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CONCRETE STRUCTURES

Commentary is opposite the text it annotates.

5.1—SCOPE

The provisions in this Section apply to the design of bridge and ancillary structures 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 design concrete compressive strengths varying from 2.4 ksi to 10.0 ksi for normal weight and lightweight concrete, except where higher strengths not exceeding 15.0 ksi are allowed for normal weight concrete. The exceptions are noted in the specific articles and tabulated in Appendix C5.

The provisions of this Section characterize regions of concrete structures by their behavior as B- (beam or Bernoulli) Regions or D- (disturbed or discontinuity) Regions, as defined in Article 5.2. The characterization of regions into B-Regions and D-Regions is discussed in Article 5.5.1.

The provisions of this Section combine and unify the requirements for reinforced and prestressed concrete.

A brief outline for the design of some routine concrete components is contained in Appendix A5.

C5.1

This section was substantially reorganized and updated in the 8th Edition. As a transitional aid in locating information retained from the 7th Edition, a cross-walk between article numbers in the 7th and 8th Editions was included in Appendix E5 and remains in the 9th Edition as a historical reference.

These specifications use kips and ksi units. Some other specifications, such as ACI 318, use pound and psi units. For most variables the conversion is obvious, but for those which have the form $N\sqrt{f'_c}$, the conversion is $\frac{N\sqrt{f'_c}}{\sqrt{1{,}000}}$. For commonly used values of *N,* the relation between psi and ksi is given below:

N, psi	*N*, ksi
1	0.0316
2	0.0632
3	0.0948
4	0.126
6	0.190
7.5	0.237
12	0.379

5.2—DEFINITIONS

Adhesive Anchor—A post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by characteristic bond of the anchor system as defined in ACI 318-14.

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

Anchor Pullout Strength—The strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

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.

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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.

Beam or Bernoulli Region (B-Region)—Regions of concrete members in which Bernoulli's hypothesis of straight-line strain profiles, linear for bending and uniform for shear, applies. (See Article 5.5.1.2 for more detail.)

Blanketed Strand—See *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 Anchor—A headed bolt, headed stud, or hooked bolt installed before placing concrete.

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 5.6.2.1.

Concrete Breakout Strength—The strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

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.

Concrete Pryout Strength—The strength corresponding to formation of a concrete spall behind short, stiff anchors displaced in the direction opposite to the applied shear force.

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

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.

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.

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

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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 2*d* or components in which a load causing more than one third of the shear at a support is closer than 2*d* from the face of the support.

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

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.

Disturbed or Discontinuity Region (D-Region)—Regions of concrete members encompassing abrupt changes in geometry or concentrated forces in which strain profiles more complex than straight lines exist (See Article 5.5.1.2 for more detail.).

Duct Stack—A vertical group of tendons in which the space between individual tendons is less than 1.5 in.

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.

Expansion Anchor—A post-installed anchor, inserted into hardened concrete, that transfers loads to or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.

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

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

Five Percent Fractile—A statistical term meaning 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.

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

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.

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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 conforming to AASHTO M 195 and having an equilibrium density not exceeding 0.135 kcf, as determined by ASTM C567.

Local Bending—The lateral flexural bending caused by curved post-tensioning tendons on the concrete cover between the internal ducts and the inside face of the curved element (usually webs).

Local Shear—The lateral shear caused by curved post-tensioning tendons on the concrete cover between the internal ducts and the inside face of the curved element (usually webs).

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

Post-Installed Anchor—An anchor installed in hardened concrete. Expansion, undercut, and adhesive anchors are examples of post-installed anchors.

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.

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.

Regional Bending—Transverse bending of a concrete box girder web due to concentrated lateral prestress forces resisted by the frame action of the box acting as a whole.

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

Reinforcement—Reinforcing bars, welded wire reinforcement, and/or prestressing steel.

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

Resal Effect—The reduction or addition of shear based on the bottom slab compression angle with the center of gravity.

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.

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 5.12.5.)

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

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

Shielded Strand—See *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 Concrete Strength—The compressive strength of concrete specified in the contract documents which may be greater than the compressive strength of concrete for use in design, f'_c.

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

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 5.12.3.4.)

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

Supplementary Anchor Reinforcement—Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into the structural member.

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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 the tension-controlled strain limit.

Tension-Controlled Strain Limit—The net tensile strain in the extreme tension steel at nominal resistance. (See Article 5.6.2.1.)

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.

Unbonded Tendon—Tendons that are effectively bonded at only their anchorages and intermediate bonded sections, such as deviators.

Undercut Anchor—A post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

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

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) (5.8.4.4.2) (5.12.5.3.2)

A_b = effective net area of a bearing plate (in.^{2}); effective bearing area (in.^{2}); area of a single bar (in.^{2}) (5.8.4.4.2) (5.8.4.5.2) (5.10.8.2.6d)

A_{btr} = cross-sectional area of an individual transverse bar crossing the potential plane of splitting (in.^{2}) (C5.10.8.2.1c)

A_c = area of core measured to the outside diameter of the spiral (in.^{2}); 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}); area of column core measured to the outside of the hoop (in.^{2}); gross area of concrete deck slab (in.^{2}) (5.6.4.6) (5.9.3.4.3a) (5.11.4.1.4) (C5.12.3.3.3)

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}) (5.9.5.6.7b)

A_{conf} = bearing area of confined concrete in the local zone (in.$^{2)}$ (5.8.4.5.2)

A_{cp} = area enclosed by outside perimeter of concrete cross section (in.^{2}) (5.7.2.1)

A_{ct} = area of concrete on the flexural tension side of the member (5.7.3.4.2) (in.^{2})

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2}) (5.7.4.2)

A_d = area of deck concrete (in.^{2}) (5.9.3.4.3d)

A_g = gross area of section (in.^{2}); gross area of bearing plate (in.^{2}) (5.6.4.2) (5.8.4.4.2)

A_h = total area of horizontal crack control reinforcement within spacing s_h (in.^{2}); area of shear reinforcement parallel to flexural tension reinforcement (in.^{2}) (5.8.2.6) (5.8.4.2.1)

A_{hr} = area of one leg of hanger reinforcement in beam ledges and inverted T-beams (in.^{2}) (5.8.4.3.5)

AI = for segmental construction: dynamic response due to accidental release or application of a precast segment load or other sudden application of an otherwise static load to be added to the dead load (kip) (5.12.5.3.2)

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A_ℓ = total area of longitudinal torsion reinforcement in a box girder (in.^{2}); area of longitudinal column reinforcement (in.^{2}) (5.7.3.6.3) (5.10.8.4.2a)

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc} (in.^{2}) (5.8.4.2.2)

A_o = area enclosed by shear flow path, including any area of holes therein (in.^{2}) (5.7.2.1)

A_{plate} = area of anchor bearing plate (5.8.4.5.2)

A_{ps} = area of prestressing steel (in.^{2}); area of prestressing steel on the flexural tension side of the member (in.^{2}) (5.6.3.1.1) (5.7.3.4.2)

A_{psb} = area of bonded prestressing steel (in.^{2}) (5.6.3.1.3b)

A_{psu} = area of unbonded prestressing steel (in.^{2}) (5.6.3.1.3b)

A_s = area of nonprestressed tension reinforcement (in.^{2}); area of nonprestressed steel on the flexural tension side of the member at the section under consideration (in.^{2}); total area of reinforcement located within the distance h/4 from the end of the beam (in.^{2}); area of reinforcement in each direction and each face (in.^{2}/ft); total area of longitudinal deck reinforcement (in.^{2}); area of reinforcement in the design width (in.^{2}) (5.6.3.1.1) (5.7.3.4.2) (5.9.4.4.1) (5.10.6) (C5.12.3.3.3) (5.12.7.3)

A'_s = area of compression reinforcement (in.^{2}) (5.6.3.1.1)

A_{sh} = total cross-sectional area of tie reinforcement, including supplementary cross-ties having a vertical spacing of s and crossing a section having a core dimension of h_c (in.^{2}) (5.11.4.1.4)

A_{sk} = area of skin reinforcement per unit height on each side face (in.^{2}) (5.6.7)

A_{sp} = cross-sectional area of spiral or hoop (in.^{2}); area of shaft spiral or transverse reinforcement (in.^{2}) (5.6.4.6) (5.10.8.4.2a)

A_{sp1} = cross-sectional area of a tendon in the larger group (in.^{2}) (C5.9.3.2.3b)

A_{sp2} = cross-sectional area of a tendon in the smaller group (in.^{2}) (C5.9.3.2.3b)

A_{st} = total area of longitudinal nonprestressed reinforcement (in.^{2}) (5.6.4.4)

$A_{s\text{-}BW}$ = area of steel in the band width (in.^{2}) (5.12.8.5)

$A_{s\text{-}SD}$ = total area of steel in short direction (in.^{2}) (5.12.8.5)

A_t = area of one leg of closed transverse torsion reinforcement in solid members, or total area of transverse torsion reinforcement in the exterior web and flange of hollow members (in.^{2}); total area of transverse torsion reinforcing in the exterior web and flange (in.^{2}) (5.7.3.6.2) (5.12.5.3.8d)

A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed (in^2); area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2}) (5.10.8.2.1c) (C5.12.3.3.3)

A_v = area of a transverse reinforcement within distance s (in.^{2}); total area of vertical crack control reinforcement within spacing s_v; (in.^{2}) total area of transverse reinforcement in all webs in the cross section within a distance s (in.^{2}) (5.7.2.5) (5.8.2.6) (5.12.5.3.8c)

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2}); area of shear-friction reinforcement (in.^{2}); total area of reinforcement, including flexural reinforcement (in.^{2}) (5.7.4.2) (5.8.4.2.2) (5.11.4.4)

A_w = area of an individual wire to be developed or spliced (in.^{2}) (5.10.8.2.5)

A_1 = area under bearing device (in.^{2}) (5.6.5)

A_2 = Notional 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}) (5.6.5)

a = depth of equivalent rectangular stress block (in.); shear span (in.); lateral dimension of the anchorage device or group of devices in the direction considered (in.); lateral dimension of the anchorage device or group of devices in the transverse direction of the slab (in.) (5.6.3.2.2) (C5.8.2.2) (5.8.4.5.3) (5.8.4.5.5)

a_{eff} = lateral dimension of the effective bearing area of the anchorage measured parallel to the larger dimension of the cross section (in.) (5.8.4.5.2)

a_f = distance from centerline of girder reaction to vertical reinforcement in backwall or stem of inverted T (in.^{2}) (5.8.4.3)

a_v = distance from face of wall to the concentrated load (in.) (5.8.4.2.1)

B_w = total web width in single cell or symmetrical two-cell hollow sections at height of the web where principal tension is being checked (in.) (5.9.2.3.3)

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.); width of corbel or ledge (in.); least width of component section (in.); width of a pier (in.); design width (in.) (5.6.3.1.1) (5.8.4.2.2) (5.10.6) (5.11.4.2) (5.12.7.3)

b_e = effective width of the shear flow path, to be take as the minimum thickness of the exterior webs or flanges comprising the closed box section (in.); the effective thickness of the shear flow path of the elements making up the space truss model resisting torsion (in.) (5.7.2.1) (5.12.5.3.8c)

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b_{eff} = lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross section (in.) (5.8.4.5.2)
b_f = the overall width of the inverted T-beam flange and bottom flange of the I-beam (in.) (5.8.4.3.5) (5.9.4.3.3)
b_o = the perimeter of critical section for shear (in.) (5.8.4.3.4)
b_v = width of web adjusted for the presence of ducts (in.); 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.) For grouted ducts, no modification is necessary. For ungrouted ducts, reduce b_v by the diameter of the duct.; effective web width taken as the minimum web width within the depth d_v (in.); effective web width taken as the total minimum width of all webs in the cross section within the depth d_v adjusted for the effect of openings or ducts (in.) (5.7.2.5) (5.7.2.8) (5.7.3.3) (5.12.5.3.8c)
b_{vi} = interface width considered to be engaged in shear transfer (in.) (5.7.4.3)
b_w = web width or diameter of a circular section (in.); gross width of web, not reduced for the presence of post-tensioning ducts (in.); width of member's web (in.); the width of the inverted T-beam stem (in.); web width at the height of the web where principal tension is being checked (in.); web width of I-beam (in.) (5.6.3.1.1) (C5.8.2.2) (5.8.4.3.4) (5.9.2.3.3) (5.9.4.3.3)
CEQ = for segmental construction: specialized construction equipment (kip) (5.12.5.3.2)
CLE = for segmental construction: longitudinal construction equipment load (kip) (5.12.5.3.2)
CLL = for segmental construction: distributed construction live load (ksf) (5.12.5.3.2)
CR = creep effects (ksi) (5.12.5.3.2)
C_1 = compression force above the shear plane associated with M_1 (kip) (C5.7.4.5)
C_{u2} = compression force above the shear plane associated with M_{u2} (kip) (C5.7.4.5)
c = distance from the extreme compression fiber to the neutral axis (in.); cohesion factor (ksi); spacing from centerline of bearing to end of beam ledge (in.); required concrete cover over the confining reinforcement (in.) (5.6.2.1) (5.7.4.3) (5.8.4.3.2) (C5.8.4.4.1)
c_b = the smaller of distance from center of bar or wire being developed to the nearest concrete surface and one-half the center-to-center spacing of the bars or wires being developed (in.) (5.10.8.2.1c)
D = external diameter of the circular member (in.); diameter of circular bearing pad (in.) (C5.7.2.8) (5.8.4.3.4)
DC = weight of supported structure (kip) (5.12.5.3.2)
$DIFF$ = for segmental construction: differential load (kip) (5.12.5.3.2)
D_r = diameter of the circle passing through the centers of the longitudinal reinforcement (in.) (C5.7.2.8)
DW = superimposed dead load (kip) or (klf) (5.12.5.3.2)
d = effective depth of the member defined as the distance between the extreme compression fiber and the centroid of the primary longitudinal reinforcement (in.); depth of a pier (in); 0.8h or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.) (5.5.1.2.1) (5.11.4.2) (5.12.5.3.8c)
d_b = nominal strand diameter (in.); nominal diameter of reinforcing bar (in.); nominal diameter of reinforcing bar or wire (in.) (5.9.4.3.2) (5.10.2.1) (5.10.8.2.1a)
d_{burst} = distance from anchorage device to the centroid of the bursting force, T_{burst} (in.) (5.8.4.5.3)
d_c = core diameter of column measured to the outside diameter of spiral or hoop (in.); thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.); minimum concrete cover over the tendon duct (in.) (5.6.4.6) (5.6.7) (5.9.5.4.4)
d_{duct} = outside diameter of post-tensioning duct (in.) (5.9.5.4.4b)
d_e = depth of center of gravity of steel (in.); effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) (5.8.4.2.2) (5.12.5.3.8d)
d_{eff} = one-half the effective length of the failure plane in shear and tension for a curved element (in.) (5.9.5.4.4b)
d_f = distance from top of ledge to the bottom longitudinal reinforcement (in.) (5.8.4.3.4)
d_ℓ = distance from the extreme compression fiber to the centroid of extreme tension steel element (in.) (5.6.7)
d_o = girder depth (ft) (C5.12.5.3.11d)
d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.) (5.6.3.1.1)
d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement measured along the centerline of the web (in.) (5.6.2.1)
d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.) (5.6.3.2.2)
d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure (in.); distance between the centroid of the

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tension steel and the mid-thickness of the slab to compute a factored interface shear stress (in.) (5.7.2.8) (5.7.4.5)

E_b = modulus of elasticity of the bearing plate material (ksi) (5.8.4.4.2)
E_c = modulus of elasticity of concrete (ksi) (5.4.2.4)
$E_{c\ deck}$ = modulus of elasticity of deck concrete (ksi) (5.9.3.4.3d)
E_{ci} = modulus of elasticity of concrete at transfer (ksi) (C5.9.3.2.3a)
E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi) (5.9.3.2.3a)
E_{eff} = effective modulus of elasticity (ksi) (C5.12.5.3.6)
EI = flexural stiffness (kip-in.^{2}) (5.6.4.3)
E_p = modulus of elasticity of prestressing steel (ksi) (5.4.4.2)
E_s = modulus of elasticity of steel reinforcement (ksi); modulus of elasticity of longitudinal steel (ksi) (5.4.3.2) (5.6.4.3)
e = minimum edge distance for anchorage devices as specified by the supplier (in.); eccentricity of the anchorage device or group of devices with respect to the centroid of the cross section, always taken as positive (in.); base of natural logarithms (C5.8.4.4.1) (5.8.4.5.3) (5.9.2.1)
e_d = eccentricity of deck with respect to the gross composite section (in.) (5.9.3.4.3d)
e_m = average prestressing steel eccentricity at midspan (in.) (C5.9.3.2.3a)
e_{pc} = eccentricity of prestressing force with respect to centroid of composite section (in.) (5.9.3.4.3a)
e_{pg} = eccentricity of prestressing force with respect to centroid of girder (in.) (5.9.3.4.2a)
F = force effect determined using the modulus of elasticity of the concrete at the time loading is applied (kip) (5.9.2.1)
F' = reduced force effect (kip) (5.9.2.1)
$F_{u\text{-}in}$ = in-plane deviation force effect per unit length of tendon (kips/ft) (5.9.5.4.4a)
$F_{u\text{-}out}$ = out-of-plane force effect per unit length of tendon (kips/ft) (5.9.5.4.5)
f_b = stress in anchor plate at a section taken at the edge of the wedge hole or holes (ksi) (5.8.4.4.2)
f'_c = compressive strength of concrete for use in design (ksi) (5.4.2.1)
f_{ca} = concrete compressive stress ahead of the anchorage devices (ksi) (5.8.4.5.2)
f_{cb} = unfactored minimum compressive stress in the region behind the anchor at service limit states and any stage of construction (ksi) (5.9.5.6.7b)
f_{cgp} = concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately at transfer and the self-weight of the member at the section of maximum moment (ksi) (5.9.3.2.3a)
f'_{ci} = design concrete compressive strength at time of prestressing for pretensioned members and at time of initial loading for nonprestressed members (ksi); design concrete strength at time of application of tendon force (ksi) (5.4.2.3.2) (5.8.4.4.2)
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) (5.6.3.3)
f_{cr} = design flexural cracking stress of the hypothetical unreinforced concrete beam consisting of the cover concrete over the inside face of a stack of horizontally-curved post-tensioned tendons (ksi) (5.9.5.4.4c)
f_{ct} = average splitting tensile strength of lightweight concrete (ksi) (5.4.2.8)
f_{cu} = limiting compressive stress at the face of a node (ksi) (5.8.2.5.1)
f_{max} = maximum principal stress in the web, compression positive (ksi) (C5.9.2.3.3)
f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the unfactored permanent loads or the unfactored permanent loads, shrinkage, and creep-induced external loads (ksi); minimum principal stress in the web, tension negative (ksi) (5.5.3.2) (5.9.2.3.3)
f_n = nominal concrete bearing stress (ksi) (5.8.4.4.2)
f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi) (C5.9.3.2.3a)
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) (5.7.2.1)
f_{pcx} = horizontal stress in the web (ksi) (5.9.2.3.3)
f_{pcy} = vertical stress in the web (ksi) (5.9.2.3.3)
f_{pe} = effective stress in prestressing steel after losses (ksi) (5.6.3.1.2)
f_{pi} = prestressing steel stress immediately prior to transfer (ksi) (5.9.3.3)
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) (5.7.3.4.2)

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f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of member is required (ksi) (5.6.3.1)
f_{psl} = stress in the strand at the service limit state. Cracked section shall be assumed (ksi) (5.12.3.3.9c)
f_{pt} = stress in prestressing strands immediately after transfer (ksi) (5.9.3.4.2c)
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) (5.12.3.3.9c)
f_{px} = design stress in pretensioned strand at nominal flexural strength at section of member under consideration (ksi) (5.9.4.3.2)
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 nonprestressed tension reinforcement at nominal flexural resistance (ksi); stress in steel (ksi) (5.6.3.1.1) (5.9.4.4.1)
f'_s = stress in the nonprestressed compression reinforcement at nominal flexural resistance (ksi) (5.6.3.1.1)
f_{ss} = calculated tensile stress in nonprestressed reinforcement at the service limit state not to exceed 0.60 f_y (ksi) (5.6.7)
f_t = direct tensile strength of concrete (ksi) (C5.4.2.7)
$f_{u\ell}$ = specified minimum tensile strength of column longitudinal reinforcement (ksi), 90 ksi for ASTM A615 and 80 ksi for ASTM A706 (5.10.8.4.2a)
f_y = specified minimum yield strength of reinforcement (ksi), note that limits on physical yield strength or on substitution limits in equations may be specified in various articles (5.5.3.2) (Appendix D5)
f_{ytr} = specified minimum yield strength of shaft transverse reinforcement (ksi) (5.10.8.4.2a)
f'_y = specified minimum yield strength of compression reinforcement (ksi) (5.6.2.1)
f_{yh} = specified minimum yield strength of spiral reinforcement (ksi); yield strength of tie or spiral reinforcement (ksi) ≤ 75.0 ksi (5.6.4.6) (5.11.4.1.4)
H = average annual ambient relative humidity (percent) (5.4.2.3.2)
h = overall thickness or depth of a member (in.); lateral dimension of the cross section in the direction considered (in.); overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.); least thickness of component section (in.) (5.6.7) (5.8.4.5.3) (5.9.4.4.1) (5.10.6)
h_a = length of the back face of an STM node (in.) (5.8.2.2)
h_c = span of the web between the top and bottom slabs measured along the axis of the web (in.); core dimension of tied column in direction under consideration (in.) (5.9.5.4.4d) (5.11.4.1.4)
h_{ds} = height of the duct stack (in.) (5.9.5.4.4c)
h_f = compression flange depth (in.); compression flange depth of an I- or T-member (in.) (5.6.3.1.1) (5.6.3.2.2)
h_{STM} = node-to-node depth of STM (C5.8.2.2)
h_1 = largest lateral dimension of member (in.) (C5.9.5.6.5b)
h_2 = least lateral dimension of member (in.) (C5.9.5.6.5b)
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}) (5.9.3.4.3a)
I_{cr} = moment of inertia of the cracked section, transformed to concrete (in.^{4}) (5.6.3.5.2)
IE = for segmental construction: dynamic load from equipment (kip) (5.12.5.3.2)
I_e = effective moment of inertia (in.^{4}) (5.6.3.5.2)
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4}) (5.6.3.5.2)
I_s = moment of inertia of the longitudinal reinforcement about the centroidal axis (in.^{4}) (5.6.4.3)
K = effective length factor for compression members; wobble friction coefficient (per ft of tendon) (5.6.4.1) (5.9.3.2.2b)
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 (5.9.3.4.3a)
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 (5.9.3.4.2a)
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 (5.9.3.4.2c)
K'_L = factor accounting for type of steel equal to 45 for low relaxation steel (C5.9.3.4.2c)
K_1 = correction factor for source of aggregate taken as 1.0 unless determined by physical test, and as approved by the Owner; fraction of concrete strength available to resist interface shear (5.4.2.4) (5.7.4.3)

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K_2 = limiting interface shear resistance (5.7.4.3)
k = factor representing the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance (5.10.8.4.2a)
k_c = factor for the effect of the volume-to-surface ratio for creep; ratio of the maximum concrete compressive stress to the design compressive strength of concrete (C5.4.2.3.2) (5.6.4.4)
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 of the component (5.4.2.3.2)
k_{td} = time development factor (5.4.2.3.2)
k_{tr} = transverse reinforcement index (5.10.8.2.1c)
L = length of bearing pad (in.); span length (ft); span length between supports (ft) (5.8.4.3.4) (5.12.2.1) (C5.12.5.3.11d)
L_{vi} = interface length considered to be engaged in shear transfer (in.) (5.7.4.3)
ℓ_a = effective length of a CTT node (in.) (5.8.2.2); embedment length beyond the center of a support or at point of inflection (in.) (C5.10.8.1.2b)
ℓ_b = length of the bearing face (in.) (5.8.2.2)
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.) (5.8.4.5.2) (5.10.8.4.5a)
ℓ_d = development length (in.) (5.9.4.3.2)
ℓ_{db} = basic development length for straight reinforcement to which modification factors are applied to determine ℓ_d (in.) (5.10.8.2.1a)
ℓ_{dh} = development length of deformed bars in tension terminating in a standard hook (in.) (5.10.8.2.4a)
ℓ_{dsh} = total length of extended strand (in.) (5.12.3.3.9c)
ℓ_e = effective tendon length (in.); embedment length between midheight of the member and the outside end of the hook (in.) (5.6.3.1.2) (5.10.8.2.6b)
ℓ_{hb} = basic development length of standard hook in tension (in.) (5.10.8.2.4a)
ℓ_i = length of tendon between anchorages (in.) (5.6.3.1.2)
ℓ_{px} = distance from free end of pretensioned strand to section of member under consideration (in.) (5.9.4.3.2)
ℓ_s = required tension lap splice length of the column longitudinal reinforcement (in.) (5.10.8.4.2a)
ℓ_u = unbraced length (in.) (5.6.4.1)
M_a = maximum moment in a component at the stage for which deformation is computed (kip-in.) (5.6.3.5.2)
M_c = magnified factored moment (kip-in.) (5.6.4.3)
M_{cr} = cracking moment (kip-in.) (5.6.3.3)
M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.) (5.6.3.3)
M_{end} = moment at the ends of a hypothetical unreinforced concrete beam consisting of the cover concrete over the inside face of a stack of horizontally-curved post-tensioned tendons (kip-in.) (5.9.5.4.4c)
M_g = midspan moment due to member self-weight (kip-in.) (C5.9.3.2.3a)
M_{mid} = moment at the midpoint of a hypothetical unreinforced concrete beam consisting of the cover concrete over the inside face of a stack of horizontally-curved post-tensioned tendons (kip-in.) (5.9.5.4.4c)
M_n = nominal flexural resistance (kip-in.) (5.6.3.2.1)
M_r = factored flexural resistance (kip-in.) (5.6.3.2.1)
M_{rx} = uniaxial factored flexural resistance of a section in the direction of the x-axis (kip-in.) (5.6.4.5)
M_{ry} = uniaxial factored flexural resistance of a section in the direction of the y-axis (kip-in.) (5.6.4.5)
M_u = factored moment at the section (kip-in.) (5.7.3.4.2)
M_{ux} = factored applied moment about the x-axis (kip-in.) (5.6.4.5)
M_{uy} = factored applied moment about the y-axis (kip-in.) (5.6.4.5)
M_{u2} = maximum factored moment at section 2 (kip-in.) (C5.7.4.5)
M_1 = smaller end moment at the strength limit state due to factored loads acting on a compression member; positive if the member is bent in single curvature and negative if bent in double curvature (kip-in.); factored moment at section 1 concurrent with M_{u2} (5.6.4.3) (C5.7.4.5)
M_2 = larger end moment at the strength limit state due to factored loads acting on a compression member; always positive (kip-in.) (5.6.4.3)
m = confinement modification factor (5.6.5)
N = total cycles of loading; number of identical prestressing tendons (5.5.3.4) (5.9.3.2.3b)
N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points (5.6.3.1.2)
N_u = factored axial force, taken as positive if tensile and negative if compressive (kip) (5.7.3.4.2)

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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) (5.8.4.2.1)
N_1 = number of tendons in the larger group (C5.9.3.2.3b)
N_2 = number of tendons in the smaller group (C5.9.3.2.3b)
n = modular ratio = E_s/E_c or E_p/E_c; projection of base plate beyond the wedge hole or wedge plate, as appropriate (in.); number of anchorages in a row; number of bars or wires developed along plane of splitting; modular ratio between deck concrete and reinforcement (5.6.1) (5.8.4.4.2) (5.8.4.5.2) (5.10.8.2.1c) (C5.12.3.3.3)
P_c = permanent net compressive force, normal to the shear plane (kip) (5.7.4.3)
P_n = nominal axial resistance (kip); nominal bearing resistance (kip); nominal resistance of a node face or tie (kip) (5.6.4.4) (5.6.5) (5.8.2.3)
P_o = nominal axial resistance of a section at 0.0 eccentricity (kip) (5.6.4.5)
P_r = factored axial resistance; factored resistance of a node face or tie (kip); factored bearing resistance of anchorages (kip); factored splitting resistance of pretensioned anchorage zones provided by reinforcement in the end of pretensioned beams (kip) (5.6.4.4) (5.8.2.3) (5.8.4.4.2) (5.9.4.4.1)
P_{rx} = factored axial resistance determined on the basis that only eccentricity e_y is present (kip) (5.6.4.5)
P_{rxy} = factored axial resistance in biaxial flexure (kip) (5.6.4.5)
P_{ry} = factored axial resistance determined on the basis that only eccentricity e_x is present (kip) (5.6.4.5)
P_s = unfactored tendon force(s) at the anchorage (kip) (5.9.5.6.7b)
P_u = factored applied axial force; factored tendon force (kip); factored tendon force on an individual anchor (kip); minimum factored axial load (kip) (5.6.4.3) (5.8.4.5.2) (5.8.4.5.5) (5.11.4.4)
p_c = length of outside perimeter of the concrete section (in.) (5.7.2.1)
p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.); perimeter of the centerline of the closed transverse torsion reinforcement for solid members, or the perimeter of the centroid of the transverse torsion reinforcement in the exterior webs and flanges for hollow members (in.); perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion (in.) (5.7.3.4.2) (5.7.3.6.3) (5.12.5.3.8d)
Q = force effect in associated units (5.12.5.3.4a)
Q_g = lesser of the first moment of gross concrete area above or below the height of the web where the principal tension is being checked (in.^{3}) (5.9.2.3.3)
R = radius of curvature of the tendon at the considered location (ft); radius of curvature of the tendon in a vertical plane at the considered location (ft) (5.9.5.4.4a) (5.9.5.4.5)
r = radius of gyration of gross cross section (in.) (5.6.4.1)
S = center-to-center spacing of bearing along a beam ledge (in.) (5.8.4.3.2)
S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3}) (5.6.3.3)
SH = shrinkage (5.12.5.3.2)
S_{max} = spacing of transverse shaft reinforcement (in.) (5.10.8.4.2a)
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}) (5.6.3.3)
s = pitch of spiral or vertical spacing of hoops (in.); average spacing of nonprestressed reinforcement in layer closest to tension face (in.); spacing of transverse reinforcement (in.); spacing of hanger reinforcing bars (in.); center-to-center spacing of anchorages (in.); anchorage spacing (in.); maximum center-to-center spacing of transverse reinforcement within ℓ_d (in.); vertical spacing of hoops, not exceeding 4.0 in. (in.); spacing of stirrups (in.) (5.6.4.6) (5.6.7) (5.7.2.5) (5.8.4.3.5) (5.8.4.5.2) (5.8.4.5.5) (5.10.8.2.1c) (5.11.4.1.4) (5.12.5.3.8c)
s_h = spacing of horizontal crack control reinforcement (in.) (5.8.2.6)
s_{max} = maximum permitted spacing of transverse reinforcement (in.) (5.7.2.6)
s_v = spacing of vertical crack control reinforcement (in.) (5.8.2.6)
s_w = spacing of wires to be developed or spliced (in.) (5.10.8.2.5)
s_x = crack spacing parameter (in.) (5.7.3.4.2)
s_{xe} = crack spacing parameter as influenced by aggregate size (in.) (5.7.3.4.2)
T = concurrent torsional moment for Service III load combination (kip-in.); thermal (°F) (5.9.2.3.3) (5.12.5.3.2)
T_{burst} = tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis (kip) (5.8.4.5.3)
T_{cr} = torsional cracking moment (kip-in.) (5.7.2.1)
T_{ia} = tie-back tension force at the intermediate anchorage (kip) (5.9.5.6.7b)

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T_n = nominal torsional resistance (kip-in.) (5.7.2.1)
T_r = factored torsional resistance (kip-in.) (5.7.2.1)
T_u = applied factored torsional moment (kip-in.); applied factored torsional moment on the girder (5.7.2.1) (5.7.3.4.2)
T_1 = edge tension force (kip) (5.8.4.5.5)
T_2 = bursting force (kip) (5.8.4.5.5)
t = maturity of concrete (day); thickness of wall (in.); average thickness of bearing plate (in.); member thickness (in.); time between strand tensioning and deck placement (day) (5.4.2.3.2) (5.6.4.7.1) (5.8.4.4.2) (5.8.4.5.2) (C5.9.3.4.2c)
t_d = age at deck placement (day) (5.9.3.4.2b)
t_f = final age (day) (5.9.3.4.2a)
t_i = age of concrete at time of initial load application (day); age of concrete at time of initial deck placement (day) (5.4.2.3.2) (5.9.3.4.2a)
U = for segmental construction: segment unbalance (kip) (5.12.5.3.2)
V = shear force for Service III load combination (kip) (5.9.2.3.3)
V_c = nominal shear resistance of the concrete (kip) (5.7.2.3)
V_{hi} = factored interface shear force per unit length (kips/length) (C5.7.4.5)
V_n = nominal shear resistance (kip); nominal punching shear resistance (kip); nominal shear resistance of two shear planes per unit length (kips/in.) (5.7.2.1) (5.8.4.3.4) (5.9.5.4.4b)
V_{ni} = nominal interface shear resistance (kip) (5.7.2.1)
V_p = component of prestressing force in the direction of the shear force (kip) (5.7.2.3)
V_r = factored shear resistance (kip); shear resistance per unit length of the concrete cover against pullout by deviation forces (kip-in.) (5.7.2.1) (5.9.5.4.4b)
V_{ri} = factored interface shear resistance (kip) (5.7.4.3)
V/S = volume-to-surface ratio (5.4.2.3.2)
V_s = shear resistance provided by transverse reinforcement (kip) (5.7.3.3)
V_u = factored shear force (kip); factored shear force for the girder or for the web under consideration (kip) (5.7.2.3) (5.7.3.4.2)
V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations (kip); factored interface shear force for a concrete girder/slab bridge (kip/ft) (5.7.4.3) (5.7.4.5)
V_{u1} = maximum factored vertical shear at section 1 (kip) (5.7.4.2)
V_1 = factored vertical shear at section 1 concurrent with M_{u2} (kip) (C5.7.4.5)
v = concrete efficiency factor (5.8.2.5.3a)
v_u = shear stress (ksi) (5.7.2.6)
W = width of bearing plate or pad (in.) (5.8.4.3.2)
W/CM = water/cementitious materials ratio designated in earlier practice as water–cement ratio (5.4.2.1)
WE = for segmental construction: horizontal wind load on equipment (kip) (5.12.5.3.2)
WS = for segmental construction: horizontal wind on structure (ksf) (5.12.5.3.2)
WUP = for segmental construction: wind uplift on cantilever (ksf) (5.12.5.3.2)
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.) (5.6.4.7.1)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft) (5.9.3.2.2b)
y_t = distance from the neutral axis to the extreme tension fiber (in.) (5.6.3.5.2)
α = angle of inclination of transverse reinforcement to longitudinal axis (degrees); fraction defining the bearing face length of a portion of a nodal region; angle of inclination of a tendon force with respect to the centerline of the member, positive for concentric tendons or if the anchor force points toward the centroid of the section, negative if the anchor force points away from the centroid of the section (degrees); total angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.7.3.3) (5.8.2.2) (5.8.4.5.3) (5.9.3.2.2b)
α_h = total horizontal angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.3.2.2b)
α_v = total vertical angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.3.2.2b)
α_1 = stress block factor taken as the ratio of equivalent rectangular concrete compressive stress block intensity to the compressive strength of concrete used in design (5.6.2.2)
β = factor indicating the ability of diagonally cracked concrete to transmit tension and shear; ratio of long side to short side of footing (5.7.3.3) (5.12.8.5)

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β_b = ratio of the area of reinforcement cutoff to the total area of tension reinforcement at the section (C5.10.8.1.2a)
β_c = ratio of the long side to the short side of the rectangle through which the concentrated load or reaction force is transmitted (5.12.8.6.3)
β_d = ratio of maximum factored permanent moments to maximum factored total load moment; always positive (5.6.4.3)
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 (5.6.7)
β_1 = stress block factor taken as the 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 (5.6.2.2)
γ = load factor (5.5.3.1)
γ_e = exposure factor (5.6.7)
γ_h = correction factor for relative humidity of the ambient air (5.9.3.3)
γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member (5.9.3.3)
Δf = force effect, live load stress range due to the passage of the 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) (5.9.3.4.3b)
Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi) (5.9.3.4.3d)
Δf_{cdp} = change in concrete stress at center of gravity of prestressing steel due to all dead loads, except dead load acting at the time the prestressing force is applied (ksi) (5.9.3.4.3)
Δf_{pA} = loss due to anchorage set (ksi) (5.9.3.1)
Δf_{pCD} = change in prestress due to creep of girder concrete between time of deck placement and final time (ksi) (5.9.3.4.1)
Δf_{pCR} = prestress loss due to creep of girder concrete between transfer and deck placement (ksi) (5.9.3.4.1)
Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi) (5.9.3.1)
Δf_{pF} = loss due to friction (ksi) (5.9.3.1)
Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi) (5.9.3.1)
Δf_{pR1} = prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (ksi) (5.9.3.4.1)
Δf_{pR2} = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi) (5.9.3.4.1)
Δf_{pSD} = prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi) (5.9.3.4.1)
Δf_{pSR} = prestress loss due to shrinkage of girder concrete between transfer and deck placement (ksi) (5.9.3.4.1)
Δf_{pSS} = prestress gain due to shrinkage of deck in composite section (ksi) (5.9.3.4.1)
Δf_{pT} = total loss (ksi) (5.9.3.1)
$\Delta \ell$ = unit length segment of girder (in.) (C5.7.4.5)
δ = duct diameter correction factor, taken as 2.0 for grouted ducts (5.7.3.3)
ε_{bdf} = shrinkage strain of girder between time of deck placement and final time (in./in.) (5.9.3.4.3a)
ε_{bid} = concrete shrinkage strain of girder between time of transfer and deck placement (in./in.) (5.9.3.4.2a)
ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.) (5.5.4.2)
ε_{cu} = failure strain of concrete in compression (in./in.) (5.6.4.4)
ε_{ddf} = shrinkage strain of deck concrete between placement and final time (in./in.) (5.9.3.4.3d)
$\varepsilon_{effective}$ = effective concrete shrinkage strain (in./in.) (C5.12.3.3.3)
ε_s = net longitudinal tensile strain in the section at the centroid of the tension reinforcement (in./in.) (5.7.3.4.2)
ε_{sh} = concrete shrinkage strain at a given time (in./in.); unrestrained shrinkage strain for deck concrete (in./in.) (5.4.2.3.3) (C5.12.3.3.3)
ε_t = net tensile strain in extreme tension steel at nominal resistance (in./in.) (5.5.4.2)
ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.) (5.5.4.2)
ε_x = longitudinal strain at the mid-depth of the member (in./in.) (B5.2)
θ = angle of inclination of diagonal compressive stresses (degrees) (5.7.3.3)
θ_s = angle between strut and longitudinal axis of the member (degrees) (5.8.2.2)

κ = correction factor for closely spaced anchorages; multiplier for strand development length (5.8.4.5.2) (5.9.4.3.2)
λ = concrete density modification factor (5.4.2.8)
λ_{cf} = coating factor (5.10.8.2.1a)
λ_{duct} = shear strength reduction factor accounting for the reduction in the shear resistance provided by transverse reinforcement due to the presence of a grouted post-tensioning duct, taken as 1.0 for ungrouted post-tensioning ducts and with a reduced web or flange width to account for the presence of ungrouted duct. (5.7.3.3)
λ_{er} = excess reinforcement factor (5.10.8.2.1a)
λ_{rc} = reinforcement confinement factor (5.10.8.2.1a)
λ_{rl} = reinforcement location factor (5.10.8.2.1a)
λ_w = wall slenderness ratio for hollow columns (5.6.4.7.1)
μ = friction factor (5.7.4.3)
ρ_h = ratio of area of horizontal shear reinforcement to area of gross concrete area of a vertical section (5.11.4.2)
ρ_s = ratio of spiral reinforcement to total volume of column core (5.6.4.6)
ρ_v = ratio of area of vertical shear reinforcement to area of gross concrete area of a horizontal section (5.11.4.2)
τ = shear stress in web vertical shear (5.9.2.3.3)
ϕ = resistance factor (5.5.4.2)
ϕ_{cont} = girder web continuity factor (5.9.5.4.4d)
ϕ_{duct} = diameter of post-tensioning duct present in the girder web within depth d_v (in.) (5.7.3.3)
ϕ_w = hollow column reduction factor (5.6.4.7.2c)
$\Psi(t, t_i)$ = creep coefficient at time t for loading applied at t_i (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 (5.9.3.4.2b)
$\Psi_b(t_f, t_d)$ = 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.) (5.9.3.4.3b) (5.9.3.4.3d)
$\Psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer (5.9.3.4.2a)

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.

5.4.2—Normal Weight and Lightweight Concrete

5.4.2.1—Compressive Strength

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

Design concrete compressive 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. Concrete with

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

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compressive strengths used in design below 2.4 ksi should not be used in structural applications.

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

For lightweight concrete, physical properties in addition to compressive strength shall be specified in the contract documents.

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 the design concrete compressive strengths and their current upper limit.

It is common practice that the compressive strength of concrete for use in design, or the Owner's specified strength if higher, 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.

It is recommended that the classes of concrete shown in the *AASHTO LRFD Bridge Construction Specifications*, Section 8, be used wherever appropriate.

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).

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

The sum of Portland cement and other cementitious materials shall be specified not to exceed 800 lb/yd^3, except for Class P (HPC) concrete where the sum of Portland cement and other cementitious materials shall be specified not to exceed 1000 lb/yd^3.

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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 (1992), ACI 343 (1995) and ACI 213 (2014).

5.4.2.3—Creep and Shrinkage

5.4.2.3.1—General

Values of creep and shrinkage, specified herein and in Articles 5.9.3.3 and 5.9.3.4, shall be used to determine the effects of creep and shrinkage 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 5.6.3.5.2, may be used to determine the effects of shrinkage and creep on deflections.

These provisions shall be applicable for design concrete compressive 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.

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

- Articles 5.4.2.3.2 and 5.4.2.3.3,
- The FIP Model Code for Concrete Structures (CEB 2010),
- CEB/FIP Model Code for Concrete Structures (CEB 1990), or
- ACI 209.

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

- specific materials,
- structural dimensions,
- site conditions,
- construction methods, and
- concrete age at various stages of erection.

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.

Values of modulus of elasticity, creep factors, and shrinkage factors should be taken from a consistent source.

5.4.2.3.2—Creep

The creep coefficient may be taken as:

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

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),

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in which:

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

$$k_{hc} = 1.56 - 0.008H \tag{5.4.2.3.2-3}$$

$$k_f = \frac{5}{1 + f'_{ci}} \tag{5.4.2.3.2-4}$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} \tag{5.4.2.3.2-5}$$

where:

H = average annual ambient relative humidity (percent). 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
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} = design concrete compressive strength 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_s may be taken as 1.00.

Rizkalla et al. (2007), and Collins and Mitchell (1991). These methods are based on the recommendation of ACI Committee 209 as modified by additional 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 the following:

- 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. 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:

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

For shrinkage:

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

(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:

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

in which:

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

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.

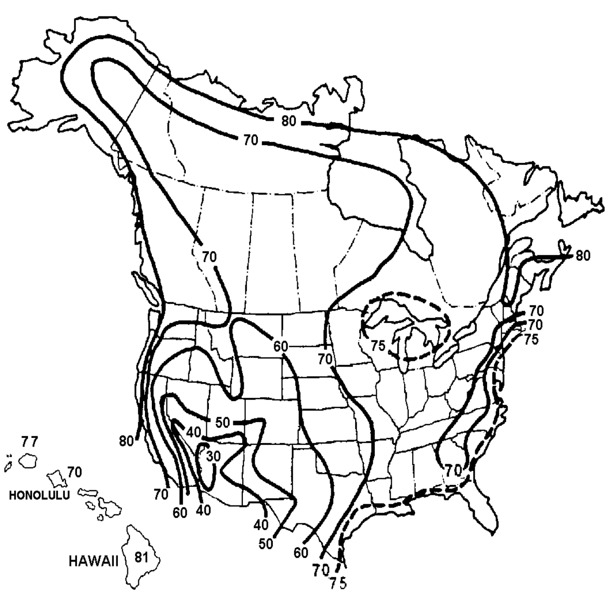

Figure 5.4.2.3.3-1—Average Annual Ambient Relative Humidity, in Percent

C5.4.2.3.3

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.

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5.4.2.4—Modulus of Elasticity

In the absence of measured data, the modulus of elasticity, E_c, for normal weight concrete with design compressive strengths up to 15.0 ksi and lightweight concrete up to 10.0 ksi, with unit weights between 0.090 and 0.155 kcf, may be taken as:

$$E_c = 120{,}000 K_1 w_c^{2.0} f_c'^{0.33} \quad (5.4.2.4\text{-}1)$$

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 Owner

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

f'_c = compressive strength of concrete for use in design (ksi)

C5.4.2.4

See commentary on strength in Article 5.4.2.1.

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

$$E_c = 2{,}500 f_c'^{0.33} \quad (C5.4.2.4\text{-}1)$$

Eqs. 5.4.2.4-1 and C5.4.2.4-1 are based upon the research of Greene and Graybeal (2013).

For normal weight concrete with w_c = 0.145 kcf and design compressive strengths up to 10.0 ksi, E_c may be determined from either of the following:

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'} \quad (C5.4.2.4\text{-}2)$$

$$E_c = 1{,}820 \sqrt{f'_c} \quad (C5.4.2.4\text{-}3)$$

Eqs. C5.4.2.4-2 and C5.4.2.4-3 are the traditional equations, which do not fully reflect lightweight concrete and higher compressive strengths.

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 lightweight concrete with design compressive strengths up to 10.0 ksi and for normal weight concrete with design compressive strengths up to 15.0 ksi. 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, for lightweight concrete with specified compressive strengths up to 10.0 ksi and normal weight concrete with specified compressive strengths up to 15.0 ksi may be taken as $0.24\,\lambda\,\sqrt{f'_c}$ where λ is the concrete density modification factor as specified in Article 5.4.2.8.

Where 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

Most modulus of rupture test data on normal weight concrete is between $0.24\sqrt{f'_c}$ and $0.37\sqrt{f'_c}$ (ksi) (Walker and Bloem, 1960; Khan, Cook, and Mitchell, 1996). A value of $0.37\sqrt{f'_c}$ has been recommended for the prediction of the tensile strength of high-strength concrete (ACI 363, 2010). However, the modulus of rupture is sensitive to curing methods and nearly all of the test units in the dataset mentioned previously were moist cured until testing. Carrasquillio et al. (1981) noted a 26 percent reduction in the 28-day modulus of rupture if high-strength units were allowed to dry after 7 days of moist curing over units that were moist cured until testing.

The flexural cracking stress of concrete members has been shown to significantly reduce with increasing member depth. Shioya et al. (1989) observed that the flexural cracking strength is proportional to $H^{-0.25}$ where H is the overall depth of the flexural member in inches. Based on this observation, a 36.0 in. deep girder should achieve a flexural cracking stress that is 36 percent lower than that of a 6.0 in. deep modulus of rupture test.

Since modulus of rupture units were either 4.0 or 6.0 in. deep and moist cured up to the time of testing, the modulus of rupture should be significantly greater than that of an average size bridge member composed of the same concrete. Therefore, $0.24\sqrt{f'_c}$ is appropriate for checking minimum reinforcement in Article 5.6.3.3.

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 splitting tensile strength method in accordance with AASHTO T 198 (ASTM C496).

C5.4.2.7

For normal weight concrete with design concrete compressive strengths up to 10.0 ksi, the direct tensile strength may be estimated as $f_t = 0.23\sqrt{f'_c}$.

5.4.2.8—Concrete Density Modification Factor

The concrete density modification factor, λ, shall be determined as:

- Where the splitting tensile strength, f_{ct}, is specified:

$$\lambda = 4.7\frac{f_{ct}}{\sqrt{f'_c}} \leq 1.0 \tag{5.4.2.8-1}$$

- Where f_{ct} is not specified:

$$0.75 \leq \lambda = 7.5\,w_c \leq 1.0 \tag{5.4.2.8-2}$$

- Where normal weight concrete is used, λ shall be taken as 1.0.

