes the highest shear. Although it is sometimes convenient to design for the highest torsion combined with the highest shear, it is only necessary to design for the highest shear and its concurrent torsion, and the highest torsion and its concurrent shear.

5.8.3.6.2—Torsional Resistance

The nominal torsional resistance shall be taken as:

$$T_n = \frac{2A_o A_t f_y \cot\theta}{s} \qquad (5.8.3.6.2\text{-}1)$$

where:

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

A_t = area of one leg of closed transverse torsion reinforcement in solid members, or total area of transverse torsion reinforcement in the exterior web of cellular members (in.^{2})

θ = angle of crack as determined in accordance with the provisions of Article 5.8.3.4 with the modifications to the expressions for v and V_u herein (degrees)

5.8.3.6.3—Longitudinal Reinforcement

The provisions of Article 5.8.3.5 shall apply as amended, herein, to include torsion.

The longitudinal reinforcement in solid sections shall be proportioned to satisfy Eq. 5.8.3.6.3-1:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5 V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi}\right)^2} \quad (5.8.3.6.3\text{-}1)$$

In box sections, longitudinal reinforcement for torsion, in addition to that required for flexure, shall not be less than:

$$A_\ell = \frac{T_n p_h}{2 A_o f_y} \quad (5.8.3.6.3\text{-}2)$$

where:

p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.)

C5.8.3.6.3

To account for the fact that on one side of the section the torsional and shear stresses oppose each other, the equivalent tension used in the design equation is taken as the square root of the sum of the squares of the individually calculated tensions in the web.

5.8.4—Interface Shear Transfer—Shear Friction

5.8.4.1—General

Interface shear transfer shall be considered across a given plane at:

- An existing or potential crack,
- An interface between dissimilar materials,
- An interface between two concretes cast at different times, or
- The interface between different elements of the cross-section.

C5.8.4.1

Shear displacement along an interface plane may be resisted by cohesion, aggregate interlock, and shear-friction developed by the force in the reinforcement crossing the plane of the interface. Roughness of the shear plane causes interface separation in a direction perpendicular to the interface plane. This separation induces tension in the reinforcement balanced by compressive stresses on the interface surfaces.

Adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/flange interfaces in box girders to transfer flange longitudinal forces at the strength limit state. The factored design force for the interface reinforcement is calculated to account for the interface shear force, ΔF, as shown in Figure C5.8.4.1-1, as well as any localized shear effects due to the prestressing force anchorages at the section.

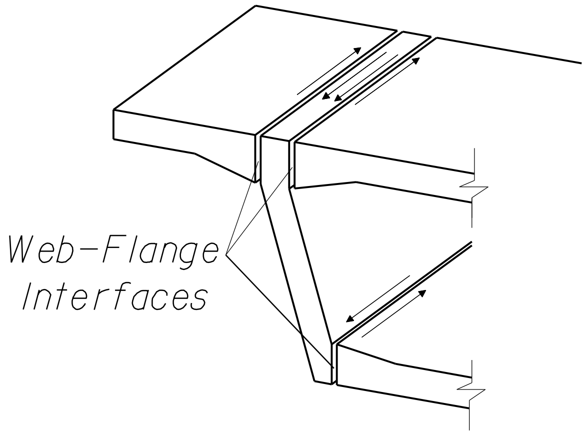


Figure C5.8.4.1-1—Longitudinal Shear Transfer between Flanges and Webs of Box Girder Bridges

Reinforcement for interface shear may consist of single bars, multiple leg stirrups, or welded wire fabric.

All reinforcement present where interface shear transfer is to be considered shall be fully developed on both sides of the interface by embedment, hooks, mechanical methods such as headed studs or welding to develop the design yield stress.

Any reinforcement crossing the interface is subject to the same strain as the designed interface reinforcement. Insufficient anchorage of any reinforcement crossing the interface could result in localized fracture of the surrounding concrete.

When the required interface shear reinforcement in girder/slab design exceeds the area required to satisfy vertical (transverse) shear requirements, additional reinforcement must be provided to satisfy the interface shear requirements. The additional interface shear reinforcement need only extend into the girder a sufficient depth to develop the design yield stress of the reinforcement rather than extending the full depth of the girder as is required for vertical shear reinforcement.

The minimum area of interface shear reinforcement specified in Article 5.8.4.4 shall be satisfied.

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \quad (5.8.4.1\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \quad (5.8.4.1\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)

V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip), and

ϕ = resistance factor for shear specified in Article 5.5.4.2.1. In cases where different weight concretes exist on the two sides of an interface, the lower of the two values of ϕ shall be used.

Total load shall include all noncomposite and composite loads.

For the extreme limit state event ϕ may be taken as 1.0.

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \qquad (5.8.4.1\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}, \text{ or} \qquad (5.8.4.1\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \qquad (5.8.4.1\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \qquad (5.8.4.1\text{-}6)$$

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})

b_{vi} = interface width considered to be engaged in shear transfer (in.)

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in Article 5.8.4.3 (ksi)

μ = friction factor specified in Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

A pure shear friction model assumes interface shear resistance is directly proportional to the net normal clamping force ($A_{vf} f_y + P_c$), through a friction coefficient (μ). Eq. 5.8.4.1-3 is a modified shear-friction model accounting for a contribution, evident in the experimental data, from cohesion and/or aggregate interlock depending on the nature of the interface under consideration given by the first term. For simplicity, the term “cohesion factor” is used throughout the body of this Article to capture the effects of cohesion and/or aggregate interlock such that Eq. 5.8.4.1-3 is analogous to the vertical shear resistance expression of $V_c + V_s$.

Eq. 5.8.4.1-4 limits V_{ni} to prevent crushing or shearing of aggregate along the shear plane.

Eqs. 5.8.4.1-3 and 5.8.4.1-4 are sufficient, with an appropriate value for K_1, to establish a lower bound for the available experimental data; however, Eq. 5.8.4.1-5 is necessitated by the sparseness of available experimental data beyond the limiting K_2 values provided in Article 5.8.4.3.

The interface shear strength Eqs. 5.8.4.1-3, 5.8.4.1-4, and 5.8.4.1-5 are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi to 6.0 ksi; and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

Composite section design utilizing full-depth precast deck panels is not addressed by these provisions. Design specifications for such systems should be established by, or coordinated with, the Owner.

A_{vf} used in Eq. 5.8.4.1-3 is the interface shear reinforcement within the interface area A_{cv}. For a girder/slab interface, the area of the interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.1-3 with $12b_{vi}$ and P_c corresponding to the same one foot of girder length.

In consideration of the use of stay-in-place deck panels, or any other interface details, the Designer shall determine the width of interface, b_{vi}, effectively acting to resist interface shear.

The interface reinforcement is assumed to be stressed to its design yield stress, f_y. However, f_y used in determining the interface shear resistance is limited to 60 ksi because interface shear resistance computed using higher values have overestimated the interface shear resistance experimentally determined in a limited number of tests of pre-cracked specimens.

P_c = permanent net compressive force normal to the shear plane; if force is tensile, $P_c = 0.0$ (kip)

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3.

K_2 = limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

It is conservative to neglect P_c if it is compressive, however, if included, the value of P_c shall be computed as the force acting over the area, A_{cv}. If P_c is tensile, additional reinforcement is required to resist the net tensile force as specified in Article 5.8.4.2.

5.8.4.2—Computation of the Factored Interface Shear Force, V_{ui}, for Girder/Slab Bridges

Based on consideration of a free body diagram and utilizing the conservative envelope value of V_{u1}, the factored interface shear stress for a concrete girder/slab bridge may be determined as:

$$V_{ui} = V_{u1} \div b_{vi} d_v \quad (5.8.4.2\text{-}1)$$

where:

d_v = the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress

The factored interface shear force in kips/ft for a concrete girder/slab bridge may be determined as:

$$V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_v \quad (5.8.4.2\text{-}2)$$

If the net force, P_c, across the interface shear plane is tensile, additional reinforcement, A_{vpc}, shall be provided as:

$$A_{vpc} = P_c \div \phi f_y \quad (5.8.4.2\text{-}3)$$

For beams and girders, the longitudinal spacing of the rows of interface shear transfer reinforcing bars shall not exceed 24.0 in.

C5.8.4.2

The following illustrates a free body diagram approach to computation of interface shear in a girder/slab bridge. In reinforced concrete, or prestressed concrete, girder bridges, with a cast-in-place slab, horizontal shear forces develop along the interface between the girders and the slab. The classical strength of materials approach, which is based on elastic behavior of the section, has been used successfully in the past to determine the design interface shear force. As an alternative to the classical elastic strength of materials approach, a reasonable approximation of the factored interface shear force at the strength or extreme event limit state for either elastic or inelastic behavior and cracked or uncracked sections, can be derived with the defined notation and the free body diagram shown in Figure C5.8.4.2-1 as follows:

M_{u2} = maximum factored moment at section 2

V_1 = the factored vertical shear at section 1 concurrent with M_{u2}

M_1 = the factored moment at section 1 concurrent with M_{u2}

Δl = unit length segment of girder

C_1 = compression force above the shear plane associated with M_1

C_{u2} = compression force above the shear plane associated with M_{u2}

$$M_{u2} = M_1 + V_1 \Delta l \quad (C5.8.4.2\text{-}1)$$

$$C_{u2} = M_{u2} \div d_v \quad (C5.8.4.2\text{-}2)$$

$$C_{u2} = M_1 \div d_v + V_1 \Delta l \div d_v \quad (C5.8.4.2\text{-}3)$$

$$C_1 = M_1 \div d_v \qquad \text{(C5.8.4.2-4)}$$

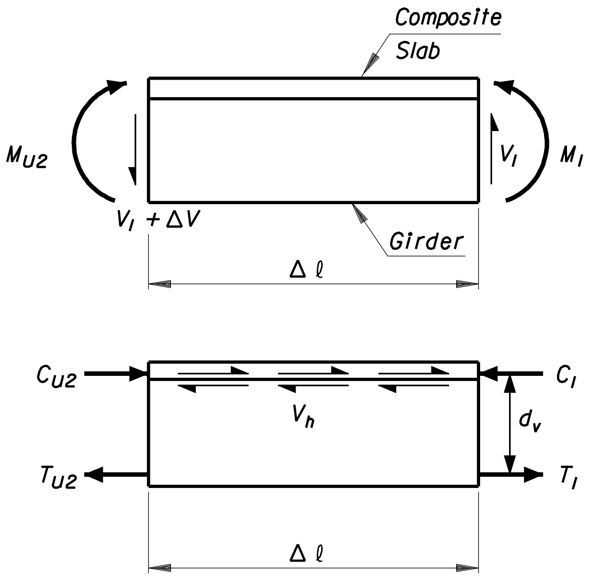


Figure C5.8.4.2-1—Free Body Diagrams

$$V_h = C_{u2} - C_1 \qquad \text{(C5.8.4.2-5)}$$

$$V_h = V_1 \Delta 1 \div d_v \qquad \text{(C5.8.4.2-6)}$$

Such that for a unit length segment:

$$V_{hi} = V_1 \div d_v \qquad \text{(C5.8.4.2-7)}$$

where:

V_{hi} = factored interface shear force per unit length (kips/length)

The variation of V_1 over the length of any girder segment reflects the shear flow embodied in the classical strength of materials approach. For simplicity of design, V_1 can be conservatively taken as V_{u1} (since V_{u1}, the maximum factored vertical shear at section 1, is not likely to act concurrently with the factored moment at section 2); and further, the depth, d_v, can be taken as the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress.

For design purposes, the computed factored interface shear stress of Eq. 5.8.4.2-1 is converted to a resultant interface shear force computed with Eq. 5.8.4.2-1 acting over an area, A_{cv}, within which the computed area of reinforcement, A_{vf}, shall be located. The resulting area of reinforcement, A_{vf}, then defines the area of interface reinforcement required per foot of girder for direct comparison with vertical shear reinforcement requirements.

5.8.4.3—Cohesion and Friction Factors

The following values shall be taken for cohesion, c, and friction factor, μ:

- For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.

c = 0.28 ksi
μ = 1.0
K_1 = 0.3
K_2 = 1.8 ksi for normal-weight concrete
= 1.3 ksi for lightweight concrete

- For normal-weight concrete placed monolithically:

c = 0.40 ksi
μ = 1.4
K_1 = 0.25
K_2 = 1.5 ksi

- For lightweight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.0 ksi

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.5 ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

C5.8.4.3

The values presented provide a lower bound of the substantial body of experimental data available in the literature (Loov and Patnaik, 1994; Patnaik, 1999; Mattock, 2001; Slapkus and Kahn, 2004). Furthermore, the inherent redundancy of girder/slab bridges distinguishes this system from other structural interfaces.

The values presented apply strictly to monolithic concrete. These values are not applicable for situations where a crack may be anticipated to occur at a Service Limit State.

The factors presented provide a lower bound of the experimental data available in the literature (Hofbeck, Ibrahim, and Mattock, 1969; Mattock, Li, and Wang, 1976; Mitchell and Kahn, 2001).

Available experimental data demonstrates that only one modification factor is necessary, when coupled with the resistance factors of Article 5.5.4.2, to accommodate both all-lightweight and sand-lightweight concrete. Note this deviates from earlier specifications that distinguished between all-lightweight and sand-lightweight concrete.

Due to the absence of existing data, the prescribed cohesion and friction factors for nonmonolithic lightweight concrete are accepted as conservative for application to monolithic lightweight concrete.

Tighter constraints have been adopted for roughened interfaces, other than cast-in-place slabs on roughened girders, even though available test data does not indicate more severe restrictions are necessary. This is to account for variability in the geometry, loading and lack of redundancy at other interfaces.

- For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint:

  c = 0.025 ksi
  μ = 0.7
  K_1 = 0.2
  K_2 = 0.8 ksi

For brackets, corbels, and ledges, the cohesion factor, c, shall be taken as 0.0.

Since the effectiveness of cohesion and aggregate interlock along a vertical crack interface is unreliable the cohesion component in Eq. 5.8.4.1-3 is set to 0.0 for brackets, corbels, and ledges.

5.8.4.4—Minimum Area of Interface Shear Reinforcement

Except as provided herein, the cross-sectional area of the interface shear reinforcement, A_{vf}, crossing the interface area, A_{cv}, shall satisfy:

$$A_{vf} \geq \frac{0.05 A_{cv}}{f_y} \qquad (5.8.4.4\text{-}1)$$

For a cast-in-place concrete slab on clean concrete girder surfaces free of laitance, the following provisions shall apply:

- The minimum interface shear reinforcement, A_{vf}, need not exceed the lesser of the amount determined using Eq. 5.8.4.4-1 and the amount needed to resist $1.33V_{ui}/\phi$ as determined using Eq. 5.8.4.1-3.

- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, v_{ui} of Eq. 5.8.4.2-1, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored in the slab.

C5.8.4.4

For a girder/slab interface, the minimum area of interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.4-1 with $12b_{vi}$.

Previous editions of these specifications and of the AASHTO Standard Specifications have required a minimum area of reinforcement based on the full interface area; similar to Eq. 5.8.4.4-1, irrespective of the need to mobilize the strength of the full interface area to resist the applied factored interface shear. In 2006, the additional minimum area provisions, applicable only to girder/slab interfaces, were introduced. The intent of these provisions was to eliminate the need for additional interface shear reinforcement due simply to a beam with a wider top flange being utilized in place of a narrower flanged beam.

The additional provision establishes a rational upper bound for the area of interface shear reinforcement required based on the interface shear demand rather than the interface area as stipulated by Eq. 5.8.4.4-1. This treatment is analogous to minimum reinforcement provisions for flexural capacity where a minimum additional overstrength factor of 1.33 is required beyond the factored demand.

With respect to a girder/slab interface, the intent is that the portion of the reinforcement required to resist vertical shear which is extended into the slab also serves as interface shear reinforcement.

5.8.5—Principal Stresses in Webs of Segmental Concrete Bridges

The provisions specified herein shall apply to all types of segmental bridges with internal and/or external tendons.

The principal tensile stress resulting from the long-term residual axial stress and maximum shear and/or maximum shear combined with shear from torsion stress at the neutral axis of the critical web shall not exceed the tensile stress limit of Table 5.9.4.2.2-1 at the Service III limit state of Article 3.4.1 at all stages during the life of the structure, excluding those during construction. When investigating principal stresses during construction, the tensile stress limits of Table 5.14.2.3.3-1 shall apply.

The principal stress shall be determined using classical beam theory and the principles of Mohr's Circle. The width of the web for these calculations shall be measured perpendicular to the plane of the web.

Compressive stress due to vertical tendons provided in the web shall be considered in the calculation of the principal stress. The vertical force component of draped longitudinal tendons shall be considered as a reduction in the shear force due to the applied loads.

Local tensions produced in webs resulting from anchorage of tendons as discussed in Article 5.10.9.2 shall be included in the principal tension check.

Local transverse flexural stress due to out-of-plane flexure of the web itself at the critical section may be neglected in computing the principal tension in webs.

C5.8.5

This principal stress check is introduced to verify the adequacy of webs of segmental concrete bridges for longitudinal shear and torsion.

5.8.6—Shear and Torsion for Segmental Box Girder Bridges

5.8.6.1—General

Where it is reasonable to assume that plane sections remain plane after loading, the provisions presented herein shall be used for the design of segmental post-tensioned concrete box girder bridges for shear and torsion in lieu of the provisions of Article 5.8.3.

The applicable provisions of Articles 5.8.1, 5.8.2, 5.8.4, and 5.8.5 may apply, as modified by the provisions herein.

Discontinuity regions (where the plane sections assumption of flexural theory is not applicable) shall be designed using the provisions of Article 5.8.6.2 and the strut-and-tie model approach of Article 5.6.3. The provisions of Article 5.13.2 shall apply to special discontinuity regions such as deep beams, brackets and corbels, as appropriate.

The effects of any openings or ducts in members shall be considered. In determining the effective web or flange thickness, b_e, the diameters of ungrouted ducts or one-half the diameters of grouted ducts shall be subtracted from the web or flange thickness at the location of these ducts.

The values of $\sqrt{f'_c}$ used in any part of Article 5.8.6 shall not exceed 3.16.

C5.8.6.1

For types of construction other than segmental box girders, the provisions of Article 5.8.3 may be applied in lieu of the provisions of Article 5.8.6.

Discontinuity regions where the plane sections assumption of flexural theory is not applicable include regions adjacent to abrupt changes in cross-sections, openings, dapped ends, regions where large concentrated loads, reactions, or post-tensioning forces are applied or deviated, diaphragms, deep beams, corbels or joints.

The effects of using concrete with $\sqrt{f'_c} > 3.16$ on the allowable stress limits is not well known.