C5.4.2.8

The concrete density modification factor was developed based on available test data. There is a lack of data for concrete mix designs wherein a large majority of the fine aggregate is lightweight and a large majority of the coarse aggregate is normal weight. The concrete density modification factor, λ, is based on work on mechanical properties, development of reinforcement, and shear by Greene and Graybeal (2013), (2014), and (2015), respectively.

The determination of λ as defined in Eq. 5.4.2.8-2 is illustrated in Figure C5.4.2.8-1.

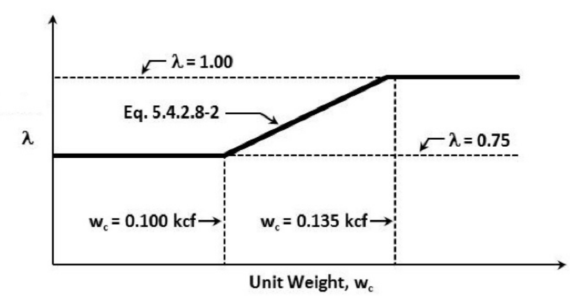

Figure C5.4.2.8-1—Illustration of λ as a Function of Unit Weight in Eq. 5.4.2.8-2

5.4.3—Reinforcing Steel

5.4.3.1—General

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

C5.4.3.1

Unlike reinforcing bars with yield strengths below 75.0 ksi, reinforcing bars with yield strengths exceeding 75.0 ksi usually do not have well-defined yield plateaus. Consequently, different methods are used in different standards to establish yield strengths. These include the 0.2 percent offset and the 0.35 percent or 0.50 percent extension methods. For design purposes, the value of f_y should be the same as the specified minimum yield strength defined in the material standard. Based on research by Shahrooz et al. (2011), certain articles now allow the use of reinforcement with yield strengths up to 100 ksi for all elements and connections in Seismic Zone 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. Yield strengths in excess of 75.0 ksi up to 100 ksi may be used for design permitted by Article 5.4.3.3. 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/A706M, "Standard Specification for Deformed and Plain Low-Alloy Steel Deformed Bars for Concrete Reinforcement," should be specified.

ASTM A706 reinforcement should be considered for seismic design because of its well defined yield plateau.

5.4.3.2—Modulus of Elasticity

The modulus of elasticity, E_s, of steel reinforcement shall be assumed as 29,000 ksi for specified minimum yield strengths up to 100 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.

Where permitted by specific articles, reinforcement with specified minimum yield strengths of less than or equal to 100 ksi may be used for all elements and connections in Seismic Zone 1.

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.

Reinforcement with a minimum specified yield strength between 75.0 and 100 ksi may be used in seismic applications, with the Owner's approval, only as permitted in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2011).

5.4.4—Prestressing Steel

5.4.4.1—General

Uncoated, low-relaxation, seven-wire strand, or uncoated plain or deformed, high-strength bars, shall conform to either of the following materials standards,

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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).

Requirements for these steels may be taken as specified in Table 5.4.4.1-1.

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	270 ksi	0.375 to 0.6	270	90% of f_{pu}
Bar	Type 1, Plain Type 2, Deformed	0.75 to 1.375 0.625 to 2.5	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 steel, 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

5.4.5—Post-Tensioning Anchorages and Couplers

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

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

5.4.6—Post-Tensioning Ducts

5.4.6.1—General

Ducts for tendons may be metallic or nonmetallic and shall conform to the requirements of Article 10.8.2 of the *AASHTO LRFD Bridge Construction Specifications*.

The minimum radius of curvature of tendon ducts shall take into account the tendon size, duct type and shape, and the location relative to the stressing anchorage; subject to the manufacturer’s recommendations.

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

C5.4.6.1

The use of nonmetallic duct is generally recommended in corrosive environments.

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

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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*.

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 in structural concrete shall not exceed 0.54 times the least gross concrete thickness at the duct.

C5.4.6.2

Research by Moore et. al. (2015) examined shear resistance of members with ducts and proposed a λ_{duct} factor to modify V_s for the presence of ducts. The testing and analysis in this research examined cases up to a ratio of 54 percent, a likely pragmatic maximum given cover, shear reinforcing, and duct size. The research was also limited to duct stacks and does not apply to bundled ducts. See Article 5.7.3.3.

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 AND DESIGN METHODOLOGIES

5.5.1—General

5.5.1.1—Limit-State Applicability

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

Prestressed concrete structural components shall be proportioned 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.

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

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

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5.5.1.2—Design Methodologies

5.5.1.2.1—General

Conventional beam theory based on Bernoulli's plane section hypothesis shall be considered applicable for the service and fatigue limit states. At the strength and extreme event limit states, regions of a concrete structure shall be characterized by their behavior as B-Regions (beam or Bernoulli) or D-Regions (disturbed or discontinuity). Bernoulli's hypothesis of straight-line strain profile, and therefore conventional beam theory, may be assumed to apply in B-Regions. A more complex variation in stress and strain exists in D-Regions as shown in Figure 5.5.1.2.1-1, where the effective depth of the member, *d*, is defined as the distance between the extreme compression fiber and the centroid of the primary longitudinal reinforcement.

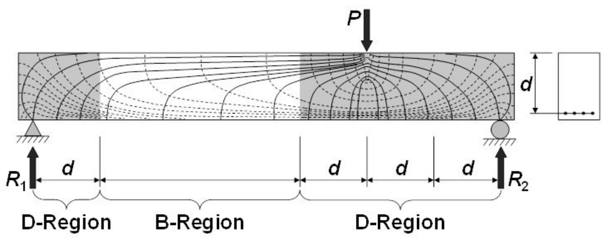

Figure 5.5.1.2.1-1— Stress Trajectories within B- and D-Regions of a Flexural Member (adapted from Birrcher et al., 2009)

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

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

C5.5.1.2.1

D-Regions occur in the vicinity of load or geometric discontinuities. In Figure 5.5.1.2.1-1, the applied load and support reactions are discontinuities that "disturb" the regions of the member near the locations at which they act. Frame corners, dapped ends, openings, and corbels are examples of geometric discontinuities which correspond to the existence of D-Regions.

The distribution of strains through the member depth in D-Regions is nonlinear, and the assumptions that underlie the sectional design procedure are therefore invalid. According to St. Venant's principle, an elastic stress analysis indicates that a linear distribution of stress can be assumed at approximately one member depth from a load or geometric discontinuity. In other words, a nonlinear stress distribution exists within one member depth from the location where the discontinuity is introduced (Schlaich et al., 1987). D-Regions are therefore assumed to extend approximately a distance *d* from the applied load and support reactions in Figure 5.5.1.2.1-1. In the case of the reaction at an interior support, the disturbed region extends a distance *d* on each side of the reaction.

B-Regions occur between D-Regions, as shown in Figure 5.5.1.2.1-1. Plane sections are assumed to remain plane within B-Regions according to the primary tenets of beam theory, implying that a linear distribution of strains occurs through the member depth. The beam is therefore dominated by sectional behavior, and design can proceed on a section-by-section basis (i.e., sectional design). For the flexural design of a B-Region, the compressive stresses (represented by solid lines in Figure 5.5.1.2.1-1) are conventionally assumed to act over a rectangular stress block, while the tensile stresses (represented by dashed lines) are assumed to be carried by the longitudinal steel reinforcement.

5.5.1.2.2—B-Regions

Design practices for B-Regions shall be based on a sectional model for behavior. Design for flexure in B-Regions shall be based on the conventional beam theory of Article 5.6 while the design for shear in B-Regions shall be based on conventional beam theory in conjunction with the truss analogy of Article 5.7.

Conventional beam theory is applicable to all limit states.

C5.5.1.2.2

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

5.5.1.2.3—D-Regions

For the strength and extreme event limit states, the strut-and-tie method (STM) of Article 5.8.2 or other methods from Article 5.8.3 or Article 5.8.4 may be applied for the design of all types of D-Regions in structural concrete.

C5.5.1.2.3

These specifications recognize three general classes of analysis methods for the design of D-Regions:

- The strut-and-tie method introduced here and explained in more detail in Article 5.8.2.

The most familiar types of D-Regions, such as beam ends, diaphragms, deep beams, brackets, corbels, and beam ledges, may be designed by the empirical approaches or the legacy detailing practices such as those found in Article 5.8.4.

- Elastic analysis introduced in Article 5.8.3.
- Legacy practices which are empirical and other approximate methods usually developed before refined analysis methods such as the STM became more widely used and are described in Article 5.8.4. Use of these methods is not preferred and is expected to decline in time.

Plasticity-based methods, which include STM, can be classified into two categories: upper-bound and lower-bound methods. The upper-bound solutions approach the resistance from the unconservative side, while lower-bound solutions approach the resistance from the conservative side. STM is a lower-bound design method. As such, it adheres to these principles: (1) the truss model is in equilibrium with external forces, and (2) the concrete element has enough deformation capacity to accommodate the assumed distribution of forces (Schlaich et al., 1987). Proper anchorage of the reinforcement is required. Additionally, the compressive forces in the concrete must not exceed the factored strut capacities, and the tensile forces within the strut-and-tie model must not exceed the factored tie capacities.

The STM recognizes the significance of how the loads are introduced into a disturbed region and how that region is supported, and is based on the assumption of straight line trajectories of internal stresses due to significant cracking beyond service loads. This method is also applicable to both B- and D-Regions, but it is typically not practical to apply the method to B-Regions.

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 5.6.7, 5.6.3.5, and 5.9.2.3, 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 unfactored permanent loads and prestress in reinforced 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.

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 1.75 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.

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Fatigue of the reinforcement need not be checked for prestressed components designed to have extreme fiber tensile stress due to Service III limit state within the tensile stress limit specified in Table 5.9.2.3.2b-1. Structural components with a combination of prestressing strands and reinforcing bars that allow the tensile stress in the concrete to exceed the Service III limit specified in Table 5.9.2.3.2b-1 shall be checked for fatigue.

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 prestressed components, the net concrete stress is usually significantly less than the concrete tensile stress limit specified in Table 5.9.2.3.2b-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 fatigue considerations, concrete members shall satisfy:

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

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)

For prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of the unfactored 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 and Welded Wire Reinforcement

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:

$$\left(\Delta F\right)_{TH} = 26 - \frac{22 f_{min}}{f_y} \qquad (5.5.3.2\text{-}1)$$

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:

$$\left(\Delta F\right)_{TH} = 18 - 0.36 f_{\min} \qquad (5.5.3.2\text{-}2)$$

C5.5.3.2

With the permitted use of steel reinforcement having yield stresses above 75.0 ksi, the value of f_{min} is expected to increase. In previous versions of Eq. 5.5.3.2-1, an increase in f_{min} would result in a decrease in $(\Delta F)_{TH}$, regardless of the yield strength of the bar. Current data indicates that steel with a higher yield strength actually has a higher fatigue limit (DeJong and MacDougall, 2006). Eq. 5.5.3.2-1 has been calibrated such that there is no change to the value of $(\Delta F)_{TH}$ from earlier versions of this equation for cases of f_y = 60.0 ksi, but it now provides more reasonable values of $(\Delta F)_{TH}$ for higher-strength reinforcing bars. The values of 60.0 and 100 ksi are limits of substitution into Eq. 5.5.3.2-1, not a prohibition on providing reinforcement with other yield strengths.

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where:

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

f_y = specified minimum yield strength of reinforcement, not to be taken less than 60.0 ksi nor greater than 100 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.

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 Helgason et al. (1976). The simplified form in this edition replaces the (r/h) parameter with the default value 0.3 recommended by Helgason et al. (1976). Inclusion of limits for welded wire reinforcement is based on research by Hawkins et al. (1971, 1987) and Amorn et al. (2007).

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

5.5.3.3—Prestressing Steel

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for prestressing steel 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	18.0 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	12.0 ksi
All other types of splices	4.0 ksi

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 4.0 ksi constant amplitude stress range. This lower limit is a close lower bound for the splice fatigue data obtained in NCHRP Project 10-35, “Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel,” 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 5.10.8.4.2b and 5.10.8.4.2c 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 5.10.8.4.3b 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

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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.

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, 5.11, 5.12, and 5.13, unless another limit state is specifically identified, and the resistance factor 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

The provisions of this article are applicable to prestressed concrete sections and to reinforced concrete sections with nonprestressed reinforcement having specified minimum yield strengths up to 100 ksi for elements and connections specified in Article 5.4.3.3. Where no distinction is made for density the values given shall be taken to apply to normal weight and lightweight concrete.

Resistance factor ϕ shall be taken as:

- For tension-controlled reinforced concrete sections as specified in Article 5.6.2.1:
  - normal weight concrete 0.90
  - lightweight concrete 0.90
- For tension-controlled prestressed concrete sections with bonded strand or tendons as specified in Article 5.6.2.1:
  - normal weight concrete 1.00
  - lightweight concrete 1.00
- For tension-controlled post-tensioned concrete sections with unbonded strand or tendons as specified in Article 5.6.2.1:
  - normal weight concrete 0.90
  - lightweight concrete 0.90

C5.5.4.2

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 primary prestressing forces are not included.

In editions of and interims to the AASHTO *LRFD Bridge Design Specifications* prior to 2005, the provisions specified the magnitude of the resistance factor for cases of axial load or flexure, or both, in 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 318-14.

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.

The use of debonded strand in a nontension-controlled zone qualifies for $\phi = 1.00$.

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.

- For shear and torsion in reinforced concrete sections:
  - normal weight concrete 0.90
  - lightweight concrete 0.90

- For shear and torsion in monolithic prestressed concrete sections and prestressed concrete sections with cast-in-place closures or with match cast and epoxied joints having bonded strands or tendons:
  - normal weight concrete 0.90
  - lightweight concrete 0.90

- For shear and torsion in monolithic prestressed concrete sections and prestressed concrete sections with cast-in-place closures or with match cast and epoxied joints having unbonded or debonded strands or tendons:
  - normal weight concrete 0.85
  - lightweight concrete 0.85

- For compression-controlled sections with spirals or ties, as specified in Article 5.6.2.1, except as specified in Articles 5.11.3 and 5.11.4.1.2 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

- For tension in strut-and-tie models:
  - reinforced concrete…….……………...……..0.90
  - prestressed concrete……………………...….1.00

- For compression in anchorage zones:
  - normal weight concrete 0.80
  - lightweight concrete 0.80

- For tension in steel in anchorage zones............. 1.00

- For resistance during pile driving 1.00

Compression-controlled and tension-controlled sections are specified in Article 5.6.2.1 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 the tension-controlled strain limit, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal resistance between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Figure C5.5.4.2-1. The concept of net tensile strain ε_t is discussed in Article C5.6.2.1. 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.

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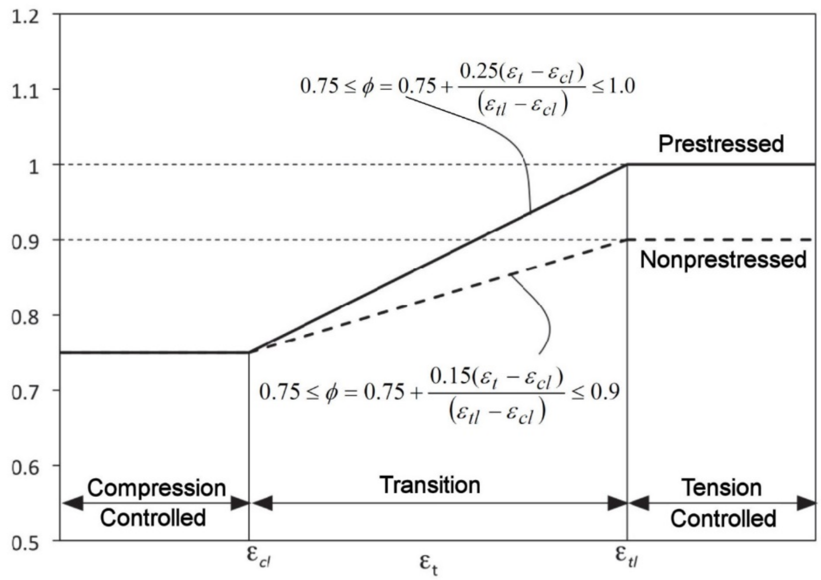

Figure C5.5.4.2-1—Variation of ϕ with Net Tensile Strain ε_t for Nonprestressed Reinforcement and for Prestressing Steel

For sections in which the net tensile strain in the extreme tension steel at nominal resistance is between the compression-controlled strain limit, ε_{cl} and tension-controlled strain limit, ε_{tl}, the value of ϕ associated with net tensile strain may be obtained by a linear interpolation from 0.75 to that for tension-controlled sections.

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

$$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0 \qquad (5.5.4.2\text{-}1)$$

and for nonprestressed members such that:

$$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9 \qquad (5.5.4.2\text{-}2)$$

where:

ε_t = net tensile strain in the extreme tension steel at nominal resistance (in./in.)
ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.)
ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.)

Where combinations of different grades of nonprestressed reinforcement are used in design, the lowest resistance factor calculated for each grade of reinforcement shall be used.

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

The ϕ-factor of 0.8 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 design of intermediate anchorages, anchorages, diaphragms, and multiple slab anchorages are addressed in Breen et al. (1994).

The typical cross section of a continuous concrete box girder often has both conventional bar reinforcing and post-tensioning ducts. This superstructure, however, is first designed to satisfy the service limit state by determining the number of tendons required to satisfy allowable stress limits. Then, the strength limit state is checked. Nonprestressed reinforcement may or may not need to be added. If nonprestressed reinforcement is required to satisfy the strength but not the service limit state, the member is still considered prestressed for the purpose of determining the appropriate resistance factor.

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Joints between precast units shall be either cast-in-place closures or match cast and epoxied joints. In selecting resistance factors for flexure, ϕ_f, and shear and torsion, ϕ_v, the bonding condition of the post-tensioning system shall be considered. In order for a tendon to be considered as bonded at a section, it should be fully developed at that section for a development length not less than that required by Article 5.9.4.3. Shorter embedment lengths may be permitted if demonstrated by full-size tests and approved by the Engineer.

Comprehensive tests of a large continuous three-span model of a twin-cell box girder bridge built from precast segments with 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.

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

C5.5.4.3

Stability during handling, transportation, and erection can govern the design of precast, prestressed girders. Precast members should be designed such that safe storage, handling, and erection can be accomplished by the contractor. This consideration does not make the designer responsible for the contractor's means and methods for construction, as discussed in Article 2.5.3.

Lateral bending stability analysis should be based on the "Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders", Precast Concrete Institute, Publication CB-02-16-E. A detailed design example is presented in Seguirant, Brice, and Khaleghi (2009).

5.5.5—Extreme Event Limit State

5.5.5.1—General

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.

5.5.5.2—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 5.11.3 and 5.11.4.1.2.

5.6—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS—B REGIONS

Reference to nonprestressed reinforcement in this article shall apply to steel with specified minimum yield strengths up to 100 ksi for elements and connections specified in Article 5.4.3.3.

C5.6

Reinforcement with specified minimum yield strengths between 75.0 and 100 ksi may be used in seismic applications, with the Owner's approval, only as permitted in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2011).

5.6.1—Assumptions for Service and Fatigue Limit States

The following assumptions may be used in the design of reinforced and prestressed concrete components for all design concrete compressive strengths permitted by these specifications:

- Prestressed concrete resists tension at sections that are uncracked, except as specified in Article 5.6.6.
- The strains in the concrete vary linearly, except in components or regions of components for which conventional strength of materials is inappropriate.
- Where transformed section analysis is used to assess the elastic shortening at transfer or post-tensioning and elastic response due to transient loads in prestressed components, or the determination of elastic stresses in reinforced concrete components, the transformed area properties may be calculated by replacing the steel area with an equivalent concrete area equal to the steel area multiplied by a modular ratio defined as:
  - E_s / E_c for reinforcing bars
  - E_p / E_c for prestressing tendons
- Where transformed section analysis is used to assess the time-dependent response to permanent loads, an age adjusted effective modular ratio that accounts for creep properties of concrete may be employed using the provisions of Article 5.4.2.3.2.

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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.

Transformed section properties are used in the working stress methods based on elastic and time-dependent analysis, for instantaneous and creep effects, respectively. The methods are applicable for service and fatigue limit states. Approximate analysis using gross section properties may be adequate in some designs as long as volume change effects are recognized, and prestress losses are accounted for as specified in Article 5.9.3.

Where the applied load is constant over time, the age adjusted modulus of elasticity may be taken as the modulus of elasticity divided by 1 plus the creep coefficient given by Eq. 5.4.2.3.2-1. Using this method to include the effect of creep puts the emphasis on the material subject to creep as opposed to previous approaches that adjusted the steel rather than the concrete. It is also more consistent with Article 5.9.3.4, Refined Estimates of Time-Dependent Losses.

5.6.2—Assumptions for Strength and Extreme Event Limit States

The following assumptions may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

5.6.2.1—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 bonded reinforcement or prestressing, or in the bonded length of debonded strands, strain is directly proportional to the distance from the neutral axis, except for deep members that shall satisfy the requirements for disturbed regions.
- In components with unbonded prestressing tendons, i.e., not debonded strands, the difference in strain between the tendons and the concrete section and

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The first paragraph of Article C5.6.1 applies.

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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 reinforcement 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 reinforcement and prestressing elements and transfer of pretensioning are considered.
- Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.
- Sections are compression-controlled where the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit, ε_{cl}, 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 nonprestressed reinforcement with a specified minimum yield strength of $f_y \leq 60.0$ ksi, ε_{cl} is taken as f_y/E_s but not greater than 0.002. For nonprestressed reinforcement with a specified minimum yield strength of 100 ksi, the compression-controlled strain limit may be taken as $\varepsilon_{cl} = 0.004$. For nonprestressed reinforcement with a specified minimum yield strength between 60.0 and 100 ksi, the compression-controlled strain limit may be determined by linear interpolation based on specified minimum yield strength. For all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.
- Sections are tension-controlled where the net tensile strain in the extreme tension steel is equal to or greater than the tension-controlled strain limit, ε_{tl} 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 the

The results of Rizkalla et al. (2007) have shown that the maximum usable strain at the extreme concrete compression fiber of 0.003 is valid for flexural members with design compressive strengths up to 18.0 ksi for normal weight concrete even though the provisions are currently limited to 15.0 ksi.

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 usable 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.

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 C5.6.2.1-1, using similar triangles.

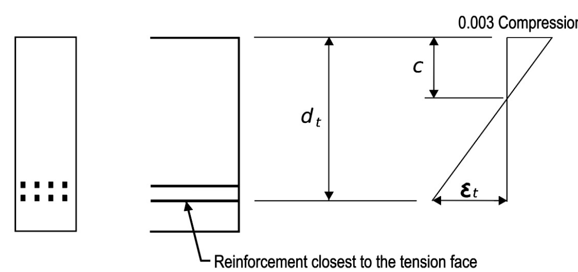

Figure C5.6.2.1-1—Strain Distribution and Net Tensile Strain

Where the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than the tension-controlled strain limit), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. Where the net tensile strain in the extreme tension steel is small (less than or equal to the compression-tension-controlled strain limit constitute a transition region between compression-controlled and tension-controlled sections. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.005 for nonprestressed reinforcement with a specified minimum yield strength, $f_y \leq 75.0$ ksi, and prestressed reinforcement. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.008 for nonprestressed reinforcement with a specified minimum yield strength, $f_y = 100$ ksi. For nonprestressed reinforcement with a specified minimum yield strength between 75.0 and 100 ksi, the tension-controlled strain limit shall be determined by linear interpolation based on specified minimum yield strength.

- The use of compression reinforcement in conjunction with additional tension reinforcement is permitted to increase the strength of flexural members.

- In the approximate flexural resistance equations of Articles 5.6.3.1 and 5.6.3.2, f_y and f'_y may replace f_s and f'_s, respectively, subject to the following conditions:
  - f_s may be replaced by f_y in the calculations in the cited articles as long as the resulting ratio c/d_s does not exceed:

$$\frac{c}{d_s} \leq \frac{0.003}{0.003 + \varepsilon_{cl}} \qquad (5.6.2.1\text{-}1)$$

where:

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

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

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 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 nonprestressed reinforcement with a specified minimum yield strength of 75.0 ksi or less and prestressed reinforcement. Research by Shahrooz et al. (2011) and Mast (2008) supports the values stated for compression- and tension-controlled strain limits for steel with higher specified minimum yield strengths.

Unless unusual amounts of ductility are required, the tension-controlled strain 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 5.6.3.4 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 $1.5\varepsilon_{cl}$.

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.

Where the approximate flexural resistance equations in Articles 5.6.3.1 and 5.6.3.2 are used, it is important to assure that both the nonprestressed tension and compression 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 nonprestressed tension reinforcement would yield at nominal flexural resistance, but this limit was eliminated in the 2006 interim revisions. The current limit on c/d_s assures that the nonprestressed tension reinforcement will be at or near yield. The ratio $c \geq 3d'_s$ assures that nonprestressed compression reinforcement with $f_y \leq 60.0$ ksi will yield. For yield strengths above 60.0 ksi, the yield strain is close to or exceeds 0.003, so the compression steel may not yield. It is conservative to ignore the compression steel where calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to

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ε_{cl} = compression-controlled strain limit as specified above

If c/d_s exceeds this limit, strain compatibility shall be used to determine the stress in the nonprestressed tension reinforcement.

- f'_s may be replaced by f'_y in the calculations in the cited articles as long as $c \geq 3d'_s$ and $f_y \leq$ 60.0 ksi. If $c < 3d'_s$ or $f_y > 60.0$ ksi, strain compatibility shall be used to determine the stress in the nonprestressed compression reinforcement. Alternatively, the compression reinforcement may be conservatively ignored, i.e., $A'_s = 0$.

- Where strain compatibility is used, the calculated stress in nonprestressed reinforcement having a specified minimum yield strength between 75.0 and 100 ksi shall not be taken as greater than the specified minimum yield strength.

Additional limitations on the maximum usable extreme concrete compressive strain in hollow rectangular compression members shall be investigated as specified in Article 5.6.4.7.

use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

Values of the compression- and tension-controlled strain limits are given in Table C5.6.2.1-1 for common values of specified minimum yield strengths.

Table C5.6.2.1-1—Strain Limits for Nonprestressed Reinforcement

Specified Minimum Yield Strength, ksi	Strain Limits: Compression Control ε_{cl}	Strain Limits: Tension Control ε_{tl}
60	0.0020	0.0050
75	0.0028	0.0050
80	0.0030	0.0056
100	0.0040	0.0080

The nonprestressed 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.

5.6.2.2—Rectangular Stress Distribution

The natural relationship between concrete stress and strain may be considered satisfied by an equivalent rectangular concrete compressive stress block of $\alpha_1 f'_c$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$ from the extreme compression fiber. The distance c shall be measured perpendicular to the neutral axis. The stress block factor α_1 shall be taken as 0.85 for design compressive strengths of concrete not exceeding 10.0 ksi. For design compressive strengths of concrete exceeding 10.0 ksi, α_1 shall be reduced at a rate of 0.02 for each 1.0 ksi of strength in excess of 10.0 ksi, except that α_1 shall not be taken to be less than 0.75. The stress block factor β_1 shall be taken as 0.85 for design compressive strengths of concrete not exceeding 4.0 ksi. For design compressive strengths of concrete 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.

Additional limitations on the use of the rectangular stress block when applied to hollow rectangular

C5.6.2.2

For practical design, the rectangular compressive stress distribution defined in this article may be used in lieu of a more exact concrete stress distribution. This rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but in many practical cases it does provide essentially the same results as those obtained in tests. All strength equations presented in Article 5.6.3 are based on the rectangular stress block.

Rizkalla et al. (2007) determined that α_1 gradually decreases for compressive strengths of concrete used in design in excess of 10.0 ksi.

The stress block 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).

compression members shall be investigated as specified in Article 5.6.4.7.

5.6.3—Flexural Members

The following assumptions may be used for normal weight concrete with design compressive strengths up to 15.0 ksi and lightweight concrete up to 10.0 ksi.

5.6.3.1—Stress in Prestressing Steel at Nominal Flexural Resistance

5.6.3.1.1—Components with Bonded Tendons

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in Article 5.6.2.2 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:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \tag{5.6.3.1.1-1}$$

in which:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \tag{5.6.3.1.1-2}$$

for T-section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w)h_f}{\alpha_1 f'_c \beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}} \tag{5.6.3.1.1-3}$$

for rectangular section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} \tag{5.6.3.1.1-4}$$

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)
A_s = area of nonprestressed tension reinforcement (in.^{2})
A'_s = area of compression reinforcement (in.^{2})
f_s = stress in the nonprestressed tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1
f'_s = stress in the nonprestressed compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1

C5.6.3.1.1

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 nonprestressed 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 5.6.2.1. The background and basis for Eqs. 5.6.3.1.1-1 and 5.6.3.1.2-1 can be found in Loov (1988), Naaman (1989), and Naaman (1990–1992).

Values of f_{py}/f_{pu} are defined in Table C5.6.3.1.1-1. Therefore, the values of k from Eq. 5.6.3.1.1-2 depend only on the type of tendon used.

Table C5.6.3.1.1-1—Values of *k*

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

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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.)
h_f = compression flange depth (in.)
d_p = distance from extreme compression fiber to the centroid of the prestressing force (in.)
c = distance from the extreme compression fiber to the neutral axis (in.)
α_1 = stress block factor specified in Article 5.6.2.2
β_1 = stress block factor specified in Article 5.6.2.2

5.6.3.1.2—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 5.6.4.5, where the approximate stress distribution specified in Article 5.6.2.2 is used, the average stress in unbonded prestressing steel may be taken as:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py} \qquad (5.6.3.1.2\text{-}1)$$

in which:

$$\ell_e = \left(\frac{2\,\ell_i}{2 + N_s}\right) \qquad (5.6.3.1.2\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w} \qquad (5.6.3.1.2\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b} \qquad (5.6.3.1.2\text{-}4)$$

where:

c = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded, given by Eqs. 5.6.3.1.2-3 and 5.6.3.1.2-4 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 plastic hinges at supports in an assumed failure mechanism crossed by the tendon between anchorages or discretely bonded points assumed as:

- For simple spans……………………….0
- End spans of continuous units…………1
- Interior spans of continuous units……..2

C5.6.3.1.2

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

$$f_{ps} = f_{pe} + 15.0 \text{ (ksi)} \qquad (C5.6.3.1.2\text{-}1)$$

In order to solve for the value of f_{ps} in Eq. 5.6.3.1.2-1, 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.

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f_{py} = yield strength of prestressing steel (ksi)
f_{pe} = effective stress in prestressing steel after losses (ksi)

5.6.3.1.3—Components with Both Bonded and Unbonded Tendons

5.6.3.1.3a—Detailed Analysis

Except as specified in Article 5.6.3.1.3b, 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.

5.6.3.1.3b—Simplified Analysis

In lieu of the detailed analysis described in Article 5.6.3.1.3a, 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. 5.6.3.1.1-1 through 5.6.3.1.1-4, with the term $A_{ps}f_{pu}$ in Eqs. 5.6.3.1.1-3 and 5.6.3.1.1-4 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. 5.6.3.2.2-1, 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.

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5.6.3.2—Flexural Resistance

5.6.3.2.1—Factored Flexural Resistance

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

$$M_r = \phi M_n \tag{5.6.3.2.1-1}$$

where:

M_n = nominal flexural resistance (kip-in.)
ϕ = resistance factor as specified in Article 5.5.4.2

C5.6.3.2.1

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.

5.6.3.2.2—Flanged Sections

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.6.4.5, where the approximate stress distribution specified in Article 5.6.2.2 is used and where the compression flange depth is less than $a = \beta_1 c$, as determined in accordance with Eqs. 5.6.3.1.1-3, 5.6.3.1.1-4, 5.6.3.1.2-3, or 5.6.3.1.2-4, the nominal flexural resistance may be taken as:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c\left(b - b_w\right) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \tag{5.6.3.2.2-1}$$

where:

A_{ps} = area of prestressing steel (in.^{2})
f_{ps} = average stress in prestressing steel at nominal bending resistance specified in Eq. 5.6.3.1.1-1 (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 nonprestressed tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1
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 nonprestressed compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1
d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.)
f'_c = design concrete compressive strength (ksi)

C5.6.3.2.2

In previous editions and interims of the AASHTO *LRFD Bridge Design Specifications*, the stress block factor β_1 was applied to the flange overhang term of Eqs. 5.6.3.1.1-3, 5.6.3.1.2-3, and 5.6.3.2.2-1. 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 AASHTO *LRFD Bridge Design 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).

α_1 = stress block factor specified in Article 5.6.2.2
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 5.6.2.2
h_f = compression flange depth of an I- or T-member (in.)
a = $c\beta_1$; depth of the equivalent stress block (in.)

5.6.3.2.3—Rectangular Sections

For rectangular sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.6.4.5, where the approximate stress distribution specified in Article 5.6.2.2 is used and where the compression flange depth is not less than $a = \beta_1 c$ as determined in accordance with Eqs. 5.6.3.1.1-4 or 5.6.3.1.2-4, the nominal flexural resistance M_n may be determined by using Eqs. 5.6.3.1.1-1 through 5.6.3.2.2-1, in which case b_w shall be taken as b.

5.6.3.2.4—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 5.6.2. The requirements of Article 5.6.3.3 shall apply.

5.6.3.2.5—Strain Compatibility Approach

Alternatively, the strain compatibility approach may be used if more precise calculations are required. The appropriate provisions of Article 5.6.2.1 shall apply.

The stress and corresponding strain in any given layer of reinforcement may be taken from any representative stress–strain formula or graph for nonprestressed reinforcement and prestressing strands. Minimum specified properties shall be used in the stress–strain formula or graph for nonprestressed reinforcement with yield strengths between 75.0 and 100 ksi.

5.6.3.2.6—Composite Girder Sections

For composite girder sections in which the neutral axis is located below the deck and within the prestressed high-strength concrete girder, the nominal flexural resistance, M_n, may be determined by Eq. 5.6.3.2.2-1, based on the concrete compressive strength of the deck.

C5.6.3.2.6

Test results from Rizkalla et al. (2007) show that, in lieu of detailed analysis with two different compressive strengths of concrete used in design in the compression zone, the use of lower concrete compressive strength of the deck provides a sufficiently accurate yet conservative estimate of the nominal flexural resistance.

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5.6.3.3—Minimum Reinforcement

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

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1;
- $M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$

(5.6.3.3-1)

where:

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

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

The following factors shall be used to account for variability in the flexural cracking strength of concrete, variability of prestress, and the ratio of nominal yield stress of reinforcement to ultimate:

γ_1 = flexural cracking variability factor
= 1.2 for precast segmental structures
= 1.6 for all other concrete structures

γ_2 = prestress variability factor
= 1.1 for bonded tendons
= 1.0 for unbonded tendons

C5.6.3.3

Minimum reinforcement provisions are intended to reduce the probability of brittle failure by providing flexural capacity greater than the cracking moment.

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

For precast segmental construction, cracking generally starts at the segment joints. Research at the University of California, San Diego has shown that flexure cracks occur adjacent to the epoxy-bonded match-cast face, where the accumulation of fines reduces the tensile strength (Megally et al., 2003). Based on this observation, a reduced γ_1 factor of 1.2 is justified.

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γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement
= 0.67 for AASHTO M 31 (ASTM A615), Grade 60 reinforcement
= 0.75 for AASHTO M 31 (ASTM A615), Grade 75 reinforcement
= 0.76 for AASHTO M 31 (ASTM A615), Grade 80 reinforcement
= 0.75 for ASTM A706, Grade 60 reinforcement
= 0.80 for ASTM A706, Grade 80 reinforcement
= 0.67 for AASHTO M 334 (ASTM A1035), Grade 100 reinforcement

For prestressing steel, γ_3 shall be taken as 1.0.

The provisions of Article 5.10.6 shall apply.

Testing of a large number of lightly reinforced and prestressed concrete members at the University of Illinois demonstrated that significant inelastic displacements can be achieved, and none of the beams tested failed without large warning deflections (Freyermuth and Alami, 1997). If these experiments were conducted in load control, a number of specimens would have failed without warning because the ultimate strength (including the effects of strain hardening) was less than the cracking strength. Based on this observation, the ultimate strength should be used instead of the nominal strength as a true measure of brittle response. The ratio of steel stress at yield to ultimate γ_3 sufficiently approximates the nominal to ultimate strength for lightly reinforced concrete members.

5.6.3.4—Moment Redistribution

In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.10.8 is provided at the internal supports of continuous spans, 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 where ε_t is equal to or greater than $1.5\varepsilon_{tl}$ at the section at which moment is reduced, where ε_{tl} is the tension-controlled strain limit specified in Article 5.6.2.1.

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

C5.6.3.4

In editions and interims to the AASHTO *LRFD Bridge Design Specifications* prior to 2005, Article 5.6.3.4 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.

Mast (1992) provides an equation for the minimum strain in the tensile steel required for moment redistribution, which is based on the yield strain of the reinforcing steel and the assumption that $\rho/\rho_{bal} = 0.5$. Using reasonable values of yield strain for reinforcement with $f_y = 100$ ksi, the minimum tensile strain for moment redistribution can be found as 0.012, which is $1.5\varepsilon_{tl}$ (Shahrooz et al., 2011). Previous versions of this article set the minimum tensile strain at 0.0075 where the tension-controlled strain limit was set to 0.005. Thus, $1.5\varepsilon_{tl}$ gives the same value of minimum tensile strain for moment redistribution for reinforcement with $f_y \leq 75.0$ ksi as in previous editions, but also provides a reasonable value of minimum tensile strain for moment redistribution for higher strength reinforcement.

5.6.3.5—Deformations

5.6.3.5.1—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.

C5.6.3.5.1

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.

5.6.3.5.2—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 5.9.3.6 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. 5.6.3.5.2-1:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g \qquad (5.6.3.5.2\text{-}1)$$

in which:

$$M_{cr} = f_r \frac{I_g}{y_t} \qquad (5.6.3.5.2\text{-}2)$$

where:

M_{cr} = cracking moment (kip-in.)
M_a = maximum moment in a component at the stage for which deformation is computed (kip-in.)
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})
I_{cr} = moment of inertia of the cracked section, transformed to concrete (in.^{4})
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.)

For prismatic members, effective moment of inertia may be taken as the value obtained from Eq. 5.6.3.5.2-1 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. 5.6.3.5.2-1 for the critical positive and negative moment sections.

C5.6.3.5.2

Camber is the deflection that is built into a member, other than prestressing, in order to achieve the desired roadway geometry.

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 5.12.5.3.6.

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Unless a more exact determination is made, for nonprestressed members, the long-term 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: $3.0 - 1.2\ (A'_s/A_s) \geq 1.6$

where:

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

5.6.3.5.3—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.

5.6.4—Compression Members

5.6.4.1—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 5.6.4.3.
where:

K = effective length factor for compression members specified in Article 4.6.2.5
ℓ_u = unbraced length (in.)
r = radius of gyration of gross cross section (in.)

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 (2010), may also be used.

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The requirements of this article shall be supplemented and modified for structures in Seismic Zones 2, 3, and 4, as specified in Article 5.11.

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.

5.6.4.2—Limits for Reinforcement

The following reinforcement limits may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

Additional limits on reinforcement for compression members in Seismic Zones 2, 3, and 4 shall be considered as specified in Articles 5.11.3 and 5.11.4.1.1.

The maximum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall satisfy the following:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \tag{5.6.4.2-1}$$

and the area of prestressed longitudinal reinforcement shall also satisfy the following:

$$\frac{A_{ps} f_{pe}}{A_g f'_c} \leq 0.30 \tag{5.6.4.2-2}$$

Except as specified herein, the minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall satisfy the following:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \geq 0.135 \frac{f'_c}{f_y} \tag{5.6.4.2-3}$$

For design compressive strengths of normal weight concrete up to 15.0 ksi where the unfactored permanent loads do not exceed 0.4 $A_g f'_c$, the reinforcement ratio calculated by Eq. 5.6.4.2-3 need not be greater than:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.015 \tag{5.6.4.2-4}$$

where:

A_s = area of nonprestressed tension reinforcement (in.^{2})
A_g = gross area of section (in.^{2})
A_{ps} = area of prestressing steel (in.^{2})

C5.6.4.2

According to current American Concrete Institute (ACI) codes, the area of longitudinal reinforcement for nonprestressed noncomposite compression components should be not less than 0.01 A_g. Analyses by Rizkalla et al. (2007) showed that the reinforcement ratio calculated by Eq. 5.6.4.2-3 need not be greater than 0.015 for the condition indicated, which is typically the case encountered in design. The limit of 0.015 is based on a reinforcement yield strength of 60.0 ksi and is applicable for specified yield strengths of 60.0 ksi and greater. For prestressed members, current codes specify a minimum average prestress of 0.225 ksi. Here also the influence of the design concrete compressive strength is not accounted for. A design compressive strength of concrete of 5.0 ksi was used as a basis for these provisions, and a weighted averaging procedure was used to arrive at the equation.

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f_{pu} = specified tensile strength of prestressing steel (ksi)
f_y = specified minimum yield strength of reinforcement (ksi)
f_{pe} = effective stress in the prestressing steel after losses (ksi)
f'_c = compressive strength of concrete for use in design (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.

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 the total of prestressed and nonprestressed longitudinal reinforcement of the reduced effective area is to be the greater of one percent or the value obtained from Eq. 5.6.4.2-3. Both the reduced effective area and the gross area must be capable of resisting all applicable load combinations from Table 3.4.1-1.

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 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. 5.6.4.2-3 is not recommended because of the potential for significant transfer of load from the concrete to the reinforcement.

5.6.4.3—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, at the strength limit state due to factored loads and the term (M_1/M_2) is positive if the member is bent in single curvature and negative if bent in double curvature.

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 applied axial force, P_u, determined by elastic analysis and a magnified factored moment, M_c, as specified in Article 4.5.3.2.2b.
- The unbraced 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.

C5.6.4.3

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.

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- 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 determined as the larger of the following:

$$EI = \frac{\frac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \tag{5.6.4.3-1}$$

$$EI = \frac{\frac{E_c I_g}{2.5}}{1+\beta_d} \tag{5.6.4.3-2}$$

where:

EI = flexural stiffness (kip-in.^{2})
E_c = modulus of elasticity of concrete (ksi)
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})
E_s = modulus of elasticity of longitudinal steel (ksi)
I_s = moment of inertia of longitudinal reinforcement 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.

5.6.4.4—Factored Axial Resistance

The following assumptions may be used for normal weight concrete with design compressive strengths up to 15.0 ksi and lightweight concrete up to 10.0 ksi.

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

$$P_r = \phi P_n \tag{5.6.4.4-1}$$

in which:

- For members with spiral reinforcement:

$$P_n = 0.85\left[\begin{array}{l} k_c f'_c\left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} \\ -A_{ps}\left(f_{pe} - E_p\varepsilon_{cu}\right)\end{array}\right] \tag{5.6.4.4-2}$$

C5.6.4.4

The values of 0.85 and 0.80 in Eqs. 5.6.4.4-2 and 5.6.4.4-3 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,

- For members with tie reinforcement:

$$P_n = 0.80\begin{bmatrix} k_c f'_c\left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} \\ -A_{ps}\left(f_{pe} - E_p\varepsilon_{cu}\right) \end{bmatrix} \quad (5.6.4.4\text{-}3)$$

The factor k_c shall be taken as 0.85 for design compressive strengths not exceeding 10.0 ksi. For design compressive strengths exceeding 10.0 ksi, k_c shall be reduced at a rate of 0.02 for each 1.0 ksi of strength in excess of 10.0 ksi, except that k_c shall not be less than 0.75.

where:

P_r = factored axial resistance, with or without flexure (kip)
ϕ = resistance factor specified in Article 5.5.4.2
P_n = nominal axial resistance, with or without flexure (kip)
k_c = ratio of the maximum concrete compressive stress to the design compressive strength of concrete
f'_c = compressive strength of concrete for use in design (ksi)
A_g = gross area of section (in.^{2})
A_{st} = total area of longitudinal nonprestressed reinforcement (in.^{2})
A_{ps} = area of prestressing steel (in.^{2})
f_y = specified minimum yield strength of reinforcement (ksi)
f_{pe} = effective stress in prestressing steel after losses (ksi)
E_p = modulus of elasticity of prestressing steel (ksi)
ε_{cu} = failure strain of concrete in compression (in./in.)

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.

Research by Shahrooz et al. (2011) and Ward (2008) showed that these provisions are applicable to columns using reinforcement with specified minimum yield strengths up to 100 ksi. Designers should account for the fact that columns using high-strength reinforcement may have smaller areas of steel or smaller gross dimensions for the same resistance, or both. These reductions may affect axial deformations, slenderness effects, and the effects of creep and shrinkage.

5.6.4.5—Biaxial Flexure

The following assumptions may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

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 greater than or equal to 0.10 $\phi f'_c A_g$:

C5.6.4.5

Eqs. 5.6.3.2.1-1 and 5.6.4.4-1 relate factored resistances, given in Eqs. 5.6.4.5-1 and 5.6.4.5-2 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. 5.6.4.5-1 and 5.6.4.5-2, these specifications implicitly include the resistance factor by using factored resistances in the denominators.

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{\phi P_o} \qquad (5.6.4.5\text{-}1)$$

in which:

$$P_o = k_c f'_c \left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \qquad (5.6.4.5\text{-}2)$$

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

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \qquad (5.6.4.5\text{-}3)$$

where:

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)
P_{ry} = factored axial resistance determined on the basis that only eccentricity e_x is present (kip)
ϕ = resistance factor for members in axial compression
P_o = nominal axial resistance of a section at 0.0 eccentricity (kip)
P_u = factored applied axial force (kip)
k_c = ratio of the maximum concrete compressive stress to the design compressive strength of concrete
M_{ux} = factored applied moment about the x-axis (kip-in.)
M_{uy} = factored applied moment about the y-axis (kip-in.)
M_{rx} = uniaxial factored flexural resistance of a section in the direction of the x-axis (kip-in.)
M_{ry} = uniaxial factored flexural resistance of a section in the direction of 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.)

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. 5.6.4.4-2 or 5.6.4.4-3, as appropriate.

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.

5.6.4.6—Spirals, Hoops, and Ties

The following assumptions may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

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 5.11.

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Where the area of spiral reinforcement or hoops is not controlled by:

- Seismic requirements,
- Shear or torsion as specified in Article 5.7, or
- Minimum requirements as specified in Article 5.10.4.

The ratio of spiral reinforcement to total volume of concrete core, ρ_s, measured out-to-out of spirals or hoops, shall be determined as:

$$\rho_s = \frac{4A_{sp}}{(d_c s)} \geq 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}} \qquad (5.6.4.6\text{-}1)$$

where:

A_{sp} = cross-sectional area of spiral or hoop (in.^{2})
d_c = core diameter of column measured to the outside diameter of spiral or hoop (in.)
s = pitch of spiral or vertical spacing of hoops (in.)
A_g = gross area of section (in.^{2})
A_c = area of core measured to the outside diameter of the spiral (in.^{2})
f'_c = compressive strength of concrete for use in design (ksi)
f_{yh} = specified minimum yield strength of spiral reinforcement (ksi) $\leq$100 ksi for elements and connections specified in Article 5.4.3.3; $\leq$75.0 ksi otherwise

Other details of spiral and tie reinforcement shall conform to the provisions of Articles 5.10.4 and 5.11.

5.6.4.7—Hollow Rectangular Compression Members

5.6.4.7.1—Wall Slenderness Ratio

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

$$\lambda_w = \frac{X_u}{t} \qquad (5.6.4.7.1\text{-}1)$$

where:

λ_w = wall slenderness ratio for hollow columns
X_u = clear length of the constant thickness portion of a wall between other walls or fillets between walls (in.)
t = thickness of wall (in.)

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.

C5.6.4.7.1

The definition of the parameter X_u is illustrated in Figure C5.6.4.7.1-1, taken from Taylor et al. (1990).