The design yield strength of transverse shear or torsion reinforcement shall be in accordance with Article 5.8.2.8.

5.8.6.2—Loading

Design for shear and torsion shall be performed at the strength limit state load combinations as defined in Article 3.4.1.

The shear component of the primary effective longitudinal prestress force acting in the direction of the applied shear being examined, V_p, shall be added to the load effect, with a load factor of 1.0.

The secondary shear effects from prestressing shall be included in the *PS* load defined in Article 3.3.2.

The vertical component of inclined tendons shall only be considered to reduce the applied shear on the webs for tendons which are anchored or fully developed by anchorages, deviators, or internal ducts located in the top or bottom 1/3 of the webs.

The effects of factored torsional moments, T_u, shall be considered in the design when their magnitude exceeds the value specified in Article 5.8.6.3.

In a statically indeterminate structure where significant reduction of torsional moment in a member can occur due to redistribution of internal forces upon cracking, the applied factored torsion moment at a section, T_u, may be reduced to ϕT_{cr}, provided that moments and forces in the member and in adjoining members are adjusted to account for the redistribution.

where:

T_u = factored torsional moment (kip-in.)

T_{cr} = torsional cracking moment calculated using Eq. 5.8.6.3-2 (kip-in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

In lieu of a more refined analysis, the torsional loading from a slab may be assumed as linearly distributed along the member.

The effects of axial tension due to creep, shrinkage, and thermal effects in restrained members shall be considered wherever applicable.

C5.8.6.2

Design of prestressed concrete segmental bridges for shear and torsion is based on the strength limit state conditions because little information is available concerning actual shear stress distributions at the service limit state.

This load effect should only be added to the box girder analysis and not transferred into the substructure. Some designers prefer to add this primary prestress force shear component to the resistance side of the equation.

For members subjected to combined shear and torsion, the torsional moments produce shear forces in different elements of the structure that, depending on the direction of torsion, may add to or subtract from the shear force in the element due to vertical shear. Where it is required to consider the effects of torsional moments, the shear forces from torsion need to be added to those from the vertical shear when determining the design shear force acting on a specific element. The possibility of the torsional moment reversing direction should be investigated.

The component of inclined flexural compression or tension, in the direction of the applied shear, in variable depth members shall be considered when determining the design factored shear force.

5.8.6.3—Regions Requiring Consideration of Torsional Effects

For normal weight concrete, torsional effects shall be investigated where:

$$T_u > 1/3\ \phi T_{cr} \qquad (5.8.6.3\text{-}1)$$

in which:

$$T_{cr} = 0.0632 K \sqrt{f'_c}\, 2 A_o b_e \qquad (5.8.6.3\text{-}2)$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\sqrt{f'_c}}} \leq 2.0 \qquad (5.8.6.3\text{-}3)$$

where:

T_u = factored torsional moment (kip-in.)

T_{cr} = torsional cracking moment (kip-in.)

K = stress variable K shall not be taken greater than 1.0 for any section where the stress in the extreme tension fiber, calculated on the basis of gross section properties, due to factored load and effective prestress force after losses exceeds $0.19\sqrt{f'_c}$ in tension.

A_o = area enclosed by the shear flow path of a closed box section, including any holes therein (in.^{2})

b_e = effective width of the shear flow path, but not exceeding the minimum thickness of the webs or flanges comprising the closed box section (in.). b_e shall be adjusted to account for the presence of ducts as specified in Article 5.8.6.1.

p_c = the length of the outside perimeter of the concrete section (in.)

f_{pc} = unfactored compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi)

ϕ = resistance factor for shear specified in Article 5.5.4.2

In lieu of a more refined analysis, b_e may be taken as A_{cp}/P_e, where A_{cp} is the area enclosed by the outside perimeter of the concrete cross-section and P_c is the outside perimeter of the concrete cross-section.

When calculating K for a section subject to factored axial force, N_u, f_{pc} shall be replaced with $f_{pc} - N_u/A_g$. N_u shall be taken as a positive value when the axial force is tensile and negative when it is compressive.

5.8.6.4—Torsional Reinforcement

Where consideration of torsional effects is required by Article 5.8.6.3, torsion reinforcement shall be provided, as specified herein. This reinforcement shall be in addition to the reinforcement required to resist the factored shear, as specified in Article 5.8.6.5, flexure and axial forces that may act concurrently with the torsion.

The longitudinal and transverse reinforcement required for torsion shall satisfy:

$$T_u \leq \phi T_n \qquad (5.8.6.4\text{-}1)$$

The nominal torsional resistance from transverse reinforcement shall be based on a truss model wit 45-degree diagonals and shall be computed as:

$$T_n = \frac{2A_o A_v f_y}{s} \qquad (5.8.6.4\text{-}2)$$

The minimum additional longitudinal reinforcement for torsion, A_ℓ, shall satisfy:

$$A_\ell \geq \frac{T_u\, p_h}{2\phi A_o f_y} \qquad (5.8.6.4\text{-}3)$$

where:

A_v = area of transverse shear reinforcement (in.^{2})

A_ℓ = total area of longitudinal torsion reinforcement in the exterior web of the box girder (in.^{2})

T_u = applied factored torsional moment (kip-in.)

p_h = perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion. p_h may be taken as the perimeter of the centerline of the outermost closed stirrups (in.)

A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2})

f_y = yield strength of additional longitudinal reinforcement (ksi)

C5.8.6.4

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing is taken into account by considering it equivalent to an area of reinforcing steel with a yield force equal to the effective prestressing force.

ϕ = resistance factor for shear specified in Article 5.5.4.2

A_ℓ shall be distributed around the perimeter of the closed stirrups in accordance with Article 5.8.6.6.

Subject to the minimum reinforcement requirements of Article 5.8.6.6, the area of additional longitudinal torsion reinforcement in the flexural compression zone may be reduced by an amount equal to:

$$\frac{M_u}{(0.9 d_e f_y)} \qquad (5.8.6.4\text{-}4)$$

where:

M_u = the factored moment acting at that section concurrent with T_u (kip-in.)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

f_y = specified minimum yield strength of reinforcing bars (ksi)

5.8.6.5—Nominal Shear Resistance

In lieu of the provisions of Article 5.8.3, the provisions herein shall be used to determine the nominal shear resistance of post-tensioned concrete box girders in regions where it is reasonable to assume that plane sections remain plane after loading.

Transverse reinforcement shall be provided when $V_u > 0.5\phi V_c$, where V_c is computed by Eq. 5.8.6.5-4.

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

$$V_n = V_c + V_s \qquad (5.8.6.5\text{-}1)$$

$$V_n = 0.379\sqrt{f'_c}\, b_v d_v \qquad (5.8.6.5\text{-}2)$$

and, where the effects of torsion are required to be considered by Article 5.8.6.2, the cross-sectional dimensions shall be such that:

$$V_c = 0.0632\ K\sqrt{f'_c}\ b_v d_v \qquad (5.8.6.5\text{-}3)$$

in which:

$$V_s = \frac{A_v f_y d_v}{s} \qquad (5.8.6.5\text{-}4)$$

$$\left(\frac{V_u}{b_v\, d_v}\right) + \left(\frac{T_u}{2\,A_o\, b_e}\right) \leq 0.474\sqrt{f'_c} \qquad (5.8.6.5\text{-}5)$$

C5.8.6.5

The expression for V_c has been checked against a wide range of test data and has been found to be a conservative expression.

Eq. 5.8.6.5-4 is based on an assumed 45-degree truss model.

Eq. 5.8.6.4-5 is only used to establish appropriate concrete section dimensions.

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.6.1 (in.)

d_v = 0.8h or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.)

s = spacing of stirrups (in.)

K = stress variable computed in accordance with Article 5.8.6.3.

A_v = area of shear reinforcement within a distance s (in.^{2})

V_u = factored design shear including any normal component from the primary prestressing force (kip)

T_u = applied factored torsional moment (kip-in.)

A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2})

b_e = the effective thickness of the shear flow path of the elements making up the space truss model resisting torsion calculated in accordance with Article 5.8.6.3 (in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

The factored nominal shear resistance, ϕV_n, shall be greater than or equal to V_u.

The applied factored shear, V_u, in regions near supports may be computed at a distance h/2 from the support when the support reaction, in the direction of the applied shear, introduces compression into the support region of the member and no concentrated load occurs within a distance, h, from the face of the support.

5.8.6.6—Reinforcement Details

In addition to the provisions herein, the provisions of Article 5.10 and 5.11 shall also apply to segmental post-tensioned box girders, as applicable.

At any place on the cross-section where the axial tension due to torsion and bending exceeds the axial compression due to prestressing and bending, either supplementary tendons to counter the tension or local longitudinal reinforcement, which is continuous across the joints between segments, shall be required.

Where supplementary tendons are added, they shall be located to provide compression around the perimeter of the closed box section.

Where local longitudinal reinforcement is added, the bars shall be distributed around the perimeter formed by the closed stirrups. Perimeter bar spacing shall not exceed 18.0 in. At least one longitudinal bar shall be placed in each corner of the stirrups. The minimum diameter of the corner bars shall be 1/24 of the stirrup spacing but no less than that of a #5 bar.

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

- If $v_u < 0.19\sqrt{f'_c}$, then:

$$s_{max} = 0.8d \leq 36.0 \text{ in.} \tag{5.8.6.6-1}$$

- If $v_u \geq 0.19\sqrt{f'_c}$, then:

$$s_{max} = 0.4d \leq 18.0 \text{ in.} \tag{5.8.6.6-2}$$

where:

v_u = the shear stress calculated in accordance with Eq. 5.8.6.5-5 (ksi)

d_v = effective shear depth as defined in Article 5.8.6.5 (in.)