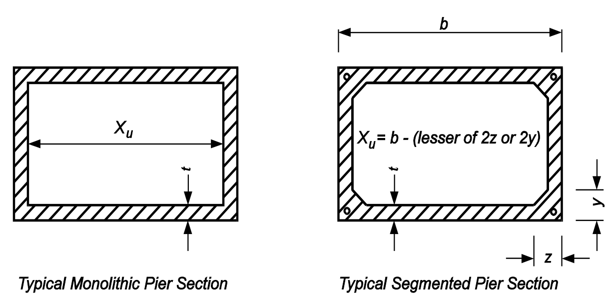

Figure C5.6.4.7.1-1—Illustration of X_u

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.

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

5.6.4.7.2a—General

Except as specified in Article 5.6.4.7.2c, 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.

5.6.4.7.2b—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 reinforcement 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 5.6.3 applied with anticipated stress–strain curves for the types of material to be used.

5.6.4.7.2c—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 5.6.4.7.2a and 5.6.4.7.2b 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 multiplied by a factor ϕ_w taken as:

- If $\lambda_w \leq 15$, then $\phi_w = 1.0$ (5.6.4.7.2c-1)

- If $15 < \lambda_w \leq 25$, then $\phi_w = 1 - 0.025(\lambda_w - 15)$ (5.6.4.7.2c-2)

- If $25 < \lambda_w \leq 35$, then $\phi_w = 0.75$ (5.6.4.7.2c-3)

5.6.5—Bearing

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

$$P_r = \phi P_n \tag{5.6.5-1}$$

in which:

$$P_n = 0.85 f'_c A_1 m \tag{5.6.5-2}$$

where:

P_n = nominal bearing resistance (kip)
A_1 = area under bearing device (in.^{2})
m = confinement modification factor

Where the supporting surface is wider than the loaded area on all sides, the modification factor, *m*, may be determined as follows:

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

$$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0 \tag{5.6.5-3}$$

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

$$m = 0.75\sqrt{\frac{A_2}{A_1}} \leq 1.50 \tag{5.6.5-4}$$

where:

A_2 = notional area defined as shown in Figure 5.6.5-1 (in.^{2})

Otherwise, where the supporting surface is not wider than the loaded area on all sides, *m* shall be taken as unity.

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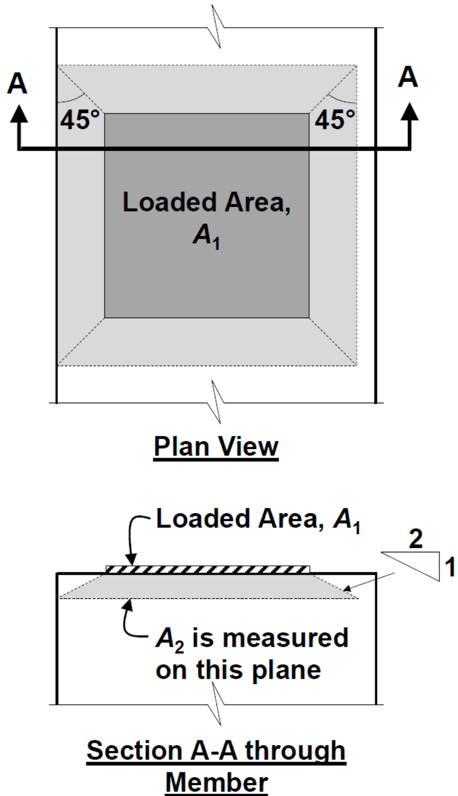

Figure 5.6.5-1—Determination of Notional Area

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 as shown in Figure 5.6.5-2.

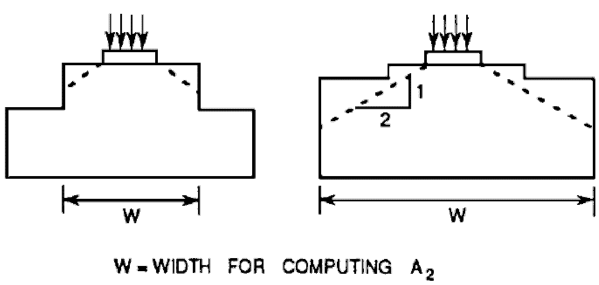

Figure 5.6.5-2—Determination of A_2 for a Stepped Support

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 5.9.5.6.

5.6.6—Tension Members

5.6.6.1—Resistance to Tension

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 5.10.8.4.4 shall apply.

The factored resistance to uniform tension shall be taken as:

$$P_r = \phi P_n \tag{5.6.6.1-1}$$

where:

ϕ = resistance factor specified in Article 5.5.4.2
P_n = nominal resistance of a tie as specified in Article 5.8.2.4.1

5.6.6.2—Resistance to Combined 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 5.6.2.

5.6.7—Control of Cracking by Distribution of Reinforcement

Except for deck slabs designed in accordance with Article 9.7.2, the provisions specified herein shall apply to the reinforcement of all concrete components 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 nonprestressed reinforcement in the layer closest to the tension face shall satisfy the following:

C5.6.7

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.

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \tag{5.6.7-1}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \tag{5.6.7-2}$$

where:

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

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

f_{ss} = calculated tensile stress in nonprestressed reinforcement at the service limit state not to exceed $0.60 f_y$ (ksi)

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

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

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

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_{ss} used in Eq. 5.6.7-1, 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.

Eq. 5.6.7-1 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. 5.6.7-1 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 Owner. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Owner 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 Owner 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. 5.6.7-1 may be provided by decreasing the permeability of the concrete, waterproofing the exposed surface, or both.

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.

Research by Shahrooz et al. (2011) indicated that Eq. 5.6.7-1 can be applied to reinforcement with specified minimum yield strengths up to 100 ksi but the tensile stress in the steel reinforcement at the service limit state, f_{ss}, cannot exceed 60.0 ksi with f_y = 100 ksi. A limit of $f_{ss} \leq 0.60 f_y$ was adopted.

In certain situations involving higher-strength reinforcement or large concrete cover, the use of Eq. 5.6.7-1 can result in small or negative values for s. Where higher strength reinforcement is used, an analysis of five crack-control equations suggests a bar spacing not less than 5.0 in. for control of flexural cracking.

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Unless justified by successful past practice, the actual concrete cover thickness should be used in the computation of d_c.

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 following:

- The effective flange width, specified in Article 4.6.2.6,
- A width equal to one tenth of the average of adjacent spans between bearings.

If the effective flange width exceeds one tenth 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 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:

$$A_{sk} \geq 0.012\ (d_\ell - 30) \qquad (5.6.7\text{-}3)$$

where:

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

However, the total area of longitudinal skin reinforcement (per face) need not exceed one fourth of the required flexural tensile reinforcement.

The maximum spacing of the skin reinforcement shall not exceeed the lesser of:

- $d_\ell/6$;
- 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.

Where large concrete cover is used, past successful practice suggests a value of d_c not greater than 2.0 in. plus the bar radius for calculation purposes. The limit of 2.0 in. plus the bar radius is the limit that was used in the specifications prior to the introduction of the current version of Eq. 5.6.7-1 in 2005.

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.

The one tenth 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 on work reported in Frantz and Breen (1978). For beams with a depth, d_ℓ, greater than 3.0 ft, 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.

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5.7—DESIGN FOR SHEAR AND TORSION—B-REGIONS

5.7.1—Design Procedures

5.7.1.1—Flexural Regions

Where it is reasonable to assume that plane sections remain plane after loading, regions of components may be designed for shear and torsion using either the sectional model as specified in Article 5.7.3 or the strut-and-tie method as specified in Article 5.8.2. The requirements of Article 5.7.2 shall apply.

In lieu of the provisions of Article 5.7.3, segmental post-tensioned concrete box girder bridges may be designed for shear and torsion using the provisions of Article 5.12.5.3.8.

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 one half (one third 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 5.8.2 apply.

C5.7.1.1

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 traditional approach applies even to the ends of typical beams provided that the provisions of Article 5.7.3.5 are satisfied and conventional detailing practices are followed. 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 method can be applied to flexural regions, it is more appropriate for regions near discontinuities where the actual flow of forces should be considered in more detail.

5.7.1.2—Regions near Discontinuities

Regions of members where the plane sections assumption of flexural theory is not valid should be considered to be D-Regions and should be designed for shear and torsion using the strut-and-tie method, as specified in Article 5.8.2, or the legacy methods of Article 5.8.4.

C5.7.1.2

The response of regions adjacent to abrupt changes in cross section, openings, dapped ends, diaphragms, 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.

5.7.1.3—Interface Regions

Interfaces between elements shall be designed for shear transfer in accordance with the provisions of Article 5.7.4.

5.7.1.4—Slabs and Footings

Slab-type regions shall be designed for shear in accordance with the provisions of Article 5.7.3 or Article 5.12.8.6.

5.7.1.5—Webs of Curved Post-Tensioned Box Girder Bridges

Curved post-tensioned box girders having an overall clear height, h_c, in excess of 4.0 ft shall be designed for all of the following combined effects before and after losses:

C5.7.1.5

Transverse web bending is a function of the vertical loads, restoring effect of the longitudinal prestressing, the Resal effect, and any transverse prestressing. Considering global web shear and regional web

- The combined effects of global shear resulting from vertical shear and torsion,
- Transverse web regional bending resulting from lateral prestress force, and
- Transverse web bending from vertical loads and transverse post-tensioning.

The applicable provisions of Articles 5.9.5.4.3 and 5.9.5.4.4 shall also be considered.

transverse bending alone will tend to underestimate the amount of vertical reinforcement required in the webs. More rigorous approaches that consider the interaction of these combined forces are presented in Menn (1990) and Nutt (2008).

5.7.2—General Requirements

5.7.2.1—General

Except where principal tensile stresses are investigated under the provisions of Article 5.9.2.3.3, design for shear and torsion shall be performed at the strength or extreme event limit state load combinations as specified in Article 3.4.1.

The secondary shear effects from prestressing shall be included in the *PS* load defined in Article 3.3.2.

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.

The vertical component of inclined tendons or strands shall only be considered to reduce the applied shear on the webs for tendons which extend through the web depth, engage both the flexural compression and flexural tension zones, and are fully developed or anchored by one of the following:

- anchorages,
- deviators, or
- internal ducts located in the top or bottom one third of the webs.

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 factored shear force where its effect is detrimental (increase in shear load) but may be considered if its effect is beneficial (decrease in shear load).

The values of the compressive strength of concrete for use in design shall not exceed 15.0 ksi for shear using the provisions of Article 5.7.3 or 10.0 ksi for torsion or for torsion and shear.

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

$$V_r = \phi V_n \quad (5.7.2.1\text{-}1)$$

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

$$T_r = \phi T_n \quad (5.7.2.1\text{-}2)$$

where:

C5.7.2.1

See Appendix C5 for limits of the maximum design concrete compressive strength applicable to affected articles. An exception to the listed assumptions is the use of the Plastic Stress Distribution Method (PSDM) for composite concrete filled steel tubes, which is addressed in Section 6.

Torsion is often designated as either compatibility torsion or equilibrium torsion as illustrated in Figure C5.7.2.1-1. Equilibrium torsion is required for stable equilibrium by the topology of the structure and must be addressed in the design. All torsion in a statically determinant structure is equilibrium torsion. Torsion in a statically indeterminate structure may be either compatibility or equilibrium torsion. Torsion which results from rotational restraint but which is not required to keep the applied loads in stable equilibrium because other load paths exist to support the loads is called compatibility torsion. It is not necessary to design for compatibility torsion as long as the other load paths are properly designed for the redistributed forces. Consideration should be given to the aesthetic issues that may be caused by cracking that could be associated with not designing for compatibility torsion.

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ϕ = resistance factor as specified in Article 5.5.4.2
V_n = nominal shear resistance specified in Article 5.7.3.3 (kip)
T_n = nominal torsional resistance specified in Article 5.7.3.6 (kip-in.)

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 torsional 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.

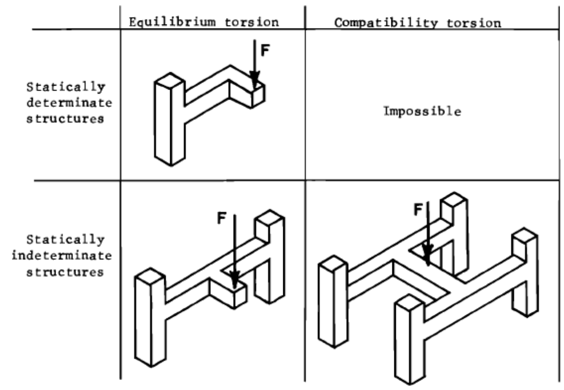

Figure C5.7.2.1-1 Equilibrium and Compatibility Torsion (Rameriz and Breen, 1983)

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

If the factored torsional moment is less than the indicated fraction 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.

Torsional effects shall be investigated where:

$$T_u > 0.25\phi T_{cr} \quad (5.7.2.1\text{-}3)$$

- For solid shapes:

$$T_{cr} = 0.126K\lambda\sqrt{f'_c}\frac{A_{cp}^{\,2}}{p_c} \quad (5.7.2.1\text{-}4)$$

- For hollow shapes:

$$T_{cr} = 0.126K\lambda\sqrt{f'_c}\,2A_o b_e \quad (5.7.2.1\text{-}5)$$

in which:

$$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f'_c}}} \leq 2.0 \quad (5.7.2.1\text{-}6)$$

where:

T_u = applied factored torsional moment (kip-in.)
ϕ = resistance factor specified in Article 5.5.4.2
T_{cr} = torsional cracking moment (kip-in.)
λ = concrete density modification factor as specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design (ksi)
A_{cp} = area enclosed by outside perimeter of concrete cross section (in.^{2})

For hollow shapes of typical proportions, A_o can be taken as the area enclosed by the centerlines of the exterior webs and flanges that form the closed section.

See Article C5.9.2.3.3 for additional discussion of shear flow in hollow shapes.

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p_c = length of outside perimeter of the concrete section (in.)
A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})
b_e = effective width of the shear flow path taken as the minimum thickness of the exterior webs or flanges comprising the closed box section (in.). b_e shall be adjusted to account for the presence of ducts.
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)

b_e defined above shall not exceed A_{cp}/p_c, unless a more refined analysis is utilized to determine a larger value.

The effects of any openings or ducts in members shall be considered. 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 exceed $0.19\,\lambda\sqrt{f'_c}$ in tension.

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 as a negative value when it is compressive.

An example of a more refined analysis would be a plate model of the cross section subjected to a torsional load.

The current recommendation for determining the effective web or flange thickness, b_e, is that the diameters of corrugated metal or plastic ungrouted ducts or one half the diameters of grouted ducts be subtracted from the web or flange thickness at the location of these ducts (AASHTO, 1999). For determining the shear capacity of girders containing grouted ducts in the web, no reduction in effective web or flange thickness is required provided the λ_{duct} factor of Article 5.7.3.3 is employed in calculating V_s. This approach is consistent with the fundamentals of MCFT, which is rooted in the ability of cracked concrete to transfer shear.

5.7.2.2—Transfer and Development Lengths

The provisions of Article 5.9.4.3 shall be considered for longitudinal reinforcement resisting tension caused by shear.

C5.7.2.2

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 5.7.3.5.

5.7.2.3—Regions Requiring Transverse Reinforcement

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

- $$V_u > 0.5\phi\left(V_c + V_p\right) \quad (5.7.2.3\text{-}1)$$

  or

- Where consideration of torsion is required by Eq. 5.7.2.1-3.

where:

V_u = factored shear force (kip)
ϕ = resistance factor specified in Article 5.5.4.2
V_c = nominal shear resistance of the concrete (kip)
V_p = component of prestressing force in the direction of the shear force

C5.7.2.3

Transverse reinforcement is required in all regions where there is a significant chance of diagonal cracking.

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5.7.2.4—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 gauge 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.

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 5.10.8.2.6d;
- A closed cage of welded wire reinforcement with transverse wires perpendicular to the longitudinal axis of the member; or
- Spirals or hoops.

C5.7.2.4

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 reinforcement, 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. Care must be taken to avoid excessive loss of prestress due to anchorage slip or seating losses because the tendons are short. 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).

5.7.2.5—Minimum Transverse Reinforcement

Where transverse reinforcement is required as specified in either Article 5.7.2.3 or Article 5.12.5.3.8c, and nonprestressed reinforcement is used to satisfy that requirement, the area of steel shall satisfy:

$$A_v \geq 0.0316\ \lambda\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad (5.7.2.5\text{-}1)$$

where:

C5.7.2.5

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.

Testing by Shahrooz et al. (2011) has verified these

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A_v = area of transverse reinforcement within distance s (in.^{2})
λ = concrete density modification factor as specified in Article 5.4.2.8
b_v = width of web adjusted for the presence of ducts as specified in Article 5.7.2.8 (in.)
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi) ≤100 ksi

minimum values of transverse reinforcement for reinforcing steel with specified minimum yield strengths up to 100 ksi for both prestressed and nonprestressed members subjected to flexural shear without torsion for applications in Seismic Zone 1. See Article C5.4.3.3 for additional information.

The design yield strength of prestressed transverse reinforcement in Eq. 5.7.2.5-1 shall be taken as the effective stress, after allowance for all prestress losses, plus 60.0 ksi, but not greater than f_{py}.

For segmental post-tensioned concrete box girder bridges where transverse reinforcement is not required by Article 5.12.5.3.8c, 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.

Previous editions of these specifications had a minimum transverse reinforcement requirement for segmentally-constructed, post-tensioned concrete box girders that was not a function of concrete strength. Observations in shear testing of prestressed concrete girders indicate that higher strength concrete with lightly reinforced girders can fail soon after cracking. Since the limit for segmental structures was both less conservative and not tied to concrete strength, it was eliminated.

5.7.2.6—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:

$$s_{max} = 0.8d_v \leq 24.0 \text{ in.} \qquad (5.7.2.6\text{-}1)$$

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

$$s_{max} = 0.4d_v \leq 12.0 \text{ in.} \qquad (5.7.2.6\text{-}2)$$

where:

v_u = shear stress calculated in accordance with Article 5.7.2.8 (ksi)
d_v = effective shear depth as defined in Article 5.7.2.8 (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.

C5.7.2.6

Some research (NCHRP Report 579) indicates that in prestressed girders the angle of diagonal cracking can be sufficiently steep that a transverse bar reinforcement spacing of $0.8d_v$ could result in no stirrups intersecting and impeding the opening of a diagonal crack. A limit of $0.6d_v$ may be appropriate in some situations. Reducing the transverse bar reinforcement diameter is another approach taken by some.

These spacing requirements were verified by Shahrooz et al. (2011) for transverse reinforcement with specified minimum yield strengths up to 100 ksi for prestressed and nonprestressed members subjected to flexural shear without torsion for applications in Seismic Zone 1. See Article C5.4.3.3 for additional information.

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5.7.2.7—Design and Detailing Requirements

Transverse reinforcement shall be anchored at both ends in accordance with the provisions of Article 5.10.8.2.6. 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 5.10.8.2.6 are satisfied.

For elements and connections specified in Article 5.4.3.3 subjected to flexural shear without torsion, the design yield strength of nonprestressed transverse reinforcement shall be taken as the specified minimum yield strength, but not to exceed 100 ksi. For all other elements and connections, the design yield strength of nonprestressed transverse reinforcement shall be taken equal to the specified yield strength where 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.

Where welded wire reinforcement is used as transverse reinforcement, it shall be anchored at both ends in accordance with Article 5.10.8.2.6c. No welded joints other than those required for anchorage shall be permitted.

C5.7.2.7

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.

Some of the provisions of Article 5.7.3 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. Previous versions limited the design yield strength of nonprestressed transverse reinforcement to 75.0 ksi or a stress corresponding to a strain of 0.0035 to provide control of crack widths at service limit state. Shahrooz et al. (2011) compared the performance of transverse reinforcement with specified yield strengths of 60.0 and 100 ksi in prestressed and nonprestressed members subjected to flexural shear only, under Seismic Zone 1 conditions. The results do not show any discernable difference between the performance of the two types of transverse reinforcement at either service or strength limit states. 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.

5.7.2.8—Shear Stress on Concrete

The shear stress on the concrete shall be determined as:

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v} \qquad (5.7.2.8\text{-}1)$$

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,

C5.7.2.8

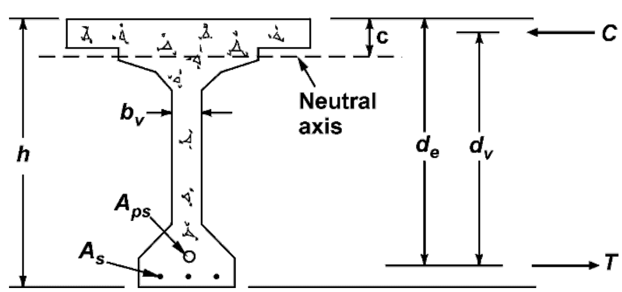

Figure C5.7.2.8-1—Illustration of the Terms b_v and d_v

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

modified for the presence of ducts where applicable (in.) For grouted ducts, no modification is necessary. For ungrouted ducts, reduce b_v by the diameter of the duct.

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

in which:

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \quad (5.7.2.8\text{-}2)$$

The provisions of Article 5.7.2.1 regarding deductions for post-tensioning ducts shall apply.

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}} \quad (C5.7.2.8\text{-}1)$$

For girders with debonded strands, d_v is calculated by neglecting the area of debonded strands for the length over which strands are debonded. For regions where debonded strands are bonded for distances less than ℓ_d, the value of d_v is calculated accounting for the lack of full development of the strand according to Article 5.9.4.3.2. Determine ℓ_d using Eq. 5.9.4.3.2-1 with the value of κ taken as 2.0.

For girders designed as individual I-girders, or as individual web lines, b_v is taken as the minimum width of the single web of the girder being considered. For box girders designed as a single unified section, b_v is taken as the total minimum width of all the webs in the cross section.

In continuous members near the point of inflection, if Eq. C5.7.2.8-1 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.

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 C5.7.2.8-1, 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 were 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. C5.7.2.8-1 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:

$$d_e = \frac{D}{2} + \frac{D_r}{\pi} \quad (C5.7.2.8\text{-}2)$$

where:

D = external diameter of the circular member (in.)
D_r = diameter of the circle passing through the centers of the longitudinal reinforcement (in.)

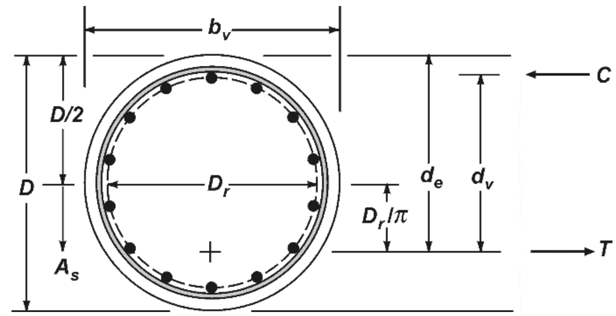

Figure C5.7.2.8-2—Illustration of Terms b_v, d_v, and d_e for Circular Sections

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 mid-depth 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.

5.7.3—Sectional Design Model

5.7.3.1—General

The sectional design model may be used for shear design where permitted in accordance with the provisions of Article 5.7.1. The provisions herein may be used for normal weight concrete with compressive strength of concrete used for design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.

C5.7.3.1

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.

The extension of these shear provisions to normal weight concretes with compressive strengths up to 15.0 ksi is based on the work presented in NCHRP Report 579 (Hawkins and Kuchma, 2007 and Kuchma et al., 2008).

See Articles 5.11.3 and 5.11.4.1.3 for additional requirements for Seismic Zones 2, 3, and 4, and Articles 5.7.1.2 and 5.7.3.2 for additional requirements for member end regions.

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.

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).

5.7.3.2—Sections near Supports

The provisions of Article 5.7.1.2 shall be considered.

In those cases where the sectional design model is used and a concentrated load exists within d_v from the

C5.7.3.2

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

face of a support, the shear load and shear resistance shall be calculated at the face of the support.

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 and the shear reinforcement required at the critical section shall be extended to the support.

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 5.9.5.6. For pretensioned beams, a reinforcement cage confining the ends of strands shall be provided as specified in Article 5.9.4.4. 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.

If the shear stress at the design section calculated in accordance with Article 5.7.2.8 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 method specified in Article 5.8.2.

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 $0.18f'_c$, 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 method.

5.7.3.3—Nominal Shear Resistance

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

$$V_n = V_c + V_s + V_p \tag{5.7.3.3-1}$$

$$V_n = 0.25 f'_c b_v d_v + V_p \tag{5.7.3.3-2}$$

in which:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\, b_v d_v \tag{5.7.3.3-3}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}\lambda_{duct} \tag{5.7.3.3-4}$$

$$\lambda_{duct} = 1 - \delta\left(\frac{\phi_{duct}}{b_w}\right)^2 \tag{5.7.3.3-5}$$

where:

C5.7.3.3

As noted in Article 5.7.2.3 for members subjected to flexural shear without torsion, transverse reinforcement with specified minimum yield strengths up to 100 ksi is permitted for elements and connections specified in Article 5.4.3.3.

The limit in Article 5.7.3.3 was derived from the Modified Compression Field Theory (Vecchio and Collins, 1986) and has been validated by numerous experiments on prestressed and nonprestressed concrete members (Saleh and Tadros, 1997; Lee et al., 2010).

The upper limit of V_n, given by Eq. 5.7.3.3-2, 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. 5.7.3.3-4 reduces to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}\lambda_{duct} \tag{C5.7.3.3-1}$$

V_p = component of prestressing force in the direction of the shear force; positive if resisting the applied shear

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.7.2.8 (in.)

d_v = effective shear depth as determined in Article 5.7.2.8 (in.)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.7.3.4

λ = concrete density modification factor as specified in Article 5.4.2.8

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

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.7.3.4 (degrees)

α = angle of inclination of transverse reinforcement to longitudinal axis (degrees)

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)

λ_{duct} = shear strength reduction factor accounting for the reduction in the shear resistance provided by transverse reinforcement due to the presence of a grouted post-tensioning duct. Taken as 1.0 for ungrouted post-tensioning ducts and with a reduced web or flange width to account for the presence of ungrouted duct.

δ = duct diameter correction factor, taken as 2.0 for grouted ducts

ϕ_{duct} = diameter of post-tensioning duct present in the girder web within depth d_v (in.)

b_w = gross width of web, not reduced for the presence of post-tensioning ducts (in.)

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:

$$V_s = A_v f_y (\sin\alpha)\lambda_{duct} \le 0.095\lambda\sqrt{f'_c}b_v d_v \qquad (5.7.3.3\text{-}6)$$

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.

For girders designed as individual girders, or as individual webs lines, A_v is taken as the reinforcing in the single web of the girder being considered. For box girders designed as a single unified section, A_v is taken as the total reinforcing in all the webs in the cross section.

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

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 as shown in Figure C5.7.3.3-1.

In situations where a significant amount of the load is applied below the mid-depth of the member, such as inverted T-beam pier caps, and the section model is used to design for shear, it is more appropriate to use the traditional approach to the design of transverse reinforcement shown in Figure C5.7.3.3-1.

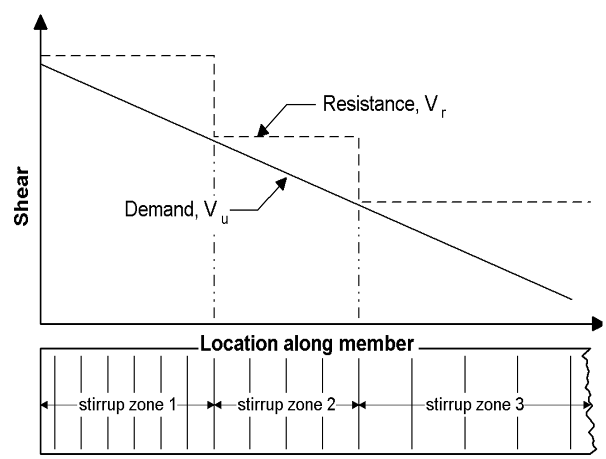

Figure C5.7.3.3 -1—Traditional Shear Design

For typical cases where the applied load acts at or above the mid-depth of the member, it is practical to take the traditional approach as shown in Figure C5.7.3.3-1 or a more liberal yet conservative approach as shown in Figure C5.7.3.3-2 which has the effect of extending the required stirrup spacing for a distance of $0.5d_v \cot\theta$ toward the bearing.

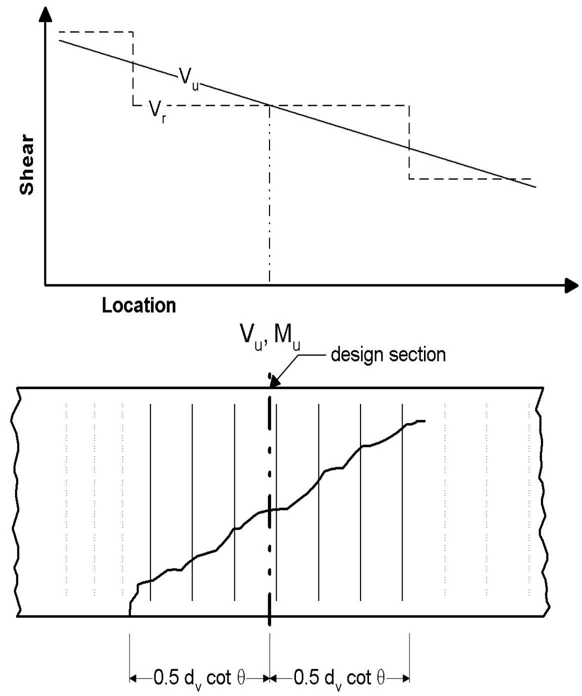

Figure C5.7.3.3-2—Simplified Design Section for Loads Applied at or above the Mid-Depth of the Member

Requirements for bent bars were added to make the provisions consistent with those in AASHTO (2002).

The shear strength reduction factor λ_{duct} is based on research that conducted and examined full-scale tests of post-tensioned concrete girder bridges. Research by Moore et al. (2015) examined both plastic and metal grouted ducts used in spliced girders. If the λ_{duct} factor is used in determining the shear capacity of post-tensioned girders containing ducts in the web, no reduction in effective web thickness is required (i.e. $b_e = b_w$). For elements with ungrouted ducts, λ_{duct} factor must be taken as 1.0 and b_v must be reduced to account for the duct diameter. This is consistent with methods used in past editions of the *LRFD Bridge Design Specifications* (Moore et al., 2015).

While Moore et al. (2015) and Williams et al. (2015) did not conduct tests to study torsional behavior, it is reasonable to apply λ_{duct} to the capacity calculation by Eq. 5.7.3.6.2-1 to be consistent with the fundamentals of the Modified Compression Field Theory. In this way, shear resulting from torsion and that resulting from direct shear can be combined consistently in webs or flanges of members that contain post-tensioning ducts embedded in concrete.

5.7.3.4—Procedures for Determining Shear Resistance Parameters β and θ

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

C5.7.3.4

Two complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.7.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in

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Article 5.7.3.4.2, 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 5.7.3.4.2, and an evaluation using tabularized values presented in Appendix B5. The approaches to Method 2 may be considered statistically equivalent.

5.7.3.4.1—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 5.7.2.5, or having an overall depth of less than 16.0 in., the following values may be used:

$\beta = 2.0$
$\theta = 45$ degrees

C5.7.3.4.1

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.

5.7.3.4.2—General Procedure

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 5.7.2.5, the value of β may be determined by Eq. 5.7.3.4.2-1:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} \tag{5.7.3.4.2-1}$$

When sections do not contain at least the minimum amount of shear reinforcement, the value of β may be as specified in Eq. 5.7.3.4.2-2:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})} \tag{5.7.3.4.2-2}$$

The value of θ in both cases may be as specified in Eq. 5.7.3.4.2-3:

$$\theta = 29 + 3500\varepsilon_s \tag{5.7.3.4.2-3}$$

In Eqs. 5.7.3.4.2-1 through 5.7.3.4.2-3, ε_s is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement as shown in Figures 5.7.3.4.2-1 and 5.7.3.4.2-2. In lieu of more involved procedures, ε_s may be determined by Eq. 5.7.3.4.2-4:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \tag{5.7.3.4.2-4}$$

C5.7.3.4.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.7.3.1. Such an analysis, see Figure C5.7.3.4.2-1, 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, 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 C5.7.3.4.2-2.

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 noniterative 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 appropriate for use in the *AASHTO LRFD Bridge Design Specifications* (Hawkins et al., 2006, 2007).

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

Where consideration of torsion is required by the provisions of Article 5.7.2.1, V_u in Eq. 5.7.3.4.2-4 shall be replaced by V_{eff}.

For solid sections:

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \quad (5.7.3.4.2\text{-}5)$$

For hollow sections:

$$V_{eff} = V_u + \frac{T_u d_s}{2A_o} \quad (5.7.3.4.2\text{-}6)$$

where:

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

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

V_u = factored shear force for the girder in Eq. 5.7.3.4.2-5 and for the web under consideration in Eq. 5.7.3.4.2-6 (kip)

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure 5.7.3.4.2-1 (in.^{2})

f_{po} = a parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel 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

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

T_u = applied factored torsional moment on the girder (kip-in.)

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

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

A_{ct} = area of concrete on the flexural tension side of the member as shown in Figure 5.7.3.4.2-1 (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 5.7.3.4.2-1 (in.^{2})

The crack spacing parameter as influenced by aggregate size, s_{xe}, shall be determined as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \quad (5.7.3.4.2\text{-}7)$$

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. 5.7.3.4.2-5. 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 hollow girder, where it is required to consider torsion, 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. The possibility of reversed torsion should be investigated. In the controlling web, the second term in Eq. 5.7.3.4.2-6 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.

The longitudinal strain, ε_s, can be determined by the procedure illustrated in Figure C5.7.3.4.2-3. 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_{ct}. 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 equal to 1.0 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_s and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure C5.7.3.4.2-4.

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.

in which:

$$12.0 \text{ in.} \leq s_{xe} \leq 80.0 \text{ in.}$$

where:

s_x = crack spacing parameter, taken as 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 5.7.3.4.2-3 (in.)
a_g = maximum aggregate size (in.)

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 5.7.3.4.2-1.

In the use of Eqs. 5.7.3.4.2-1 through 5.7.3.4.2-4, 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. 5.7.3.4.2-4 is negative, it should be taken as zero or the value should be recalculated with the denominator of Eq. 5.7.3.4.2-4 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 unless there is a concentrated load within d_v from the support, in which case ε_s should be calculated at the face of the support.
- The area of concrete on the flexural tension side of the member, A_{ct}, is shown in Figure 5.7.3.4.2-1 (in.^{2})
- If the axial tension is large enough to crack the flexural compression face of the section, the value calculated from Eq. 5.7.3.4.2-4 should be doubled.
- It is permissible to determine β and θ from Eqs. 5.7.3.4.2-1 through 5.7.3.4.2-3 using a value of ε_s which is greater than that calculated from Eq. 5.7.3.4.2-4 However, ε_s should not be taken greater than 6.0×10^{-3}.

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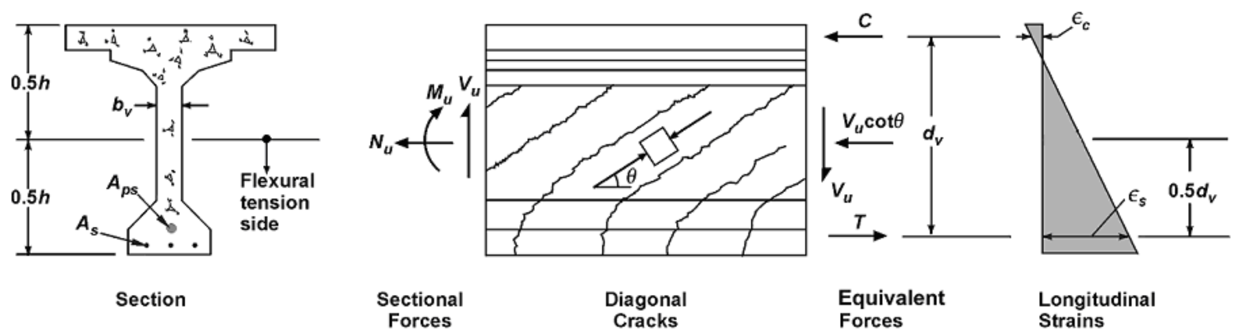

Figure 5.7.3.4.2-1—Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, $V_p = 0$

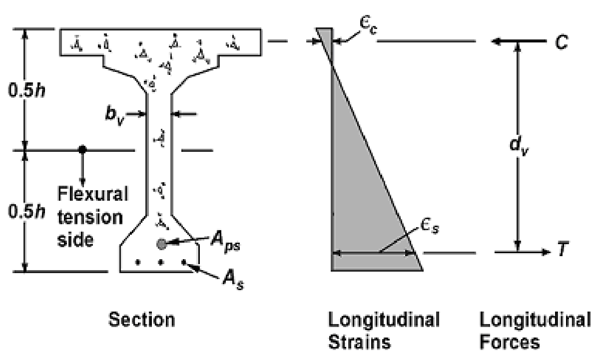

Figure 5.7.3.4.2-2—Longitudinal Strain, ε_s, for Sections Containing Less than the Minimum Amount of Transverse Reinforcement

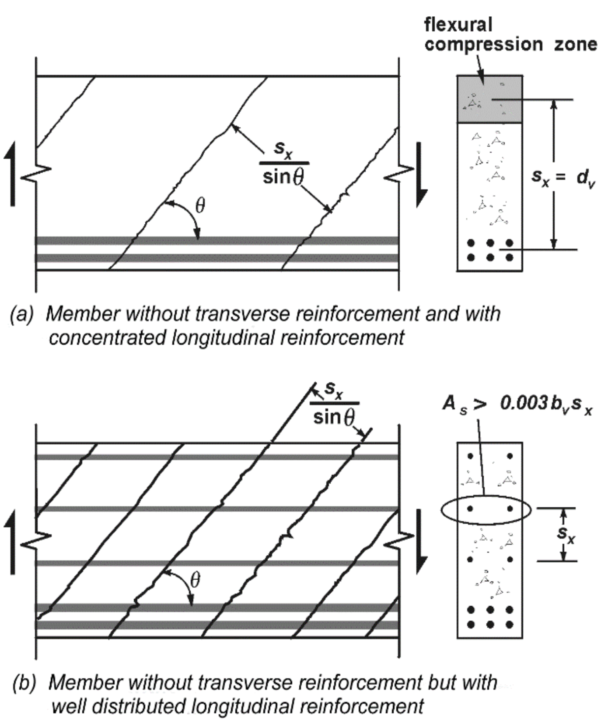

Figure 5.7.3.4.2-3—Definition of Crack Spacing Parameter, s_x

The relationships for evaluating β and θ in Eqs. 5.7.3.4.2-1 and 5.7.3.4.2-2 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 5.7.3.4.2-3. 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.

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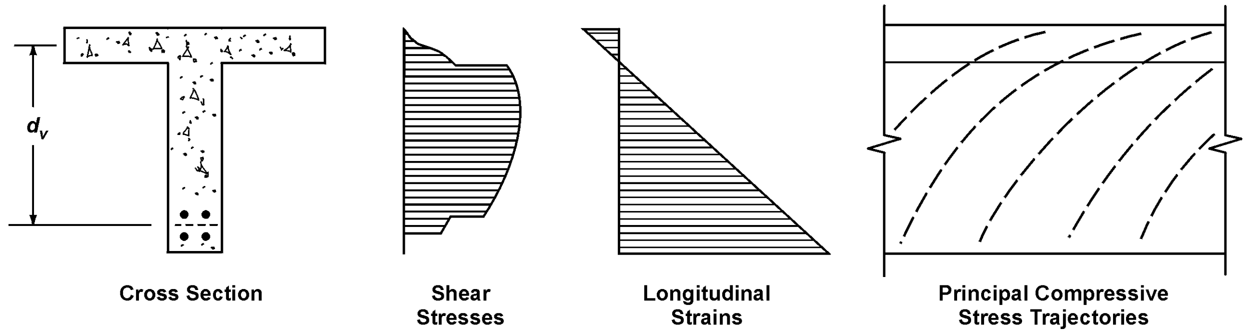

Figure C5.7.3.4.2-1—Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.7.3.1

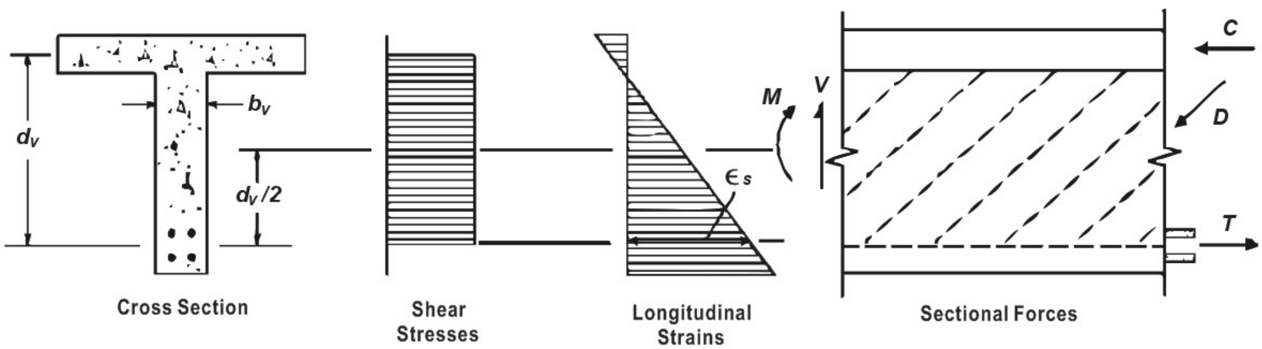

Figure C5.7.3.4.2-2—More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.7.3.4.2

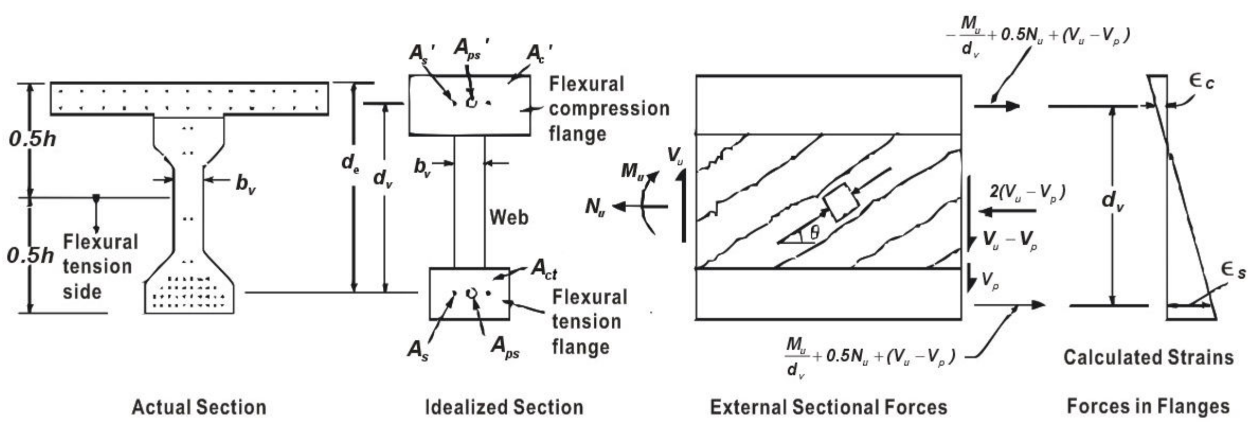

Figure C5.7.3.4.2-3—More Accurate Calculation Procedure for Determining ε_s

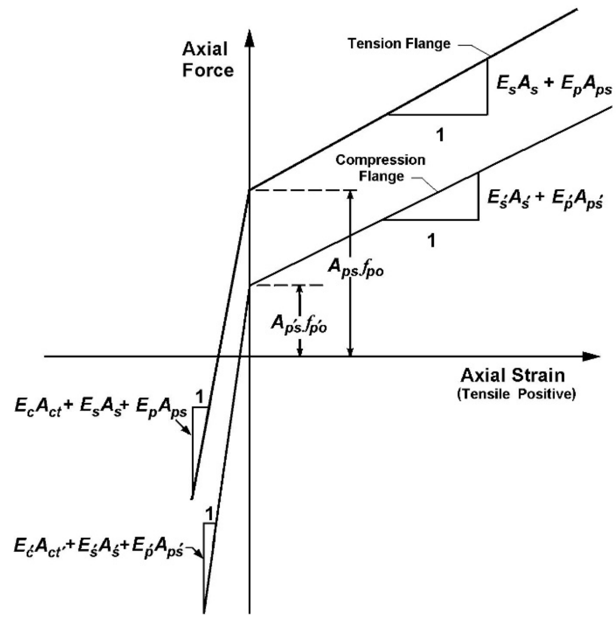

Figure C5.7.3.4.2-4—Assumed Relationships between Axial Force in Flange and Axial Strain of Flange

5.7.3.5—Longitudinal Reinforcement

Where consideration of torsion is required these provisions shall be amended as specified in Article 5.7.3.6.3.

Except as specified herein, at each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_{ps}f_{ps} + A_sf_y \geq \frac{|M_u|}{d_v\phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta \quad (5.7.3.5\text{-}1)$$

where:

ϕ_f, ϕ_v, ϕ_c = resistance factors taken from Article 5.5.4.2 as appropriate for moment, shear, and axial resistance

V_s = shear resistance provided by transverse reinforcement at the section under investigation as given by Eq. 5.7.3.3-4, except V_s shall not be taken as greater than V_u/ϕ_v in Eqs. 5.7.3.5-1 and 5.7.3.5-2 (kip)

C5.7.3.5

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 C5.7.3.5-1. Taking moments about Point 0 in Figure C5.7.3.5-1, 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.

θ = angle of inclination of diagonal compressive stresses used in determining the nominal shear resistance of the section under investigation as determined by Article 5.7.3.4 (degrees)

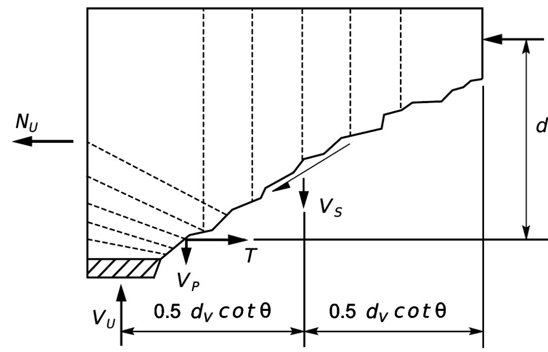

Figure C5.7.3.5-1—Forces Assumed in Resistance Model Caused by Moment and Shear

Eq. 5.7.3.5-1 shall be evaluated where simply-supported girders are made continuous for live loads and where longitudinal reinforcement is discontinuous.

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:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad (5.7.3.5\text{-}2)$$

Eqs. 5.7.3.5-1 and 5.7.3.5-2 shall be taken to apply to sections not subjected to torsion. Any lack of full development shall be accounted for.

For pretensioned sections, including those with debonded strands, the tensile force in the prestressing reinforcement ($A_{ps}f_{ps}$) shall exceed the tensile forces of the nonprestressed reinforcement (A_sf_y) at all sections. Development of straight and bent-up strands as well as overhangs, if present, shall be considered for determining the value of $A_{ps}f_{ps}$ and A_sf_y.

Except as may be required by Article 5.7.3.6.3, where the reaction force or the load at the maximum moment location introduces direct compression into the flexural compression face of the member, 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.

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 5.10.8.2.1a or the bi-linear variation of resistance over the transfer and development length of Article 5.9.4.3.2 may be assumed.

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 C5.7.3.5-2. 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 $d_v \cot\theta$ or as specified in Article 5.10.8, whichever is greater.

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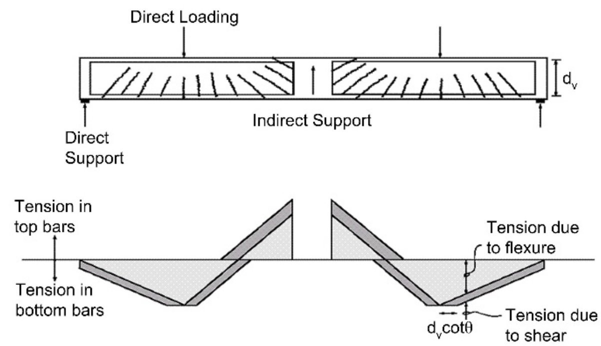

Figure C5.7.3.5-2—Force Variation in Longitudinal Reinforcement Near Maximum Moment Locations

Longitudinal or transverse reinforcing steel, or a combination thereof, with specified minimum yield strengths up to 100 ksi, may be used in elements and connections specified in Article 5.4.3.3.

This provision allows the use of longitudinal and transverse reinforcement with specified minimum yield strengths up to 100 ksi for elements and connections specified in Article 5.4.3.3; however, the use of higher strength longitudinal reinforcement may not be practical due to longer required development lengths.

5.7.3.6—Sections Subjected to Combined Shear and Torsion

5.7.3.6.1—Transverse Reinforcement

The transverse reinforcement shall not be less than the sum of that required for shear, as specified in Article 5.7.3.3, and for the concurrent torsion, as specified in Articles 5.7.2.1 and 5.7.3.6.2.

C5.7.3.6.1

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.

5.7.3.6.2—Torsional Resistance

The nominal torsional resistance shall be taken as:

$$T_n = \frac{2A_oA_tf_y\cot\theta}{s}\lambda_{duct} \qquad (5.7.3.6.2\text{-}1)$$

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 and flange of hollow members (in.^{2})

C5.7.3.6.2

See Article 5.7.2.4 for limits of the yield strength of transverse reinforcement.

For solid sections, A_o may be taken as the area enclosed by the centerline of the effective width b_e determined as A_{cp}/p_c, with A_{cp} and p_c taken as defined in Article 5.7.2.1.

For hollow members, the total area of transverse reinforcing, A_t must be placed in each exterior web and each flange that forms the closed shape.

θ = angle of inclination of diagonal compressive stresses as determined in accordance with the provisions of Article 5.7.3.4 with the modifications to the expressions for v and V_u herein (degrees)

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)

λ_{duct} = shear strength reduction factor as defined in Eq. 5.7.3.3-5

5.7.3.6.3—Longitudinal Reinforcement

The provisions of Article 5.7.3.5 shall apply as amended, herein, to include torsion. At least one bar or tendon shall be placed in the corners of the stirrups.

The longitudinal reinforcement in solid sections shall be proportioned to satisfy Eq. 5.7.3.6.3-1:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5 V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi}\right)^2} \quad (5.7.3.6.3\text{-}1)$$

In box sections, longitudinal reinforcement for torsion, in addition to that required for flexure, shall not be less than:

$$A_\ell = \frac{T_n p_h}{2 A_o f_y} \quad (5.7.3.6.3\text{-}2)$$

where:

p_h = perimeter of the centerline of the closed transverse torsion reinforcement for solid members, or the perimeter through the centroids of the transverse torsion reinforcement in the exterior webs and flanges for hollow members (in.)

A_ℓ shall be distributed around the outermost webs and top and bottom slabs of the box girder.

C5.7.3.6.3

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.

Torsion addressed in this article, St. Venant's Torsion, causes an axial tensile force. In a nonprestressed beam, this force is resisted by longitudinal reinforcement having an axial tensile strength of $A_\ell f_y$. This steel is in addition to the flexural reinforcement and is to be distributed uniformly around the perimeter so that the resultant acts along the axis of the member. In a prestressed beam, the same approach (providing additional reinforcing bars with strength $A_\ell f_y$) can be followed, or the longitudinal torsion reinforcement can be comprised of normal reinforcing bars and any portion of the longitudinal prestressing steel in excess of that required for cross-sectional flexural resistance at the strength limit states.

For box girder construction, interior webs should not be considered in the calculation of the longitudinal torsion reinforcement required by this article. The values of p_h and A_ℓ should be for the box shape defined by the outer-most webs and the top and bottom slabs of the box girder. Warping torsion is known to exist in cross section (d) shown in Table 4.6.2.2.1-1, but many structures of this type have performed successfully without consideration of warping.

The longitudinal tension due to torsion may be considered to be offset in part by compression at a cross-section resulting from longitudinal flexure, allowing a reduction in the longitudinal torsion steel in longitudinally compressed portions of the cross section at strength limit states.

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5.7.4—Interface Shear Transfer—Shear Friction

5.7.4.1—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.

Where the required interface shear reinforcement in girder/slab design exceeds the area required to satisfy flexural shear requirements, additional reinforcement shall 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.

Reinforcement for interface shear may consist of single bars, multiple leg stirrups, or welded wire reinforcement.

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.

C5.7.4.1

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.

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.

5.7.4.2—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:

$$A_{vf} \geq \frac{0.05 A_{cv}}{f_y} \tag{5.7.4.2-1}$$

where:

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})
A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})
f_y = yield stress of reinforcement but design value not to exceed 60.0 (ksi)

For a cast-in-place concrete slab on clean concrete girder surfaces free of laitance, the following provisions shall apply:

C5.7.4.2

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. 5.7.4.2-1 with $12b_{vi}$.

Previous editions of these specifications have required a minimum area of reinforcement based on the full interface area; similar to Eq. 5.7.4.2-1, irrespective of the need to mobilize the strength of the full interface

- The minimum interface shear reinforcement, A_{vf}, need not exceed the lesser of the amount determined using Eq. 5.7.4.2-1 and the amount needed to resist $1.33V_{ui}/\phi$ as determined using Eq. 5.7.4.3-3.

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. 5.7.4.2-1. 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.

- 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. 5.7.4.5-1, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.7.2.5 is extended across the interface and adequately anchored in the slab.

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.

5.7.4.3—Interface Shear Resistance

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \quad (5.7.4.3\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \quad (5.7.4.3\text{-}2)$$

where:

ϕ = resistance factor for shear specified in Article 5.5.4.2. For the extreme limit state, event ϕ may be taken as 1.0.

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)

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \quad (5.7.4.3\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not exceed either of the following:

$$V_{ni} \leq K_1 f'_c A_{cv} \quad (5.7.4.3\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \quad (5.7.4.3\text{-}5)$$

C5.7.4.3

Total load shall include all noncomposite and composite loads appropriate to the interface being investigated.

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. 5.7.4.3-3 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. 5.7.4.3-3 is analogous to the vertical shear resistance expression of $V_c + V_s$.

in which:

$$A_{cv} = b_{vi} L_{vi} \quad (5.7.4.3\text{-}6)$$

where:

c = cohesion factor specified in Article 5.7.4.4 (ksi)
μ = friction factor specified in Article 5.7.4.4
b_{vi} = interface width considered to be engaged in shear transfer (in.)
L_{vi} = interface length considered to be engaged in shear transfer (in.)
P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)
f'_c = design concrete 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 5.7.4.4.
K_2 = limiting interface shear resistance specified in Article 5.7.4.4 (ksi)

If a member has transverse reinforcement with a specified minimum yield strength greater than 60.0 ksi for flexural shear resistance, interface reinforcement may be provided by extending the transverse reinforcement across the interface zone. In this case, the value of f_y in Eq. 5.7.4.3-3 shall not be taken as greater than 60.0 ksi.

Eq. 5.7.4.3-4 limits V_{ni} to prevent crushing or shearing of aggregate along the shear plane.

Eqs. 5.7.4.3-3 and 5.7.4.3-4 are sufficient, with an appropriate value for K_1, to establish a lower bound for the available experimental data; however, Eq. 5.7.4.3-5 is necessitated by the sparseness of available experimental data beyond the limiting K_2 values provided in Article 5.7.4.4.

The interface shear strength Eqs. 5.7.4.3-3, 5.7.4.3-4, and 5.7.4.3-5 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; and lightweight concrete strengths from 2.0 ksi to 6.0 ksi.

A_{vf} used in Eq. 5.7.4.3-3 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. 5.7.4.3-3 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 60.0 ksi because interface shear resistance computed using higher values have overestimated the interface shear resistance experimentally determined in a limited number of tests of precracked specimens.

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 5.7.4.5.

5.7.4.4—Cohesion and Friction Factors

The following values shall be taken for cohesion, c, and friction factor, μ:

C5.7.4.4

The values presented provide a lower bound of the substantial body of experimental data available in the literature (Loov and Patnaik, 1994; Patnaik, 1999;

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- 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 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 = 1.0 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

- 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.

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 lightweight concrete with lightweight fine aggregate and normal weight fine aggregate. This deviates from earlier specifications that distinguished between different types of lightweight concrete.

Due to the absence of existing data, the prescribed cohesion and friction factors for lightweight concrete placed against hardened 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.

Since the effectiveness of cohesion and aggregate interlock along a vertical crack interface is unreliable,

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the cohesion component in Eq. 5.7.4.3-3 is set to 0.0 for brackets, corbels, and ledges.

5.7.4.5—Computation of the Factored Interface Shear Force 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:

$$v_{ui} = \frac{V_{u1}}{b_{vi} d_v} \quad (5.7.4.5\text{-}1)$$

where:

d_v = distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress (in.)

The factored interface shear force in kips/ft for a concrete girder/slab bridge may be determined as:

$$V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_{vi} \quad (5.7.4.5\text{-}2)$$

If the net force, P_c, across the interface shear plane is tensile, additional reinforcement, A_{vpc}, shall be provided as:

$$A_{vpc} = \frac{P_c}{\phi f_y} \quad (5.7.4.5\text{-}3)$$

C5.7.4.5

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 C5.7.4.5-1 as follows:

M_{u2} = maximum factored moment at section 2 (kip-in.)
V_1 = factored vertical shear at section 1 concurrent with M_{u2} (kip)
M_1 = factored moment at section 1 concurrent with M_{u2} (kip-in.)
$\Delta\ell$ = unit length segment of girder (in.)
C_1 = compression force above the shear plane associated with M_1 (kip)
C_{u2} = compression force above the shear plane associated with M_{u2} (kip)

$$M_{u2} = M_1 + V_1 \Delta\ell \quad (C5.7.4.5\text{-}1)$$

$$C_{u2} = \frac{M_{u2}}{d_v} \quad (C5.7.4.5\text{-}2)$$

$$C_{u2} = \frac{M_1}{d_v} + \frac{V_1 \Delta\ell}{d_v} \quad (C5.7.4.5\text{-}3)$$

$$C_1 = \frac{M_1}{d_v} \quad (C5.7.4.5\text{-}4)$$

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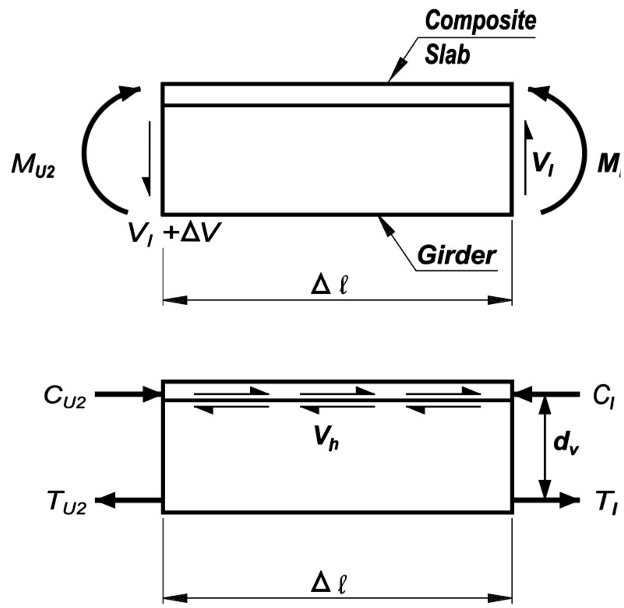

Figure C5.7.4.5-1—Free Body Diagrams

$$V_h = C_{u2} - C_1 \qquad \text{(C5.7.4.5-5)}$$

$$V_h = \frac{V_1 \Delta\ell}{d_v} \qquad \text{(C5.7.4.5-6)}$$

Such that for a unit length segment:

$$V_{hi} = \frac{V_1}{d_v} \qquad \text{(C5.7.4.5-7)}$$

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. 5.7.4.5-1 is converted to a resultant interface shear force computed with Eq. 5.7.4.5-2 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.

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For beams or girders, the longitudinal center-to-center spacing of nonwelded interface shear connectors shall not exceed 48.0 in. or the depth of the member, *h*. For cast-in-place box girders, the longitudinal center-to-center spacing of nonwelded interface shear connectors shall not exceed 24.0 in.

Recent research (Markowski et al., 2005; Tadros and Girgis, 2006; Badie and Tadros, 2008; Sullivan et al., 2011) has demonstrated that increasing interface shear connector spacing from 24.0 to 48.0 in. has resulted in no deficiency in composite action for the same resistance of shear connectors per foot, and girder and deck configurations. These research projects have independently demonstrated no vertical separation between the girder top and the deck under cyclic or ultimate loads. However, the research did not investigate relatively shallow members; hence, the additional limitation related to the member depth is provided.

As the spacing of connector groups increases, the capacities of the concrete and grout in their vicinity become more critical and need to be carefully verified. This applies to all connected elements at the interface. Eqs. 5.7.4.3-2 and 5.7.4.3-3 are intended to ensure that the capacity of the concrete component of the interface is adequate. Methods to enhance that capacity, if needed, include use of high-strength materials and of localized confinement reinforcement.

5.7.4.6—Interface Shear in Box Girder Bridges

Adequate shear transfer reinforcement shall be provided at the web/flange interfaces in box girders to transfer flange longitudinal forces at the strength limit state. Consideration shall be given to any construction joints at the interfaces, as well as the amount of reinforcing present in the flange and web for other design purposes.

C5.7.4.6

The factored design force for the interface reinforcement is calculated to account for the interface shear force as well as any localized shear effects due to the prestressing force anchorages at the section.

Historically, vertical planes at the web/flange interface have been checked for segmental bridges. If a horizontal construction joint is present, the interface shear check should also include the horizontal planes at the construction joints. This is a check that enough reinforcing is present. Additional reinforcing need not be added if there is already enough reinforcing present for other purposes, such as global shear and torsion in the cross section and transverse bending.

5.8—DESIGN OF D-REGIONS

5.8.1—General

Refined analysis methods or strut-and-tie method may be used to determine internal force effects in disturbed regions such as those near supports and the points of application of concentrated loads at strength and extreme event limit states.

5.8.2—Strut-and-Tie Method (STM)

5.8.2.1—General

The STM 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 STM should be considered for the design of deep footings and pile caps or other situations in which

C5.8.2.1

Traditional section-by-section design is based on the assumption that the reinforcement required at a particular section depends only on the independent values of the factored section force effects V_u, M_u, and T_u and does not consider the manner in which the loads and reactions that generate these sectional forces are the distance between the centers of applied load and the supporting reactions is less than two times the member depth.

The provisions of these articles apply to components with reinforcement yield strengths not exceeding 75.0 ksi and normal weight concrete compressive strengths for use in design up to 15.0 ksi.

The STM provisions herein were developed for nonhydrostatic nodes and shall not be applied to strut-and-tie models utilizing hydrostatic nodes.

Where the STM is selected for structural analysis, Articles 5.8.2.2 through 5.8.2.6 shall apply. For post-tensioning anchorage zones, diaphragms, deep beams, brackets, beam ledges and corbels, Articles 5.8.2.7 through 5.8.2.9 and 5.8.4 shall also apply.

applied. The traditional method further assumes that the shear stress distribution is essentially uniform over the depth and that the longitudinal strains will vary linearly over the depth of the beam.

For members such as deep beams, these assumptions are not valid. For example, the shear stresses on a section near a support will be concentrated near the bottom face. The behavior of a component can be predicted more accurately if the flow of forces through the complete structure is studied.

The limitation on the use of the STM for only normal weight concrete components is based solely on a lack of suitable experimental verification.

The STM provisions herein allow a more seamless transition between B- and D-Regions of a given structure. This is accomplished by replacing the previous strain-based strut efficiency calculations with newly developed efficiency factors based primarily upon the *fib Model Code for Concrete Structures 2010*. The new efficiency factors are simpler to use; less open to misinterpretation; and exhibit better statistical parameters, bias, and coefficients of variation, leading to improved accuracy and precision in a variety of applications documented by Birrcher et al. (2009), Williams et al. (2012), and Larson et al. (2013). These provisions and five other design methods are compared to 179 experimental results in Birrcher et al. (2009).

The geometry of each node should be defined prior to conducting the strength checks. Nodes may be proportioned in two ways: (1) as hydrostatic nodes or (2) as nonhydrostatic nodes. Hydrostatic nodes are proportioned in a manner that causes the stresses applied to each face to be equal. Nonhydrostatic nodes, however, are proportioned based on the origin of the applied stress. For example, the faces of a nonhydrostatic node may be sized to match the depth of the equivalent rectangular compression stress block of a flexural member or may be based upon the desired location of the longitudinal reinforcement (see Figure C5.8.2.1-1). This proportioning technique allows the geometry of the nodes to closely correspond to the actual stress concentrations at the nodal regions. In contrast, the use of hydrostatic nodes can sometimes result in unrealistic nodal geometries and impractical reinforcement layouts as shown in Figure C5.8.2.1-1. Thus, nonhydrostatic nodes are preferred in design and are used throughout these specifications.

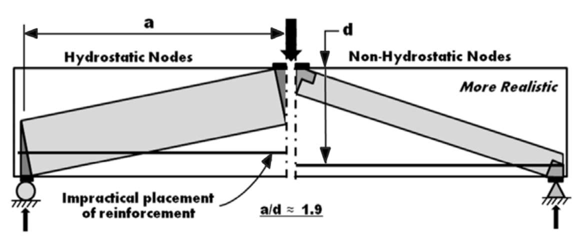


Figure C5.8.2.1-1—Hydrostatic and Nonhydrostatic Nodes

The basic steps in the STM may be taken as:

1. Determine the locations of the B- and D-Regions.
2. Define load cases.
3. Analyze structural components.
4. Size structural components using the shear serviceability check, given by Eq. C5.8.2.2-1.
5. Develop a strut-and-tie model. See Article 5.8.2.2.
6. Proportion ties.
7. Perform nodal strength checks. See Article 5.8.2.5.
8. Proportion crack control reinforcement. See Article 5.8.2.6.
9. Provide necessary anchorage for ties.

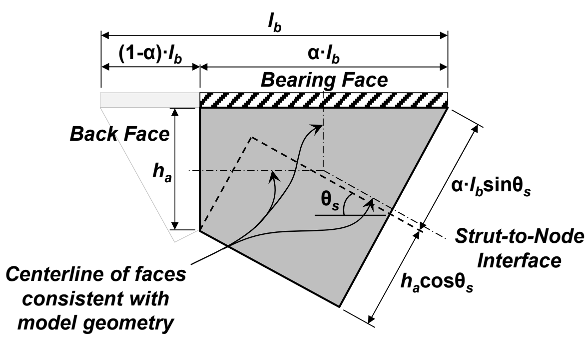
More detailed information on this method is given by Schlaich et al. (1987), Collins and Mitchell (1991), Martin and Sanders (2007), Birrcher et al. (2009), Mitchell and Collins (2013), Williams et al. (2012), and Larson et al. (2013).

5.8.2.2—Structural Modeling

The structure, and a component or region thereof, may be modeled as an assembly of steel ties and concrete struts interconnected at nodes to form a truss capable of carrying all the applied loads to the supports. As illustrated in Figure 5.8.2.2-1, nodes may be characterized as:

- CCC: nodes where only struts intersect
- CCT: nodes where a tie intersects the node in only one direction
- CTT: nodes where ties intersect in two different directions

The angle between the axes of a strut and tie should be limited to angles greater than 25 degrees.



(a) CCC Node

Figure 5.8.2.2-1—Nodal Geometries

C5.8.2.2

Cracked reinforced concrete carries load principally by compressive stresses in the concrete and tensile stresses in the reinforcement. The principal compressive stress trajectories in the concrete can be approximated by straight struts. Ties are used to model the primary reinforcement.

The following general guidelines for model development should be considered:

- At the interface of a B-Region and a D-Region, ensure the internal forces and moment within the B-Region are applied correctly to the D-Region.
- A tie must be located at the centroid of the reinforcement that carries the tensile force.
- Minimize the number of vertical ties between a load and a support using the least number of truss panels possible while still satisfying the 25-degree minimum, as shown in Figure C5.8.2.2-3.
- The strut-and-tie model must be in external and internal equilibrium.
- Ensure proper reinforcement detailing.

The angular limits are provided in order to mitigate wide crack openings and excessive strain in the reinforcement. When the angular limit cannot be satisfied, the truss configuration should be altered as appropriate.

Where a strut passes through a cold joint in the member, the joint should be investigated to determine that it has sufficient shear-friction capacity. If a D-Region is built in stages, forces imposed by each stage of construction on previously completed portions of the structure must be carried through appropriate strut-and-tie models.

A curved bar node, resulting from bending larger bars such as No. 11 and 18, is discussed in the literature

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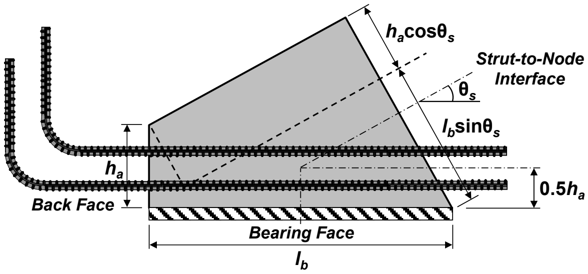
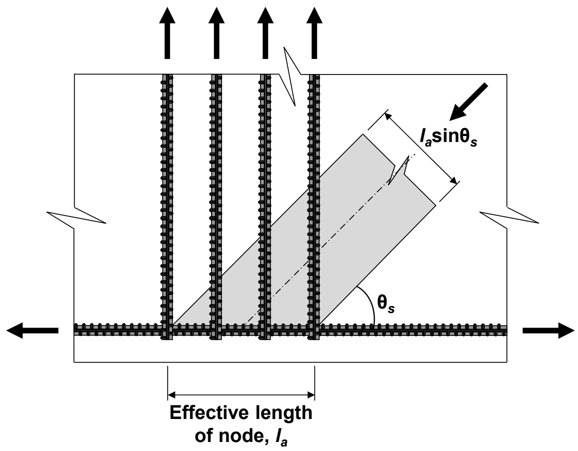


(b) CCT Node



(c) CTT Node

h_a = length of the back face of an STM node (in.)
l_a = effective length of a CTT node (in.)
l_b = length of the bearing face (in.)
α = fraction defining the bearing face length of a portion of a nodal region
θ_s = angle between strut and longitudinal axis of the member (degrees)

Figure 5.8.2.2-1 (continued)—Nodal Geometries

The configuration of a truss is dependent on the geometry of the nodal regions which shall be detailed as shown in Figures 5.8.2.2-1 and 5.8.2.2-2. Proportions of nodal regions should be based on the bearing dimensions, reinforcement location, and depth of the compression zone as illustrated in Figure 5.8.2.2-1.

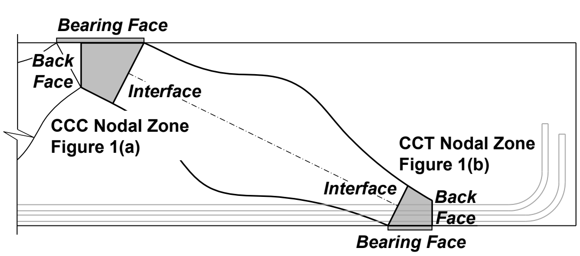


Figure 5.8.2.2-2—Strut-and-Tie Model for a Deep Beam

Where a strut is anchored only by reinforcement as in a CTT node, the effective concrete area of the node may be taken as shown in Figure 5.8.2.2-1c.

and may also be considered where a review indicates that it is appropriate (Williams et al., 2012). This type of node, shown in Figure C5.8.2.2-1, is not yet included in the specification as it is not as well vetted by experimental data as the CCC, CCT, and CTT nodes.

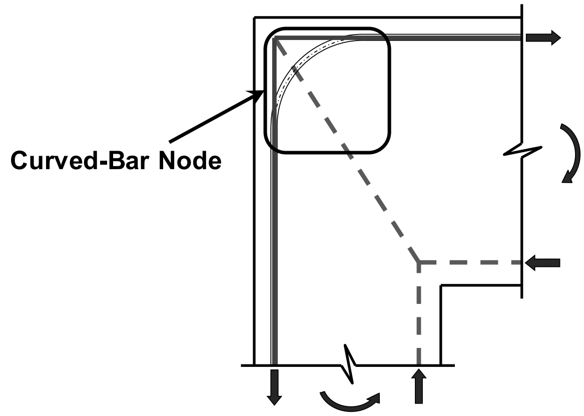


Figure C5.8.2.2-1—Curved-Bar Node

For a CCT or CTT node, the tie width, or the available length, l_a, over which the stirrups considered to carry the force in the tie can be spread, is indicated in Figure C5.8.2.2-2. The diagonal struts extending from both the load and the support are assumed to spread to form the fan shapes shown in this figure. The stirrups engaged by the fan-shaped struts are included in the vertical tie. For additional information, see Wight and Parra-Montesinos (2003). The junction of the narrow end of the fan and a CCC or CCT node may be designed using the provisions for those nodes specified herein.

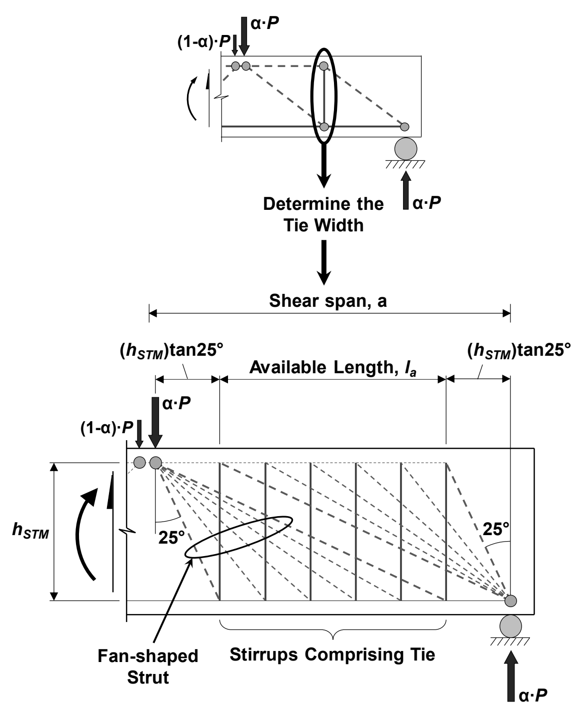


Figure C5.8.2.2-2—Fan-shaped Strut Engaging Transverse Reinforcement Forming a Tie

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Research has shown that a direct strut is the primary mechanism for transferring shear within a D-Region (Collins and Mitchell, 1991; Birrcher et al., 2009; Williams et al., 2012; and Larson et al., 2013). Therefore, a single-panel truss model is illustrated in Figure 5.8.2.2-2 and, according to these provisions, may be used in regions where the shear-span to depth ratio is less than 2 as may occur in transfer girders, bents, pile caps, or corbels. While these provisions limit the application to STM to a ratio of 2 as the transition between B-Regions and D-Regions, research has shown that the actual transition is gradual and occurs over the range of $a/d = 2$ to $a/d = 2.5$. The distance a, shown in Figure C5.8.2.2-2, is often referred to as the shear span.

A strut-and-tie truss model is shown in Figure C5.8.2.2-3 for a simply supported deep beam. The zones of high unidirectional compressive stress in the concrete are represented by 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.

Efficient and inefficient methods for modeling a simply supported beam are depicted in Figure C5.8.2.2-3. To satisfy the 25-degree minimum in the example shown, the least number of truss panels that can be provided between the applied loads and the supports is two, as shown on the left side of the beam. Two more vertical ties than necessary are used to model the flow of forces on the right side of the beam. On the right side, enough reinforcement must be provided to carry the forces in the three vertical ties each carrying 0.5P. On the left side, only the reinforcement required to carry the force in one tie carrying 0.5P is needed. The model used on the left side of the beam is more efficient since less reinforcement is needed and the resulting design is still safe.

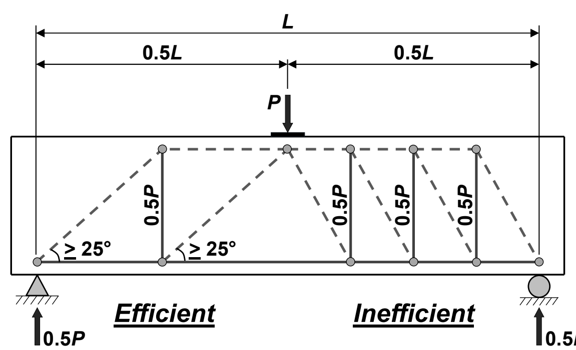

Figure C5.8.2.2-3—Effect of Modeling on the Amount of Transverse Reinforcement

The division of the load, P, to the two diagonal struts shown in Figure C5.8.2.2-3 is relatively obvious due to symmetry. A more general situation is shown in Figure C5.8.2.2-4. The truss geometry is shown in Figure C5.8.2.2-4(a). A simplification in which the two struts on the right carrying forces F_1 and F_2 are resolved into F_R is shown in Figure C5.8.2.2-4(b). The same expedient is shown on the left side of the joint with the result shown in Figure C5.8.2.2-4(c). This simplification results in two CCC nodes positioned as shown in Figure C5.8.2.2-4(d). The load, P, is divided into two statically equivalent loads assumed to act in the center of the tributary areas of the load plate. The factor, α, denotes the portion of the load supported at the right reaction in the situation shown, and (1-α) denotes the portion of the load supported at the left reaction. The amount of the load plate assigned to each CCC node is the same proportions. Note that once the load is separated into two loads, there is an associated change in the angles defining the resolved struts indicated by the change from θ_{s2} to θ_{s3}.

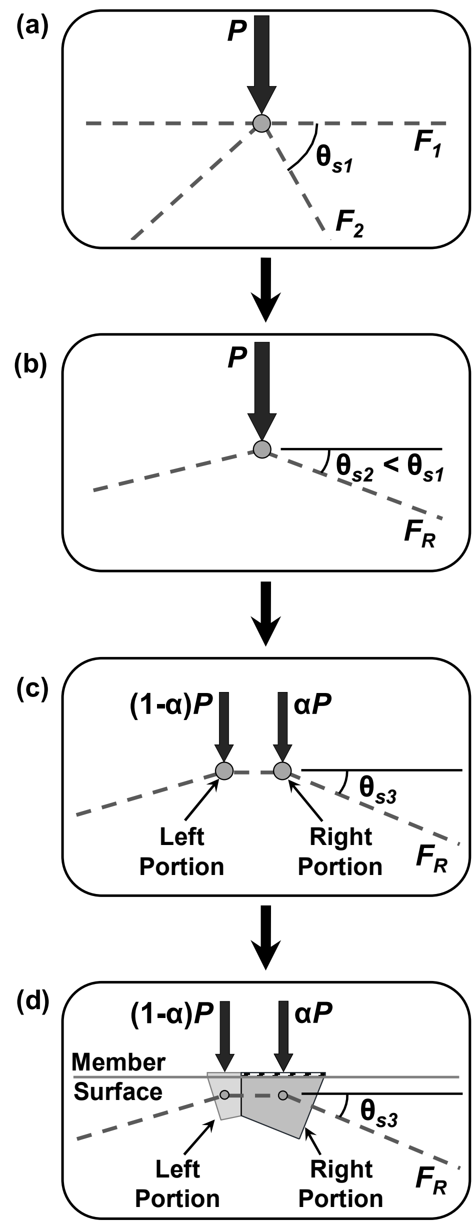

Figure C5.8.2.2-4—Division of Node when Struts Enter from Both Sides

The estimated resistance at which diagonal cracks begin to form, V_{cr}, is determined for the initial geometries of the D-Regions using the following expression (Birrcher et al., 2009):

$$V_{cr} = \left[0.2 - 0.1\left(\frac{a}{d}\right)\right]\sqrt{f_c'}b_w d \qquad \text{(C5.8.2.2-1)}$$

but not greater than $0.158\sqrt{f_c'}b_w d$ nor less than $0.0632\sqrt{f_c'}b_w d$.

where:

a = shear span (in.)
d = effective depth of the member (in.)
f'_c = compressive strength of concrete for use in design (ksi)
b_w = width of member's web (in.)

Where the shear in service is less than V_{cr}, reasonable assurance is provided that shear cracks are unlikely to form.

Stresses in a strut-and-tie model concentrate at the nodal zones. Failure of the structure may be attributed to the crushing of concrete in these critical nodal regions. For this reason, the capacity of a truss model may be directly related to the geometry of the nodal regions. Since the compressive stress will be highest at a node there is no need to investigate compression elsewhere in the strut. Where the thickness of the strut varies along its length, a more refined model, such as that shown in Figure C5.8.2.2-5 for a strut through the flanges and web of an I-Beam, may be necessary. As shown in the figure, the highest compressive stress in each strut will still be at a node.

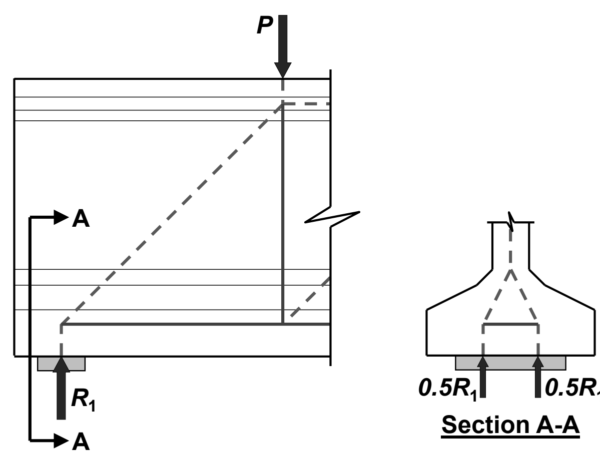

Figure C5.8.2.2-5—Refined Model for Strut with Varying Thickness along Its Length

Interior nodes that are not bounded by a bearing plate are referred to as smeared nodes, an example of which is shown in Figure 5.8.2.2-1(c). A check of concrete stresses in smeared nodes is unnecessary (Schlaich et al., 1987).

A possible application of the STM to a spread footing is shown in Figure C5.8.2.2-6. In the figure, the contact pressure is shown as a uniform load. Traditionally, crack control reinforcement as specified in Article 5.8.2.6 is often omitted from footings in which case the concrete efficiency factor, v, would be limited to 0.45 by the provisions of Article 5.8.2.5.3a. This is analogous to the exemption for transverse reinforcement permitted in Article 5.7.2.3.

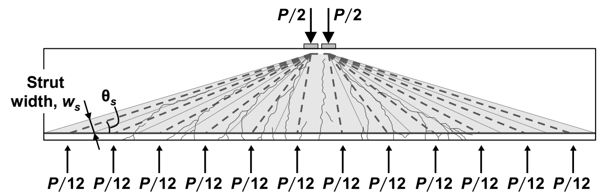

Figure C5.8.2.2-6—Application of STM to a Spread Footing

A uniform load may be treated as a series of concentrated loads as shown in Figure C5.8.2.2-7 which shows treatment of mixed beam reactions and self-weight of an inverted T-cap straddle bent.

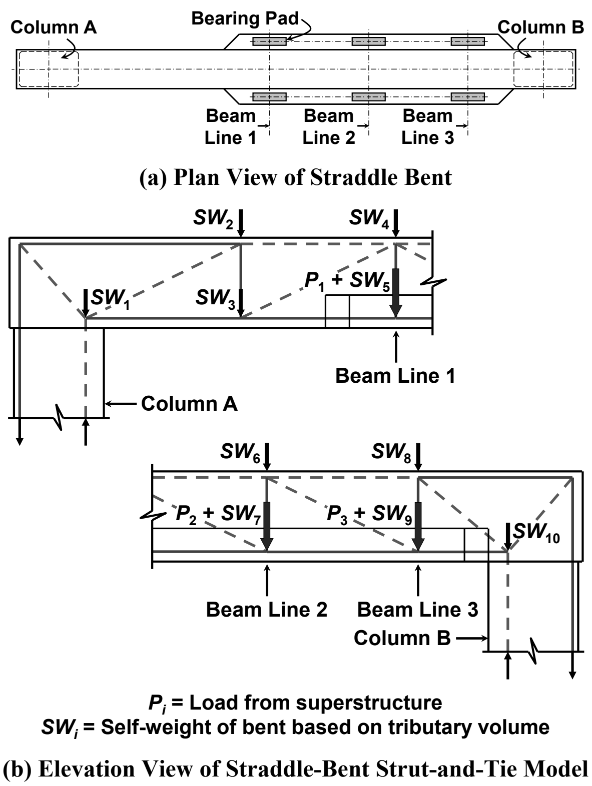

Figure C5.8.2.2-7—Application of STM to Straddle Bent with Mixed Loads

Additional guidance on treating distributed loads and reaction fanning can be found in Mitchell and Collins (2013), Martin and Sanders (2007), and Birrcher et.al. (2009). Application of STM to integral pier caps, albeit with different nodal design procedures, can be found in Martin and Sanders (2007).

5.8.2.3—Factored Resistance

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

$$P_r = \phi P_n \qquad (5.8.2.3\text{-}1)$$

where:

ϕ = resistance factor for tension or compression specified in Article 5.5.4.2, as appropriate
P_n = nominal resistance of a node face or tie (kip)

5.8.2.4—Proportioning of Ties

5.8.2.4.1—Strength of Tie

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

$$P_n = f_y A_{st} + A_{ps}\left[f_{pe} + f_y\right] \qquad (5.8.2.4.1\text{-}1)$$

where:

f_y = yield strength of nonprestressed longitudinal reinforcement (ksi)
A_{st} = total area of longitudinal nonprestressed reinforcement (in.^{2})
A_{ps} = area of prestressing steel (in.^{2})
f_{pe} = effective stress in prestressing steel after losses (ksi)

The sum of f_{pe} and f_y in Eq. 5.8.2.4.1-1 shall not be taken greater than the yield strength of the prestressing steel.

C5.8.2.4.1

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 nonprestressed reinforcement will undergo. If there is no nonprestressed reinforcement, f_y may be taken as 60.0 ksi in the second term of Eq. 5.8.2.4.1-1.

5.8.2.4.2—Anchorage of Tie

The 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 Articles 5.9.4.3 and 5.10.8.2.

C5.8.2.4.2

The ties must be properly anchored to ensure that the tie force can be fully developed and that the structure can achieve the resistance assumed by the STM. For a tie to be properly anchored at a nodal region, the yield resistance of the nonpretressed reinforcement should be developed at the point where the centroid of the bars exits the extended nodal zone as shown in Figure C5.8.2.4.2-1. In other words, the critical section for the development of the tie is taken at the location where the centroid of the bars intersects the edge of the diagonal strut.

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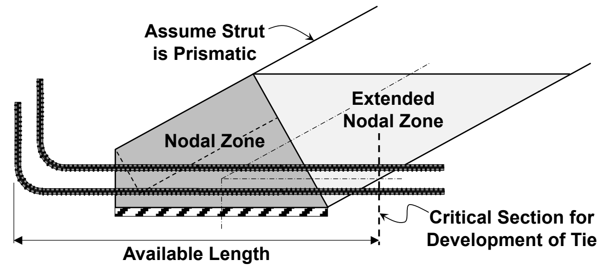

Figure C5.8.2.4.2-1—Available Development Length for Ties

5.8.2.5—Proportioning of Node Regions

5.8.2.5.1—Strength of a Node Face

The nominal resistance of the node face shall be taken as:

$$P_n = f_{cu}A_{cn} \tag{5.8.2.5.1-1}$$

where:

P_n = nominal resistance of a node face (kip)
f_{cu} = limiting compressive stress at the node face as specified in Article 5.8.2.5.3 (ksi)
A_{cn} = effective cross-sectional area of the node face as specified in Article 5.8.2.5.2 (in.^{2})

Where the back face of a CCC node contains nonprestressed compressive reinforcement, the resistance given by Eq. 5.8.2.5.1-1 may be augmented with the yield resistance of the nonprestressed reinforcement.

5.8.2.5.2—Effective Cross-Sectional Area of the Node Face

The value of A_{cn} shall be determined by considering the details of the nodal region and the in-plane dimensions illustrated in Figure 5.8.2.2-1. The out-of-plane dimension may be based on the bearing device or devices, or the dimension of the member, as appropriate.

When a strut is anchored by reinforcement, the height of the back face, h_a, of the CCT node may be considered to extend twice the distance from the exterior surface of the beam to the centroid of the longitudinal tensile reinforcement, as shown in Figure 5.8.2.2-1(b).

The depth of the back face of the CCC node, h_a, as shown in Figure 5.8.2.2-1(a), may be taken as the effective depth of the compression stress block determined from a conventional flexural analysis.

C5.8.2.5.2

Research has shown that the shear behavior of conventionally reinforced deep beams, as wide as 36.0 in., is not significantly influenced by the distribution of stirrups across the section. Beams wider than 36.0 in., or beams with a width to height aspect ratio greater than one, may benefit from distributing the area of transverse reinforcement across the width of the cross section. Based on limited research, the additional stirrup legs should be spaced transversely no more than the shear depth of the beam but no less than 30.0 in. (Birrcher et al., 2009). The 30.0 in. recommendation is based on the spacing of the legs on the transverse in the 36.0 in. wide beams, rounded down to a convenient number.

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5.8.2.5.3—Limiting Compressive Stress at the Node Face

5.8.2.5.3a—General

Unless confinement reinforcement is provided and its effect is supported by analysis or experimentation, the limiting compressive stress at the node face, f_{cu}, shall be taken as:

$$f_{cu} = mvf'_c \quad (5.8.2.5.3a\text{-}1)$$

where:

m = confinement modification factor, taken as $\sqrt{(A_2/A_1)}$ but not more than 2.0 as defined in Article 5.6.5, where:

- A_1 = area under the bearing device (in.^{2})
- A_2 = notional area specified in Article 5.6.5 (in.^{2})

v = concrete efficiency factor:

- 0.45 for structures that do not contain crack control reinforcement as specified in Article 5.8.2.6
- as shown in Table 5.8.2.5.3a-1 for structures with crack control reinforcement as specified in Article 5.8.2.6

f'_c = compressive strength of concrete for use in design (ksi)

In addition to satisfying strength criteria, the node regions shall be designed to comply with the stress and anchorage limits specified in Articles 5.8.2.4.1 and 5.8.2.4.2.

C5.8.2.5.3a

Concrete efficiency factors have been selected based on simplicity in application, compatibility with other sections of the specifications, compatibility with tests of D-Regions, and compatibility with other provisions.

Eq. 5.8.2.5.3a-1 is valid for design concrete compressive strengths up to 15.0 ksi and therefore stress-block factors, k_c and α_1, for high-strength concrete need not be applied to this equation.

The efficiency factors specified herein were derived for nonhydrostatic nodes and are shown graphically in Figure C5.8.2.5.3a-1.

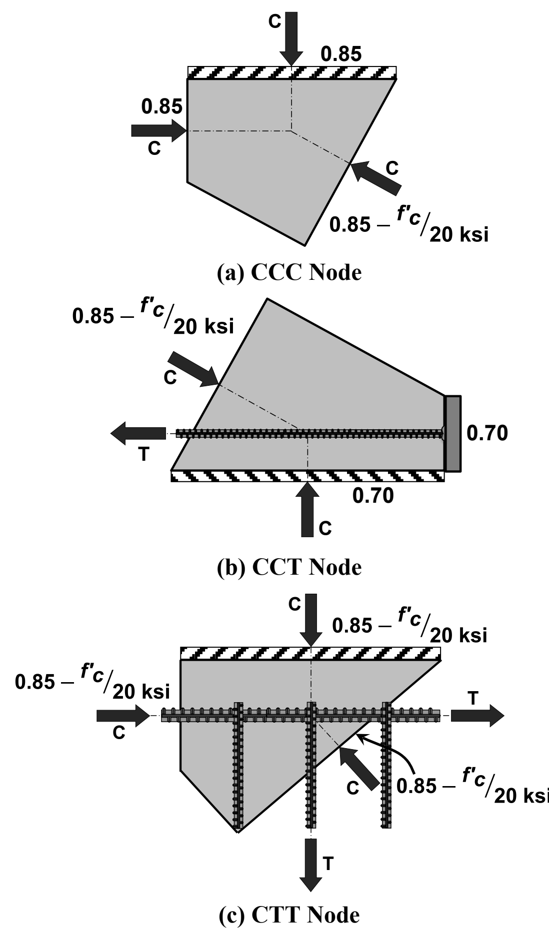


Figure C5.8.2.5.3a-1—Depiction of Efficiency Factors

Table 5.8.2.5.3a-1—Efficiency Factors for Nodes with Crack Control Reinforcement

	Node Type		
Face	**CCC**	**CCT**	**CTT**
Bearing Face Back Face	0.85	0.70	
Strut-to-Node Interface	$0.85 - \frac{f'_c}{20\text{ ksi}}$ $0.45 \le v \le 0.65$	$0.85 - \frac{f'_c}{20\text{ ksi}}$ $0.45 \le v \le 0.65$	$0.85 - \frac{f'_c}{20\text{ ksi}}$ $0.45 \le v \le 0.65$

5.8.2.5.3b—Back Face of a CCT Node

Bond stresses resulting from the force in a developed tie as shown in Figure 5.8.2.5.3b-1 need not be applied to the back face of the CCT node.

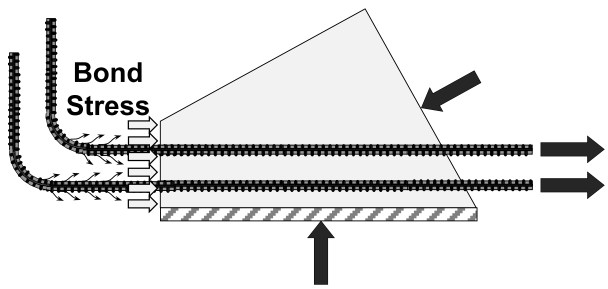
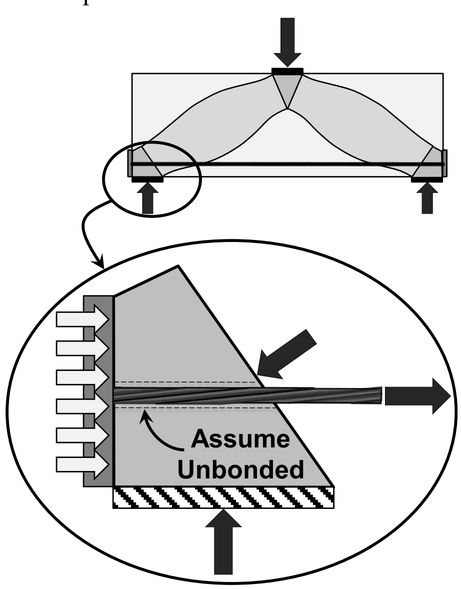


Figure 5.8.2.5.3b-1—Bond Stress Resulting from the Anchorage of a Developed Tie

The bearing stresses resulting from an anchor plate or headed bar, or an external indeterminacy such as that which occurs at a node over a continuous support as shown in Figure 5.8.2.5.3b-2 shall be investigated using the applicable provisions of Article 5.8.2.5.



(a) Bearing Stress Applied from an Anchor Plate or Headed Bar

Figure 5.8.2.5.3b-2—Stress Condition at the Back Face of a CCT Node

C5.8.2.5.3b

An examination of experimental studies in the literature did not reveal any cases where the back face of a CCT node controlled the capacity in test specimens where deformed bars were used as tie steel. (Birrcher et al., 2009)

If the stress applied to the back face of a CCT node is from an anchor plate or headed bar, a check of the back face stresses should be made assuming that the bar is unbonded and all of the tie force is transferred to the anchor plate or bar head.

If the stress applied to the back face of a CCT node is the result of a combination of both anchorage and a discrete force from another strut, it is only necessary to proportion the node to resist the direct compression stresses. It is not necessary to apply the bonding stresses to the back face, provided the tie is adequately anchored.