Transverse reinforcement for shear and torsion shall be provided for a distance at least $h/2$ beyond the point they are theoretically required.

Interface shear transfer reinforcement shall be provided as specified in Article 5.8.4.

5.9—PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1—General Design Considerations

5.9.1.1—General

The provisions herein specified shall apply to structural concrete members reinforced with any combination of prestressing tendons and conventional reinforcing bars acting together to resist common force effects. Prestressed and partially prestressed concrete structural components shall be designed for both initial and final prestressing forces. They shall satisfy the requirements at service, fatigue, strength, and extreme event limit states, as specified in Article 5.5, and in accordance with the assumptions provided in Articles 5.6, 5.7, and 5.8.

Unstressed prestressing tendons or reinforcing bars may be used in combination with stressed tendons, provided it is shown that performance of the structure satisfies all limit states and the requirements of Articles 5.4 and 5.6.

Compressive stress limits, specified in Article 5.9.4, shall be used with any applicable service load combination in Table 3.4.1-1, except Service Load Combination III, which shall not apply to the investigation of compression.

C5.9.1.1

The introduction of partial prestressing permits the development of a unified theory of concrete structures in which conventional reinforced and prestressed concrete become boundary cases.

The background material in this Article is based on previous editions of the Standard Specifications and ACI 343, ACI 318, and the *Ontario Highway Bridge Design Code*, the provisions of which are extended herein to accommodate partial prestressing.

Prestressing tendons of high-strength steel bars or strands are generally used, but other materials satisfying desired strength, stiffness, and ductility requirements could also be used, provided that they meet the intent of Article 5.4.1.

Partial prestressing can be considered a design concept that allows one or a combination of the following design solutions:

- A concrete member reinforced with a combination of prestressed and nonprestressed reinforcement designed to simultaneously resist the same force effects, Tensile stress limits, specified in Article 5.9.4, shall be used with any applicable service load combination in Table 3.4.1-1. Service Load Combination III shall apply when investigating tension under live load.

- A prestressed concrete member designed to crack in tension under service load, and
- A prestressed concrete member in which the effective prestress in the prestressed reinforcement is purposely kept lower than its maximum allowable value.

5.9.1.2—Specified Concrete Strengths

The specified strengths, f'_c and f'_{ci}, shall be identified in the contract documents for each component. Stress limits relating to specified strengths shall be as specified in Article 5.9.4.

Concrete strength at transfer shall be adequate for the requirements of the anchorages or for transfer through bond as well as for camber or deflection requirements.

5.9.1.3—Buckling

Buckling of a member between points where concrete and tendons are in contact, buckling during handling and erection, and buckling of thin webs and flanges shall be investigated.

5.9.1.4—Section Properties

For section properties prior to bonding of post-tensioning tendons, effects of loss of area due to open ducts shall be considered.

For both pretensioned or post-tensioned members after bonding of tendons, section properties may be based on either the gross or transformed section.

C5.9.1.4

Bonding means that the grout in the duct has attained its specified strength.

5.9.1.5—Crack Control

Where cracking is permitted under service loads, crack width, fatigue of reinforcement, and corrosion considerations shall be investigated in accordance with the provisions of Articles 5.5, 5.6, and 5.7.

5.9.1.6—Tendons with Angle Points or Curves

The provisions of Article 5.4.6 for the curvature of ducts shall apply.

The provisions of Article 5.10.4 shall apply to the investigation of stress concentrations due to changes in the direction of prestressing tendons.

For tendons in draped ducts that are not nominally straight, consideration shall be given to the difference between the center of gravity of the tendon and the center of gravity of the duct when determining eccentricity.

C5.9.1.6

Vertically draped strand tendons should be assumed to be at the bottom of the duct in negative moment areas and at the top of the duct in positive moment areas. The location of the tendon center of gravity, with respect to the centerline of the duct, is shown for negative moment in Figure C5.9.1.6-1.

Duct Size (IN)	Z (IN)
3 OD and Less	1/2
Over 3 OD to 4	3/4
Over 4 OD	1

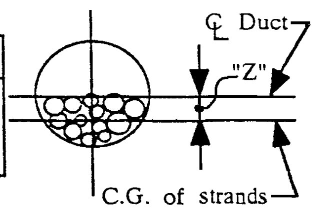


Figure C5.9.1.6-1—Location of Tendon in Duct

5.9.2—Stresses Due to Imposed Deformation

The effects on adjoining elements of the structure of elastic and inelastic deformations due to prestressing shall be investigated. The restraining forces produced in the adjoining structural elements may be reduced due to the effects of creep.

In monolithic frames, force effects in columns and piers resulting from prestressing the superstructure may be based on the initial elastic shortening.

For conventional monolithic frames, any increase in column moments due to long-term creep shortening of the prestressed superstructure is considered to be offset by the concurrent relaxation of deformation moments in the columns due to creep in the column concrete.

The reduction of restraining forces in other members of a structure that are caused by the prestress in a member may be taken as:

- For suddenly imposed deformations

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right), \text{ or} \qquad (5.9.2\text{-}1)$$

- For slowly imposed deformations

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right) / \psi\left(t, t_i\right) \qquad (5.9.2\text{-}2)$$

where:

F = force effect determined using the modulus of elasticity of the concrete at the time loading is applied (kip)

F' = reduced force effect (kip)

$\Psi(t, t_i)$ = creep coefficient at time t for loading applied at time t_i as specified in Article 5.4.2.3.2

e = base of Napierian logarithms

C5.9.2

Additional information is contained in Leonhardt (1964).

5.9.3—Stress Limitations for Prestressing Tendons

The tendon stress due to prestress or at the service limit state shall not exceed the values:

- Specified in Table 5.9.3-1, or
- Recommended by the manufacturer of the tendons or anchorages.

The tendon stress at the strength and extreme event limit states shall not exceed the tensile strength limit specified in Table 5.4.4.1-1.

C5.9.3

For post-tensioning, the short-term allowable of $0.90f_{py}$ may be allowed for short periods of time prior to seating to offset seating and friction losses, provided that the other values in Table 5.9.3-1 are not exceeded.

Table 5.9.3-1—Stress Limits for Prestressing Tendons

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Tendon Type: Low Relaxation Strand	Tendon Type: Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70f_{pu}$	$0.75f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$
At anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.70f_{pu}$	$0.70f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.74f_{pu}$	$0.70f_{pu}$
At service limit state after losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

5.9.4—Stress Limits for Concrete

5.9.4.1—For Temporary Stresses before Losses—Fully Prestressed Components

5.9.4.1.1—Compression Stresses

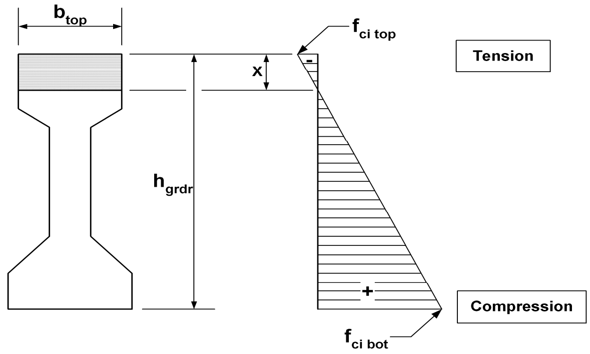
The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0.60 f'_{ci} (ksi).

5.9.4.1.2—Tension Stresses

The limits in Table 5.9.4.1.2-1 shall apply for tensile stresses.

C5.9.4.1.2

Where bonded reinforcement is provided to allow use of the increased tensile limiting stress in areas with bonded reinforcement, the tensile force must be computed. The first step in computing the tensile force, T, is to determine the depth of the tensile zone using the extreme fiber stresses at the location being considered, $f_{ci\ top}$ and $f_{ci\ bot}$. An area is then defined over which the average tensile stress is assumed to act. The tensile force is computed as the product of the average tensile stress and the computed area, as illustrated below. The required area of reinforcement, A_s, is computed by dividing the tensile force by the permitted stress in the reinforcement.



$$T = \frac{f_{ci\ top}}{2} b_{top} x$$

$$A_s = \frac{T}{f_s}$$

where $f_s = 0.5\, f_y \leq 30$ ksi

Figure C5.9.4.1.2-1—Calculation of Tensile Force and Required Area of Reinforcement

Table 5.9.4.1.2-1—Temporary Tensile Stress Limits in Prestressed Concrete before Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	• In precompressed tensile zone without bonded reinforcement	N/A
	• In areas other than the precompressed tensile zone and without bonded reinforcement	$0.0948\sqrt{f'_{ci}} \leq 0.2$ (ksi)
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi.	$0.24\sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	$0.158\sqrt{f'_{ci}}$ (ksi)
Segmentally Constructed Bridges	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5f_y$; with internal tendons or external tendons	$0.0948\sqrt{f'_{ci}}$ maximum tension (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through the joints	No tension
	Transverse Stresses through Joints	
	• For any type of joint	$0.0948\sqrt{f'_{ci}}$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded nonprestressed reinforcement	No tension
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5\ f_y$, not to exceed 30 ksi.	$0.19\sqrt{f'_{ci}}$ (ksi)
	Principal Tensile Stress at Neutral Axis in Web	
	• All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures	$0.110\sqrt{f'_{ci}}$ (ksi)

5.9.4.2—For Stresses at Service Limit State after Losses—Fully Prestressed Components

5.9.4.2.1—Compression Stresses

Compression shall be investigated using the Service Limit State Load Combination I specified in Table 3.4.1-1. The limits in Table 5.9.4.2.1-1 shall apply.