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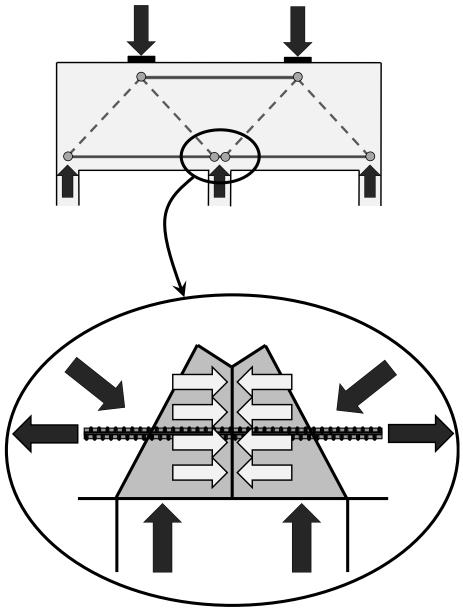

(b) Interior Node Over a Continuous Support

Figure 5.8.2.5.3b-2 (continued)—Stress Condition at the Back Face of a CCT Node

5.8.2.6—Crack Control Reinforcement

Structures and components or regions thereof, except for slabs and footings, which have been designed using the efficiency factor of Table 5.8.2.5.3a-1, shall contain orthogonal grids of bonded reinforcement. 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 direction shall satisfy the following:

$$\frac{A_v}{b_w s_v} \geq 0.003 \qquad (5.8.2.6\text{-}1)$$

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

$$\frac{A_h}{b_w s_h} \geq 0.003 \qquad (5.8.2.6\text{-}2)$$

where:

A_v = total area of vertical crack control reinforcement within spacing s_v (in.^{2})
b_w = width of member's web (in.)
s_v, s_h = spacing of vertical and horizontal crack control reinforcement, respectively (in.)
A_h = total area of horizontal crack control reinforcement within spacing s_h (in.^{2})

Where provided, crack control reinforcement shall be distributed evenly near the side faces of the strut.

C5.8.2.6

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. This behavior is consistent with the plasticity basis of STM as discussed in Article C5.5.1.2.3.

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 C5.8.2.6-1. Crack-control reinforcement is intended to provide restraint for the spread of compression present in a bottle-shaped strut. It does not serve as the primary tie shown in Figure C5.8.2.6-1. For thinner members, this crack control reinforcement will consist of two grids of reinforcement, one near each face. For thicker members, multiple grids of reinforcement through the thickness may be required in order to achieve a practical layout. Further discussion on the distribution of shear reinforcement through the thickness of thin and thick members can be found in Article C5.8.2.5.2.

An investigation of the effect of various amounts of crack control reinforcement indicated that beyond a reinforcement ratio corresponding to 0.003 times the effective area of the strut there continued to be a reduction of crack widths but the efficacy of the increasing steel reached a point of diminishing returns. Specimens tested by Birrcher et al. (2009) and Larson et al. (2013) showed that the width of the first diagonal crack forming in a deep beam was unacceptably large (i.e. greater than 0.016 in.) if crack control

Where necessary, interior layers of crack control reinforcement may be used.

reinforcement provided in that specimen was less than 0.003 times the effective area of the strut.

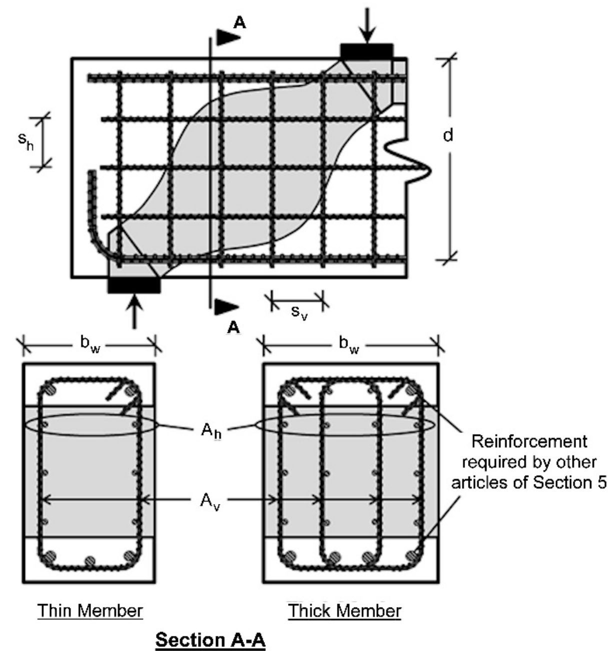

Figure C5.8.2.6-1—Distribution of Crack Control Reinforcement in Struts

5.8.2.7—Application to the Design of the General Zones of Post-Tensioning Anchorages

5.8.2.7.1—General

The flow of forces in the anchorage zone may be approximated by a strut-and-tie model as specified herein.

All forces acting on the anchorage zone shall be considered in the selection of a strut-and-tie model which should follow a load path from the anchorages to the end of the anchorage zone.

C5.8.2.7.1

A conservative estimate of the resistance of a concrete structure or member may be obtained by application of the lower bound theorem of the theory of plasticity of structures. If sufficient ductility is present in the system, strut-and-tie models fulfill the conditions for the application of the above-mentioned theorem. Figure C5.8.2.7.1-1 shows the linear elastic stress field and a corresponding strut-and-tie model for the case of an anchorage zone with two eccentric anchors (Schlaich et al., 1987).

Because of the limited ductility of concrete, strut-and-tie models, which are not greatly different from the elastic solution in terms of stress distribution, should be selected. This procedure will reduce the required stress redistributions in the anchorage zone and ensure that reinforcement is provided where cracks are most likely to occur. Strut-and-tie models for some typical load cases for anchorage zones are shown in Figure C5.8.2.7.1-2.

Figure C5.8.2.7.1-3 shows the strut-and-tie model for the outer regions of general anchorage zones with eccentrically located anchorages. The anchorage local zone becomes a node for the strut-and-tie model and the adequacy of the node must be checked by appropriate analysis or full-scale testing.

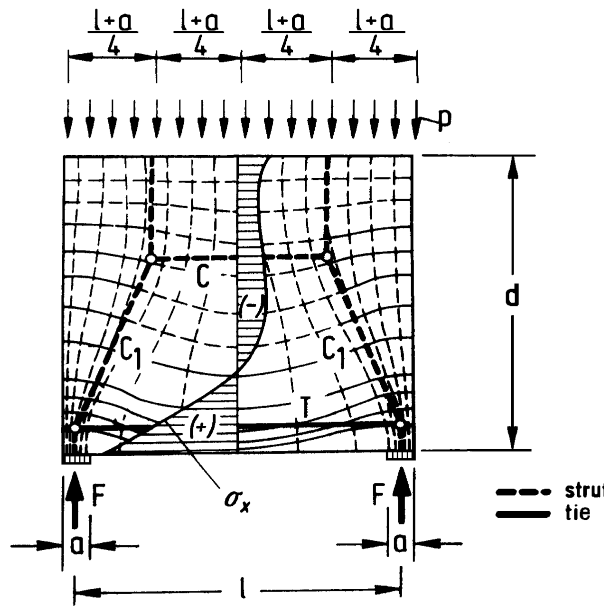

Figure C5.8.2.7.1-1—Principal Stress Field and Superimposed Strut-and-Tie Model

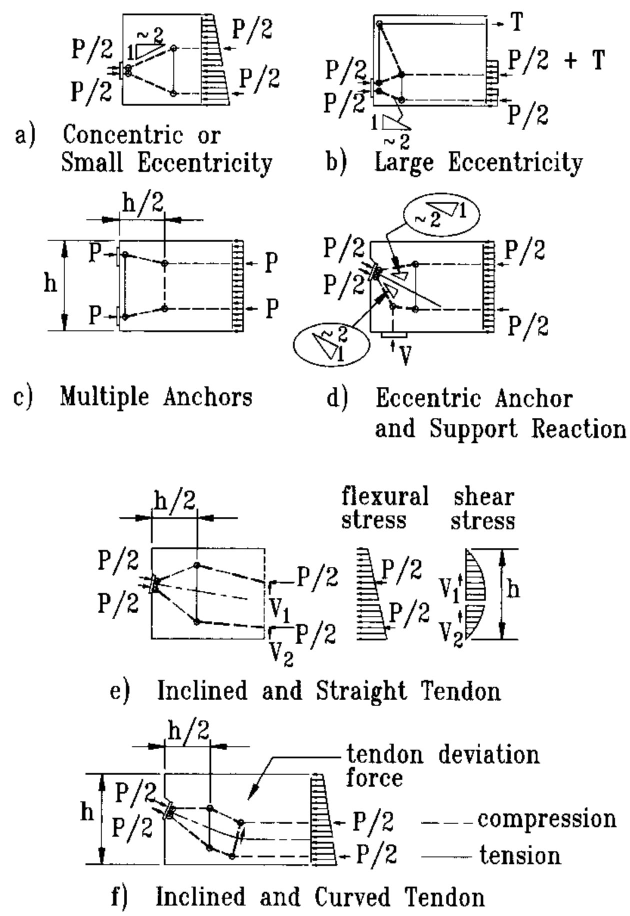

Figure C5.8.2.7.1-2—Strut-and-tie Models for Selected Anchorage Zones

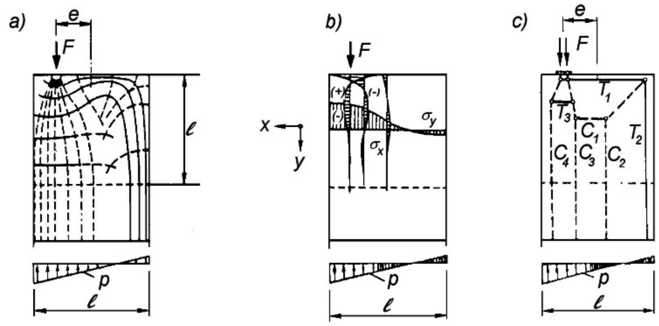

Figure C5.8.2.7.1-3—Strut-and-Tie Model for the Outer Regions of the General Zone

5.8.2.7.2—Nodes

Local zones that satisfy the requirements of Article 5.8.4.4 of these specifications or Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications* may be considered as properly detailed and are adequate nodes. The other nodes in the anchorage zone may be considered adequate if the effective concrete stresses in the struts satisfy the requirements of Article 5.8.2.7.3, and the ties are detailed to develop the full yield strength of the reinforcement.

C5.8.2.7.2

Nodes are critical elements of the strut-and-tie model. The entire local zone constitutes the most critical node or group of nodes for anchorage zones. In Article 5.8.4.4 of these specifications, the adequacy of the local zone is ensured by limiting the bearing pressure under the anchorage device. Alternatively, this limitation may be exceeded if the adequacy of the anchorage device is proven by the acceptance test of Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications.*

5.8.2.7.3—Struts

The factored compressive stress shall not exceed the limits specified in Article 5.9.5.6.5a.

C5.8.2.7.3

For strut-and-tie models oriented on the elastic stress distribution, the design concrete compressive strength specified in Article 5.9.5.6.5a is adequate. However, if the selected strut-and-tie model deviates considerably from the elastic stress distribution, large plastic deformations are required and the usable concrete strength should also be reduced if the concrete is cracked due to other load effects.

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

Ordinarily, the geometry of the local zone node and, thus, of the interface between strut and local zone, is determined by the size of the bearing plate and the selected strut-and-tie model, as indicated in Figure 5.8.2.2-1(a). Based on the acceptance test of Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications*, the stresses on special anchorage devices should be investigated at a larger distance from the node, assuming that the width of the strut increases with the distance from the local zone.

For thin members, the dimension of the strut in the direction of the thickness of the member may be approximated by assuming that the thickness of the strut varies linearly from the transverse lateral dimension of the anchor at the surface of the concrete to the total thickness of the section at a depth equal to the thickness of the section.

The compression stresses should be assumed to act parallel to the axis of the strut and to be uniformly distributed over its cross section.

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5.8.2.7.4—Ties

Ties consisting of nonprestressed or prestressed reinforcement shall resist the total tensile force.

Ties shall extend beyond the nodes to develop the full-tie force at the node. The reinforcement layout should follow as closely as practical the paths of the assumed ties in the strut-and-tie model.

C5.8.2.7.4

Because of the unreliable strength of concrete in tension, it is prudent to neglect it entirely in resisting tensile forces.

In the selection of a strut-and-tie model, only practical reinforcement arrangements should be considered. The reinforcement layout, actually detailed on the plans, should be in agreement with the selected strut-and-tie model.

5.8.2.8—Application to the Design of Pier Diaphragms

The flow of forces in pier diaphragms may be approximated by the strut-and-tie method.

C5.8.2.8

Figure C5.8.2.8-1 illustrates the application of the strut-and-tie method to analysis of forces in a prestressed interior diaphragm of a box girder bridge.

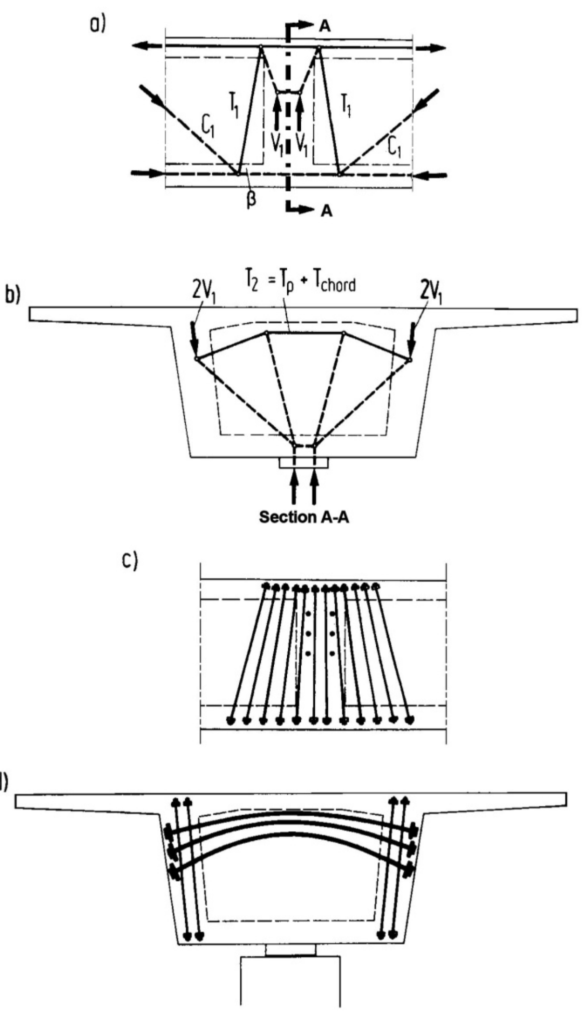

Figure C5.8.2.8-1—Diaphragm of a Box Girder Bridge: (a) Disturbed Regions and Model of the Web near the Diaphragm; (b) Diaphragm and Model; (c) and (d) Prestressing of the Web and the Diaphragm (Schlaich et al., 1987)

5.8.2.9—Application to the Design of Brackets and Corbels

The flow of forces in corbels and beam ledges may be approximated by the strut-and-tie method as an alternative to the provisions of Article 5.8.4.2.2. Detailing shall conform to the applicable provisions of Article 5.8.4.2.

C5.8.2.9

Figure C5.8.2.9-1 illustrates the application of strut-and-tie models to analysis of brackets and corbels.

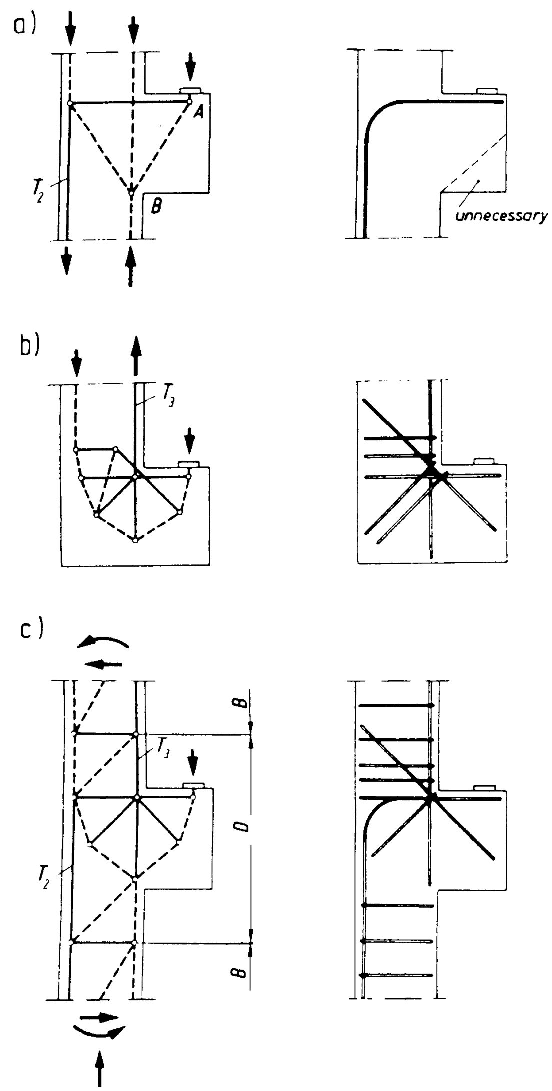

Figure C5.8.2.9-1—Different Support Conditions Leading to Different Strut-and-Tie Models and Different Reinforcement Arrangements of Corbels and Beam Ledges (Schlaich et al., 1987)

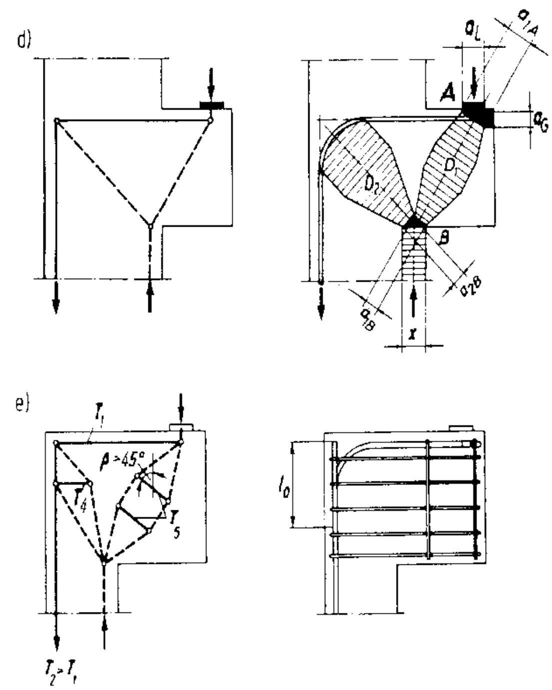

Figure C5.8.2.9-1 (continued) —Different Support Conditions Leading to Different Strut-and-Tie Models and Different Reinforcement Arrangements of Corbels and Beam Ledges (Schlaich et al., 1987)

5.8.3—Elastic Stress Analysis

5.8.3.1—General

Analyses based on elastic material properties, equilibrium of forces and loads, and compatibility of strains may be used for the analysis and design of anchorage zones.

5.8.3.2—General Zones of Post-Tensioning Anchorages

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

5.8.4—Approximate Stress Analysis and Design

5.8.4.1—Deep Components

Although the strut-and-tie method of Article 5.8.2 is the preferred method for designing deep components, legacy methods that have served an Owner well may be used provided that all of the following are met:

- The provisions of Article 5.8.2.6 specifying the amount and spacing of crack control reinforcement are met as a minimum;

C5.8.4.1

ACI 318-14 contains further information on, and references for, deep component legacy methods. No particular method is specified or recommended although crack control reinforcement and a limit on shear capacity is specified.

Many of the treatments of deep components in the literature deal with situations where the load is applied on top of the component and the support is located on

- A limit on usable shear capacity is specified;
- The loading is placed at the appropriate depth of the component relative to the reactions in a manner consistent with the legacy method being used;
- The method of analysis reflects the disturbed stress field, the behavior of cracked concrete, and other nonlinear behavior anticipated at the strength or extreme event limit states.

the bottom. Unless a legacy method expressly treats situations in which loads are applied along the web or near the bottom of the component, the behavior can be accurately accounted for only by the strut-and-tie method or a finite element analysis utilizing nonlinear material properties that also accounts for cracked concrete.

5.8.4.2—Brackets and Corbels

5.8.4.2.1—General

The section at the face of the support for brackets and corbels may be designed in accordance with either the strut-and-tie method specified in Article 5.8.2 or the alternative to strut-and-tie model specified in Article 5.8.4.2.2.

Components in which a_v, as shown in Figure 5.8.4.2.1-1, is less than d shall be considered to be brackets or corbels. If a_v is greater than d, the component shall be designed as a cantilever beam.

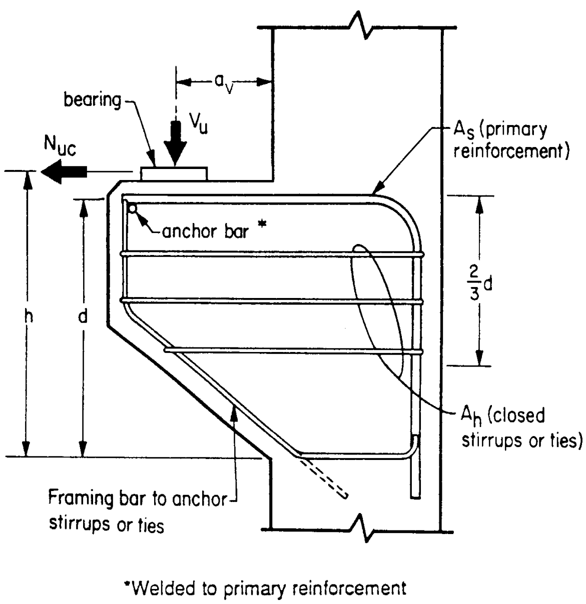

Figure 5.8.4.2.1-1—Typical Bracket or Corbel

The section at the face of support shall be designed to resist simultaneously a factored shear force, V_u, a factored moment, M_u, and the concurrent factored tensile force, N_{uc}, where:

$$M_u = V_u a_v + N_{uc}(h-d) \qquad (5.8.4.2.1\text{-}1)$$

and a concurrent factored horizontal tensile force, N_{uc}. Unless special provisions are made to prevent the tensile force, N_{uc}, from developing, it shall not be taken to be less than $0.2V_u$. N_{uc} shall be regarded as a live load, even where it results from creep, shrinkage, or temperature change.

C5.8.4.2.1

The steel ratio of A_s/bd at the face of the support shall not be less than $0.04f'_c/f_y$, where d is measured at the face of the support.

The total area, A_h, of the closed stirrups or ties shall not be less than 50 percent of the area, A_s, of the primary tensile tie reinforcement. Stirrups or ties shall be uniformly distributed within two thirds of the effective depth adjacent to the primary tie reinforcement.

At the front face of a bracket or corbel, the primary tension reinforcement shall be anchored to develop the specified yield strength, f_y.

Anchorages for developing reinforcement may include:

- a structural weld to a transverse bar of at least equal size;
- bending the primary bars down to form a continuous loop; or
- some other positive means of anchorage.

The bearing area on a bracket or corbel shall not project either beyond the straight portion of the primary tension bars or beyond the interior face of any transverse anchor bar.

The depth at the outside edge of the bearing area shall not be less than half the depth at the face of the support.

5.8.4.2.2—Alternative to Strut-and-Tie Model

Where the provisions of Article 5.8.4.2.1 are invoked, all of the following shall apply:

- Design of shear-friction reinforcement, A_{vf}, to resist the factored shear force, V_u, shall be as specified in Article 5.7.4, except that:

  For normal weight concrete, nominal shear resistance, V_n, shall be determined as the lesser of the following:

  $$V_n = 0.2 f'_c b d_e \qquad (5.8.4.2.2\text{-}1)$$

  $$V_n = 0.8 b d_e \qquad (5.8.4.2.2\text{-}2)$$

  For lightweight concrete, nominal shear resistance, V_n, shall be determined as the lesser of the following:

  $$V_n = \left(\frac{0.2 - 0.07 a_v}{d_e}\right) f'_c b d_e \qquad (5.8.4.2.2\text{-}3)$$

  $$V_n = \left(\frac{0.8 - 0.28\ a_v}{d_e}\right) b d_e \qquad (5.8.4.2.2\text{-}4)$$

- Reinforcement, A_s, to resist the factored force effects shall be determined as for ordinary members subjected to flexure and axial load.

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- Area of primary tension reinforcement, A_s, shall satisfy:

$$A_s \geq \frac{2A_{vf}}{3} + A_n \text{, and} \qquad (5.8.4.2.2\text{-}5)$$

- The area of closed stirrups or ties placed within a distance equal to $2d_e/3$ from the primary reinforcement shall satisfy:

$$A_h \geq 0.5\left(A_s - A_n\right) \qquad (5.8.4.2.2\text{-}6)$$

in which:

$$A_n \geq \frac{N_{uc}}{\phi f_y} \qquad (5.8.4.2.2\text{-}7)$$

where:

b = width of corbel or ledge (in.)
d_e = depth of center of gravity of steel (in.)
A_{vf} = area of shear-friction reinforcement (in.^{2})
A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc} (in.^{2})

5.8.4.3—Beam Ledges

5.8.4.3.1—General

As illustrated in Figure 5.8.4.3.1-1, beam ledges shall resist:

- Flexure, shear, and horizontal forces at the location of Crack 1;
- Tension force in the supporting element at the location of Crack 2;
- Punching shear at points of loading at the location of Crack 3; and
- Bearing force at the location of Crack 4.

C5.8.4.3.1

Beam ledges may be distinguished from brackets and corbels in that their width along the face of the supporting member is greater than ($W + 5a_f$), as shown in Figure 5.8.4.3.3-1. In addition, beam ledges are supported primarily by ties to the supporting member, whereas corbels utilize a strut penetrating directly into the supporting member. Beam ledges are generally continuous between points of application of bearing forces. Daps should be considered to be inverted beam ledges.

Examples of beam ledges include hinges within spans and inverted T-beam caps, as illustrated in Figure C5.8.4.3.1-1. In the case of an inverted T-beam pier cap, the longitudinal members are supported by the flange of the T-beam acting as beam ledges.

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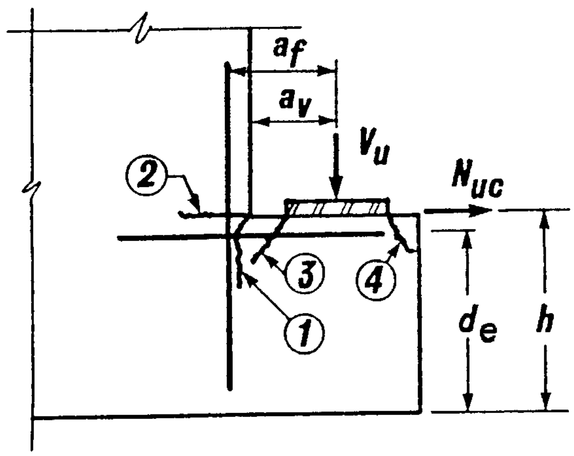

Figure 5.8.4.3.1-1—Notation and Potential Crack Locations for Ledge Beams

Beam ledges may be designed in accordance with either the strut-and-tie method or the provisions of Articles 5.8.4.3.2 through 5.8.4.3.6. Bars shown in Figures 5.8.4.3.2-1 through 5.8.4.3.5-2 shall be properly developed in accordance with Article 5.10.8.1.1.

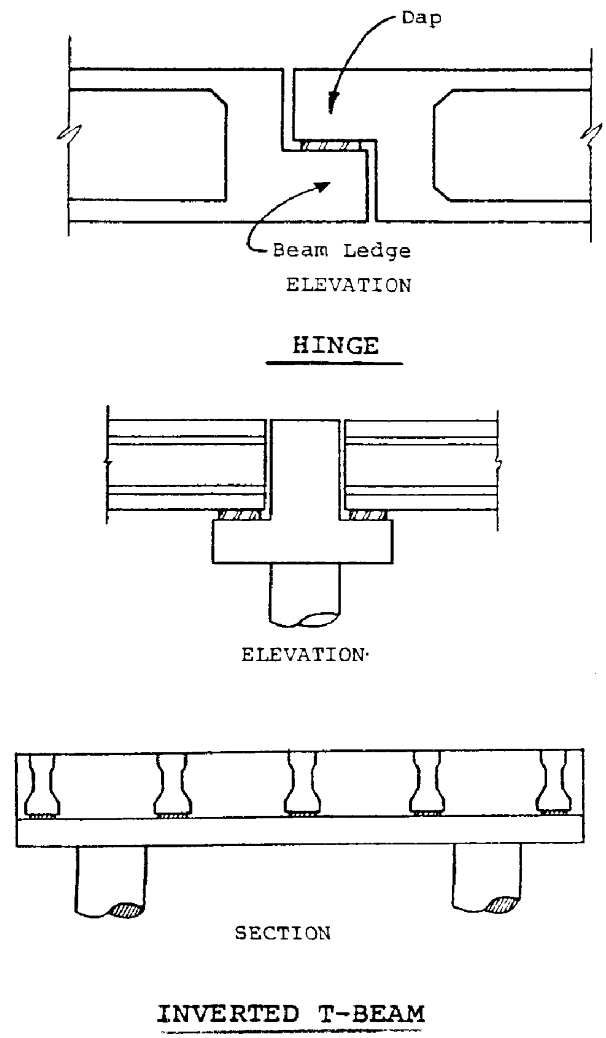

Figure C5.8.4.3.1-1—Examples of Beam Ledges

5.8.4.3.2—Design for Shear

Design of beam ledges for shear shall be in accordance with the requirements for shear friction specified in Article 5.7.4. Nominal interface shear resistance shall satisfy Eqs. 5.8.4.2.2-1 through 5.8.4.2.2-4 wherein the width of the concrete face, b, assumed to participate in resistance to shear shall not exceed the lesser of the following:

- $(W + 4a_v)$, or $2c$, as appropriate to the situation illustrated in Figure 5.8.4.3.2-1
- S

where:

W = width of bearing plate or pad (in.)

c = spacing from centerline of bearing to end of beam ledge (in.)

S = center-to-center spacing of bearings along a beam ledge (in.)

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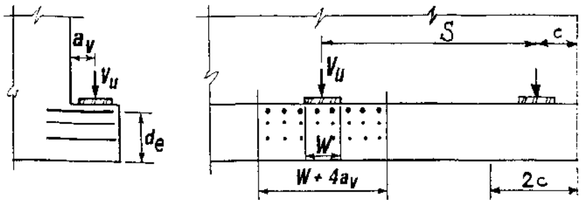

Figure 5.8.4.3.2-1—Design of Beam Ledges for Shear

5.8.4.3.3—Design for Flexure and Horizontal Force

The section at the face of support shall be designed to resist a factored horizontal force, N_u, and a factored moment, M_u, acting simultaneously, similar to that required for corbels by Eq. 5.8.4.2.1-1.

The area of total primary tension reinforcement, A_s, shall satisfy the requirements of Article 5.8.4.2.2.

The primary tension reinforcement shall be spaced uniformly within the region (W + $5a_f$) or $2c$, as illustrated in Figure 5.8.4.3.3-1, except that the widths of these regions shall not overlap.

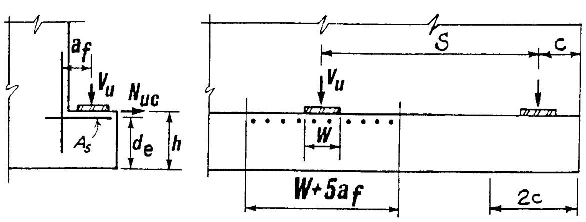

Figure 5.8.4.3.3-1—Design of Beam Ledges for Flexure and Horizontal Force

5.8.4.3.4—Design for Punching Shear

Notation used in this article shall be taken as shown in Figures 5.8.4.3.4-1 through 5.8.4.3.4-3.

The truncated pyramids assumed as failure surfaces for punching shear, as illustrated in Figure 5.8.4.3.4-1, shall not overlap.

To prevent overlap of failure surfaces:

- Between adjacent bearings on a beam ledge or inverted T-beam:

$$S > 2d_f + W \tag{5.8.4.3.4-1}$$

- Between bearings on opposite ledges of an inverted T-beam web:

$$b_w + 2a_v > L + 2d_f \tag{5.8.4.3.4-2}$$

where:

S = center-to-center spacing of bearing along a beam ledge (in.)
d_f = distance from top of ledge to the bottom longitudinal reinforcement (in.)

C5.8.4.3.4

The area of concrete resisting the punching shear for each concentrated load is shown in Figure 5.8.4.3.4-1. The area of the truncated pyramid is approximated as the average of the perimeter of the bearing plate or pad and the perimeter at depth d, assuming 45-degree slopes. If the pyramids overlap, an investigation of the combined surface areas will be necessary.

W = width of bearing plate or pad (in.)
b_w = the width of the inverted T-beam stem (in.)
a_v = distance from face of wall to the concentrated load (in.)
L = length of bearing pad (in.)

Nominal punching shear resistance, V_n, in kips, shall be taken as:

$$V_n = 0.125\lambda\sqrt{f'_c}b_o d_f \quad (5.8.4.3.4\text{-}3)$$

where:

λ = concrete density modification factor as specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design (ksi)
b_o = the perimeter of the critical section for shear enclosing the bearing pad or plate (in.)

This perimeter need not approach closer than a distance of 0.5 d_f to the edges of the bearing pad or plate.

For rectangular or circular pads with failure surfaces that do not overlap, b_o shall be taken as:

- At interior rectangular pads:

$$b_o = W + 2L + 2d_f \quad (5.8.4.3.4\text{-}4)$$

- At exterior rectangular pads:

$$b_o = 0.5W + L + d_f + c \leq W + 2L + 2d_f \quad (5.8.4.3.4\text{-}5)$$

- At interior circular pads:

$$b_o = \frac{\pi}{2}\left(D + d_f\right) + D \quad (5.8.4.3.4\text{-}6)$$

- At exterior circular pads:

$$b_o = \frac{\pi}{4}\left(D + d_f\right) + \frac{D}{2} + c \leq \frac{\pi}{2}\left(D + d_f\right) + D \quad (5.8.4.3.4\text{-}7)$$

where:

W = width of bearing plate or pad (in.)
L = length of bearing pad (in.)
c = spacing from the centerline of bearing to the end of beam ledge (in.)
D = diameter of circular bearing pad (in.)

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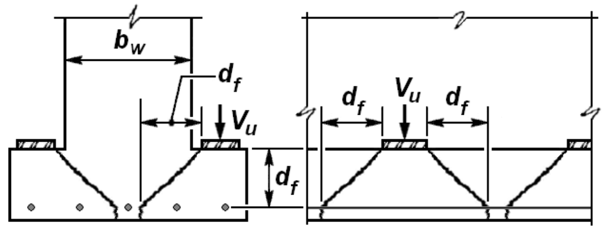

Figure 5.8.4.3.4-1—Failure Surfaces for Punching Shear

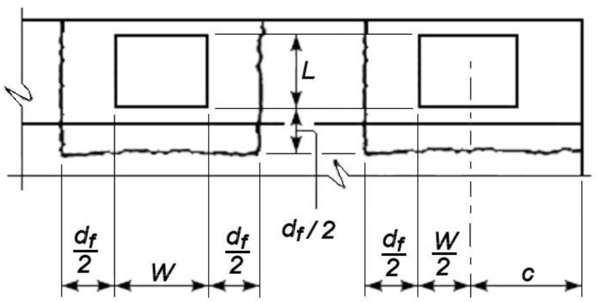

Figure 5.8.4.3.4-2—Critical Section for Rectangular Bearing Pads

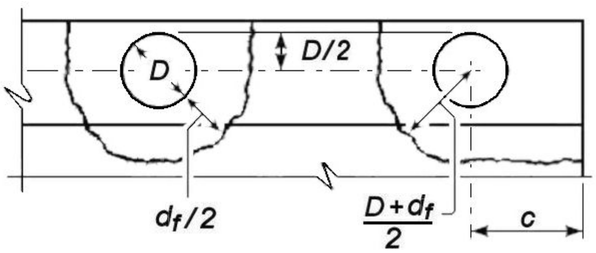

Figure 5.8.4.3.4-3—Critical Section for Circular Bearing Pads

5.8.4.3.5—Design of Hanger Reinforcement

The hanger reinforcement specified herein shall be provided in addition to the shear reinforcement required on either side of the beam reaction being supported.

The arrangement for hanger reinforcement, A_{hr}, in single-beam ledges shall be as shown in Figure 5.8.4.3.5-1. Using the notation in Figure 5.8.4.3.5-1, the nominal shear resistance, V_n, in kips shall be taken as:

- For the service limit state:

$$V_n = \frac{0.5 A_{hr} f_y \left(W + 3a_v\right)}{s} \qquad (5.8.4.3.5\text{-}1)$$

in which $(W + 3a_v)$ shall not exceed either S or $2c$.

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- For the strength limit state:

$$V_n = \frac{A_{hr} f_y}{s} S \qquad (5.8.4.3.5\text{-}2)$$

in which S shall not exceed $2c$.

where:

A_{hr} = area of one leg of hanger reinforcement as illustrated in Figure 5.8.4.3.5-1 (in.^{2})
S = center-to-center spacing of bearing along a beam ledge (in.)
s = spacing of hanger reinforcing bars (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
a_v = distance from face of wall to the concentrated load as illustrated in Figure 5.8.4.3.5-1 (in.)

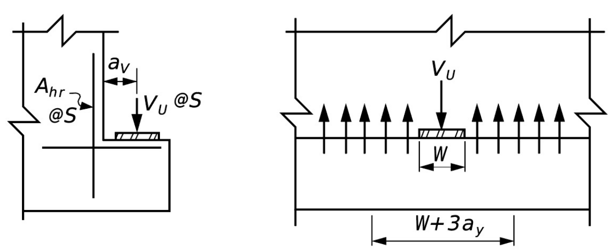

Figure 5.8.4.3.5-1—Single-Ledge Hanger Reinforcement

Using the notation in Figure 5.8.4.3.5-2, the nominal shear resistance of the ledges of inverted T-beams shall be the lesser of that specified by Eq. 5.8.4.3.5-2 and Eq. 5.8.4.3.5-3.

$$V_n = \left(0.063\lambda\sqrt{f_c'}b_f d_f\right) + \frac{A_{hr} f_y}{s}\left(W + 2d_f\right) \qquad (5.8.4.3.5\text{-}3)$$

where:

b_f = the overall width of the inverted T-beam flange as illustrated in Figure 5.8.4.3.5-2
d_f = distance from top of ledge to the bottom longitudinal reinforcement as illustrated in Figure 5.8.4.3.5-2 (in.)
λ = concrete density modification factor as specified in Article 5.4.2.8

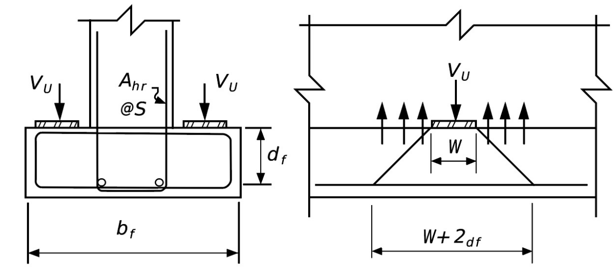

Figure 5.8.4.3.5-2—Inverted T-Beam Hanger Reinforcement

Inverted T-beams shall satisfy the torsional moment provisions as specified in Articles 5.7.3.6 and 5.7.2.1.

5.8.4.3.6—Design for Bearing

For the design for bearings supported by beam ledges, the provisions of Article 5.6.5 shall apply.

5.8.4.4—Local Zones

5.8.4.4.1—Dimensions of Local Zone

Where either:

- the manufacturer has not provided edge distance recommendations; or
- edge distances have been recommended by the manufacturer, but they have not been independently verified,

then the transverse dimensions of the local zone in each direction shall be taken as the greater of:

- the corresponding bearing plate size, plus twice the minimum concrete cover required for the particular application and environment; and
- the outer dimension of any required confining reinforcement, plus the required concrete cover over the confining reinforcement for the particular application and environment.

The cover required for corrosion protection shall be as specified in Article 5.10.1.

Where the manufacturer has recommendations for minimum cover, spacing, and edge distances for a particular anchorage device, and where these dimensions have been independently verified, the transverse dimensions of the local zone in each direction shall be taken as the lesser of:

- twice the edge distance specified by the anchorage device supplier; and
- the center-to-center spacing of anchorages specified by the anchorage device supplier.

C5.8.4.4.1

The provisions of this article are to ensure adequate concrete strength in the local zone. They are not intended to be guidelines for the design of the actual anchorage hardware.

The local zone is the highly stressed region immediately surrounding the anchorage device. It is convenient to define this region geometrically, rather than by stress levels. Figure C5.8.4.4.1-1 illustrates the local zone.

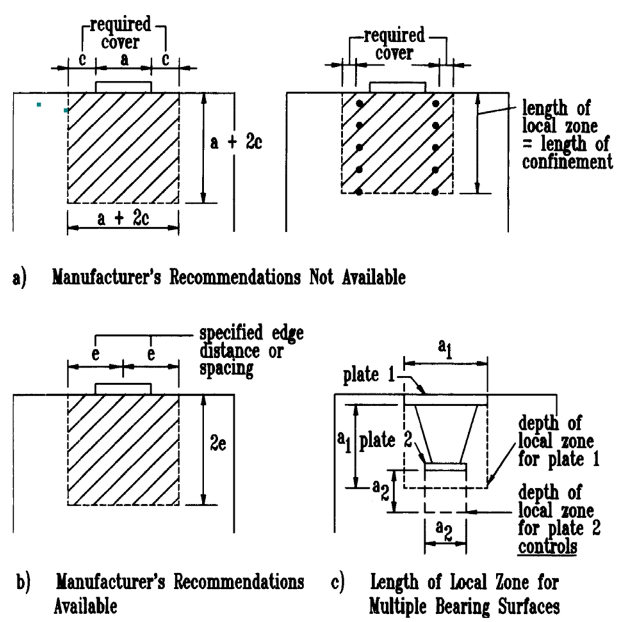

Figure C5.8.4.4.1-1—Geometry of the Local Zone

Recommendations for spacing and edge distance of anchorages provided by the manufacturer shall be taken as minimum values.

The length of the local zone along the tendon axis shall not be taken less than any of the following:

- the maximum width of the local zone;
- the length of the anchorage device confining reinforcement; and
- for anchorage devices with multiple bearing surfaces, the distance from the loaded concrete surface to the bottom of each bearing surface, plus the maximum dimension of that bearing surface.

The length of the local zone shall not be taken as greater than 1.5 times the width of the local zone.

For closely spaced anchorages, an enlarged local zone enclosing all individual anchorages should also be considered.

5.8.4.4.2—Bearing Resistance

Normal anchorage devices shall comply with the requirements specified herein. Special anchorage devices shall comply with the requirements specified in Article 5.8.4.4.3.

Where general zone reinforcement satisfying Article 5.9.5.6.5b is provided, and the extent of the concrete along the tendon axis ahead of the anchorage device is at least twice the length of the local zone as defined in Article 5.8.4.4.1, the factored bearing resistance of anchorages shall be taken as:

$$P_r = \phi f_n A_b \quad (5.8.4.4.2\text{-}1)$$

for which f_n is determined as the lesser of the following:

$$f_n = 0.7\, f'_{ci} \sqrt{\frac{A}{A_g}} \quad (5.8.4.4.2\text{-}2)$$

$$f_n = 2.25\, f'_{ci} \quad (5.8.4.4.2\text{-}3)$$

where:

ϕ = resistance factor specified in Article 5.5.4.2
f_n = nominal concrete bearing stress (ksi)
A_b = effective net area of the bearing plate calculated as the area A_g, minus the area of openings in the bearing plate (in.^{2})
f'_{ci} = design concrete compressive strength at time of application of tendon force (ksi)
A = maximum area of the portion of the supporting surface that is similar to the loaded area and concentric with it and does not overlap similar areas for adjacent anchorage devices (in.^{2})
A_g = gross area of the bearing plate calculated in accordance with the requirements herein (in.^{2})

The full bearing plate area may be used for A_g and the calculation of A_b if the plate material does not yield at the factored tendon force and the slenderness of the bearing plate, n/t, shall satisfy:

C5.8.4.4.2

References to construction specifications herein and in Article C5.8.4.4.3 refer to Articles 10.3.2.3 and 10.3.2.4 of the *AASHTO LRFD Bridge Construction Specifications* and other applicable Owner contract documents.

These specifications provide bearing pressure limits for anchorage devices, called normal anchorage devices, that are not to be tested in accordance with the acceptance test of the construction specifications. Alternatively, these limits may be exceeded if an anchorage system passes the acceptance test. Figures C5.8.4.4.2-1, C5.8.4.4.2-2, and C5.8.4.4.2-3 illustrate the specifications of Article 5.8.4.4.2 (Roberts, 1990).

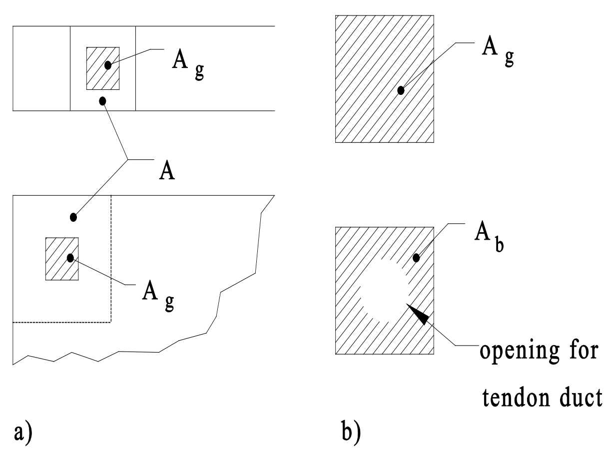

Figure C5.8.4.4.2-1—Area of Supporting Concrete Surface in Eq. 5.8.4.4.2-2

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$$n/t \leq 0.08\left(\frac{E_b}{f_b}\right)^{0.33} \quad (5.8.4.4.2\text{-}4)$$

where:

n = projection of base plate beyond the wedge hole or wedge plate, as appropriate (in.)
t = average thickness of the bearing plate (in.)
E_b = modulus of elasticity of the bearing plate material (ksi)
f_b = stress in anchor plate at a section taken at the edge of the wedge hole or holes (ksi)

For anchorages with separate wedge plates, n may be taken as the largest distance from the outer edge of the wedge plate to the outer edge of the bearing plate. For rectangular bearing plates, this distance shall be measured parallel to the edges of the bearing plate. If the anchorage has no separate wedge plate, n may be taken as the projection beyond the outer perimeter of the group of holes in the direction under consideration.

For bearing plates that do not meet the slenderness requirement specified herein, the effective gross bearing area, A_g, shall be taken as:

- **For anchorages with separate wedge plates**—the area geometrically similar to the wedge plate, with dimensions increased by twice the bearing plate thickness.
- **For anchorages without separate wedge plates**—the area geometrically similar to the outer perimeter of the wedge holes, with dimension increased by twice the bearing plate thickness.

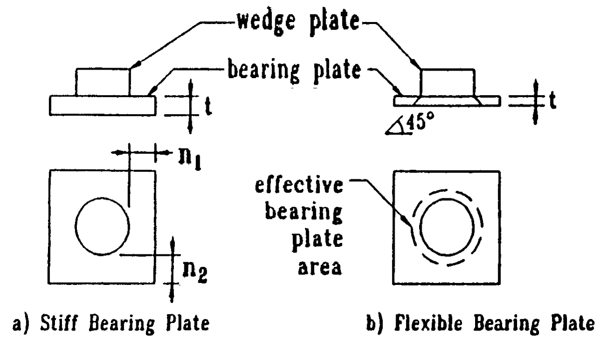

Figure C5.8.4.4.2-2—Effective Bearing Plate Area for Anchorage Devices with Separate Wedge Plate

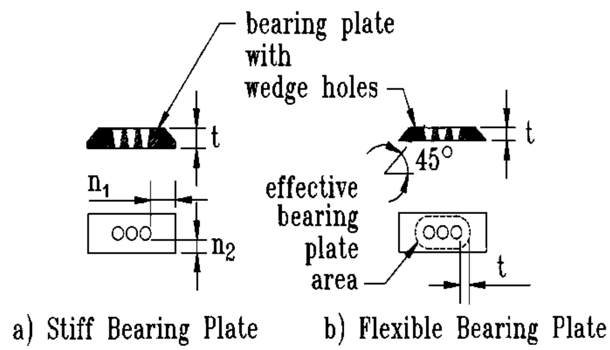

Figure C5.8.4.4.2-3—Effective Bearing Plate Area for Anchorage Device without Separate Wedge Plate

A larger effective bearing area may be calculated by assuming an effective area and checking the new f_b and n/t values.

5.8.4.4.3—Special Anchorage Devices

Special anchorage devices that do not satisfy the requirements specified in Article 5.8.4.4.2 may be used, provided that they have been tested by an independent testing agency acceptable to the Engineer and have met the acceptance criteria specified in Articles 10.3.2 and 10.3.2.3.10 of *AASHTO LRFD Bridge Construction Specifications*.

C5.8.4.4.3

Most anchorage devices fall in this category and still have to pass the acceptance test of the construction specifications. However, many of the anchorage systems currently available in the United States have passed equivalent acceptance tests. The results of these tests may be considered acceptable if the test procedure is generally similar to that specified in the construction specifications.

Local anchorage zone reinforcement supplied as part of a proprietary post-tensioning system shall be shown on post-tensioning shop drawings. Adjustment of general anchorage zone tensile reinforcement due to reinforcement supplied as part of a proprietary post-tensioning system may be considered as part of the shop drawing approval process. The responsibility for design of general anchorage zone reinforcement shall remain with the Engineer of Record.

For a series of similar special anchorage devices, tests may only be required for representative samples, unless tests for each capacity of the anchorages in the series are required by the Engineer of Record.

In addition to any required confining reinforcement provided in the acceptance test of special anchorage devices, supplementary skin reinforcement is permitted by the construction specifications. Equivalent reinforcement should also be placed in the actual structure. Other general zone reinforcement in the corresponding portion of the anchorage zone may be counted toward this reinforcement requirement.

5.8.4.5—General Zone of Post-Tensioning Anchorages

5.8.4.5.1—Limitations of Application

Concrete compressive stresses ahead of the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated using Eqs. 5.8.4.5.2-1 through 5.8.4.5.3-2, provided that:

- the member has a rectangular cross section and its longitudinal extent is not less than the larger transverse dimension of the cross section;
- the member has no discontinuities within or ahead of the anchorage zone;
- the minimum edge distance of the anchorage in the main plane of the member is not less than 1.5 times the corresponding lateral dimension, a, of the anchorage device;
- only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone; and
- the angle of inclination of the tendon, as specified in Eqs. 5.8.4.5.3-1 and 5.8.4.5.3-2, is between −5.0 degrees and +20.0 degrees.

C5.8.4.5.1

The equations specified herein are based on the analysis of members with rectangular cross sections and on an anchorage zone at least as long as the largest dimension of that cross section. For cross sections that deviate significantly from a rectangular shape, for example I-girders with wide flanges, the approximate equations should not be used.

Discontinuities, such as web openings, disturb the flow of forces and may cause higher compressive stresses, bursting forces, or edge tension forces in the anchorage zone. Figure C5.8.4.5.1-1 compares the bursting forces for a member with a continuous rectangular cross section and for a member with a noncontinuous rectangular cross section. The approximate equations may be applied to standard I-girders with end blocks if the longitudinal extension of the end block is at least one girder height and if the transition from the end block to the I-section is gradual.

Anchorage devices may be treated 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.

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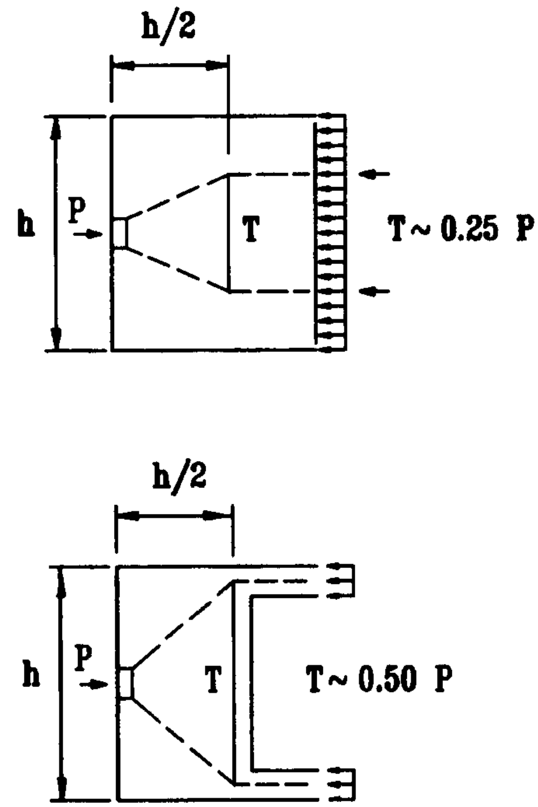

Figure C5.8.4.5.1-1—Effect of Discontinuity in Anchorage Zone

The approximate equations for concrete compressive stresses are based on the assumption that the anchor force spreads in all directions. The minimum edge distance requirement satisfies this assumption and is illustrated in Figure C5.8.4.5.1-2. The approximate equations for bursting forces are based on finite element analyses for a single anchor acting on a rectangular cross section. Eq. 5.8.4.5.3-1 gives conservative results for the bursting reinforcement, even if the anchors are not closely spaced, but the resultant of the bursting force is located closer to the anchor than indicated by Eq. 5.8.4.5.3-2.

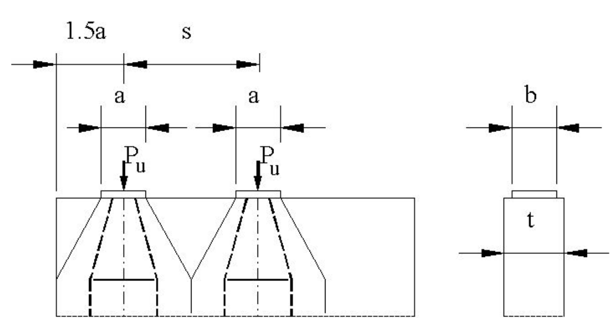

Figure C5.8.4.5.1-2—Edge Distances and Notation

5.8.4.5.2—Compressive Stresses

The concrete compressive stress ahead of the anchorage devices, f_{ca}, calculated using Eq. 5.8.4.5.2-1, shall not exceed the limit specified in Article 5.9.5.6.5a:

$$f_{ca} = \frac{0.6P_u\kappa}{A_b\left(1+\ell_c\left(\frac{1}{b_{eff}}-\frac{1}{t}\right)\right)} \qquad (5.8.4.5.2\text{-}1)$$

in which:

if $a_{eff} \le s < 2a_{eff}$, then:

$$\kappa = 1 + \left(2-\frac{s}{a_{eff}}\right)\left(0.3 + \frac{n}{15}\right) \qquad (5.8.4.5.2\text{-}2)$$

if $s \ge 2a_{eff}$, then:

$$\kappa = 1 \qquad (5.8.4.5.2\text{-}3)$$

where:

P_u = factored tendon force (kip)
κ = correction factor for closely spaced anchorages
A_b = effective bearing area (in.^{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.)
b_{eff} = lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross section (in.)
t = member thickness (in.)
a_{eff} = lateral dimension of the effective bearing area of the anchorage measured parallel to the larger dimension of the cross section (in.)
s = center-to-center spacing of anchorages (in.)
n = number of anchorages in a row

The effective bearing area, A_b, in Eq. 5.8.4.5.2-1 shall be taken as the larger of the anchor bearing plate area, A_{plate}, or the bearing area of the confined concrete in the local zone, A_{conf}, with the following limitations:

C5.8.4.5.2

This check of concrete compressive stresses is not required for basic anchorage devices satisfying Article 5.8.4.4.2.

Eqs. 5.8.4.5.2-1 and 5.8.4.5.2-2 are based on a strut-and-tie model for a single anchor with the concrete stresses determined as indicated in Figure C5.8.4.5.2-1 (Burdet, 1990), with the anchor plate width, b, and member thickness, t, being equal. Eq. 5.8.4.5.2-1 was modified to include cases with values of $b < t$.

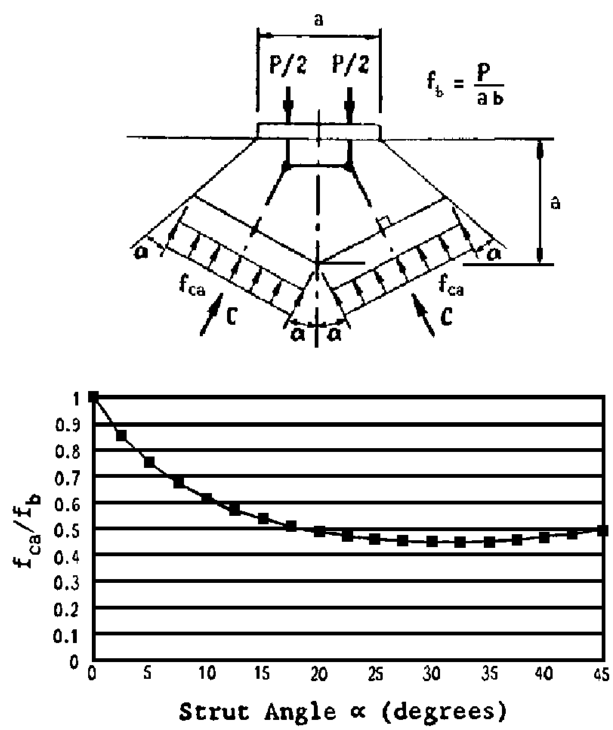

Figure C5.8.4.5.2-1—Local Zone and Strut Interface

For multiple anchorages spaced closer than $2a_{eff}$, a correction factor, κ, is necessary. This factor is based on an assumed stress distribution at a distance of one anchor plate width ahead of the anchorage device, as indicated in Figure C5.8.4.5.2-2.

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- If A_{plate} controls, A_{plate} shall not be taken larger than $(4/\pi)\ A_{conf}$.
- If A_{conf} controls, and either the maximum dimension of A_{conf} is more than either twice the maximum dimension of A_{plate} or three times the minimum dimension of A_{plate}, the effective bearing area, A_b, shall be based on A_{plate}.
- Deductions shall be made for the area of the duct in the determination of A_b.

If a group of anchorages is closely spaced in two directions, the product of the correction factors, κ, for each direction shall be used, as specified in Eq. 5.8.4.5.2-1.

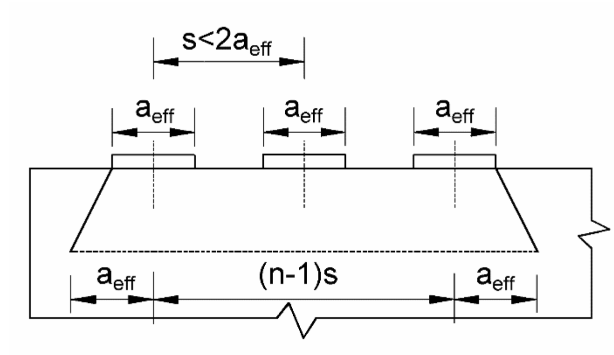

Figure C5.8.4.5.2-2—Closely Spaced Multiple Anchorages

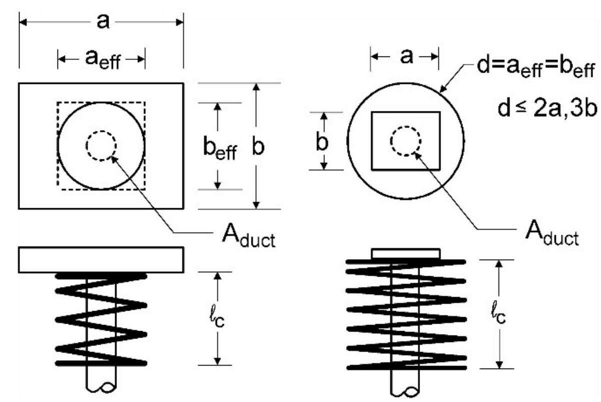

Figure C5.8.4.5.2-3—Effective Bearing Area

5.8.4.5.3—Bursting Forces

The bursting forces in anchorage zones, T_{burst}, may be taken as:

$$T_{burst} = 0.25 \sum P_u \left(1 - \frac{a}{h}\right) + 0.5\ \left|\Sigma\left(P_u \sin\alpha\right)\right| \qquad (5.8.4.5.3\text{-}1)$$

The location of the bursting force, d_{burst}, may be taken as:

$$d_{burst} = 0.5\,(h - 2e) + 5e\sin\alpha \qquad (5.8.4.5.3\text{-}2)$$

where:

T_{burst} = tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis (kip)
P_u = factored tendon force (kip)
a = lateral dimension of the anchorage device or group of devices in the direction considered (in.)
h = lateral dimension of the cross section in the direction considered (in.)

C5.8.4.5.3

Eqs. 5.8.4.5.3-1 and 5.8.4.5.3-2 are based on the results of linear elastic stress analyses (Burdet, 1990). Figure C5.8.4.5.3-1 illustrates the terms used in the equations.

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α = angle of inclination of a tendon force with respect to the centerline of the member; positive for concentric tendons or if the anchor force points toward the centroid of the section; negative if the anchor force points away from the centroid of the section (degrees)

d_{burst} = distance from anchorage device to the centroid of the bursting force, T_{burst} (in.)

e = eccentricity of the anchorage device or group of devices with respect to the centroid of the cross section; always taken as positive (in.)

The location and distribution of bursting reinforcement shall satisfy the requirements of Article 5.9.5.6.5b.

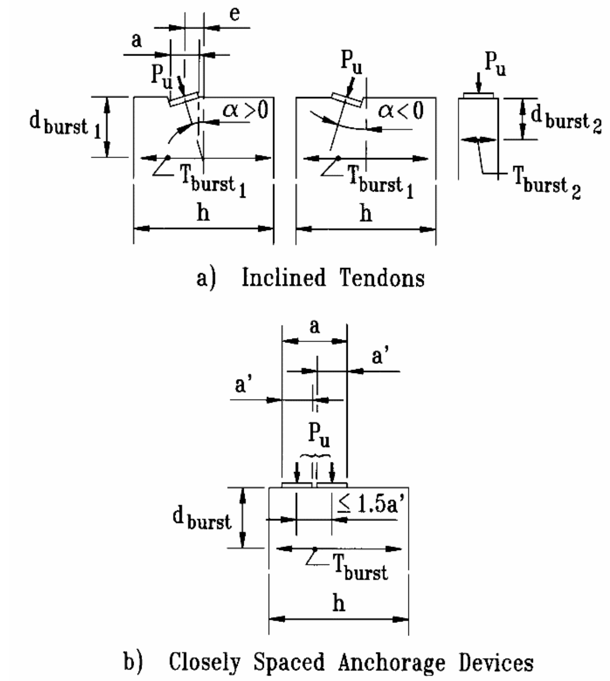

Figure C5.8.4.5.3-1—Notation for Eqs. 5.8.4.5.3-1 and 5.8.4.5.3-2

5.8.4.5.4—Edge Tension Forces

The longitudinal edge tension force may be determined from an analysis of a section located at one-half the depth of the section away from the loaded surface taken as a beam subjected to combined flexure and axial load. The spalling force may be taken as equal to the longitudinal edge tension force but not less than that specified in Article 5.9.5.6.5b.

C5.8.4.5.4

If the centroid of all tendons is located outside of the kern of the section, both spalling forces and longitudinal edge tension forces are induced. The determination of the edge tension forces for eccentric anchorages is illustrated in Figure C5.8.4.5.4-1. Either type of axial-flexural beam analysis is acceptable. As in the case for multiple anchorages, this reinforcement is essential for equilibrium of the anchorage zone. It is important to consider stressing sequences that may cause temporary eccentric loadings of the anchorage zone.

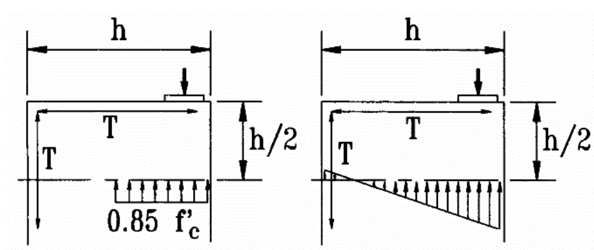

Figure C5.8.4.5.4-1—Determination of Edge Tension Forces for Eccentric Anchorages

5.8.4.5.5—Multiple Slab Anchorages

Unless a more detailed analysis is made, the minimum reinforcement specified herein to resist bursting force and edge tension force shall be provided.

Reinforcement shall be provided to resist the bursting force. This reinforcement shall be anchored close to the faces of the slab with standard hooks bent

C5.8.4.5.5

Reinforcement to resist bursting force is provided in the direction of the thickness of the slab and normal to the tendon axis in accordance with Article 5.9.5.6.5b.

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around horizontal bars or equivalent. Minimum reinforcement should be two No. 3 bars per anchor located at a distance equal to one-half the slab thickness ahead of the anchor.

Reinforcement shall be provided to resist edge tension forces, T_1, between anchorages and bursting forces, T_2, ahead of the anchorages. Edge tension reinforcement shall be placed immediately ahead of the anchors and shall effectively tie adjacent anchors together. Bursting reinforcement shall be distributed over the length of the anchorage zones.

Reinforcement to resist edge tension force is placed in the plane of the slab and normal to the tendon axis.

$$T_1 = 0.10\, P_u \left(1 - \frac{a}{s}\right) \qquad (5.8.4.5.5\text{-}1)$$

$$T_2 = 0.20\, P_u \left(1 - \frac{a}{s}\right) \qquad (5.8.4.5.5\text{-}2)$$

where:

T_1 = edge tension force (kip)
P_u = factored tendon force on an individual anchor (kip)
a = lateral dimension of the anchorage device or group of devices in the transverse direction of the slab (in.)
s = the anchorage spacing (in.)
T_2 = bursting force (kip)

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

The use of hairpins provides better confinement to the edge region than the use of straight bars.

5.9—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 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 and 5.7.

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 5.5.

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Compressive stress limits, specified in Article 5.9.2.3, 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.

Tensile stress limits, specified in Article 5.9.2.3, 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.

5.9.1.2—Design Concrete Strengths

The design strengths, f'_c and f'_{ci}, shall be identified in the contract documents for each component. Stress limits relating to design strengths shall be as specified in Article 5.9.2.3.

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—Section Properties

For section properties prior to bonding of post-tensioning tendons, where the open ducts reduce the area of the either flange or either or both web(s) by more than 5 percent, the stresses shall be checked using section properties that account for the presence of the ducts. For both pretensioned or post-tensioned members after bonding of tendons, section properties may be based on either the gross or transformed section.

C5.9.1.3

The 5 percent allowance reflects current common practice. Bonding means that the grout in the duct has attained its specified strength.

5.9.1.4—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 and 5.6.

5.9.1.5—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.

5.9.1.6—Tendons with Angle Points or Curves

The provisions of Article 5.4.6 for the curvature of ducts shall apply.

The provisions of Article 5.9.5.4 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.

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The provisions of Article 5.7.1.5 for the webs of curved post-tensioned box girder bridges shall apply.

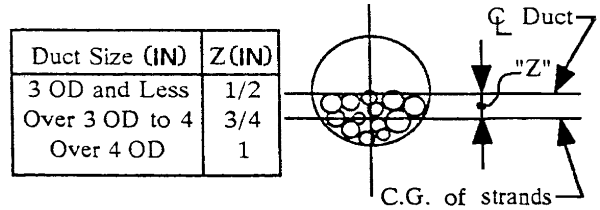

Figure C5.9.1.6-1—Location of Tendon in Duct

5.9.2—Stress Limitations

5.9.2.1—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

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right); \text{ or} \quad (5.9.2.1\text{-}1)$$

- For slowly imposed deformations

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right)/\psi\left(t,t_i\right) \quad (5.9.2.1\text{-}2)$$

where:

F' = reduced force effect (kip)
F = force effect determined using the modulus of elasticity of the concrete at the time loading is applied (kip)
e = base of natural logarithms
$\Psi(t, t_i)$ = creep coefficient at time t for loading applied at time t_i as specified in Article 5.4.2.3.2

C5.9.2.1

Additional information is contained in Leonhardt (1964).

5.9.2.2—Stress Limitations for Prestressing Steel

The steel stress due to prestress or at the service limit state shall not exceed the lesser of the following two values:

- Specified in Table 5.9.2.2-1

C5.9.2.2

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 5.9.2.2-1 are not exceeded.

- Recommended by the manufacturer of the tendons or anchorages

The steel stress at the strength and extreme event limit states shall not exceed the tensile strength limit specified in Table 5.4.4.1-1.

Table 5.9.2.2-1—Stress Limits for Prestressing Steel

Condition	Tendon Type: 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})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

5.9.2.3—Stress Limits for Concrete

5.9.2.3.1—For Temporary Stresses before Losses

5.9.2.3.1a—Compressive Stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be $0.65f'_{ci}$ (ksi).

C5.9.2.3.1a

Previous research (Hale and Russell, 2006) suggests that the concrete stress limit for prestressed concrete components can safely exceed $0.60f'_{ci}$. However, concrete in the precompressed tensile zone subjected to compressive stresses at release greater than $0.65f'_{ci}$ can experience microcracking, leading to unconservative predictions of the external load to cause cracking (Birrcher and Bayrak, 2007; Heckmann and Bayrak, 2008; Schnittker and Bayrak, 2008; and Birrcher et al., 2010).

5.9.2.3.1b—Tensile Stresses

The limits in Table 5.9.2.3.1b-1 shall apply for tensile stresses.

C5.9.2.3.1b

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.

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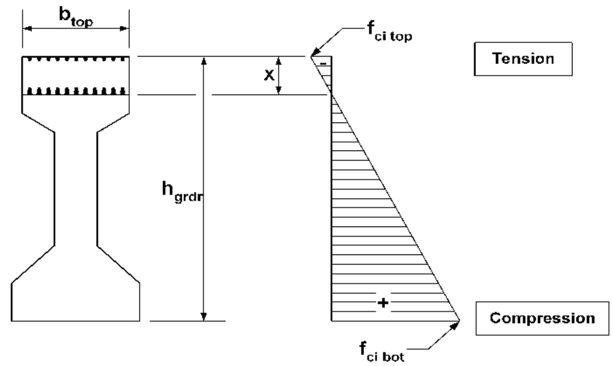

$$T = \frac{f_{ci\ top}}{2} b_{top} x$$

$$A_s = \frac{T}{f_s}$$

where $f_s = 0.5f_y \leq 30.0$ ksi

Figure C5.9.2.3.1b-1—Calculation of Tensile Force and Required Area of Reinforcement

Table 5.9.2.3.1b-1—Temporary Tensile Stress Limits in Prestressed Concrete before Losses

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	$0.0948\lambda\sqrt{f'_{ci}} \leq 0.2$ (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 30.0 ksi.	$0.24\lambda\sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	$0.158\lambda\sqrt{f'_{ci}}$ (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	$0.0948\lambda\sqrt{f'_{ci}}$ (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through the joints	No tension
	Transverse Stresses	
	• For any type of joint	$0.0948\lambda\sqrt{f'_{ci}}$ (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 30.0 ksi.	$0.19\lambda\sqrt{f'_{ci}}$ (ksi)

5.9.2.3.2—For Stresses at Service Limit State after Losses

5.9.2.3.2a—Compressive Stresses

Compression shall be investigated using the Service Limit State Load Combination I specified in Table 3.4.1-1. The limits in Table 5.9.2.3.2a-1 shall apply. These limits may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

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 5.6.4.7.1, 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 5.6.4.7.2.

C5.9.2.3.2a

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 5.6.4.7.2 to flanged, strutted, and variable thickness elements requires some judgment. Consideration of appropriate lengths of wall-type element is illustrated in Figure C5.9.2.3.2a-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 C5.9.2.3.2a-1.

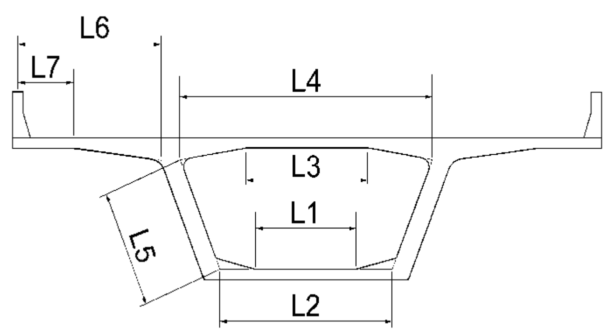

Figure C5.9.2.3.2a-1—Suggested Choices for Wall Lengths to be Considered

Table 5.9.2.3.2a-1—Compressive Stress Limits in Prestressed Concrete at Service Limit State after Losses

Location	Stress Limit
• 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)

5.9.2.3.2b—Tensile Stresses

For longitudinal 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. Load combination Service I should be investigated for load combinations that involve traffic loadings in transverse analyses of box girder bridges.

The limits in Table 5.9.2.3.2b-1 shall apply.

C5.9.2.3.2b

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

See Figure C5.9.2.3.1b-1 for calculation of required area of bonded reinforcement.

Table 5.9.2.3.2b-1—Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	Tension in the Precompressed Tensile Zone, Assuming Uncracked Sections • For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions • For components with unbonded prestressing tendons	 $0.19\lambda\sqrt{f'_c} \leq 0.6$ (ksi) $0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi) No tension
Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	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 $0.5f_y$; internal tendons or external tendons • Joints without the minimum bonded auxiliary reinforcement through joints	 $0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi) No tension
	Transverse Stresses • Tension in the transverse direction in precompressed tensile zone	 $0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi)
	Stresses in Other Areas • For areas without bonded reinforcement • 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 $0.5f_y$, not to exceed 30.0 ksi	 No tension $0.19\lambda\sqrt{f'_c}$ (ksi)

5.9.2.3.3—Principal Tensile Stresses in Webs

The provisions specified herein shall apply to all types of post-tensioned superstructures with internal and/or external tendons. The provisions specified herein shall also apply to pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. As maximum principal tensions may not occur at the neutral axis, various locations along the height of the web should be checked.

At sections where internal tendons cross near the depth at which the maximum principal tension is being checked, the provisions of Article 5.7.2.1 shall apply.

The principal tensile stresses in webs shall not exceed $0.110\lambda\sqrt{f'_c}$ when the superstructure element is subjected to the loadings of Service III limit state of Article 3.4.1, both before and after all losses and redistribution of forces.

The principal tensile stresses shall be determined using the combination of axial and shear stress which

C5.9.2.3.3

The principal stress check is introduced to limit web cracking at the service limit state for all types of post-tensioned superstructures with internal and/or external tendons and pretensioned girders with a compressive strength of concrete for use in design greater than f'_c = 10.0 ksi. Experience has shown that the cracking in the webs of conventional pretensioned girders with a compressive strength of concrete for use in design up to 10.0 ksi has not been a problem and the check may be omitted.

Since this investigation is made at the service limit state Eqs. 5.9.2.3.3-2 and 5.9.2.3.3-3 for shear stress are linear combinations of the shear stresses from shear and torsion. No relief from redistribution of stresses is assumed as might be appropriate from the strength or extreme event limit states. Likewise, no adjustment for the area or perimeter of hollow sections resulting from spalling of cover has been assumed.

produces the greatest principal tensile stress. Vertical compressive or tensile stresses, if present, shall also be considered. Live loads for shear stress computations for I- or bulb-tee sections may be derived using the distribution factors determined from Article 4.6.2.2. Torsion need not be considered for typical I- or bulb-tee sections. For solid rectangular girders, box girders, and other hollow sections, shear stresses shall be determined using vertical shear and concurrent torsion.

Where consideration of torsion may be neglected by the provisions of Article 5.7.2 the torsional term in Eqs. 5.9.2.3.3-2 and 5.9.2.3.3-3 may be neglected.

Shear stress may be determined as:

- For open sections that may be considered thin walled such as typical I-girders and bulb-tee girders:

$$\tau = \frac{VQ_g}{I_g b_w} \tag{5.9.2.3.3-1}$$

- For stocky solid sections:

$$\tau = \frac{VQ_g}{I_g b_w} + \frac{Tp_c}{{A_{cp}}^2} \tag{5.9.2.3.3-2}$$

- For single cell and symmetrical two-cell hollow sections:

$$\tau = \frac{VQ_g}{I_g B_w} + \frac{T}{2A_o b_w} \tag{5.9.2.3.3-3}$$

where:

τ = shear stress in web vertical shear (ksi)
V = shear force for Service III load combination (kip)
Q_g = the first moment about the neutral axis of gross concrete area above or below the height of the web where the principal tension is being checked (in.^{3})
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})
b_w = web width at height of the web where principal tension is being checked (in.)
T = concurrent torsional moment for Service III load combination (kip-in.)
p_c = length of outside perimeter of the concrete section (in.)
A_{cp} = area enclosed by the outside perimeter of the concrete cross section (in.^{2})
B_w = total web width in single cell or symmetrical two-cell hollow sections at height of the web where principal tension is being checked (in.)
A_o = area enclosed by shear flow path, including any area of holes therein (in.^{2})

For precast sections made composite with a cast-in-place deck, the principal tensile stress at both the noncomposite and composite neutral axes should be checked using the shear stress and axial stress at each location.

For substructure elements, such as pier caps and straddle bents where beam theory applies, a principal tensile stress check may be made at the discretion of the designer.

Shear flow in hollow sections with more than three webs may be determined by analytic methods or finite element analysis.

For hollow sections the shear flow due to torsion is added to the shear flow from shear forces in the controlling web. The cross-hatched area enclosed by the shear flow path is defined as shown in Figure C.5.9.2.3.3-1. The individual web widths for box girders, b_w, are measured perpendicular to the web axis.

For symmetrical two-cell box girder superstructures, the area enclosed by the outer perimeter of the box girder as shown in Figure C5.9.2.3.3-1 may be used for the computation of A_o. For unsymmetrical two-cell box girders, or box girders with more than three webs, consideration of the interaction of the shear flow in internal webs should be included in the computing of shear stresses. For a treatment of torsion in concrete multi-cell box girder superstructures see Chapter 7 and Appendix B of FHWA report *Post-Tensioned Box Girder Manual* (Corven, 2016).

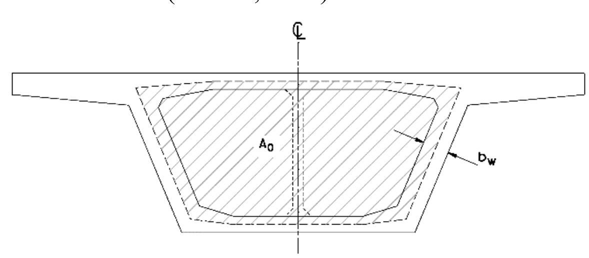

Figure C5.9.2.3.3-1—Area Enclosed by Shear Flow Path and Web Width for Single Cell Box and Symmetrical Two-Cell Girder Superstructure

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Principal tensile stresses may be determined based on the analysis using Mohr's Circle, where:

$$f_{\min} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) - \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right) \tag{5.9.2.3.3-4}$$

where:

f_{min} = minimum principal stress in the web, tension negative (ksi)
f_{pcx} = horizontal stress in the web (ksi)
f_{pcy} = vertical stress in the web (ksi)

Values of f_{max} and f_{min} in Eqs. 5.9.2.3.3-4 and C5.9.2.3.3-1 are depicted in Figure C5.9.2.3.3-2.

$$f_{\max} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) + \sqrt{\left(f_{pcx} - f_{pcy}\right)^2 + \left(2\tau\right)^2}\right) \tag{C5.9.2.3.3-1}$$

where:

f_{max} = maximum principal stress in the web, compression positive (ksi)

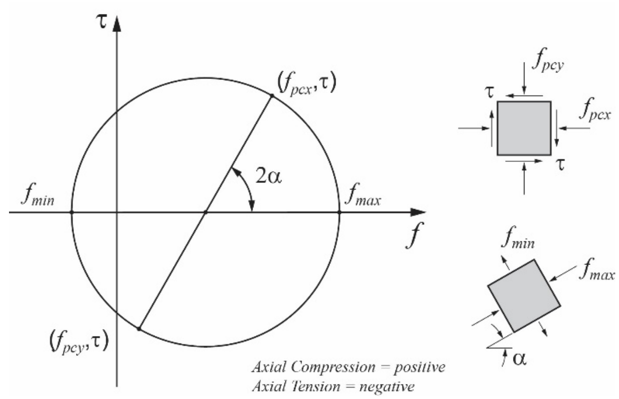

Figure C5.9.2.3.3-2—Mohr's Circle Representation of Principal Stresses

The vertical normal stress in webs is typically equal to zero. Vertical compression shown in Figure C5.9.2.3.3-2 represents the general, but less frequent, case of biaxial stress in the webs. An example of biaxial compression would be a post-tensioned concrete box girder superstructure with vertical prestressing in the webs.

5.9.3—Prestress Losses

5.9.3.1—Total Prestress Loss

Values of prestress losses specified herein shall be applicable to normal weight concrete only and may be used for concrete compressive strengths for use in design 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:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \tag{5.9.3.1-1}$$

- In post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \tag{5.9.3.1-2}$$

C5.9.3.1

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.

The loss across stressing hardware and anchorage devices has been measured from 2 to 6 percent (Roberts,

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where:

Δf_{pT} = total loss (ksi)
Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)
Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)
Δf_{pF} = loss due to friction (ksi)
Δf_{pA} = loss due to anchorage set (ksi)

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.

5.9.3.2—Instantaneous Losses

5.9.3.2.1—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.

C5.9.3.2.1

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 5.9.3.2.3 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.

5.9.3.2.2—Friction

5.9.3.2.2a—Pretensioned Members

For draped prestressing tendons, losses that may occur at the hold-down devices should be considered.

5.9.3.2.2b—Post-Tensioned Members

Losses due to friction between the internal prestressing tendons and the duct wall may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right) \qquad (5.9.3.2.2b\text{-}1)$$

C5.9.3.2.2b

Where large discrepancies occur between measured and calculated tendon elongations, in-place friction tests are required.

Losses due to friction between the external tendon across a single deviator pipe may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha+0.04)}\right) \quad (5.9.3.2.2b\text{-}2)$$

where:

f_{pj} = stress in the prestressing steel at jacking (ksi)
e = base of natural logarithms
K = wobble friction coefficient (per ft of tendon)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft)
μ = friction factor
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.)

The 0.04 radians in Eq. 5.9.3.2.2b-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.

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 5.9.3.2.2b-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 C5.12.5.3.7 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 of the prestressing steel path from jacking end to a point under investigation, α_v, and the total horizontal angular change, α_h, of the prestressing steel path from jacking end to a point under investigation.

When the developed elevation and plan of the tendons are parabolic or circular, the α can be computed from:

$$\alpha = \sqrt{\alpha_v^2 + \alpha_h^2} \quad (C5.9.3.2.2b\text{-}1)$$

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:

$$\alpha = \Sigma\Delta\alpha = \Sigma\sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2} \quad (C5.9.3.2.2b\text{-}2)$$

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 factor ($\mu = 0.25$) 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 ±0.375 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 5.9.3.2.2b-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

5.9.3.2.3—Elastic Shortening

5.9.3.2.3a—Pretensioned Members

The loss due to elastic shortening in pretensioned members shall be taken as:

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \qquad (5.9.3.2.3a\text{-}1)$$

where:

E_p = modulus of elasticity of prestressing steel (ksi)
E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)
f_{cgp} = 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).

C5.9.3.2.3a

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. 5.9.3.2.3a-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. C5.9.3.2.3a-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.

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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:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}} \tag{C5.9.3.2.3a-1}$$

where:

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

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

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

e_m = average prestressing steel eccentricity at midspan (in.)

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

M_g = midspan moment due to member self-weight (kip-in.)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing steel (ksi)

5.9.3.2.3b—Post-Tensioned Members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as:

$$\Delta f_{pES} = \frac{N-1}{2N} \frac{E_p}{E_{ci}} f_{cgp} \tag{5.9.3.2.3b-1}$$

C5.9.3.2.3b

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be determined by the following alternative equation:

$$\Delta f_{pES} = \frac{N-1}{2N} \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}} \tag{C5.9.3.2.3b-1}$$

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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. 5.9.3.2.3b-1.

where:

N = number of identical prestressing tendons
A_{ps} = area of prestressing steel (in.^{2})
f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in Table 5.9.2.2-1 (ksi)
I_g = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (in.^{4})
e_m = average prestressing steel eccentricity at midspan (in.)
A_g = gross area of section (in.^{2})
M_g = midspan moment due to member self-weight (kip-in.)
E_{ci} = modulus of elasticity of concrete at transfer (ksi)
E_p = modulus of elasticity of prestressing steel (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. C5.9.3.2.3b-1.

For post-tensioned construction, Δf_{pES} losses can be further reduced below those implied by Eq. 5.9.3.2.3b-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 determined as:

$$N = N_1 + N_2 \frac{A_{sp2}}{A_{sp1}} \tag{C5.9.3.2.3b-2}$$

where:

N_1 = number of tendons in the larger group
N_2 = number of tendons in the smaller group
A_{sp2} = cross-sectional area of a tendon in the smaller group (in.^{2})
A_{sp1} = cross-sectional area of a tendon in the larger group (in.^{2})

5.9.3.2.3c—Combined Pretensioning and Post-Tensioning

In applying the provisions of Articles 5.9.3.2.3a and 5.9.3.2.3b 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.

C5.9.3.2.3c

See Castrodale and White (2004) for information on computing the effect of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons.

5.9.3.3—Approximate Estimate of Time-Dependent Losses

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

- members are made from normal weight concrete;
- the concrete is either steam- or moist-cured;
- prestressing is by bars or strands with 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:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad (5.9.3.3\text{-}1)$$

in which:

$$\gamma_h = 1.7 - 0.01H \qquad (5.9.3.3\text{-}2)$$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} \qquad (5.9.3.3\text{-}3)$$

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

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 and in accordance with manufacturers recommendation for other types of strand (ksi)

H = average annual ambient relative humidity (percent)

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 5.9.3.4.

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 5.9.3.4 or computer time-step methods shall be used.

C5.9.3.3

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. 5.9.3.3-1 are intended for sections with composite decks only. The losses in Eq. 5.9.3.3-1 were derived as approximations of the terms in the refined method for a wide range of standard precast prestressed concrete I-beams and inverted tee beams. 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. 5.9.3.3-1 corresponds to creep losses, the second term to shrinkage losses, and the third to relaxation losses.

The commentary to Article 5.9.3.4.2 also gives an alternative relaxation loss prediction method.

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5.9.3.4—Refined Estimates of Time-Dependent Losses

5.9.3.4.1—General

For nonsegmental prestressed members, more accurate values of creep-, shrinkage-, and relaxation-related losses than those specified in Article 5.9.3.3 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 5.9.3.4.4 and 5.9.3.4.5, respectively, shall be considered before applying the provisions of this article.

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 5.9.3, 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:

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df} \quad (5.9.3.4.1\text{-}1)$$

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_{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_{pR2} = prestress loss due to relaxation of prestressing strands in composite section 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)

C5.9.3.4.1

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 5.9.3.3. 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. 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.

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$(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$
= sum of time-dependent prestress losses after deck placement (ksi)

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.3, including consideration of the time-dependent construction method and schedule shown in the contract documents.

5.9.3.4.2—Losses: Time of Transfer to Time of Deck Placement

5.9.3.4.2a—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:

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} \quad (5.9.3.4.2a\text{-}1)$$

in which:

$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_g} \left(1 + \frac{A_g e_{pg}^2}{I_g}\right) \left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \quad (5.9.3.4.2a\text{-}2)$$

where:

ε_{bid} = concrete shrinkage strain of girder between the time of transfer and deck placement per Eq. 5.4.2.3.3-1 (in./in.)
E_p = modulus of elasticity of prestressing steel (ksi)
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 (day)
t_i = age of concrete at time of transfer (day)

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5.9.3.4.2b—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:

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \Psi_b \left(t_d, t_i\right) K_{id} \qquad (5.9.3.4.2b\text{-}1)$$

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 (day)

5.9.3.4.2c—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:

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right) \qquad (5.9.3.4.2c\text{-}1)$$

where:

f_{pt} = stress in prestressing strands immediately after transfer, taken not less than $0.55f_{py}$ in Eq. 5.9.3.4.2c-1

K_L = factor accounting for type of steel taken as 30 for low relaxation strands and 7.0 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.

C5.9.3.4.2c

Eqs. 5.9.3.4.2c-1 and 5.9.3.4.3c-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):

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K'_L}\frac{\log(t)}{\log(t_i)}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right] K_{id} \qquad (C5.9.3.4.2c\text{-}1)$$

where:

t = time between strand tensioning and deck placement (day)

K'_L = factor accounting for type of steel, equal to 45 for low relaxation steel

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 5.9.3.4.2c 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

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5.9.3.4.3—Losses: Time of Deck Placement to Final Time

5.9.3.4.3a—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:

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df} \qquad (5.9.3.4.3a\text{-}1)$$

in which:

$$K_{df} = \frac{1}{1 + \dfrac{E_p}{E_{ci}} \dfrac{A_{ps}}{A_c} \left(1 + \dfrac{A_c e_{pc}^2}{I_c}\right)\left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \qquad (5.9.3.4.3a\text{-}2)$$

where:

ε_{bdf} = shrinkage strain of girder between time of deck placement and final time per Eq. 5.4.2.3.3-1 (in./in.)
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
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})
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
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})

5.9.3.4.3b—Creep of Girder Concrete

The change in 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:

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left[\psi_b\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b\left(t_f, t_d\right)K_{df} \qquad (5.9.3.4.3b\text{-}1)$$

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

5.9.3.4.3c—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:

$$\Delta f_{pR2} = \Delta f_{pR1} \qquad (5.9.3.4.3c\text{-}1)$$

C5.9.5.4.3c

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.

5.9.3.4.3d—Shrinkage of Deck Concrete

The prestress gain due to shrinkage of deck in composite section, Δf_{pSS}, shall be determined as:

$$\Delta f_{pSS} = \frac{E_p}{E_c} \Delta f_{cdf} K_{df} \left[1 + 0.7\psi_b\left(t_f, t_d\right)\right] \qquad (5.9.3.4.3d\text{-}1)$$

in which:

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{c\ deck}}{\left[1 + 0.7\psi_d\left(t_f, t_d\right)\right]} \left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right) \qquad (5.9.3.4.3d\text{-}2)$$

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 (in./in.)

A_d = area of deck concrete (in.^{2})

$E_{c\ deck}$ = modulus of elasticity of deck concrete (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

$\Psi_d(t_f, t_d)$ = 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

e_d = eccentricity of deck with respect to the gross composite section, positive in typical construction where deck is above girder (in.)

C5.9.3.4.3d

For the typical condition where the center of gravity of the prestressing force is below the NA, deck shrinkage increases the prestressing force and is considered a “gain”, hence the negative value of Δf_{pSS} calculated from Eq. 5.9.3.4.3d-1 is substituted into Eq. 5.9.3.4.1-1 as a positive value.

5.9.3.4.4—Precast Pretensioned Girders without Composite Topping

The equations in Article 5.9.3.4.2 and Article 5.9.3.4.3 are applicable to girders with noncomposite deck or topping, or with no topping. The values for time of deck placement in Article 5.9.3.4.2 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 5.9.3.4.3 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.

5.9.3.4.5—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 5.9.3.4.1 through 5.9.3.4.4. In Eq. 5.9.3.4.1-1, the value of the term $(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$ shall be taken as zero.

5.9.3.5—Losses in Multi-Stage Prestressing

For staged construction or 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 evaluating losses incrementally for each stage of construction.

C5.9.3.5

See references cited in Article C5.4.2.3.2.

Staged construction refers to any bridge whose structural statical scheme undergoes changes before reaching construction completion. An example would be segmental balanced cantilever construction where adjacent cantilevers are joined to make a new, continuous structure. Staged construction can also refer to changes in the cross section of superstructure members through composite action, where prestressing is stressed both on the noncomposite and then composite cross sections. An example would be spliced girder construction, with prestressing stressed in a first stage to make the girders continuous, and then in a second stage after the casting of the deck slab.

5.9.3.6—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.

5.9.4—Details for Pretensioning

5.9.4.1—Minimum Spacing of Pretensioning Strand

The distance between pretensioning strands, including debonded ones, at each end of a member within the transfer length, as specified in Article 5.9.4.3.1, 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 5.9.4.1-1.

Table 5.9.4.1-1—Center-to-Center Spacings

Strand Size (in.)	Spacing (in.)
0.6 0.5625 Special 0.5625	2.00
0.5000 0.4375 0.50 Special	1.75
0.3750	1.50

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 the greater of the following:

- 1.33 times the maximum size of the aggregate
- 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.

C5.9.4.1

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 Owners limit the clear distance between pretensioning strands to not less than twice the nominal size of aggregate to facilitate placing and compaction of concrete.

5.9.4.2—Maximum Spacing of Pretensioning Strand in Slabs

Pretensioning strands for precast slabs shall be spaced symmetrically and uniformly and shall not be farther apart than the lesser of the following:

- 1.5 times the total composite slab thickness; or
- 18.0 in.

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5.9.4.3—Development of Pretensioning Strand

5.9.4.3.1—General

In determining the resistance of pretensioned concrete components in their end zones, the gradual buildup of the strand force in the transfer and development lengths shall be taken into account.

The stress in the prestressing steel may be assumed to vary linearly from 0.0 at the point where bonding commences to the effective stress after losses, f_{pe}, at the end of the transfer length.

Between the end of the transfer length and the development length, the strand stress may be assumed to increase linearly, reaching the stress at nominal resistance, f_{ps}, at the development length.

For the purpose of this article, the transfer length may be taken as 60 strand diameters and the development length shall be taken as specified in Article 5.9.4.3.2.

The effects of debonding shall be considered as specified in Article 5.9.4.3.3.

The provisions of Article 5.9.4.3 may be used for normal weight concrete with design concrete compressive strengths up to 10.0 ksi at transfer (f'_{ci}) and up to 15.0 ksi for design (f'_c).

C5.9.4.3.1

Between the end of the transfer length and development length, the strand stress grows from the effective stress in the prestressing steel after losses to the stress in the strand at nominal resistance of the member.

The extension of the transfer and development length provisions to normal weight concrete with design concrete compressive strengths up to 10.0 and 15.0 ksi for f'_{ci} and f'_c, respectively, is based on the work presented in NCHRP Report 603 (Ramirez and Russell, 2008).

5.9.4.3.2—Bonded Strand

Pretensioning strand shall be bonded beyond the section required to develop f_{ps} for a development length, ℓ_d, in in., where ℓ_d shall satisfy:

$$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \qquad (5.9.4.3.2\text{-}1)$$

where:

κ = 1.0 for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24.0 in.

κ = 1.6 for pretensioned members with a depth greater than 24.0 in.

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (ksi)

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

d_b = nominal strand diameter (in.)

The variation of design stress in the pretensioned strand from the free end of the strand may be calculated as follows:

C5.9.4.3.2

An October, 1988 FHWA memorandum mandated a 1.6 multiplier on Eq. 5.9.4.3.2-1 in the specifications. The corrected equation is conservative in nature, but accurately reflects the worst-case characteristics of strands shipped prior to 1997. To eliminate the need for this multiplier, Eq. 5.9.4.3.2-1 has been modified by the addition of the κ factor.

The correlation between steel stress and the distance over which the strand is bonded to the concrete can be idealized by the relationship shown in Figure C5.9.4.3.2-1. This idealized variation of strand stress may be used for analyzing sections within the transfer and development length at the end of pretensioned members.

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- From the point where bonding commences to the end of transfer length:

$$f_{px} = \frac{f_{pe}\ell_{px}}{60d_b} \tag{5.9.4.3.2-2}$$

- From the end of the transfer length and to the end of the development of the strand:

$$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{(\ell_d - 60d_b)}(f_{ps} - f_{pe}) \tag{5.9.4.3.2-3}$$

where:

f_{px} = design stress in pretensioned strand at nominal flexural strength at section of member under consideration (ksi)

ℓ_{px} = distance from free end of pretensioned strand to section of member under consideration (in.)