C5.9.4.2.1

The reduction factor, ϕ_w, shall be taken to be equal to 1.0 when the web and flange slenderness ratios, calculated according to Article 5.7.4.7.1, are not greater than 15. When either the web or flange slenderness ratio is greater than 15, the reduction factor, ϕ_w, shall be calculated according to Article 5.7.4.7.2.

Unlike solid rectangular beams that were used in the development of concrete design codes, the unconfined concrete of the compression sides of box girders are expected to creep to failure at a stress far lower than the nominal strength of the concrete. This behavior is similar to the behavior of the concrete in thin-walled columns. The reduction factor, ϕ_w, was originally developed to account for the reduction in the usable strain of concrete in thin-walled columns at the strength limit state. The use of ϕ_w to reduce the stress limit in box girders at the service limit state is not theoretically correct. However, due to the lack of information about the behavior of the concrete at the service limit state, the use of ϕ_w provides a rational approach to account for the behavior of thin components.

The application of Article 5.7.4.7.2 to flanged, strutted, and variable thickness elements requires some judgment. Consideration of appropriate lengths of wall-type element is illustrated in Figure C5.9.4.2.1-1. For constant thickness lengths, the wall thickness associated with that length should be used. For variable thickness lengths, e.g., L_4, an average thickness could be used. For multilength components, such as the top flange, the highest ratio should be used. The beneficial effect of support by struts should be considered. There are no effective length factors shown. The free edge of the cantilever overhang is assumed to be supported by the parapet in Figure C5.9.4.2.1-1.

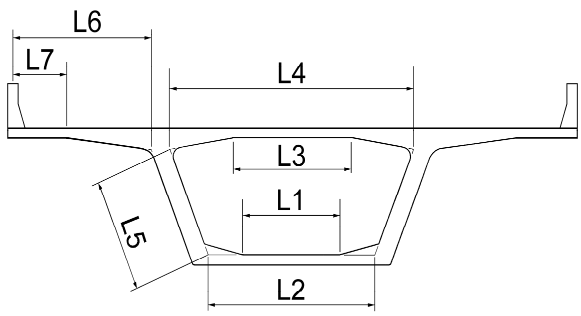


Figure C5.9.4.2.1-1—Suggested Choices for Wall Lengths to be Considered

Table 5.9.4.2.1-1—Compressive Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components

Location	Stress Limit
• In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45f'_c$ (ksi)
• In segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45f'_c$ (ksi)
• Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0.60\ \phi_w f'_c$ (ksi)

5.9.4.2.2—Tension Stresses

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

The limits in Table 5.9.4.2.2-1 shall apply.

C5.9.4.2.2

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

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

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

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.19\sqrt{f'_c}$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.0948\sqrt{f'_c}$ (ksi)
	• For components with unbonded prestressing tendons	No tension
Segmentally Constructed Bridges	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 fy; internal tendons or external tendons	$0.0948\sqrt{f'_c}$ (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through joints	No tension
	Transverse Stresses through Joints	
	• Tension in the transverse direction in precompressed tensile zone	$0.0948\sqrt{f'_c}$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded reinforcement	No tension
	• In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5 f_y$, not to exceed 30 ksi	$0.19\sqrt{f'_c}$ (ksi)
	Principal Tensile Stress at Neutral Axis in Web	
	• All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures.	$0.110\sqrt{f'_c}$ (ksi)

5.9.4.3—Partially Prestressed Components

Compression stresses shall be limited as specified in Articles 5.9.4.1.1 and 5.9.4.2.1 for fully prestressed components.

Cracking in the precompressed tensile zone may be permitted. The design of partially prestressed members should be based on a cracked section analysis with various service limit states being satisfied. Tensile stress in reinforcement at the service limit state shall be as specified in Article 5.7.3.4, in which case f_s shall be interpreted as the change in stress after decompression.

5.9.5—Loss of Prestress

5.9.5.1—Total Loss of Prestress

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

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

- In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \tag{5.9.5.1-1}$$

- In post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \tag{5.9.5.1-2}$$

where:

Δf_{pT} = total loss (ksi)

Δf_{pF} = loss due to friction (ksi)

Δf_{pA} = loss due to anchorage set (ksi)

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)

C5.9.5.1

For segmental construction, lightweight concrete construction, multi-stage prestressing, and bridges where more exact evaluation of prestress losses is desired, calculations for loss of prestress should be made in accordance with a time-step method supported by proven research data. See references cited in Article C5.4.2.3.2.

Data from control tests on the materials to be used, the methods of curing, ambient service conditions, and pertinent structural details for the construction should be considered.

Accurate estimate of total prestress loss requires recognition that the time-dependent losses resulting from creep, shrinkage, and relaxation are also interdependent. However, undue refinement is seldom warranted or even possible at the design stage because many of the component factors are either unknown or beyond the control of the Designer.

Losses due to anchorage set, friction, and elastic shortening are instantaneous, whereas losses due to creep, shrinkage, and relaxation are time-dependent.

This Article has been revised on the basis of new analytical investigations. The presence of a substantial amount of nonprestressed reinforcement, such as in partially prestressed concrete, influences stress redistribution along the section due to creep of concrete with time, and generally leads to smaller loss of prestressing steel pretension and larger loss of concrete precompression.

The loss across stressing hardware and anchorage devices has been measured from two to six percent (Roberts, 1993) of the force indicated by the ram pressure times the calibrated ram area. The loss varies depending on the ram and the anchor. An initial design value of three percent is recommended.

The extension of the provisions to 15.0 ksi was based on Tadros (2003), which only included normal weight concrete. Consequently, the extension to 15.0 ksi is only valid for members made with normal weight concrete.

5.9.5.2—Instantaneous Losses

5.9.5.2.1—Anchorage Set

The magnitude of the anchorage set shall be the greater of that required to control the stress in the prestressing steel at transfer or that recommended by the manufacturer of the anchorage. The magnitude of the set assumed for the design and used to calculate set loss shall be shown in the contract documents and verified during construction.

C5.9.5.2.1

Anchorage set loss is caused by the movement of the tendon prior to seating of the wedges or the anchorage gripping device. The magnitude of the minimum set depends on the prestressing system used. This loss occurs prior to transfer and causes most of the difference between jacking stress and stress at transfer. A common value for anchor set is 0.375 in., although values as low as 0.0625 in. are more appropriate for some anchorage devices, such as those for bar tendons.

For wedge-type strand anchors, the set may vary between 0.125 in. and 0.375 in., depending on the type of equipment used. For short tendons, a small anchorage seating value is desirable, and equipment with power wedge seating should be used. For long tendons, the effect of anchorage set on tendon forces is insignificant, and power seating is not necessary. The 0.25-in. anchorage set value, often assumed in elongation computations, is adequate but only approximate.

Due to friction, the loss due to anchorage set may affect only part of the prestressed member.

Losses due to elastic shortening may also be calculated in accordance with Article 5.9.5.2.3 or other published guidelines (PCI 1975; Zia et. al. 1979). Losses due to elastic shortening for external tendons may be calculated in the same manner as for internal tendons.

5.9.5.2.2—Friction

5.9.5.2.2a—Pretensioned Construction

For draped prestressing tendons, losses that may occur at the hold-down devices should be considered.

5.9.5.2.2b—Post-Tensioned Construction

Losses due to friction between the internal prestressing tendons and the duct wall may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right) \tag{5.9.5.2.2b-1}$$

Losses due to friction between the external tendon across a single deviator pipe may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha + 0.04)}\right) \tag{5.9.5.2.2b-2}$$

where:

f_{pj} = stress in the prestressing steel at jacking (ksi)

x = length of a prestressing tendon from the jacking end to any point under consideration (ft)

C5.9.5.2.2b

Where large discrepancies occur between measured and calculated tendon elongations, in-place friction tests are required.

The 0.04 radians in Eq. 5.9.5.2.2b-2 represents an inadvertent angle change. This angle change may vary depending on job-specific tolerances on deviator pipe placement and need not be applied in cases where the deviation angle is strictly controlled or precisely known, as in the case of continuous ducts passing through separate longitudinal bell-shaped holes at deviators. The inadvertent angle change need not be considered for calculation of losses due to wedge seating movement.

K = wobble friction coefficient (per ft of tendon)

μ = coefficient of friction

α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad.)

e = base of Napierian logarithms

Values of K and μ should be based on experimental data for the materials specified and shall be shown in the contract documents. In the absence of such data, a value within the ranges of K and μ as specified in Table 5.9.5.2.2b-1 may be used.

For slender members, the value of x may be taken as the projection of the tendon on the longitudinal axis of the member. A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes. See also Article C5.14.2.3.7 for further discussion of friction and wobble coefficients.

For tendons confined to a vertical plane, α shall be taken as the sum of the absolute values of angular changes over length x.

α_v and α_h may be taken as the sum of absolute values of angular changes over length, x, of the projected tendon profile in the vertical and horizontal planes, respectively.

The scalar sum of α_v and α_h may be used as a first approximation of α.

For tendons curved in three dimensions, the total tridimensional angular change α shall be obtained by vectorially adding the total vertical angular change, α_v, and the total horizontal angular change, α_h.

When the developed elevation and plan of the tendons are parabolic or circular, the α can be computed from:

$$\alpha = \sqrt{\alpha_v^2 + \alpha_h^2} \qquad \text{(C5.9.5.2.2b-1)}$$

When the developed elevation and the plan of the tendon are generalized curves, the tendon may be split into small intervals, and the above formula can be applied to each interval so that:

$$\alpha = \Sigma\Delta\alpha = \Sigma\sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2} \qquad \text{(C5.9.5.2.2b-2)}$$

As an approximation, the tendon may be replaced by a series of chords connecting nodal points. The angular changes, $\Delta\alpha_v$ and $\Delta\alpha_h$, of each chord may be obtained from its slope in the developed elevation and in plan.

Field tests conducted on the external tendons of a segmental viaduct in San Antonio, Texas, indicate that the loss of prestress at deviators is higher than the usual friction coefficient ($\mu = 0.25$) would estimate.

This additional loss appears to be due, in part, to the tolerances allowed in the placement of the deviator pipes. Small misalignments of the pipes can result in significantly increased angle changes of the tendons at the deviation points. The addition of an inadvertent angle change of 0.04 radians to the theoretical angle change accounts for this effect based on typical deviator length of 3.0 ft and placement tolerance of ±3/8 in. The 0.04 value is to be added to the theoretical value at each deviator. The value may vary with tolerances on pipe placement.

The measurements also indicated that the friction across the deviators was higher during the stressing operations than during the seating operations.

See Podolny (1986) for a general development of friction loss theory for bridges with inclined webs and for horizontally curved bridges.

Table 5.9.5.2.2b-1—Friction Coefficients for Post-Tensioning Tendons

Type of Steel	Type of Duct	*K*	μ
Wire or strand	Rigid and semirigid galvanized metal sheathing	0.0002	0.15–0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
High-strength bars	Galvanized metal sheathing	0.0002	0.30

5.9.5.2.3—Elastic Shortening

5.9.5.2.3a—Pretensioned Members

The loss due to elastic shortening in pretensioned members shall be taken as:

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \qquad (5.9.5.2.3a\text{-}1)$$

where:

f_{cgp} = the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

C5.9.5.2.3a

Changes in prestressing steel stress due to the elastic deformations of the section occur at all stages of loading. Historically, it has been conservative to account for this effect implicitly in the calculation of elastic shortening and creep losses considering only the prestress force present after transfer.

The change in prestressing steel stress due to the elastic deformations of the section may be determined for any load applied. The resulting change may be a loss, at transfer, or a gain, at time of superimposed load application. Where a more detailed analysis is desired, Eq. 5.9.5.2.3a-1 may be used at each section along the beam, for the various loading conditions.

In calculating f_{cgp}, using gross (or net) cross-section properties, it may be necessary to perform a separate calculation for each different elastic deformation to be included. For the combined effects of initial prestress and member weight, an initial estimate of prestress after transfer is used. The prestress may be assumed to be 90 percent of the initial prestress before transfer and the analysis iterated until acceptable accuracy is achieved. To avoid iteration altogether, Eq. C5.9.5.2.3a-1 may be used for the initial section. If the inclusion of an elastic gain due to the application of the deck weight is desired, the change in prestress force can be directly calculated. The same is true for all other elastic gains with appropriate consideration for composite sections.

The total elastic loss or gain may be taken as the sum of the effects of prestress and applied loads.

When calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for and Δf_{pES} should not be included in the prestressing force applied to the transformed section at transfer. Nevertheless, the effective prestress in the strands can be determined by subtracting losses (elastic and time-dependent) from the jacking stress. In other words, when using transformed section properties, the prestressing strand and the concrete are treated together as a composite section in which both the concrete and the prestressing strand are equally strained in compression by a prestressing force conceived as a fictitious external load applied at the level of the strands. To determine the effective stress in the prestressing strands (neglecting time-dependent losses for simplicity) the sum of the Δf_{pES} values considered must be included. In contrast, analysis with gross (or net) section properties involves using the effective stress in the strands at any given stage of loading to determine the prestress force and resulting concrete stresses.

The loss due to elastic shortening in pretensioned members may be determined by the following alternative equation:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}} \tag{C5.9.5.2.3a-1}$$

where:

A_{ps} = area of prestressing steel (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

e_m = average prestressing steel eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

I_g = moment of inertia of the gross concrete section (in.^{4})

M_g = midspan moment due to member self-weight (kip-in.)

5.9.5.2.3b—Post-Tensioned Members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as:

$$\Delta f_{pES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \qquad (5.9.5.2.3b\text{-}1)$$

where:

N = number of identical prestressing tendons

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force after jacking and the self-weight of the member at the sections of maximum moment (ksi)

f_{cgp} values may be calculated using a steel stress reduced below the initial value by a margin dependent on elastic shortening, relaxation, and friction effects.

For post-tensioned structures with bonded tendons, f_{cgp} may be taken at the center section of the span or, for continuous construction, at the section of maximum moment.

For post-tensioned structures with unbonded tendons, the f_{cgp} value may be calculated as the stress at the center of gravity of the prestressing steel averaged along the length of the member.

For slab systems, the value of Δf_{pES} may be taken as 25 percent of that obtained from Eq. 5.9.5.2.3b-1.

C5.9.5.2.3b

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be determined by the following alternative equation:

$$\Delta f_{pES} = \frac{N-1}{2N}\,\frac{A_{ps} f_{pbt}(I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps}(I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

(C5.9.5.2.3b-1)

where:

A_{ps} = area of prestressing steel (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

e_m = average eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in Table 5.9.3-1 (ksi)

I_g = moment of inertia of the gross concrete section (in.^{4})

M_g = midspan moment due to member self-weight (kip-in.)

N = number of identical prestressing tendons

f_{pj} = stress in the prestressing steel at jacking (ksi)

For post-tensioned structures with bonded tendons, Δf_{pES} may be calculated at the center section of the span or, for continuous construction, at the section of maximum moment.

For post-tensioned structures with unbonded tendons, Δf_{pES} can be calculated using the eccentricity of the prestressing steel averaged along the length of the member.

For slab systems, the value of Δf_{pES} may be taken as 25 percent of that obtained from Eq. C5.9.5.2.3b-1.

For post-tensioned construction, Δf_{pES} losses can be further reduced below those implied by Eq. 5.9.5.2.3b-1 with proper tensioning procedures such as stage stressing and retensioning.

If tendons with two different numbers of strand per tendon are used, N may be calculated as:

$$N = N_1 + N_2 \frac{A_{sp2}}{A_{sp1}} \qquad \text{(C5.9.5.2.3b-2)}$$

where:

N_1 = number of tendons in the larger group

N_2 = number of tendons in the smaller group

A_{sp1} = cross-sectional area of a tendon in the larger group (in.^{2})

A_{sp2} = cross-sectional area of a tendon in the smaller group (in.^{2})

5.9.5.2.3c—Combined Pretensioning and Post-Tensioning

In applying the provisions of Articles 5.9.5.2.3a and 5.9.5.2.3b to components with combined pretensioning and post-tensioning, and where post-tensioning is not applied in identical increments, the effects of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons shall be considered.

C5.9.5.2.3c

See Castrodale and White (2004) for information on computing the effect of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons.

5.9.5.3—Approximate Estimate of Time-Dependent Losses 2010 Revision

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

- members are made from normal-weight concrete,
- the concrete is either steam- or moist-cured,
- prestressing is by bars or strands with normal and low relaxation properties, and
- average exposure conditions and temperatures characterize the site,

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

$$\Delta f_{pLT} = 10.0 \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR} \qquad \text{(5.9.5.3-1)}$$

in which:

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

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

C5.9.5.3

The losses or gains due to elastic deformations at the time of transfer or load application should be added to the time-dependent losses to determine total losses. However, these elastic losses (or gains) must be taken equal to zero if transformed section properties are used in stress analysis.

The approximate estimates of time-dependent prestress losses given in Eq. 5.9.5.3-1 are intended for sections with composite decks only. The losses in Eq. 5.9.5.3-1 were derived as approximations of the terms in the refined method for a wide range of standard precast prestressed concrete I-beams, box beams, inverted tee beams, and voided slabs. The members were assumed to be fully utilized, i.e., level of prestressing is such that concrete tensile stress at full service loads is near the maximum limit. It is further assumed in the development of the approximate method that live load moments produce about one-third of the total load moments, which is reasonable for I-beam and inverted tee composite construction and conservative for noncomposite boxes and voided slabs. They were calibrated with full-scale test results and with the results of the refined method, and found to give conservative results (Al-Omaishi, 2001; Tadros, 2003). The approximate method should not be used for members of uncommon shapes, i.e., having *V/S* ratios much different from 3.5 in., level of prestressing, or construction staging. The first term in Eq. 5.9.5.3-1 corresponds to creep losses, the second term to shrinkage losses, and the third to relaxation losses.

The commentary to Article 5.9.5.4.2 also gives an alternative relaxation loss prediction method.

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = the average annual ambient relative humidity (%)

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

Δf_{pR} = an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand, 10.0 ksi for stress relieved strand, and in accordance with manufacturers recommendation for other types of strand (ksi)

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

For segmental concrete bridges, lump sum losses may be used only for preliminary design purposes.

For members of unusual dimensions, level of prestressing, construction staging, or concrete constituent materials, the refined method of Article 5.9.5.4 or computer time-step methods shall be used.