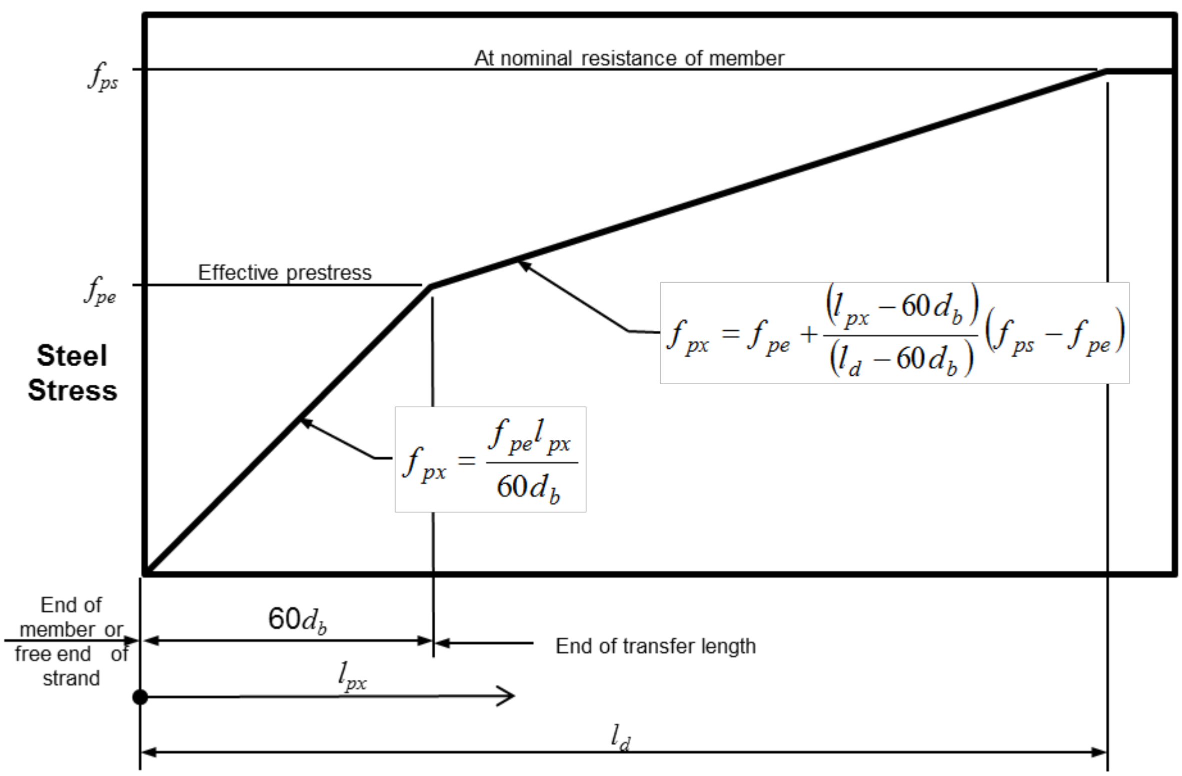

Figure C5.9.4.3.2-1—Idealized Relationship between Steel Stress and Distance from the Free End of Strand

5.9.4.3.3—Debonded Strands

Straight pretensioned strands may be debonded at the ends of beams subject to the following restrictions:

A. The number of strands debonded per row shall not exceed 45 percent of the strands provided in that row, unless otherwise approved by the Owner.

C5.9.4.3.3

Tests completed by the Florida Department of Transportation (Shahawy, Robinson, and Batchelor, 1993) (Shahawy and Batchelor, 1991) and others (Hamilton et al., 2013) (Langefeld and Bayrak, 2012) (Shahrooz et al., 2017) indicate that the anchored strength of the strands is one of the primary contributors to the shear resistance of prestressed concrete beams in

B. Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section.
C. Longitudinal spacing of debonding termination locations shall be at least $60d_b$ apart.
D. Debonded strands shall be symmetrically distributed about the vertical centerline of the cross section of the member. Debonding shall be terminated symmetrically at the same longitudinal location.
E. Alternate bonded and debonded strand locations both horizontally and vertically.
F. Where a portion or portions of a pretensioning strand are debonded and where service tension exists in the precompressed tensile zone, the development lengths, measured from the end of the debonded zone, shall be determined using Eq. 5.9.4.3.2-1 with a value of $\kappa = 2.0$.
G. For simple span precast, pretensioned girders, debonding length from the beam end should be limited to 20 percent of the span length or one half the span length minus the development length, whichever is less.
H. For simple span precast girders made continuous using positive moment connections, the interaction between debonding and restraint moments from time-dependent effects (such as creep, shrinkage and temperature variations) shall be considered. For additional guidance refer to Article 5.12.3.3.
I. For single-web flanged sections (I-beams, bulb-tees, and inverted-tees):
   - Bond all strands within the horizontal limits of the web when the total number of debonded strands exceeds 25 percent.
   - Bond all strands within the horizontal limits of the web when the bottom flange to web width ratio, b_f/b_w, exceeds 4.
   - Bond the outer-most strands in all rows located within the full-width section of the flange.
   - Position debonded strands furthest from the vertical centerline.
J. For multi-web sections having bottom flanges (voided slab, box beams and U-beams):
   - Uniformly distribute debonded strands between webs.
   - Strands shall be bonded within 1.0 times the web width projection.

their end zones. Thus, it is critical that the provisions of Article 5.7.3.5 are met. Research by Shahrooz et al.(2017) and others have shown that debonding more than the previous 25 percent limit can be achieved provided the requirement for longitudinal reinforcement (Article 5.7.3.5) is satisfied. Research by Shahrooz et al. (2017) tested a limited number of full-scale specimens with debonding limits up to 60 percent overall and 80 percent per row with satisfactory results. The recommended limit of 45 percent per row was chosen based on concern over long-term behavior of higher debonding percentages as well as detailing practicality used by several states.

When using debonded strands, the shear resistance in the region of debonding should be thoroughly investigated with due regard to the reduction in horizontal force available when considering the free body diagram in Figure C5.7.3.5-1 and to all other determinations of shear capacity provisions of this section (Articles 5.7, 5.8, and 5.9.2.3).

Research by Russell and Burns (1994) and Russell et al. (1994) recommends that the debonded strand transfer length region should not be located within the flexural cracking region to prevent reduced flexural strength performance. The research suggests that the debonding length from the beam end be limited to 15 percent of the span length. However, past practice of exceeding the 15 percent debonding length shows that limiting the debonding length from the beam end to 20 percent of the span length is sufficient in providing adequate flexural performance. The research also suggests staggering the termination location of the debonded strands to improve the bond performance.

For sections other than single-web flanged sections, the bearing conditions could be different for each end of the section, therefore adjusting the debonding detailing based on the bearing location is impractical. Shahrooz et al. (2017) provides recommendations when the same bearing conditions exist at each end of the section.

The detailing requirements are illustrated in Figures C5.9.4.3.3-1 and C5.9.4.3.3-2. Bonding the outer-most strands is necessary to minimize cracking near the surface due to the Hoyer effect and tying of mild reinforcement to the debonding material is not desirable.

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- Bond the outer-most strands within the section.

K. For all other sections:
- Debond uniformly across the width of the section.
- Bond the outer-most strands located within the section, stem, or web.

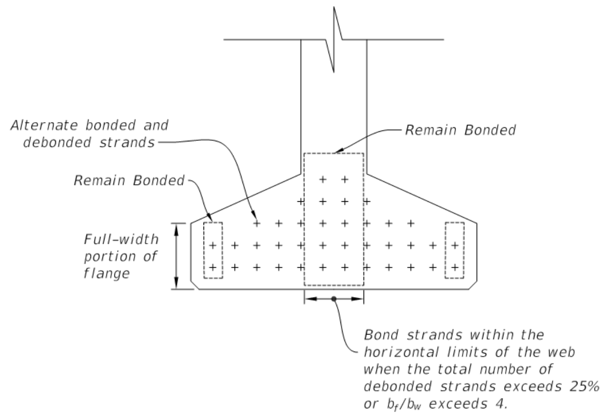

Figure C5.9.4.3.3-1—Details for Single-Web Flanged Sections

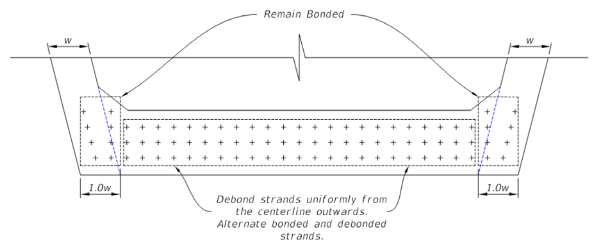

Figure C5.9.4.3.3-2—Details for Multi-Web Sections

5.9.4.4—Pretensioned Anchorage Zones

5.9.4.4.1—Splitting Resistance

The factored splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

$$P_r = f_s A_s \qquad (5.9.4.4.1\text{-}1)$$

where:

f_s = stress in steel not to exceed 20.0 ksi
A_s = total area of reinforcement located within the distance $h/4$ from the end of the beam (in.^{2})
h = overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)

For pretensioned I-girders or bulb tees, A_s shall be taken as the total area of the vertical reinforcement located within a distance of $h/4$ from the end of the member, where h is the overall height of the member (in.).

For pretensioned solid or voided slabs, A_s shall be taken as the total area of the horizontal reinforcement located within a distance of $h/4$ from the end of the member, where h is the overall width of the member (in).

For pretensioned box or tub girders, A_s shall be taken as the total area of vertical reinforcement or horizontal reinforcement located within a distance $h/4$

C5.9.4.4.1

The primary purpose of the choice of the 20-ksi steel stress limit for this provision is crack control.

Splitting resistance is of prime importance in relatively thin portions of pretensioned members that are tall or wide, such as the webs of I-girders and the webs and flanges of box and tub girders. Prestressing steel that is well distributed in such portions will reduce the splitting forces, while steel that is banded or concentrated at both ends of a member will require increased splitting resistance.

For pretensioned slab members, the width of the member is greater than the depth. A tensile zone is then formed in the horizontal direction perpendicular to the centerline member.

For tub and box girders, prestressing strands are located in both the bottom flange and webs. Tensile zones are then formed in both the vertical and horizontal directions in the webs and flanges. Reinforcement is required in both directions to resist the splitting forces.

Experience has shown that the provisions of this article generally control cracking in the end regions of pretensioned members satisfactorily; however, more reinforcement than required by this article may be necessary under certain conditions. Figures C5.9.4.4.1-1 and C5.9.4.4.1-2 show examples of splitting reinforcement for tub girders and voided slabs.

from the end of the member, where h is the lesser of the overall width or height of the member (in.).

For pretensioned members with multiple stems A_s shall be taken as the total area of vertical reinforcement, divided evenly among the webs, and located within a distance $h/4$ from the end of each web.

The resistance shall not be less than 4 percent of the total prestressing force at transfer.

The reinforcement shall be as close to the end of the beam as practicable.

Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.

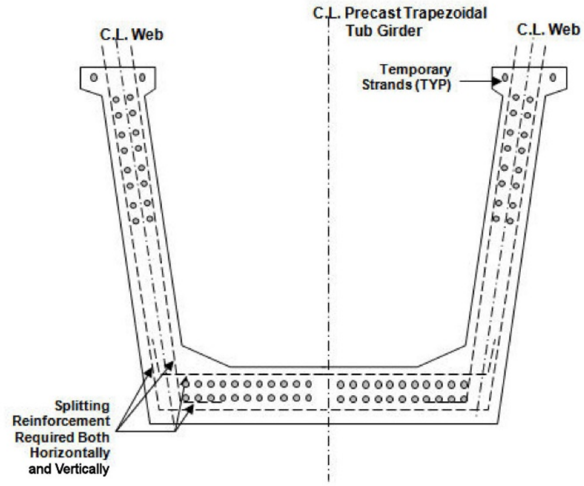

Figure C5.9.4.4.1-1—Precast Trapezoidal Tub Girder

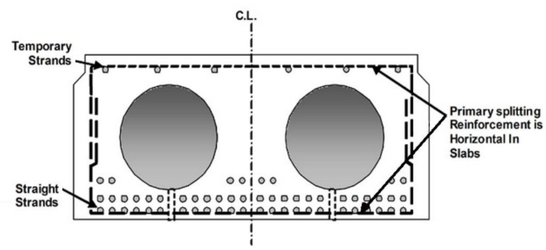

Figure C5.9.4.4.1-2—Precast Voided Slab

5.9.4.4.2—Confinement Reinforcement

For the distance of 1.5d from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands.

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

5.9.4.5—Temporary Strands

Temporary top strands may be used to control tensile stresses in precast prestressed girders during handling and transportation. These strands may be pretensioned or post-tensioned prior to lifting the girder from the casting bed or post-tensioned prior to transportation of the girder. Detensioning of temporary strands shall be shown in the construction sequence and typically occurs after the girders are securely braced and before construction of intermediate concrete diaphragms, if applicable.

C5.9.4.5

The stability of slender precast concrete girders is improved when lifting and transportation support points are moved away from the ends of the girder. The consequence of having a shorter span between support points is reduced dead load stresses to balance the stresses due to pretensioning and thus excessive tensile stresses in the top flange and compressive stresses in the bottom flange may develop. Temporary strands placed in the top flange of the girder reduce stresses and reduce the required concrete compressive strength at prestress transfer. Temporary strands in the top flange balance a

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Pretensioned temporary strands are debonded over the center portion of the girder. If pretensioned, the development length, measured from the end of the debonded zone, shall be determined as described in Article 5.9.4.3.3. No other provisions of Article 5.9.4.3.3 apply to temporary strands.

Debonded temporary strands shall be symmetrically distributed about the centerline of the member.

Debonded lengths of pairs of temporary strands that are symmetrically positioned about the centerline of the member shall be equal.

The effects of temporary strands must be considered when calculating camber and loss of prestress.

portion of the primary prestressing and reduce camber and camber growth due to creep.

Temporary top strands reduce the effectiveness of the permanent prestressing. Therefore, detensioning of the temporary top strands is typically recommended. Access to pretensioned temporary top strands is typically provided through pockets in the top surface of the top flange. Detensioning must occur before the temporary strands become inaccessible. Casting deck concrete and installation of precast deck panels will typically cover temporary top strand access points in the top flange.

Detensioning of the temporary top strands results in an upward deflection of the girder. Typically, temporary top strands are detensioned one girder at a time. This results in a differential deflection between adjacent girders that can crack intermediate concrete diaphragms, if present. To mitigate this issue, temporary strands should be detensioned after the girders are securely braced, but before intermediate concrete diaphragms are placed.

Sleeves used for debonding should be of sufficient inside diameter to mitigate binding of the strand during detensioning. Experience has shown that sleeves with inside diameter 0.18 in. to 0.25 in. larger than the strand diameter provide sufficient annular space. Access pockets should be protected to prevent water intrusion into the sleeves. Water in the sleeves can freeze and result in longitudinal cracking in the top flange that mirrors the location of the sleeves. Access pockets should be immediately patched and sealed after detensioning.

5.9.5—Details for Post-Tensioning

5.9.5.1—Minimum Spacing of Post-Tensioning Tendons and Ducts

5.9.5.1.1—Post-Tensioning Ducts—Girders Straight in Plane

Unless otherwise specified herein, the clear distance between straight post-tensioning ducts shall not be less than the greater of the following:

- 1.33 times the maximum size of the coarse aggregate; or
- 1.5 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 plane.

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The minimum vertical clear distance between bundles shall not be less than the greater of the following:

- 1.33 times the maximum size of the coarse aggregate; or
- 1.5 in.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 3.0 in.

For precast segmental construction where post-tensioning tendons extend through a segment joint the clear spacing between post-tensioning ducts shall not be less than the greater of the following:

- The duct internal diameter; or
- 4.0 in.

5.9.5.1.2—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 5.9.5.4.3. The spacing for curved ducts shall not be less than that required for straight ducts.

5.9.5.2—Maximum Spacing of Post-Tensioning Ducts in Slabs

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.

C5.9.5.2

The 4.0 times depth of slab requirement for the maximum spacing of transverse post-tensioning ducts in deck slabs reflects common practice. The composite thickness refers to slabs with bonded overlays.

5.9.5.3—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 greater of the following:

- 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.

C5.9.5.3

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.

5.9.5.4—Tendon Confinement

5.9.5.4.1—General

Tendons shall be located within the reinforcing steel stirrups in webs, and, where applicable, between layers of transverse reinforcement in flanges and slabs. For ducts in the bottom flanges of variable depth segments or

C5.9.5.4.1

This Article is based primarily on the recommendation from Breen and Kashima (1991). A possible arrangement of tendon confinement reinforcement in a bottom flange is shown in Figure components, nominal confinement reinforcement shall be provided around the duct at each segment face or joint. The reinforcement shall not be less than two rows of No. 4 tie bars at both sides of each duct, located so that they cross potential 45-degree failure planes, and with vertical dimension equal to the slab thickness, less top and bottom cover dimensions.

The effects of grouting pressure in the ducts shall be considered.

C5.9.5.4.1-1. Tie bars with two 90-degree hooks are shown. In high seismic areas the use of a 135-degree hook on one end of the tie bar should be considered in which case the bars should be placed so that the 135-degree hooks alternate top and bottom.

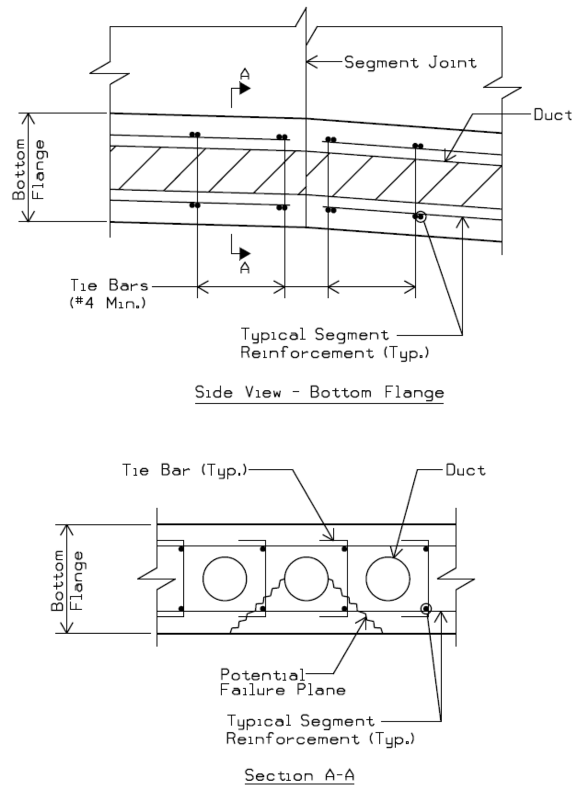

Figure C5.9.5.4.1-1—Reinforcement for Ducts in the Bottom Flanges of Variable Depth Segments

5.9.5.4.2—Wobble Effect in Slabs

For the purpose of this article, ducts spaced closer than 12.0 in. center-to-center in either direction shall be considered as closely spaced.

Where closely spaced transverse or longitudinal ducts are located in the flanges, and no provisions to minimize wobble of ducts are included in the contract documents, the top and bottom reinforcement mats should be tied together with No. 4 hairpin bars. The spacing between the hairpin bars shall not exceed the lesser of the following in each direction:

- 18.0 in.; or
- 1.5 times the slab thickness.

C5.9.5.4.2

The hairpin bars are provided to prevent slab delamination along the plane of the post-tensioning ducts.

5.9.5.4.3—Effects of Curved Tendons

Reinforcement shall be used to confine curved tendons if required by Article 5.7.1.5. The reinforcement shall be proportioned to ensure that the steel stress at service limit state does not exceed $0.6f_y$, and the assumed value of f_y shall not exceed 60.0 ksi. Unless a

C5.9.5.4.3

Curved tendons induce deviation forces that are radial to the tendon in the plane of tendon curvature. Curved tendons with multiple strands or wires also induce out-of-plane forces that are perpendicular to the plane of tendon curvature.

strut-and-tie analysis is done and indicates otherwise, spacing of the confinement reinforcement shall not exceed the lesser of the following:

- 3.0 times the outside diameter of the duct; or
- 24.0 in.

Tendons shall not be bundled in groups greater than three when girders are curved in horizontal plane.

In-plane force effects are due to a change in direction of the tendon within the plane of curvature. Resistance to in-plane forces in curved girders may be provided by increasing the concrete cover over the duct, by adding confinement tie reinforcement, or by a combination thereof. Figure C5.9.5.4.4a-1 shows an in-plane deviation in the vertical curve and Figure C5.9.5.4.4a-2 shows a potential in-plane deviation in the horizontal curve.

Out-of-plane force effects are due to the spreading of the wires or strands within the duct. Out-of-plane force effects are shown in Figure C5.9.5.4.5-1 and can be affected by ducts stacked vertically or stacked with a horizontal offset.

5.9.5.4.4—Design for In-Plane Force Effects

5.9.5.4.4a—In-Plane Force Effects

In-plane deviation force effects due to the change in direction of tendons shall be taken as:

$$F_{u-in} = \frac{P_u}{R} \quad (5.9.5.4.4a\text{-}1)$$

where:

F_{u-in} = in-plane deviation force effect per unit length of tendon (kips/ft)
P_u = factored tendon force as specified in Article 3.4.3 (kip)
R = radius of curvature of the tendon at the considered location (ft)

The maximum deviation force shall be determined on the basis that all the tendons, including provisional tendons, are stressed. The provisions of Article 5.9.5.6 shall apply to design for in-plane force effects due to tendons curved at the tendon anchorage.

C5.9.5.4.4a

In-plane forces occur, for example, in anchorage blisters or curved webs, as shown in Figures C5.9.5.4.4a-1 and C5.9.5.4.4a-2. Without adequate reinforcement, the tendon deviation forces may rip through the concrete cover on the inside of the tendon curve, or unbalanced compressive forces may push off the concrete on the outside of the curve. Small radial tensile stresses may be resisted by concrete in tension.

The load factor of 1.2 taken from Article 3.4.3 and applied to the maximum tendon jacking force results in a design load of about 96 percent of the nominal ultimate strength of the tendon. This number compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

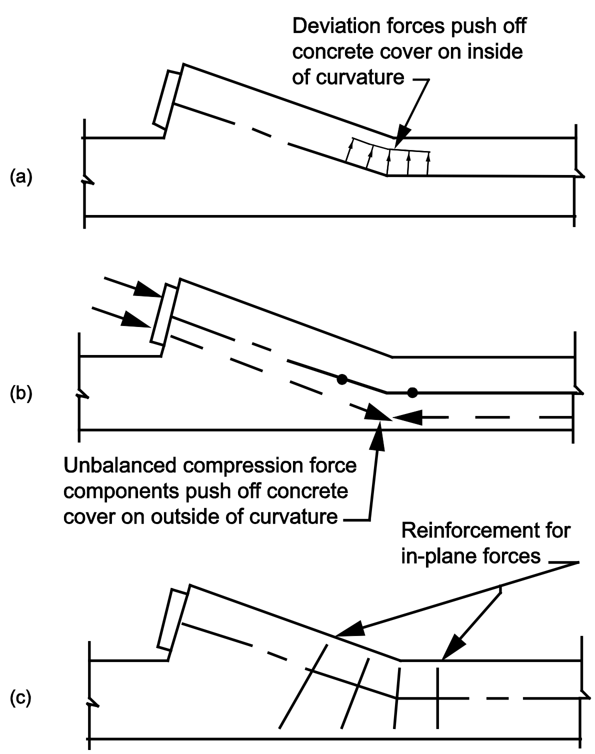

Figure C5.9.5.4.4a-1—In-Plane Forces in a Soffit Blister

The radial component from the longitudinal web stress in the concrete due to the compression in the cylindrical web must be subtracted.

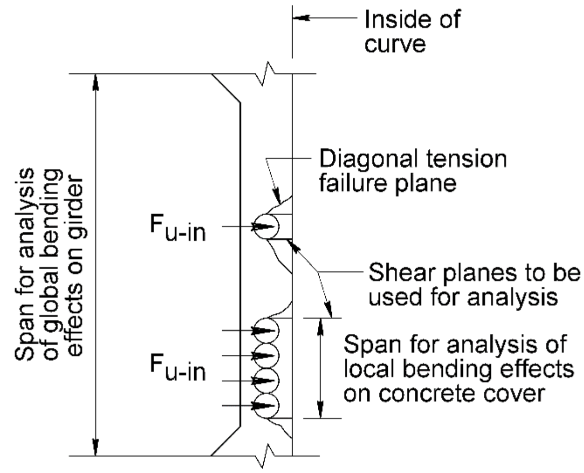

Figure C5.9.5.4.4a-2—In-Plane Force Effects in Curved Girders Due to Horizontally-curved Tendons

5.9.5.4.4b—Shear Resistance to Pullout

The shear resistance per unit length of the concrete cover against pullout by deviation forces, V_r, shall be taken as:

$$V_r = \phi V_n \qquad (5.9.5.4.4b\text{-}1)$$

in which:

$$V_n = 0.15 d_{eff} \lambda \sqrt{f'_{ci}} \qquad (5.9.5.4.4b\text{-}2)$$

where:

ϕ = resistance factor for shear, 0.75
V_n = nominal shear resistance of two shear planes per unit length (kips/in.)
d_{eff} = one-half the effective length of the failure plane in shear and tension for a curved element (in.)
λ = concrete density modification factor as specified in Article 5.4.2.8
f'_{ci} = design concrete compressive strength at time of application of tendon force (ksi)

For single duct stack or for $s_{duct} < d_{duct}$, d_{eff}, shown in Detail (a) in Figure 5.9.5.4.4b-1, shall be taken as:

$$d_{eff} = d_c + \frac{d_{duct}}{4} \qquad (5.9.5.4.4b\text{-}3)$$

For $s_{duct} \geq d_{duct}$, d_{eff} shall be taken as the lesser of the following based on Paths 1 and 2 shown in Detail (b) in Figure 5.9.5.4.4b-1:

C5.9.5.4.4b

The two shear planes for which Eq. 5.9.5.4.4b-2 gives V_n are as indicated in Figure 5.9.5.4.4b-1 for single and multiple tendons.

Where a staggered or side-by-side group of ducts is located side by side in a single web, all possible shear and tension failure planes should be considered in determining d_{eff}.

$$d_{eff} = t_w - \frac{d_{duct}}{2} \quad (5.9.5.4.4b\text{-}4)$$

$$d_{eff} = d_c + \frac{d_{duct}}{4} + \frac{\sum s_{duct}}{2} \quad (5.9.5.4.4b\text{-}5)$$

where:

d_c = minimum concrete cover over the tendon duct (in.)
d_{duct} = outside diameter of post-tensioning duct (in.)
t_w = web thickness (in.)
s_{duct} = clear distance between tendon ducts in vertical direction (in.)

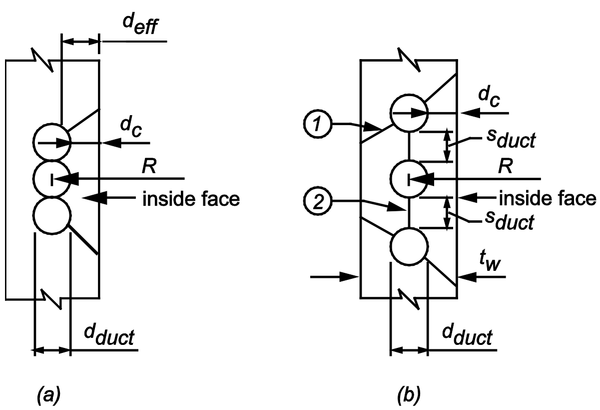

Figure 5.9.5.4.4b-1—Definition of d_{eff}

If the factored in-plane deviation force exceeds the factored shear resistance of the concrete cover, as specified in Eq. 5.9.5.4.4b-1, fully anchored stirrups and duct ties to resist the in-plane deviation forces shall be provided in the form of either nonprestressed or prestressed reinforcement. The duct ties shall be anchored beyond the ducts either by in-plane 90 degree hooks or by hooking around the vertical bar.

Additional information on deviation forces can be found in Nutt et al. (2008) and Van Landuyt (1991).

Common practice is to limit the stress in the duct ties to 36.0 ksi at the maximum unfactored tensile force.

A generic stirrup and duct tie detail is shown in Figure C5.9.5.4.4b-1. Small diameter reinforcing bars should be used for better anchorage of these bars. There have been no reported web failures where this detail has been used.

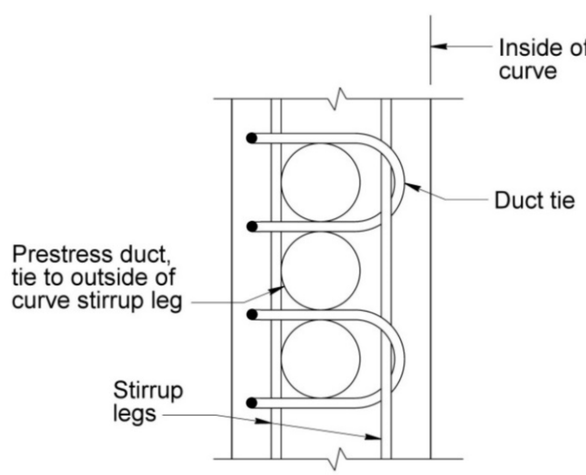

Figure C5.9.5.4.4b-1—Generic Duct Tie Detail

5.9.5.4.4c—Cracking of Cover Concrete

When the clear distance between ducts oriented in a vertical column is less than 1.5 in., the ducts shall be considered stacked. Resistance to cracking shall be investigated at the ends and at midheight of the unreinforced cover concrete.

The applied local moment per unit length at the ends of the cover shall be taken as:

$$M_{end} = \frac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{12} \qquad (5.9.5.4.4c\text{-}1)$$

and the applied local moment per unit length at the midheight of the cover shall be taken as:

$$M_{mid} = \frac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{24} \qquad (5.9.5.4.4c\text{-}2)$$

where:

h_{ds} = height of the duct stack as shown in Figure C5.9.5.4.4c-1

Tensile stresses in the unreinforced concrete cover resulting from Eqs. 5.9.5.4.4c-1 and 5.9.5.4.4c-2 shall be combined with the tensile stresses from regional bending of the web as defined in Article 5.9.5.4.4d to evaluate the potential for cracking of the cover concrete. If combined tensile stresses exceed the cracking stresses given by Eq. 5.9.5.4.4c-3, ducts shall be restrained by stirrup and duct tie reinforcement.

The design flexural cracking stress of the hypothetical unreinforced concrete beam consisting of the cover concrete over the inside face of a stack of horizontally-curved post-tensioned tendons, f_{cr}, shall be determined as follows:

$$f_{cr} = \phi f_r \qquad (5.9.5.4.4c\text{-}3)$$

in which:

ϕ = 0.85

where:

f_r = modulus of rupture of concrete (ksi)

C5.9.5.4.4c

Figure C5.9.5.4.4c-1 illustrates the concept of an unreinforced cover concrete beam to be investigated for cracking. Experience has shown that a vertical stack of more than three ducts can result in cracking of the cover concrete. When more than three ducts are required, it is recommended that at least 1.5 in. spacing be provided between the upper and lower ducts of the two stacks.

The resistance factor is based on successful performance of curved post-tensioned box girder bridges in California.

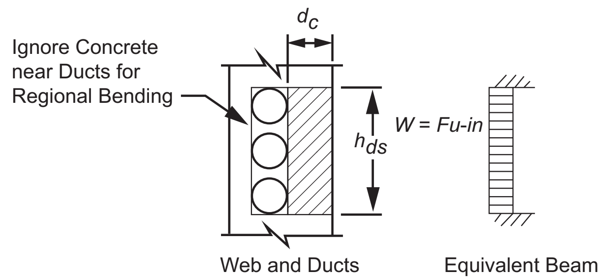

Figure C5.9.5.4.4c-1—Hypothetical Unreinforced Concrete Cover Beam

5.9.5.4.4d—Regional Bending

The regional flexural effects of in-plane forces at the strength limit state shall be taken as:

$$M_u = \frac{\phi_{cont} \Sigma F_{u-in} h_c}{4} \qquad (5.9.5.4.4d\text{-}1)$$

where:

C5.9.5.4.4d

When determining tensile stresses for the purpose of evaluating the potential for cracking of the cover concrete as specified in Article 5.9.5.4.4c, the effect of regional bending is combined with bending of the local concrete cover beam. It is recommended that the effect of stirrups in resisting bending be ignored, and that the ducts be considered as voids in the transverse section of the webs.

ϕ_{cont} = 0.6 girder web continuity factor for interior webs; 0.7 girder web continuity factor for exterior webs
h_c = span of the web between the top and bottom slabs measured along the axis of the web as shown in Figure C5.9.5.4.4a-2

For curved girders, the local flexural and shear effects of out-of-plane forces as described in Article 5.9.5.4.5 shall be evaluated.

When curved ducts for tendons other than those crossing at approximately 90 degrees are located so that the direction of the radial force from one tendon is toward another, confinement of the ducts shall be provided by:

- spacing the ducts to ensure adequate nominal shear resistance, as specified in Eq. 5.9.5.4.4b-1; or
- providing confinement reinforcement to resist the radial force.

The wedging action of strands within the duct due to vertical curvature of the tendon can exacerbate tendon pullout resulting from horizontal curvature of the tendon as described in Articles 5.9.5.4.4b and 5.9.5.4.4c.

5.9.5.4.5—Out-of-Plane Force Effects

Out-of-plane force effects due to the wedging action of strands against the duct wall may be estimated as:

$$F_{u-out} = \frac{P_u}{\pi R} \quad (5.9.5.4.5\text{-}1)$$

where:

$F_{u\text{-}out}$ = out-of-plane force effect per unit length of tendon (kip/ft)
P_u = factored tendon force, as specified in Article 3.4.3 (kip)
R = radius of curvature of the tendon in a vertical plane at the considered location (ft)

C5.9.5.4.5

Out-of-plane forces in multistrand, post-tensioning tendons are caused by the spreading of the strands or wires within the duct, as shown in Figure C5.9.5.4.5-1. Small out-of-plane forces may be resisted by concrete in shear; otherwise, spiral reinforcement is most effective to resist out-of-plane forces. In horizontally-curved bridges, out-of-plane forces due to the vertical curvature of tendons should be added to in-plane forces resulting from horizontal curvature of the tendons. Additional information can be found in Nutt (2008).

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If the factored shear resistance given by Eq. 5.9.5.4.4b-1 is not adequate, local confining reinforcement shall be provided throughout the curved tendon segments to resist all of the out-of-plane forces, preferably in the form of spiral reinforcement.

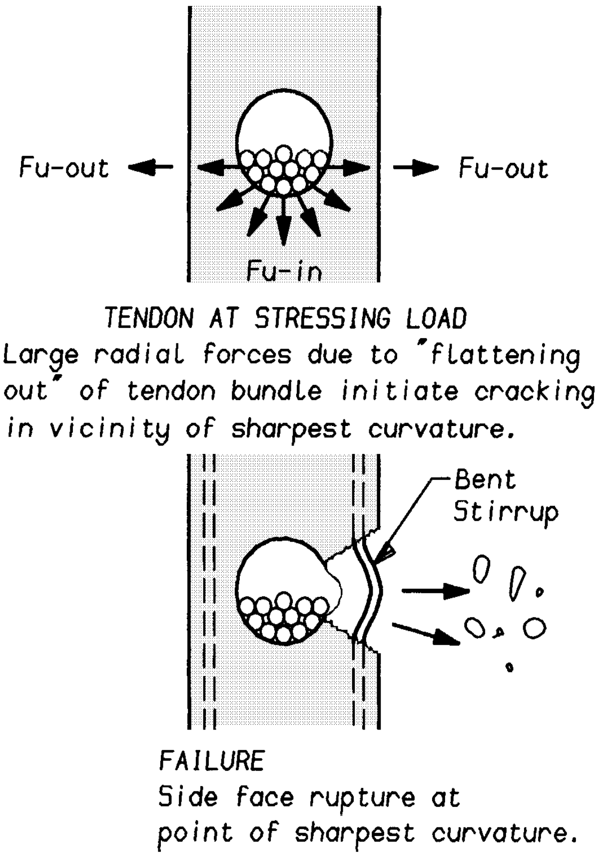

Figure C5.9.5.4.5-1—Effects of Out-of-Plane Forces

5.9.5.5—External Tendon Supports

Unless a vibration analysis indicates otherwise, the unsupported length of external tendons shall not exceed 25.0 ft. External tendon supports in curved concrete box girders shall be located far enough away from the web to prevent the free length of tendon from bearing on the web at locations away from the supports. Where deviation saddles are required for this purpose, they shall be designed in accordance with Article 5.9.5.6.9.

5.9.5.6—Post-Tensioned Anchorage Zones

5.9.5.6.1—General

For bonded systems, bond transfer lengths between anchorages and critical zones where the full prestressing force is required under strength or extreme event loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. When anchorages or couplers are located at, or near, critical sections under strength or extreme event loads, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage and coupler, tested in an unbonded state.

C5.9.5.6.1

The purpose of locating anchorages and couplers away from critical zones where the full strength of the tendon is required under strength or extreme event loads is to assure the full capacity of the tendon can be developed and is not potentially reduced by anchorages or couplers. Testing shall establish the capacity of anchorages and couplers located within a bond transfer length of any critical section as this capacity may be less than that of the tendon itself. The *AASHTO LRFD Bridge Construction Specifications* require anchorages to develop a minimum of 95 percent of actual ultimate

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Anchorages shall be designed at the strength limit states for the factored jacking forces as specified in Article 3.4.3.

For anchorage zones at the end of a component or segment, the transverse dimensions may be taken as the depth and width of the section but not larger than the longitudinal dimension of the component or segment. The longitudinal extent of the anchorage zone in the direction of the tendon shall not be less than the greater of the transverse dimensions of the anchorage zone and shall not be taken as more than one and one-half times that dimension.

For intermediate anchorages, the anchorage zone shall be considered to extend in the direction opposite to the anchorage force for a distance not less than the larger of the transverse dimensions of the anchorage zone.

tensile strength of the prestressing steel. Therefore, using 95 percent of the guaranteed ultimate tensile strength would be acceptable for design purposes in lieu of additional testing.

With slight modifications, the provisions of Article 5.9.5.6 are also applicable to the design of reinforcement under high-load capacity bearings.

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

Within the anchorage zone, the assumption that plane sections remain plane is not valid.

The dimensions of the anchorage zone are based on the principle of St. Venant. Provisions for components with a length smaller than one of its transverse dimensions were included to address cases such as transverse prestressing of bridge decks, as shown in Figure C5.9.5.6.1-1.

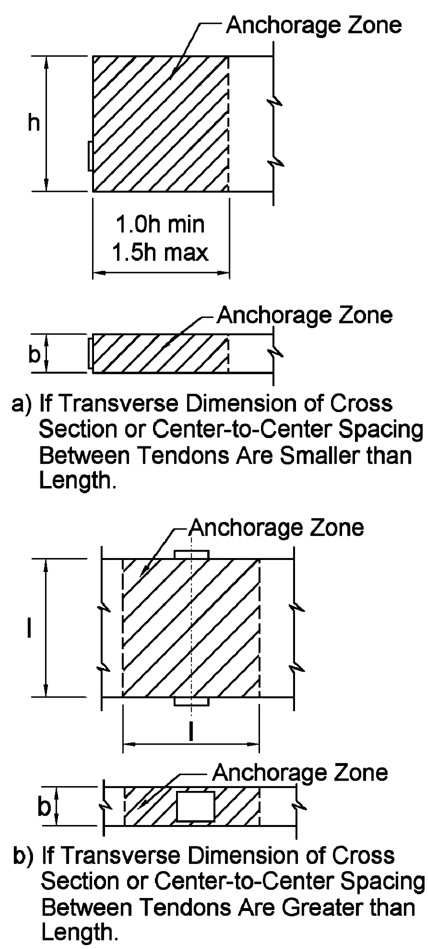


Figure C5.9.5.6.1-1—Geometry of the Anchorage Zones

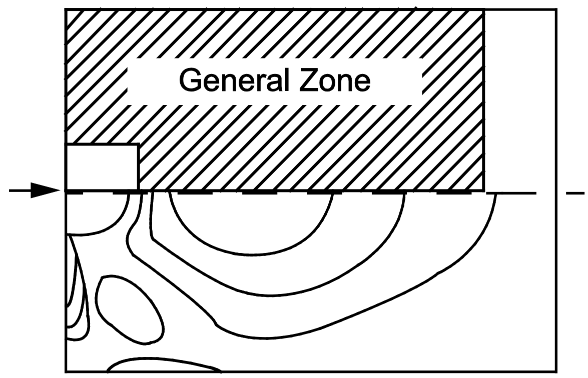
For design purposes, the anchorage zone shall be considered as comprised of two regions:

For intermediate anchorages, large tensile stresses may exist behind the anchor. These tensile stresses

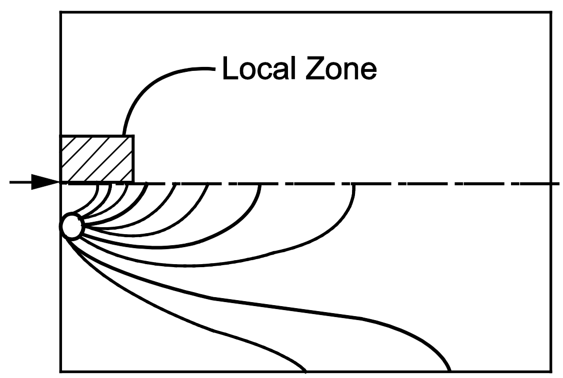
- the general zone, for which the provisions of Article 5.9.5.6.2 apply; and
- the local zone, for which the provisions of Article 5.9.5.6.3 apply.

result from the compatibility of deformations ahead of and behind the anchorage.

Figure C5.9.5.6.1-2 illustrates the distinction between the local and the general zone. The region subjected to tensile stresses due to spreading of the tendon force into the structure is the general zone (Figure C5.9.5.6.1-2a). The region of high compressive stresses immediately ahead of the anchorage device is the local zone (Figure C5.9.5.6.1-2b).



a) Principal Tensile Stresses and the General Zone



b) Principal Compressive Stresses and the Local Zone

Figure C5.9.5.6.1-2—General Zone and Local Zone

5.9.5.6.2—General Zone

The extent of the general zone shall be taken as identical to that of the overall anchorage zone including the local zone, defined in Article 5.9.5.6.1.

Design of general zones shall comply with the requirements of Article 5.9.5.6.5.

C5.9.5.6.2

In many cases, the general zone and the local zone can be treated separately, but for small anchorage zones, such as in slab anchorages, local zone effects (e.g., high bearing and confining stresses), and general zone effects (e.g., tensile stresses due to spreading of the tendon force) may occur in the same region. The designer should account for the influence of overlapping general zones.

5.9.5.6.3—Local Zone

Design of local zones shall either comply with the requirements of Article 5.8.4.4 or be based on the results of acceptance tests as specified in Article 5.8.4.4.3 and

C5.9.5.6.3

The local zone is defined as either the rectangular prism, or, for circular or oval anchorages, the equivalent rectangular prism of the concrete surrounding and

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described in Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications*.

For design of the local zone, the effects of high bearing pressure and the application of confining reinforcement shall be considered.

Anchorage devices based on the acceptance test of *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2.3, shall be referred to as special anchorage devices.

immediately ahead of the anchorage device and any integral confining reinforcement. The dimensions of the local zone are defined in Article 5.8.4.4.1.

The local zone is expected to resist the high local stresses introduced by the anchorage device and to transfer them to the remainder of the anchorage zone. The resistance of the local zone is more influenced by the characteristics of the anchorage device and its confining reinforcement than by either the geometry or the loading of the structure.

5.9.5.6.4—Responsibilities

The Engineer of Record shall be responsible for the overall design and approval of working drawings for the general zone, including the location of the tendons and anchorage devices, general zone reinforcement, the stressing sequence, and the design of the local zone for anchorage devices based on the provisions of Article 5.8.4.4. The local zone reinforcing for special anchorage devices shall be established by the Supplier of the anchorage device. The contract documents shall specify that all working drawings for the local zone must be approved by the Engineer of Record.

The anchorage device Supplier shall be responsible for furnishing anchorage devices that satisfy the anchor efficiency requirements of *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2. If special anchorage devices are used, the anchorage device Supplier shall be responsible for furnishing anchorage devices that also satisfy the acceptance test requirements of Article 5.8.4.4.3 and of *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2.3. This acceptance test and the anchor efficiency test shall be conducted by an independent testing agency acceptable to the Engineer of Record. The anchorage device supplier shall provide records of the acceptance test in conformance with *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2.3.12, to the Engineer of Record and to the Constructor and shall specify auxiliary and confining reinforcement, minimum edge distance, minimum anchor spacing, and minimum concrete strength at time of stressing required for proper performance of the local zone.

The responsibilities of the Constructor shall be as specified in the *AASHTO LRFD Bridge Construction Specifications*, Article 10.4.

C5.9.5.6.4

The Engineer of Record has the responsibility to indicate the location of individual tendons and anchorage devices. Should the Designer initially choose to indicate only total tendon force and eccentricity, he still retains the responsibility of approving the specific tendon layout and anchorage arrangement submitted by a post-tensioning specialist or the Contractor. The Engineer is responsible for the design of general zone reinforcement required by the approved tendon layout and anchorage device arrangement.

The use of special anchorage devices does not relieve the Engineer of Record from his responsibility to review the design and working drawings for the anchorage zone to ensure compliance with the anchorage device Supplier's specifications.

For special anchorage devices, the anchorage device Supplier has to provide information regarding all requirements necessary for the satisfactory performance of the local zone to the Engineer of Record and to the Contractor. Necessary local zone confinement reinforcement has to be specified by the Supplier.

5.9.5.6.5—Design of the General Zone

5.9.5.6.5a—Design Methods

For the design of general zones, the following design methods, conforming to the requirements of Article 5.9.5.6.5b, may be used:

- equilibrium-based inelastic models, generally termed the strut-and-tie method (illustrated in Article 5.8.2.7);

C5.9.5.6.5a

The design methods referred to in this article are not meant to preclude other recognized and verified procedures. In many anchorage applications where substantial or massive concrete regions surround the anchorages and where the members are essentially rectangular without substantial deviations in the force flow path, the approximate procedures of Article 5.8.4.5

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- refined elastic stress analyses as specified in Section 4; or
- other approximate methods such as those provided in Article 5.8.4.5, where applicable.

The effects of stressing sequence and three-dimensional effects due to concentrated jacking loads shall be investigated. Three-dimensional effects may be analyzed using three-dimensional analysis procedures or may be approximated by considering separate submodels for two or more planes, in which case the interaction of the submodels should be considered, and the model loads and results should be consistent.

The factored concrete compressive stress for the general zone shall not exceed $0.7\phi f'_{ci}$. In areas where the concrete may be extensively cracked at ultimate due to other force effects, or if large inelastic rotations are expected, the factored compressive stress shall be limited to $0.6\phi f'_{ci}$.

The tensile strength of the concrete shall be neglected in the design of the general zone.

The nominal tensile stress of bonded reinforcement shall be limited to f_y for both nonprestressed reinforcement and bonded prestressed reinforcement. The nominal tensile stress of unbonded or debonded prestressed reinforcement shall be limited to f_{pe} + 15,000 psi.

The contribution of any local zone reinforcement to the strength of the general zone may be conservatively neglected in the design.

can be used. However, in the post-tensioning of thin sections, flanged sections, and irregular sections or where the tendons have appreciable curvature, the more general procedures of Article 5.8.2.7 and 5.8.3 may be required.

Different anchorage force combinations have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed but also the intermediate stages.

The provision concerning three-dimensional effects was included to alert the Designer to effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs. For example, in members with thin rectangular cross sections, bursting forces not only exist in the major plane of the member but also perpendicular to it. In many cases, these effects can be determined independently for each direction, but some applications require a fully three-dimensional analysis, i.e., diaphragms for the anchorage of external tendons.