5.9.5.4—Refined Estimates of Time-Dependent Losses

5.9.5.4.1—General

For nonsegmental prestressed members, more accurate values of creep-, shrinkage-, and relaxation-related losses, than those specified in Article 5.9.5.3 may be determined in accordance with the provisions of this Article. For precast pretensioned girders without a composite topping and for precast or cast-in-place nonsegmental post-tensioned girders, the provisions of Articles 5.9.5.4.4 and 5.9.5.4.5, respectively, shall be considered before applying the provisions of this Article.

C5.9.5.4.1

See Castrodale and White (2004) for information on computing the interaction of creep effects for prestressing applied at different times.

Estimates of losses due to each time-dependent source, such as creep, shrinkage, or relaxation, can lead to a better estimate of total losses compared with the values obtained using Article 5.9.5.3. The individual losses are based on research published in Tadros (2003), which aimed at extending applicability of the provisions of these Specifications to high-strength concrete.

For segmental construction and post-tensioned spliced precast girders, other than during preliminary design, prestress losses shall be determined by the time-step method and the provisions of Article 5.9.5, including consideration of the time-dependent construction stages and schedule shown in the contract documents. For components with combined pretensioning and post-tensioning, and where post-tensioning is applied in more than one stage, the effects of subsequent prestressing on the creep loss for previous prestressing shall be considered.

The change in prestressing steel stress due to time-dependent loss, Δf_{pLT}, shall be determined as follows:

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df} \quad (5.9.5.4.1\text{-}1)$$

where:

Δf_{pSR} = prestress loss due to shrinkage of girder concrete between transfer and deck placement (ksi)

Δf_{pCR} = prestress loss due to creep of girder concrete between transfer and deck placement (ksi)

Δf_{pR1} = prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (ksi)

Δf_{pR2} = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi)

Δf_{pSD} = prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi)

Δf_{pCD} = prestress loss due to creep of girder concrete between time of deck placement and final time (ksi)

Δf_{pSS} = prestress gain due to shrinkage of deck in composite section (ksi)

$(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$
= sum of time-dependent prestress losses between transfer and deck placement (ksi)

$(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$
= sum of time-dependent prestress losses after deck placement (ksi)

The new approach additionally accounts for interaction between the precast and the cast-in-place concrete components of a composite member and for variability of creep and shrinkage properties of concrete by linking the loss formulas to the creep and shrinkage prediction formulae of Article 5.4.2.3.

For concrete containing lightweight aggregates, very hard aggregates, or unusual chemical admixtures, the estimated material properties used in this Article and Article 5.4.2.3 may be inaccurate. Actual test results should be used for their estimation.

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.5, including consideration of the time-dependent construction method and schedule shown in the contract documents.

5.9.5.4.2—Losses: Time of Transfer to Time of Deck Placement

5.9.5.4.2a—Shrinkage of Girder Concrete

The prestress loss due to shrinkage of girder concrete between time of transfer and deck placement, Δf_{pSR}, shall be determined as:

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} \tag{5.9.5.4.2a-1}$$

in which:

$$K_{id} = \frac{1}{1 + \dfrac{E_p}{E_{ci}} \dfrac{A_{ps}}{A_g} \left(1 + \dfrac{A_g e_{pg}^2}{I_g}\right) \left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \tag{5.9.5.4.2a-2}$$

where:

ε_{bid} = concrete shrinkage strain of girder between the time of transfer and deck placement per Eq. 5.4.2.3.3-1

K_{id} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement

e_{pg} = eccentricity of prestressing force with respect to centroid of girder (in.); positive in common construction where it is below girder centroid

$\Psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer per Eq. 5.4.2.3.2-1

t_f = final age (days)

t_i = age at transfer (days)

5.9.5.4.2b—Creep of Girder Concrete

The prestress loss due to creep of girder concrete between time of transfer and deck placement, Δf_{pCR}, shall be determined as:

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b \left(t_d, t_i\right) K_{id} \tag{5.9.5.4.2b-1}$$

where:

$\Psi_b(t_d, t_i)$ = girder creep coefficient at time of deck placement due to loading introduced at transfer per Eq. 5.4.2.3.2-1

t_d = age at deck placement (days)

5.9.5.4.2c—Relaxation of Prestressing Strands

The prestress loss due to relaxation of prestressing strands between time of transfer and deck placement, Δf_{pR1}, shall be determined as:

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right) \tag{5.9.5.4.2c-1}$$

where:

f_{pt} = stress in prestressing strands immediately after transfer, taken not less than $0.55f_{py}$ in Eq. 5.9.5.4.2c-1

K_L = 30 for low relaxation strands and 7 for other prestressing steel, unless more accurate manufacturer's data are available

The relaxation loss, Δf_{pR1}, may be assumed equal to 1.2 ksi for low-relaxation strands.

C5.9.5.4.2c

Eqs. 5.9.5.4.2c-1 and 5.9.5.4.3c-1 are given for relaxation losses and are appropriate for normal temperature ranges only. Relaxation losses increase with increasing temperatures.

A more accurate equation for prediction of relaxation loss between transfer and deck placement is given in Tadros et al. (2003):

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K'_L}\frac{\log\ (24t)}{\log(24t_i)}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR}\ )}{f_{pt}}\right]K_{id} \tag{C5.9.5.4.2c-1}$$

where the K'_L is a factor accounting for type of steel, equal to 45 for low relaxation steel and 10 for stress relieved steel, t is time in days between strand tensioning and deck placement. The term in the first square brackets is the intrinsic relaxation without accounting for strand shortening due to creep and shrinkage of concrete. The second term in square brackets accounts for relaxation reduction due to creep and shrinkage of concrete. The factor K_{id} accounts for the restraint of the concrete member caused by bonded reinforcement. It is the same factor used for the creep and shrinkage components of the prestress loss. The equation given in Article 5.9.5.4.2c is an approximation of the above formula with the following typical values assumed:

t_i = 0.75 day

t = 120 days

$$\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR}\ )}{f_{pt}}\right] = 0.67$$

K_{id} = 0.8

5.9.5.4.3—Losses: Time of Deck Placement to Final Time

5.9.5.4.3a—Shrinkage of Girder Concrete

The prestress loss due to shrinkage of girder concrete between time of deck placement and final time, Δf_{pSD}, shall be determined as:

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df} \tag{5.9.5.4.3a-1}$$

in which:

$$K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_c} \left(1 + \frac{A_c e_{pc}^2}{I_c}\right) \left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \tag{5.9.5.4.3a-2}$$

where:

ε_{bdf} = shrinkage strain of girder between time of deck placement and final time per Eq. 5.4.2.3.3-1

K_{df} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between deck placement and final time

e_{pc} = eccentricity of prestressing force with respect to centroid of composite section (in.), positive in typical construction where prestressing force is below centroid of section

A_c = area of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio (in.^{2})

I_c = moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio at service (in.^{4})

5.9.5.4.3b—Creep of Girder Concrete

The prestress (loss is positive, gain is negative) due to creep of girder concrete between time of deck placement and final time, Δf_{pCD}, shall be determined as:

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b\left[\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b\left(t_f, t_d\right)K_{df} \tag{5.9.5.4.3b-1}$$

where:

Δf_{cd} = change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement, combined with deck weight and superimposed loads (ksi)

$\Psi_b(t_f, t_d)$ = girder creep coefficient at final time due to loading at deck placement per Eq. 5.4.2.3.2-1

5.9.5.4.3c—Relaxation of Prestressing Strands

The prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time, Δf_{pR2}, shall be determined as:

$$\Delta f_{pR2} = \Delta f_{pR1} \quad (5.9.5.4.3c\text{-}1)$$

C5.9.5.4.3.c

Research indicates that about one-half of the losses due to relaxation occur before deck placement; therefore, the losses after deck placement are equal to the prior losses.

5.9.5.4.3d—Shrinkage of Deck Concrete

The prestress gain due to shrinkage of deck composite section, Δf_{pSS}, shall be determined as:

$$\Delta f_{pSS} = \frac{E_p}{E_c} \Delta f_{cdf} K_{df} \left[1 + 0.7\psi_b\left(t_f, t_d\right)\right] \quad (5.9.5.4.3d\text{-}1)$$

in which:

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left[1 + 0.7\psi_d\left(t_f, t_d\right)\right]} \left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right) \quad (5.9.5.4.3d\text{-}2)$$

where:

Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi)

ε_{ddf} = shrinkage strain of deck concrete between placement and final time per Eq. 5.4.2.3.3-1

A_d = area of deck concrete (in.^{2})

E_{cd} = modulus of elasticity of deck concrete (ksi)

e_d = eccentricity of deck with respect to the gross composite section, positive in typical construction where deck is above girder (in.)

$\Psi_b(t_f, t_d)$ = creep coefficient of deck concrete at final time due to loading introduced shortly after deck placement (i.e. overlays, barriers, etc.) per Eq. 5.4.2.3.2-1

5.9.5.4.4—Precast Pretensioned Girders without Composite Topping

The equations in Article 5.9.5.4.2 and Article 5.9.5.4.3 are applicable to girders with noncomposite deck or topping, or with no topping. The values for time of "deck placement" in Article 5.9.5.4.2 may be taken as values at time of noncomposite deck placement or values at time of installation of precast members without topping. Time of "deck placement" in Article 5.9.5.4.3 may be taken as time of noncomposite deck placement or values at time of installation of precast members without topping. Area of "deck" for these applications shall be taken as zero.

5.9.5.4.5—Post-Tensioned Nonsegmental Girders

Long-term prestress losses for post-tensioned members after tendons have been grouted may be calculated using the provisions of Articles 5.9.5.4.1 through 5.9.5.4.4. In Eq. 5.9.5.4.1-1, the value of the term $(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$ shall be taken as zero.

5.9.5.5—Losses for Deflection Calculations

For camber and deflection calculations of prestressed nonsegmental members made of normal weight concrete with a strength in excess of 3.5 ksi at the time of prestress, f_{cgp} and Δf_{cdp} may be computed as the stress at the center of gravity of prestressing steel averaged along the length of the member.

5.10—DETAILS OF REINFORCEMENT

5.10.1—Concrete Cover

Minimum concrete cover shall be as specified in Article 5.12.3.

5.10.2—Hooks and Bends

5.10.2.1—Standard Hooks

For the purpose of these Specifications, the term "standard hook" shall mean one of the following:

- For longitudinal reinforcement:

  (a) 180- degree bend, plus a $4.0d_b$ extension, but not less than 2.5 in. at the free end of the bar, or

  (b) 90- degree bend, plus a $12.0d_b$ extension at the free end of the bar.

C5.10.2.1

These requirements are consistent with the requirements of ACI 318 and CRSI's *Manual of Standard Practice*.

- For transverse reinforcement:

  (a) No. 5 bar and smaller—90-degree bend, plus a $6.0d_b$ extension at the free end of the bar,

  (b) No. 6, No. 7 and No. 8 bars—90-degree bend, plus a $12.0d_b$ extension at the free end of the bar; and

  (c) No. 8 bar and smaller—135-degree bend, plus a $6.0\ d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

5.10.2.2—Seismic Hooks

Seismic hooks shall consist of a 135-degree bend, plus an extension of not less than the larger of $6.0d_b$ or 3.0 in. Seismic hooks shall be used for transverse reinforcement in regions of expected plastic hinges. Such hooks and their required locations shall be detailed in the contract documents.

5.10.2.3—Minimum Bend Diameters

The diameter of a bar bend, measured on the inside of the bar, shall not be less than that specified in Table 5.10.2.3-1.

Table 5.10.2.3-1—Minimum Diameters of Bend

Bar Size and Use	Minimum Diameter
No. 3 through No. 5—General	$6.0d_b$
No. 3 through No. 5—Stirrups and Ties	$4.0d_b$
No. 6 through No. 8—General	$6.0d_b$
No. 9, No. 10, and No. 11	$8.0d_b$
No. 14 and No. 18	$10.0d_b$

The inside diameter of bend for stirrups and ties in plain or deformed welded wire fabric shall not be less than $4.0d_b$ for deformed wire larger than D6 and $2.0d_b$ for all other wire sizes. Bends with inside diameters of less than $8.0d_b$ shall not be located less than $4.0d_b$ from the nearest welded intersection.

5.10.3—Spacing of Reinforcement

5.10.3.1 Minimum Spacing of Reinforcing Bars

5.10.3.1.1—Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars,
- 1.5 times the maximum size of the coarse aggregate, or
- 1.5 in.

5.10.3.1.2—Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than:

- The nominal diameter of the bars,
- 1.33 times the maximum size of the coarse aggregate, or
- 1.0 in.

5.10.3.1.3—Multilayers

Except in decks where parallel reinforcing is placed in two or more layers, with clear distance between layers not exceeding 6.0 in., the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than 1.0 in. or the nominal diameter of the bars.

5.10.3.1.4—Splices

The clear distance limitations between bars that are specified in Articles 5.10.3.1.1 and 5.10.3.1.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

5.10.3.1.5—Bundled Bars

The number of parallel reinforcing bars bundled in contact to act as a unit shall not exceed four in any one bundle, except that in flexural members, the number of bars larger than No. 11 shall not exceed two in any one bundle.

Bundled bars shall be enclosed within stirrups or ties.

Individual bars in a bundle, cut off within the span of a member, shall be terminated at different points with at least a 40-bar diameter stagger. Where spacing limitations are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

C5.10.3.1.5

Bundled bars should be tied, wired, or otherwise fastened together to ensure that they remain in their relative position, regardless of their inclination.

5.10.3.2—Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18.0 in. The maximum spacing of spirals, ties, and temperature shrinkage reinforcement shall be as specified in Articles 5.10.6, 5.10.7, and 5.10.8.

5.10.3.3—Minimum Spacing of Prestressing Tendons and Ducts

5.10.3.3.1—Pretensioning Strand

The distance between pretensioning strands, including shielded ones, at each end of a member within the transfer length, as specified in Article 5.11.4.1, shall not be less than a clear distance taken as 1.33 times the maximum size of the aggregate nor less than the center-to-center distances specified in Table 5.10.3.3.1-1.

Table 5.10.3.3.1-1—Center-to-Center Spacings

Strand Size (in.)	Spacing (in.)
0.6 0.5625 Special 0.5625	2.000
0.5000 0.4375 0.50 Special	1.750
0.3750	1.500

If justified by performance tests of full-scale prototypes of the design, the clear distance between strands at the end of a member may be decreased.

The minimum clear distance between groups of bundled strands shall not be less than 1.33 times the maximum size of the aggregate or 1.0 in.

Pretensioning strands in a member may be bundled to touch one another in an essentially vertical plane at and between hold-down locations. Strands bundled in any manner, other than a vertical plane, shall be limited to four strands per bundle.

C5.10.3.3.1

The requirement to maintain the clear spacing within the transfer zone is to ensure the strands are separated sufficiently to properly transfer their prestressing force to the surrounding concrete and to reduce the stress concentration around the strands at the ends of pretensioned components at release.

Some jurisdictions limit the clear distance between pretensioning strands to not less than twice the nominal size of aggregate to facilitate placing and compaction of concrete.

5.10.3.3.2—Post-Tensioning Ducts—Girders Straight in Plan

Unless otherwise specified herein, the clear distance between straight post-tensioning ducts shall not be less than 1.5 in. or 1.33 times the maximum size of the coarse aggregate. For precast segmental construction when post-tensioning tendons extend through an epoxy joint between components, the clear spacing between post-tensioning ducts shall not be less than the greater of the duct internal diameter or 4.0 in.

Ducts may be bundled together in groups not exceeding three, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 3.0 ft of anchorages.

For groups of bundled ducts in construction other than segmental, the minimum clear horizontal distance between adjacent bundles shall not be less than 4.0 in. When groups of ducts are located in two or more horizontal planes, a bundle shall contain no more than two ducts in the same horizontal plan.

C5.10.3.3.2

The minimum vertical clear distance between bundles shall not be less than 1.5 in. or 1.33 times the maximum size of coarse aggregate.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 3.0 in.

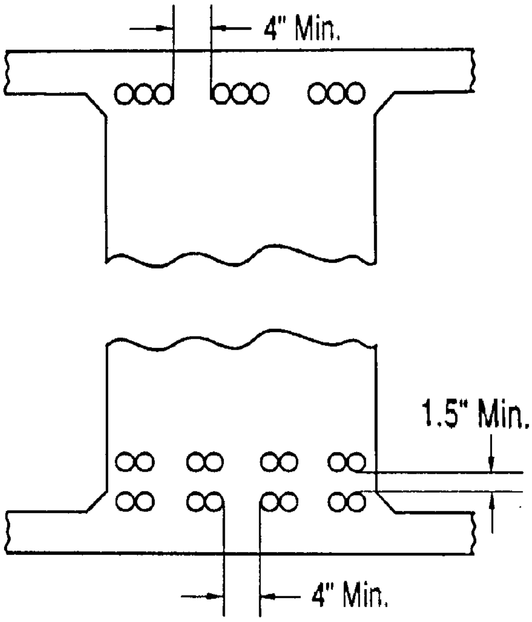


Figure C5.10.3.3.2-1—Examples of Acceptable Arrangements for Ducts Not Curved in the Horizontal Plan

5.10.3.3.3—Post-Tensioning Ducts—Girders Curved in Plan

The minimum clear distance between curved ducts shall be as required for tendon confinement as specified in Article 5.10.4.3. The spacing for curved ducts shall not be less than that required for straight ducts.

5.10.3.4—Maximum Spacing of Prestressing Tendons and Ducts in Slabs

Pretensioning strands for precast slabs shall be spaced symmetrically and uniformly and shall not be farther apart than 1.5 times the total composite slab thickness or 18.0 in.

Post-tensioning tendons for slabs shall not be farther apart, center-to-center, than 4.0 times the total composite minimum thickness of the slab.

C5.10.3.4

The 4.0 times depth of slab requirement for the maximum spacing of transverse post-tensioning ducts in deck slabs is new and reflects common practice. The composite thickness refers to slabs with bonded overlays.

5.10.3.5—Couplers in Post-Tensioning Tendons

The contract documents shall specify that not more than 50 percent of the longitudinal post-tensioning tendons be coupled at one section and that the spacing between adjacent coupler locations be not closer than the segment length or twice the segment depth. The void areas around couplers shall be deducted from the gross section area and moment of inertia when computing stresses at the time post-tensioning force is applied.

C5.10.3.5

European experience indicates that the prestressing force decreases locally in the region of a coupler. This is believed to result, in part, from increased creep caused by high compressive stresses in the reduced concrete section due to coupling of tendons. Cracking has not been observed in bridges where the number of tendons coupled at a section has been limited to 50 percent of the total number of tendons.