5.9.5.6.5b—Design Principles

Compressive stresses in the concrete ahead of basic anchorage devices shall satisfy the requirements of Article 5.8.4.4.2.

The compressive stresses in the concrete ahead of the anchorage device shall be investigated at a distance, measured from the concrete bearing surface, determined as the greater of the following:

- the depth to the end of the local confinement reinforcement; or
- the smaller lateral dimension of the anchorage device.

These compressive stresses may be determined using the strut-and-tie method of Article 5.8.2.7, an elastic stress analysis according to Article 5.8.3, or the approximate method outlined in Article 5.8.4.5.2.

The magnitude of the bursting force, T_{burst}, and its corresponding distance from the loaded surface, d_{burst}, may be determined using the strut-and-tie method of Article 5.8.2.7, an elastic stress analysis according to Article 5.8.3, or the approximate method outlined in Article 5.8.4.5.3. Three-dimensional effects shall be considered for the determination of the bursting reinforcement requirements.

C5.9.5.6.5b

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

The interface between the confined concrete of the local zone and the usually unconfined concrete of the general zone is critical. The provisions of this article define the location where concrete stresses should be investigated.

The bursting force is the tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis. Bursting forces are caused by the lateral spreading of the prestressing forces concentrated at the anchorage.

The guidelines for the arrangement of the bursting reinforcement direct the Designer toward reinforcement patterns that reflect the elastic stress distribution. The experimental test results show that this leads to satisfactory behavior at the service limit state by limiting the extent and opening of cracks and at the strength limit state by limiting the required amount of

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

Resistance to bursting forces shall be provided by nonprestressed or prestressed reinforcement or in the form of spirals, closed hoops, or anchored transverse ties. This reinforcement shall resist the total bursting force. The following guidelines for the arrangement and anchorage of bursting reinforcement should apply:

- Reinforcement is extended over the full width of the member and anchored as close to the outer faces of the member as cover permits;
- Reinforcement is distributed ahead of the loaded surface along both sides of the tendon throughout a distance taken as the lesser of 2.5 d_{burst} for the plane considered and 1.5 times the corresponding lateral dimension of the section, where d_{burst} is specified by Eq. 5.8.4.5.3-2;
- The centroid of the bursting reinforcement coincides with the distance d_{burst} used for the design; and
- Spacing of reinforcement is not greater than either 24.0 bar diameters or 12.0 in.

redistribution of forces in the anchorage zone (Sanders, 1990). A uniform distribution of the bursting reinforcement with its centroid at d_{burst}, as shown in Figure C5.9.5.6.5b-1, may be considered acceptable.

The edge tension forces may be determined using the strut-and-tie method, procedures of Article 5.8.2.7, elastic analysis according to Article 5.8.3, or approximate methods of Article 5.8.4.5.4.

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

For multiple anchorages with a center-to-center spacing of less than 0.4 times the depth of the section, the spalling force shall not be taken to be less than two percent of the total factored tendon force. For larger spacings, the spalling forces shall be determined by analysis.

The strut-and-tie method may be used for larger anchor spacings.

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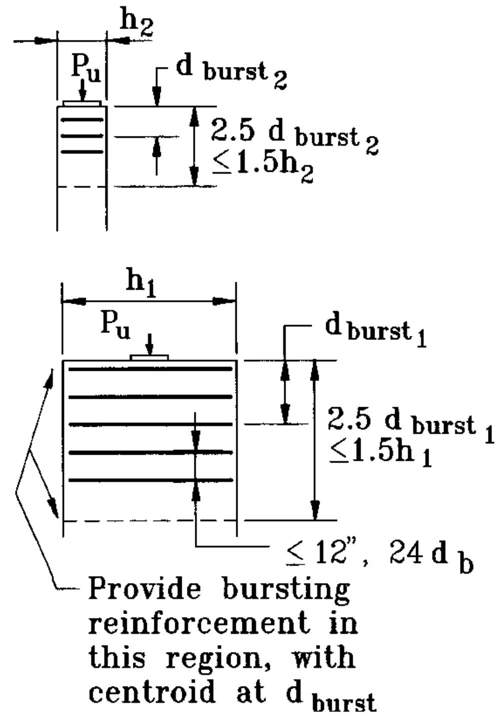


Figure C5.9.5.6.5b-1—Arrangement for Bursting Reinforcement

Spalling forces are induced in concentrically loaded anchorage zones, eccentrically loaded anchorage zones, and anchorage zones for multiple anchors. Longitudinal edge tension forces are induced where the resultant of the anchorage forces causes eccentric loading of the anchorage zone.

Resistance to edge tension forces shall be provided by reinforcement located close to the longitudinal and transverse edge of the concrete. Arrangement and anchorage of the edge tension reinforcement shall satisfy the following:

- specified spalling reinforcement is extended over the full width of the member;
- spalling reinforcement between multiple anchorage devices effectively ties the anchorage devices together; and
- longitudinal edge tension reinforcement and spalling reinforcement for eccentric anchorage devices are continuous; the reinforcement extends along the tension face over the full length of the anchorage zone and along the loaded face from the longitudinal edge to the other side of the eccentric anchorage device or group of anchorage devices.

For multiple anchorages, the spalling forces are required for equilibrium, and provision for adequate reinforcement is essential for the ultimate load capacity of the anchorage zone, as shown in Figure C5.9.5.6.5b-2. These tension forces are similar to the tensile tie forces existing between individual footings supporting deep walls. In most cases, the minimum spalling reinforcement specified herein will control.

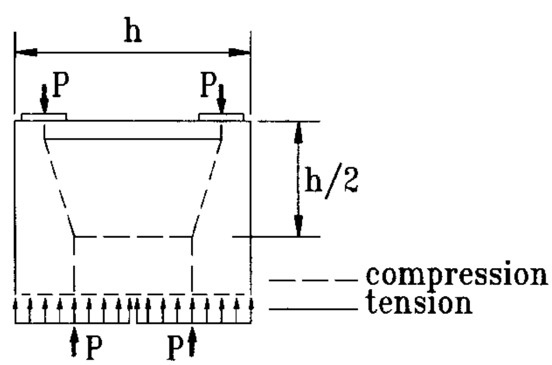


Figure C5.9.5.6.5b-2—Path of Forces for Multiple Anchorages

Figure C5.9.5.6.5b-3 illustrates the location of the edge tension forces.

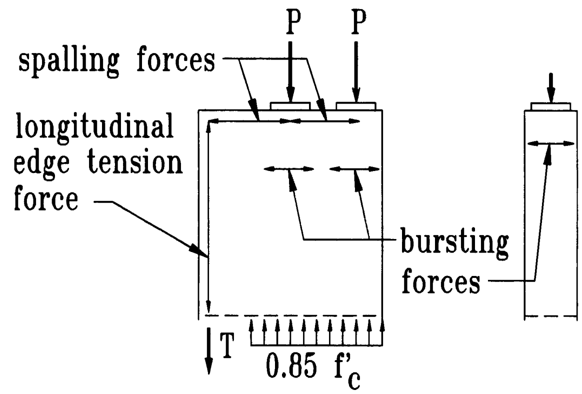


Figure C5.9.5.6.5b-3—Edge Tension Forces

The minimum spalling force for design is two percent of the total post-tensioning force. This value is smaller than the four percent proposed by Guyon (1953) and reflects both analytical and experimental findings showing that Guyon's values for spalling forces are rather conservative and that spalling cracks are rarely observed in experimental studies (Base et al., 1966; Beeby, 1983).

Figure C5.9.5.6.5b-4 illustrates the reinforcement requirements for anchorage zones.

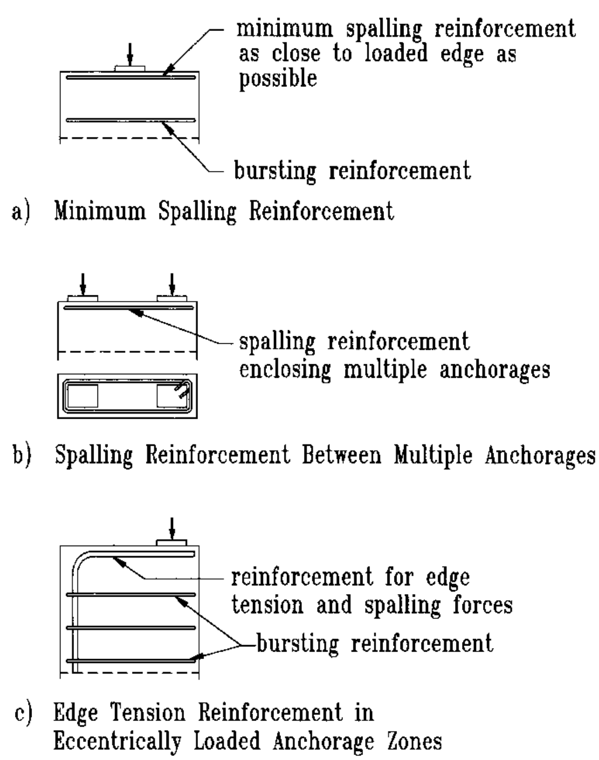


Figure C5.9.5.6.5b-4—Arrangement of Anchorage Zone Reinforcement

5.9.5.6.6—Special Anchorage Devices

Where special anchorage devices that do not satisfy the requirements of Article 5.8.4.4.2 are to be used, reinforcement similar in configuration and at least equivalent in volumetric ratio to the supplementary skin reinforcement permitted under the provisions of the *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2.3.4, shall be furnished in the corresponding regions of the anchorage zone.

5.9.5.6.7—Intermediate Anchorages

5.9.5.6.7a—General

Intermediate anchorages shall not be used in regions where significant tension is generated behind the anchor from other loads. Intermediate anchorages near unreinforced segment joints shall only be located where there is sufficient residual compression, f_{cb}, to resist 125 percent of the tie-back force behind the anchor. Wherever practical, blisters should be located in the corner between flange and webs or shall be extended over the full flange width or web height to form a continuous rib. If isolated blisters must be used on a flange or web, local shear bending, and direct force effects shall be considered in the design.

C5.9.5.6.7a

Intermediate anchorages are usually used in segmented construction. Locating anchorage blisters in the corner between flange and webs significantly reduces local force effects at intermediate anchorages. Lesser reduction in local effects can be obtained by increasing the width of the blister to match the full width of the flange or full depth of the web to which the blister is attached.

For flange thickness ranging from 5.0 to 9.0 in., an upper limit of 12, Grade 270 ksi, 0.5-in. diameter strands is recommended for tendons anchored in blisters supported only by the flange. The anchorage force of the tendon must be carefully distributed to the flange by reinforcement.

5.9.5.6.7b—Crack Control behind Intermediate Anchors

Unless otherwise specified herein, bonded reinforcement shall be provided to tie-back at least 25 percent of the tendon force behind the intermediate anchor into the concrete section at service limit states and any stage of construction. Stresses in this bonded reinforcement shall not exceed the lesser of the following:

- $0.6 f_y$; or
- 36.0 ksi.

If compressive stresses are generated behind the anchor, the amount of tie-back reinforcement may be reduced using Eq. 5.9.5.6.7b-1.

$$T_{ia} = 0.25P_s - f_{cb}A_{cb} \qquad (5.9.5.6.7b\text{-}1)$$

where:

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

P_s = unfactored tendon force(s) at the anchorage (kip)

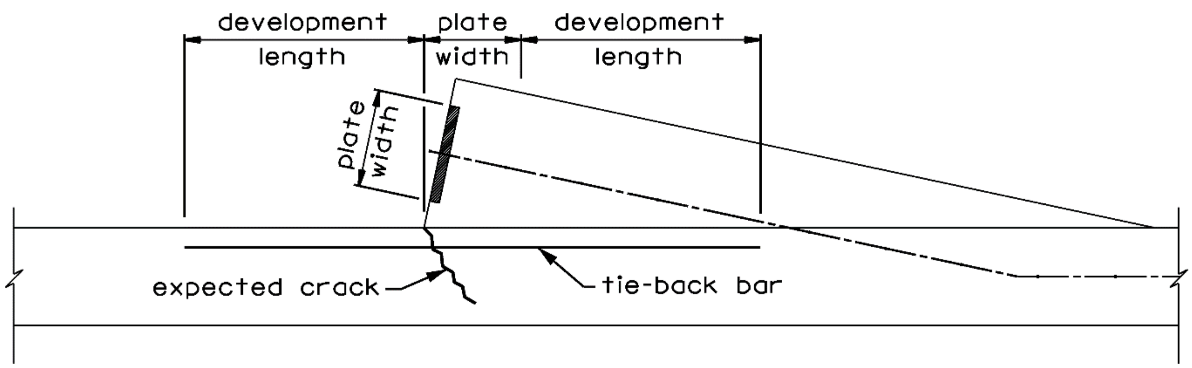
f_{cb} = unfactored minimum compressive stress in the region behind the anchor at service limit states and any stage of construction (ksi)

C5.9.5.6.7b

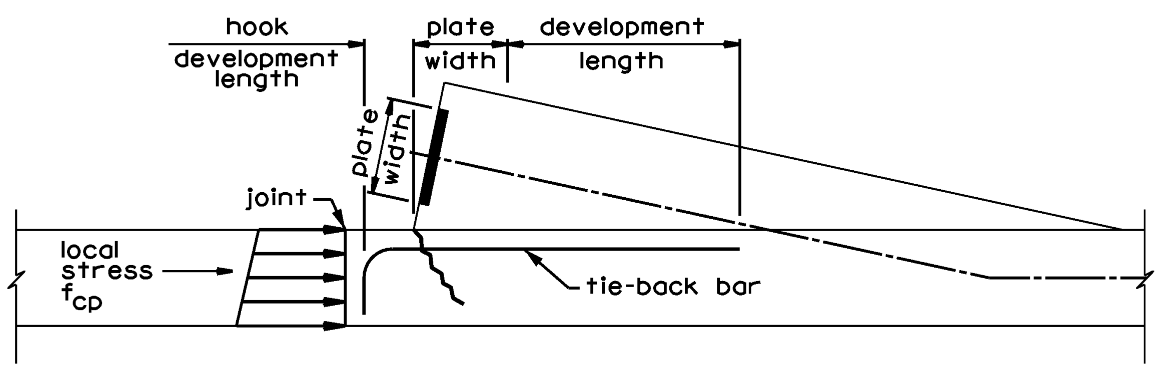
Cracks may develop in the slab or web walls, or both, immediately behind blisters and ribs due to stress concentrations caused by the anchorage force. Reinforcement proportioned to tie back 25 percent of the unfactored jacking force has been shown to provide adequate crack control (Wollmann, 1992). To ensure that the reinforcement is adequately developed at the crack location, a length of bar must be provided equal to one plate width plus one development length ahead of and one development length behind the anchor plate. This is illustrated in Figure C5.9.5.6.7b-1(a). For precast segmental bridges in which the blister is close to a joint, a hook may be used to properly develop the bar provided that the joint remains in compression locally behind the intermediate anchor for all service load combinations including any local tension behind the anchor. This is illustrated in Figure C5.9.5.6.7b-1(b).

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 as shown in Figure C5.9.5.6.7b-2 (in.^{2})

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



(a) Condition with no Nearby Precast Joints



(b) Condition with Precast Joint Near Anchor Plate

Figure C5.9.5.6.7b-1—Required Length of Crack Control Reinforcement

The amount of tie-back reinforcing can be reduced by accounting for the compression in the concrete cross section behind the anchor. Note that if the stress behind the anchor is tensile, the tie-back tension force will be $0.25P_s$ plus the tensile stress times A_{cb}. The area, A_{cb}, is illustrated in Figure C5.9.5.6.7b-2, along with other detailing requirements for the tie-back reinforcement.

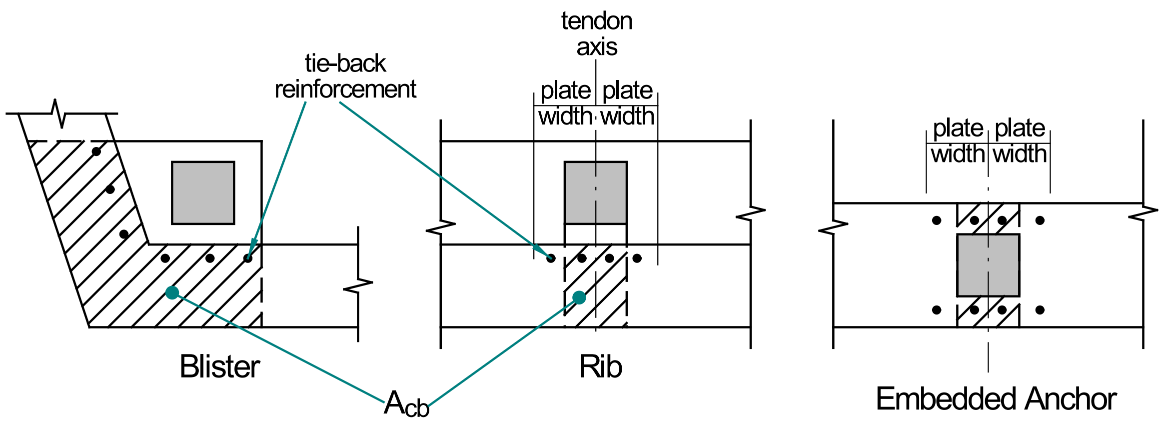


Figure C5.9.5.6.7b-2—Examples of A_{cb}

For bridges with multiple tendons and anchorages, the order of stressing should be taken into account and specified in the plans. For spans in which shorter tendons are wholly encompassed by longer tendons, it is frequently prudent to stress the longer tendons first.

5.9.5.6.7c—Blister and Rib Reinforcement

Reinforcement shall be provided throughout blisters or ribs as required for shear friction, corbel action, bursting forces, and deviation forces due to tendon curvature. This reinforcement shall extend as far as possible into the flange or web and be developed by standard hooks bent around transverse bars or equivalent. Spacing shall not exceed the smallest of blister or rib height at anchor, blister width, or 6.0 in.

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

Reinforcement, as specified in Article 5.9.5.6.5b, shall be provided to resist tensile forces due to transfer of the anchorage force from the blister or rib into the overall structure.

C5.9.5.6.7c

This reinforcement is normally provided in the form of ties or U-stirrups, which encase the anchorage and tie it effectively into the adjacent web and flange.

5.9.5.6.8—Diaphragms

For tendons anchored in diaphragms, concrete compressive stresses shall be limited within the diaphragm as specified in Article 5.9.5.6.5b. Compressive stresses shall also be investigated at the transition from the diaphragm to webs and flanges of the member.

Reinforcement shall be provided to ensure full transfer of diaphragm anchor loads into the flanges and

C5.9.5.6.8

Diaphragms anchoring post-tensioning tendons may be designed following the general guidelines of Schlaich et al. (1987), Breen and Kashima (1991), and Wollmann (1992). A typical diaphragm anchoring post-tensioning tendons usually behaves as a deep beam supported on three sides by the top and bottom flanges and the web wall. The magnitude of the bending tensile force on the face of the diaphragm opposite the anchor can be determined using the strut-and-tie method or elastic analysis. Approximate methods, such as the symmetric prism suggested by Guyon (1953), do not apply.

The more general methods of Article 5.8.2.7 or Article 5.8.3 are used to determine this reinforcement.

webs of the girder. Requirements for shear friction reinforcement between the diaphragm and web and between the diaphragm and flanges shall be checked.

Reinforcement shall also be provided to tie-back deviation forces due to tendon curvature.

5.9.5.6.9—Deviation Saddles

Deviation saddles shall be designed using the strut-and-tie method or using methods based on test results. A load factor of 1.7 shall be used with the maximum deviation force. If using a method based on test results, a resistance factor of 0.90 shall be used for direct tension and 0.85 shall be used for shear.

C5.9.5.6.9

Deviation saddles are disturbed regions of the structure and can be designed using the strut-and-tie method. Tests of scale-model deviation saddles have provided important information on the behavior of deviation saddles regions. Design and detailing guidelines are presented in Beaupre et al. (1988).

5.10—REINFORCEMENT

5.10.1—Concrete Cover

Cover for prestressing and reinforcing steel shall not be less than that specified in Table 5.10.1-1 and modified for *W*/*CM* ratio.

Modification factors for *W*/*CM* ratio shall be the following:

- For $W/CM \leq 0.40$ 0.8
- For $0.40 < W/CM < 0.50$ 1.0
- For $W/CM \geq 0.50$ 1.2

Concrete cover and placing tolerances shall be shown in the contract documents.

Cover for pretensioned prestressing strand, anchorage hardware, metal ducts for post-tensioned tendons, and mechanical connections for reinforcing bars or post-tensioned prestressing strands shall be the same as for reinforcing steel as specified in Table 5.10.1-1.

For decks exposed to tire studs or chain wear, additional cover shall be used to compensate for the expected loss in concrete depth due to abrasion, as specified in Article 2.5.2.4.

C5.10.1

Minimum cover is necessary for durability and prevention of splitting due to bond stresses and to provide for placing tolerance. Some Owners have cover requirements exceeding those in Table 5.10.1-1, especially for severe conditions such as proximity to coastal environments.

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 318-14, either or both the cover over the reinforcement is increased or the *W*/*CM* ratio is limited to 0.40. If materials, with reasonable use of admixtures, will produce a workable concrete with *W*/*CM* ratios lower than those listed in the *AASHTO LRFD Bridge Construction Specifications*, the contract documents should alter those recommendations appropriately.

The specified strengths shown in the *AASHTO Bridge LRFD Construction Specifications* are generally consistent with the *W*/*CM* ratios shown. However, it is possible to satisfy one without the other. Both are specified because *W*/*CM* 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.

The concrete cover modification factor used in conjunction with Table 5.10.1-1 recognizes the decreased permeability resulting from a lower *W*/*CM* ratio.

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Minimum cover to main bars shall be 1.0 in.

Cover to ties and stirrups may be 0.5 in. less than the values specified in Table 5.10.1-1 for main bars but shall not be less than 1.0 in. except for precast soffit form panels noted in the table below.

For bundled bars, the minimum concrete cover in noncorrosive atmosphere shall be equal to the equivalent diameter of the bundle, but need not be greater than 2.0 in.; except for concrete cast against and permanently exposed to noncorrosive soil, where the minimum cover shall be 3.0 in.

"Corrosive" water or soil contains greater than or equal to 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than or equal to 2,000 ppm and/or a pH of less than or equal to 5.5 shall be considered noncorrosive in determining minimum cover.

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Table 5.10.1-1—Minimum Cover for Main Reinforcing Steel (in.)

Situation	Reinforcing Material Category		
	A	B	C
Severe to Moderate Exposure			
Direct exposure to salt water	4.0	2.5	2.5
Cast against earth	3.0	2.0	2.0
Coastal	3.0	2.0	2.0
Exposure to deicing salts	2.5	2.0	1.5
Deck surfaces subject to tire stud or chain wear	2.5	2.5	2.0
Other than noted above	2.0	2.0	1.5
Limited Exposure			
Other than noted below			
• Up to No. 11 bar	1.5	1.0	1.0
• No. 14 and No. 18 bars	2.0	2.0	2.0
Bottom of cast-in-place slabs			
• Up to No. 11 bar	1.0	1.0	1.0
• No. 14 and No. 18 bars	2.0	2.0	2.0
Precast soffit form panels	0.8	0.8	0.8
Piling			
Precast reinforced piles			
• Noncorrosive environments	2.0	1.5	1.0
• Corrosive environments	3.0	2.5	2.0
Precast prestressed piles	2.0	1.0	1.0
Cast-in-place piles			
• Noncorrosive environments	2.0	1.5	1.5
• Corrosive environments	3.0	2.5	2.0
• Shells	2.0	1.5	1.0
• Auger-cast, tremie concrete, or slurry construction	3.0	2.5	2.0
Precast Culverts			
• Top slabs used as a driving surface	2.5	2.0	1.5
• Top slabs with less than 2.0 ft of fill	2.0	1.5	1.0
• All other members	1.0	1.0	1.0

Category A—Uncoated reinforcing steel meeting AASHTO M 31M/M 31
Category B—Epoxy coated or galvanized meeting ASTM A775/A775M
Category C—Materials meeting AASHTO M 334M/M 334

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:

C5.10.2.1

These requirements are similar to the requirements of ACI 318-14 and CRSI's *Manual of Standard Practice*.

- 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.

- 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 $6.0d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

Standard hooks may be used with reinforcing steel having a specified minimum yield strength between 75.0 and 100 ksi for elements and connections specified in Article 5.4.3.3 only if ties specified in Article 5.10.8.2.4 are provided.

5.10.2.2—Seismic Hooks

Seismic hooks meeting the requirements of Article 5.11.4.1.4 shall be used for transverse reinforcement in regions of expected plastic hinges and elsewhere as indicated 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 reinforcement 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.

Tests by Shahrooz et al. (2011) showed that standard hooks are adequate for reinforcement with specified minimum yield strengths between 75.0 and 100 ksi if transverse, confining reinforcement as specified in Article 5.10.8.2.4 is provided.

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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 the largest of the following:

- 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 largest of the following:

- 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 reinforcement 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.

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5.10.3.2—Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not be greater than the lesser of the following:

- 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 5.10.4, 5.10.5, and 5.10.6.

5.10.4—Transverse Reinforcement for Compression Members

5.10.4.1—General

The provisions of Article 5.10.8 shall also apply to design and detailing in Seismic Zones 2, 3, and 4.

Transverse reinforcement for compression members may consist of either spirals or ties.

For elements and connections specified in Article 5.4.3.3, spirals and ties may be designed for specified minimum yield strengths up to 100 ksi.

C5.10.4.1

Article 5.11.2 applies to Seismic Zone 1 but has no additional requirements for transverse reinforcement for compression members.

Spirals and ties with specified minimum yield strengths of up to 100 ksi are permitted in Seismic Zone 1 only, based on research by Shahrooz et al. (2011).

5.10.4.2—Spirals

Spiral reinforcement for compression members other than piles shall consist of one or more evenly spaced continuous spirals of either deformed or plain bar or wire with a minimum diameter of 0.375 in. The reinforcement shall be arranged so that all primary longitudinal reinforcement is contained on the inside of, and in contact with, the spirals.

The clear spacing between the bars of the spiral shall not be less than the greater of the following:

- 1.0 in.; or
- 1.33 times the maximum size of the aggregate.

The center-to-center spacing shall not exceed 6.0 times the diameter of the longitudinal bars or 6.0 in.

Except as specified in Articles 5.11.3 and 5.11.4.1 for Seismic Zones 2, 3, and 4, spiral reinforcement shall extend from the footing or other support to the level of the lowest horizontal reinforcement of the supported members.

Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of the spiral unit. For Seismic Zones 2, 3, and 4, the extension of transverse reinforcement into connecting members shall meet the requirements of Article 5.11.4.3.

Splices in spiral reinforcement may be one of the following:

- Lap splices of 48.0 uncoated bar diameters, 72.0 coated bar diameters, or 48.0 wire diameters;
- approved mechanical connectors; or
- approved welded splices.

5.10.4.3—Ties

The following requirements for transverse reinforcement may be used for normal weight concrete with design compressive strengths up to 15.0 ksi.

In tied compression members, all longitudinal bars or bundles shall be enclosed by lateral ties that shall be at least:

- No. 3 bars for No. 10 or smaller bars;
- No. 4 bars for No. 11 or larger bars; and
- No. 4 bars for bundled bars.

The spacing of ties along the longitudinal axis of the compression member with single bars or bundles of No. 9 bars or smaller shall not exceed the lesser of the following:

- The least dimension of the member; or
- 12.0 in.

Where two or more bars larger than No. 10 are bundled together, the spacing of ties along the longitudinal axis of the compression member shall not exceed the lesser of the following:

- Half the least dimension of the member; or
- 6.0 in.

Deformed wire or welded wire reinforcement of equivalent area may be used instead of bars.

C5.10.4.3

The revision to reduce the spacing, from 48.0 in. to 24.0 in., for laterally-restrained longitudinal bars or bundles is to bring the language back to the original intent of the 1980 AASHTO Standard Specifications Interim.

Figure C5.10.4.3-1 illustrates the placement of restraining ties in compression members which are not designed for plastic hinging. Cross-ties in locations not designed for plastic hinging should have standard hooks for transverse reinforcement.

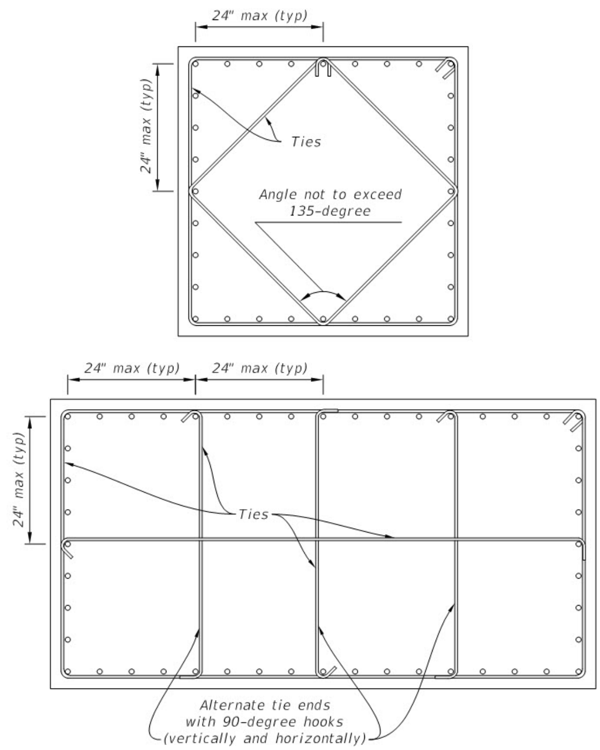

Figure C5.10.4.3-1—Acceptable Tie Arrangements in Locations not Designed for Plastic Hinging

For columns that are not designed for plastic hinging, the spacing of laterally restrained longitudinal bars or bundles shall not exceed 24.0 in. measured along the perimeter tie. A restrained bar or bundle is one that has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. Cross-ties with a 135-degree hook at one end and a 90-degree hook at the other end shall be alternated so that the 90-degree hooks are not adjacent to each other both vertically and horizontally.

Where the column design is based on plastic hinging capability, no longitudinal bar or bundle shall be farther than 6.0 in. clear on each side along the

Columns in Seismic Zones 2, 3, and 4 are designed for plastic hinging. The plastic hinge zone is defined in Article 5.11.4.1.3. Additional requirements for transverse reinforcement for bridges in Seismic Zones 2, 3, and 4 are specified in Articles 5.11.3 and 5.11.4.1. Plastic hinging may be used as a design strategy for other extreme events, such as ship collision.

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perimeter tie from such a laterally supported bar or bundle and the tie reinforcement shall meet the requirements of Articles 5.11.4.1.4 through 5.11.4.1.6.

Where the longitudinal bars or bundles are located around the periphery of a circle, a complete circular tie may be used with the splices in the circular ties staggered and without the need for cross-ties.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

5.10.5—Transverse Reinforcement for Flexural Members

Compression reinforcement in flexural members, except deck slabs, shall be enclosed by ties or stirrups satisfying the size and spacing requirements of Article 5.10.4 or by welded wire reinforcement of equivalent area.

5.10.6—Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement to ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

Reinforcement for shrinkage and temperature may be in the form of bars, welded wire reinforcement, or prestressing tendons.

For bars or welded wire reinforcement, the area of reinforcement per foot, on each face and in each direction, shall satisfy the following:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \tag{5.10.6-1}$$

except that:

$$0.11 \leq A_s \leq 0.60 \tag{5.10.6-2}$$

where:

A_s = area of reinforcement in each direction and each face (in.^{2}/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)

f_y = specified minimum yield strength of reinforcement ≤75.0 ksi

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing shall not exceed the following:

C5.10.6

The comparable equation in ACI 318-14 was written for slabs with the reinforcement being distributed equally to both surfaces of the slabs.

The requirements of this article are based on ACI 318-14 and 207.2R (2007). The coefficient in Eq. 5.10.6-1 is the product of 0.0018, 60.0 ksi, and 12.0 in./ft and, therefore, has the units kips/in.-ft.

Eq. 5.10.6-1 is written to show that the total required reinforcement, A_s = 0.0018bh, is distributed uniformly around the perimeter of the component. It provides a more uniform approach for components of any size. For example, a 30.0 ft high × 1.0 ft thick wall section requires 0.126 in.^{2}/ft in each face and each direction; a 4.0 ft × 4.0 ft component requires 0.260 in.^{2}/ft in each face and each direction; and a 5.0 ft × 20.0 ft footing requires 0.520 in.^{2}/ft in each face and each direction. For circular or other shapes, the equation becomes:

$$A_s \geq \frac{1.3A_g}{Perimeter(f_y)} \tag{C5.10.6-1}$$

Permanent prestress of 0.11 ksi is equivalent to the resistance of the steel specified in Eq. 5.10.6-1 at the strength limit state. The 0.11 ksi prestress should not be added to that required for the strength or service limit states. It is a minimum requirement for shrinkage and temperature crack control.

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- 12.0 in. for walls and footings greater than 18.0 in. thick
- 12.0 in. for other components greater than 36.0 in. thick
- For all other situations, 3.0 times the component thickness but not less than 18.0 in.

For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel shall not be required for:

- End face of walls 18.0 in. or less in thickness
- Side faces of buried footings 36.0 in. or less in thickness
- Faces of all other components, with smaller dimension less than or equal to 18.0 in.

If prestressing tendons are used as steel for shrinkage and temperature reinforcement, the tendons shall provide a minimum average compressive stress of 0.11 ksi on the gross concrete area through which a crack plane may extend, based on the effective prestress after losses. Spacing of tendons should not exceed either 72.0 in. or the distance specified in Article 5.9.4.2. Where the spacing is greater than 54.0 in., bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

The spacing of stress-relieving joints should be considered in determining the area of shrinkage and temperature reinforcement.

Surfaces of interior walls of box girders need not be considered to be exposed to daily temperature changes.

See also Article 12.14.5.8 for additional requirements for three-sided buried structures.

5.10.7—Reinforcement for Hollow Rectangular Compression Members

5.10.7.1—General

The area of longitudinal reinforcement in the cross section shall not be less than 0.01 times the gross area of concrete.

Two layers of reinforcement shall be provided in each wall of the cross section, one layer near each face of the wall. The areas of reinforcement in the two layers shall be approximately equal.

5.10.7.2—Spacing of Reinforcement

The center-to-center lateral spacing of longitudinal reinforcing bars shall be no greater than the lesser of 1.5 times the wall thickness or 18.0 in.

The center-to-center longitudinal spacing of lateral reinforcing bars shall be no greater than the lesser of 1.25 times the wall thickness or 12.0 in.

5.10.7.3—Ties

Cross-ties shall be provided between layers of reinforcement in each wall. The cross-ties shall include a standard 135-degree hook at one end and a standard 90-degree hook at the other end. Cross-ties shall be alternated so that hooks with the same bend angle are not adjacent to each other both vertically and

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horizontally. Cross-ties shall be located at bar grid intersections, and the hooks of all ties shall enclose both lateral and longitudinal bars at the intersections. Each longitudinal reinforcing bar and each lateral reinforcing bar shall be enclosed by the hook of a cross-tie at a spacing no greater than 24.0 in.

For segmentally constructed members, additional cross-ties shall be provided along the top and bottom edges of each segment. The cross-tie shall be placed so as to link the ends of each pair of internal and external longitudinal reinforcing bars in the walls of the cross section.

5.10.7.4—Splices

Lateral reinforcing bars may be joined at the corners of the cross section by overlapping 90-degree bends. Straight lap splices of lateral reinforcing bars shall not be permitted unless the overlapping bars are enclosed over the length of the splice by the hooks of at least four cross-ties located at intersections of the lateral bars and longitudinal bars.

5.10.7.5—Hoops

Where details permit, the longitudinal reinforcing bars in the corners of the cross section shall be enclosed by closed hoops. If closed hoops cannot be provided, pairs of U-shaped bars with legs at least twice as long as the wall thickness and oriented 90 degrees to one another may be used.

Post-tensioning ducts located in the corners of the cross section shall be anchored into the corner regions with closed hoops or stirrups having a 90-degree bend at each end to enclose at least one longitudinal bar near the outer face of the cross section.

5.10.8—Development and Splices of Reinforcement

5.10.8.1—General

The provisions of Articles 5.10.8.2.1, 5.10.8.2.4, and 5.10.8.4.3a are valid for No. 11 bars or smaller, in normal weight concrete with design concrete compressive strength (f'_c) of up to 15.0 ksi and lightweight concrete up to 10.0 ksi, subject to the limitations as specified in each of these articles.

C5.10.8.1

The extension of the development and splice length provisions to normal weight concrete with design concrete compressive strengths up to 15.0 ksi for f'_c is based on the work presented in NCHRP Report 603 (Ramirez and Russell, 2008). The research addressed both uncoated and epoxy-coated reinforcement with bar sizes up to No. 11.

5.10.8.1.1—Basic Requirements

This article shall be taken as applicable to nonprestressed reinforcement having a specified minimum yield strength up to 100 ksi for elements and connections specified in Article 5.4.3.3.

The calculated force effects in the reinforcement at each section shall be developed on each side of that section by embedment length, hook, or mechanical device, or a combination thereof. Hooks and mechanical

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anchorages may be used in developing bars in tension only.

5.10.8.1.2—Flexural Reinforcement

5.10.8.1.2a—General

Critical sections for development of reinforcement in flexural members shall be taken at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent.

Except at supports of simple spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than the greatest of the following:

- The effective depth of the member,
- 15 times the nominal diameter of bar, or
- $^1/_{20}$ of the clear span.

Continuing reinforcement shall extend not less than the development length, ℓ_d, specified in Article 5.10.8.2, beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

No more than 50 percent of the reinforcement shall be terminated at any section, and adjacent bars shall not be terminated in the same section.

Tension reinforcement may also be developed by either bending across the web in which it lies and terminating it in a compression area and providing the development length ℓ_d to the design section, or by making it continuous with the reinforcement on the opposite face of the member.

Supplementary anchorages shall be provided for tension reinforcement in flexural members where the reinforcement force is not directly proportional to factored moment as follows:

C5.10.8.1.2a

As a maximum, every other bar in a section may be terminated.

Past editions of the Standard Specifications required that flexural reinforcement not be terminated in a tension zone, unless one of the following conditions was satisfied:

- The factored shear force at the cutoff point did not exceed two thirds of the factored shear resistance, including the shear strength provided by the shear reinforcement.
- Stirrup area in excess of that required for shear and torsion was provided along each terminated bar over a distance from the termination point not less than three-fourths the effective depth of the member. The excess stirrup area, A_v, was not less than 0.06 $b_w s/f_y$. Spacing, s, did not exceed $0.125d/\beta_b$, where β_b was the ratio of the area of reinforcement cutoff to the total area of tension reinforcement at the section.
- For No. 11 bars and smaller, the continuing bars provided double the area required for flexure at the cutoff point, and the factored shear force did not exceed three fourths of the factored shear resistance.

These provisions are now supplemented by the provisions of Article 5.7, which account for the need to provide longitudinal reinforcement to react the horizontal component of inclined compression diagonals that contribute to shear resistance.

Supplementary anchorages may take the form of hooks or welding to anchor bars.

- sloped, stepped, or tapered footings;
- brackets;
- deep flexural members; or
- members in which tension reinforcement is not parallel to the compression face.

5.10.8.1.2b—Positive Moment Reinforcement

At least one-third the positive moment reinforcement in simple span members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6.0 in.

C5.10.8.1.2b

Past editions of the Standard Specifications required that at end supports and at points of inflection, positive moment tension reinforcement be limited to a diameter such that the development length, ℓ_d, determined for f_y by Article 5.10.8.2.1, satisfied Eq. C5.10.8.1.2b-1:

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \qquad \text{(C5.10.8.1.2b-1)}$$

where:

M_n = nominal flexural resistance, assuming all positive moment tension reinforcement at the section to be stressed to the specified yield strength f_y (kip-in.)
V_u = factored shear force at the section (kip)
ℓ_a = embedment length beyond the center of a support or at a point of inflection; taken as the greater of the effective depth of the member and 12.0 d_b (in.)

Eq. C5.10.8.1.2b-1 does not have to be satisfied for reinforcement terminating beyond the centerline of end supports by either a standard hook or a mechanical anchorage at least equivalent to a standard hook.

The value M_n/V_u in Eq. C5.10.8.1.2b-1 was to be increased by 30 percent for the ends of the reinforcement located in an area where a reaction applies transverse compression to the face of the beam under consideration.

The intent of the 30 percent provision is illustrated in Figure C5.10.8.1.2b-1.

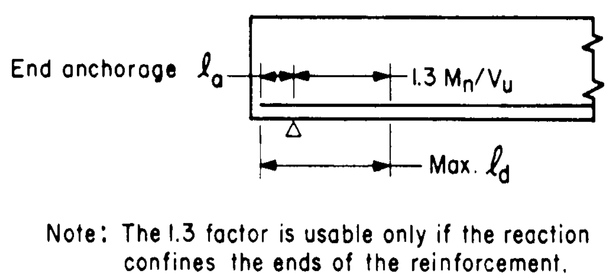

Figure C5.10.8.1.2b-1—End Confinement

These provisions are now supplemented by the provisions of Article 5.7, which account for the need to provide longitudinal reinforcement to resist the horizontal component of inclined compression diagonals that contribute to shear resistance.

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Reinforcement with specified yield strengths in excess of 75.0 ksi may require longer extensions than required by this article.

5.10.8.1.2c—Negative Moment Reinforcement

At least one third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than the greatest of the following:

- The effective depth of the member;
- 12.0 times the nominal diameter of bar; and
- $^{1}/_{16}$ of the clear span.

5.10.8.1.2d—Moment Resisting Joints

Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint.

In Seismic Zones 3 and 4, joints shall be detailed to resist moments and shears resulting from horizontal loads through the joint.

C5.10.8.1.2d

Reinforcing details for developing continuity through joints are suggested in the *ACI Detailing Manual* (ACI SP66, 2004).

5.10.8.2—Development of Reinforcement

Development lengths shall be calculated using the specified minimum yield strength of the reinforcement. Use of nonprestressed reinforcement with a specified minimum yield strength up to 100 ksi may be permitted for elements and connections specified in Article 5.4.3.3.

C5.10.8.2

Most of the provisions in the Article are adapted from ACI 318-14 and its attendant commentary. In addition, results of NCHRP Report 603 on Transfer, Development, and Splice Length for Strand/Reinforcement in High Strength Concrete (Ramirez and Russell, 2008) are incorporated to include applications with compressive strengths of concrete used in design up to 15.0 ksi. The NCHRP 603 Report examined an extensive database of previous tests compiled by ACI Committee 408. Previous tests (Azizinamini et al., 1993 and 1999) had indicated that in the case of concrete with compressive strengths between 10.0 and 15.0 ksi, a minimum amount of transverse reinforcement was needed to ensure yielding of reinforcement splices of bars with less than 12.0 in. of concrete placed below them. Although NCHRP Report 603 recommended replacing the minimum transverse reinforcement with a development modification factor of 1.2, a conservative value of 1.3 is used in this article. The bar size factor of 0.8 for No. 6 and smaller bars was recommended to be removed to generalize application to concrete strength higher than 10.0 ksi. The procedure described here is more conservative for reinforcement with yield strengths greater than 60.0 ksi than for those presented in recently published reports such as Hosny et al. (2012) and Darwin et al. (2005).

Research by Shahrooz et al. (2011) showed that calculated tensile splice lengths and calculated tensile development lengths for both straight bars and standard hooks are adequate for applications in Seismic Zone 1

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for reinforcement with yield strengths up to 100 ksi combined with design concrete compressive strengths up to 15.0 ksi.

5.10.8.2.1—Deformed Bars and Deformed Wire in Tension

The provisions herein may be used for No. 11 bars and smaller in normal weight concrete with a compressive strength of concrete for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi. Transverse reinforcement consisting of at least No. 3 bars at 12.0-in. centers shall be provided along the required development length where the design concrete compressive strength is greater than 10.0 ksi.

For straight bars having a specified minimum yield strength greater than 75.0 ksi, transverse reinforcement satisfying the requirements of Article 5.7.2.5 for beams and Article 5.10.4.3 for columns shall be provided over the required development length.

C5.10.8.2.1

The extension of this article to design concrete compressive strengths between 10.0 and 15.0 ksi is limited to No. 11 bars and smaller based on the work presented in NCHRP Report 603 (Ramirez and Russell, 2008). The requirement for minimum transverse reinforcement along the development length is based on research by Azizinamini et al. (1999). Transverse reinforcement used to satisfy the shear requirements may simultaneously satisfy this provision.

Confining reinforcement is not required in bridge slabs or decks.

5.10.8.2.1a—Tension Development Length

The modified tension development length, ℓ_d, shall not be less than the basic tension development length, ℓ_{db}, specified herein adjusted by the modification factor or factors specified in Articles 5.10.8.2.1b and 5.10.8.2.1c. The tension development length shall not be less than 12.0 in., except for development of shear reinforcement specified in Article 5.10.8.2.6.

The modified tension development length, ℓ_d, in in. shall be taken as:

$$\ell_d = \ell_{db} \times \left(\frac{\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc} \times \lambda_{er}}{\lambda} \right) \qquad (5.10.8.2.1a\text{-}1)$$

in which:

$$\ell_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}} \qquad (5.10.8.2.1a\text{-}2)$$

where:

ℓ_{db} = basic development length (in.)
λ_{rl} = reinforcement location factor
λ_{cf} = coating factor
λ_{rc} = reinforcement confinement factor
λ_{er} = excess reinforcement factor
λ = concrete density modification factor as specified in Article 5.4.2.8
d_b = nominal diameter of reinforcing bar or wire (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
f'_c = compressive strength of concrete for use in design (ksi)

Modification factors shall be applied to the basic development length to account for the various effects specified herein. They shall be taken equal to 1.0 unless they are specified to increase ℓ_d in Article 5.10.8.2.1b, or to decrease ℓ_d in Article 5.10.8.2.1c.

5.10.8.2.1b—Modification Factors which Increase ℓ_d

The basic development length, ℓ_{db}, shall be modified by the following factor or factors, as applicable:

- For horizontal reinforcement, placed such that more than 12.0 in. of fresh concrete is cast below the reinforcement, $\lambda_{rl} = 1.3$.
- For horizontal reinforcement, placed such that no more than 12.0 in. of concrete is cast below the reinforcement and f'_c is greater than 10.0 ksi, $\lambda_{rl} = 1.3$.
- For lightweight concrete, use λ as specified in Article 5.4.2.8.
- For epoxy-coated bars with cover less than $3d_b$ or with clear spacing between bars less than $6d_b$, $\lambda_{cf} = 1.5$.
- For epoxy-coated bars not covered above, $\lambda_{cf} = 1.2$.

The product $\lambda_{rl} \times \lambda_{cf}$ need not be taken greater than 1.7.

C5.10.8.2.1b

For horizontal reinforcement, placed such that no more than 12.0 in. of concrete is cast below the reinforcement and f'_c is not greater than 10.0 ksi, no modification factor is necessary or λ_{rl} could be said to be equal to 1.0.

5.10.8.2.1c—Modification Factors which Decrease ℓ_d

The basic development length, ℓ_{db}, specified in Article 5.10.8.2.1a, modified by the factors specified in Article 5.10.8.2.1b, as appropriate, may be multiplied by the following factors:

- For reinforcement being developed in the length under consideration, λ_{rc} shall satisfy the following:

$$0.4 \leq \lambda_{rc} \leq 1.0 \tag{5.10.8.2.1c-1}$$

in which:

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} \tag{5.10.8.2.1c-2}$$

$$k_{tr} = 40A_{tr}/(sn) \tag{5.10.8.2.1c-3}$$

where:

c_b = the smaller of the distance from center of bar or wire being developed to the nearest concrete surface and one-half the center-to-center spacing of the bars or wires being developed (in.)

k_{tr} = transverse reinforcement index

C5.10.8.2.1c

The provisions in this article are adapted from ACI 318-14.

The parameters c_b and n in Eqs. 5.10.8.2.1c-2 and 5.10.8.2.1c-3, as well as assumed crack locations, are shown in Figure C5.10.8.2.1c-1.

In tests to determine development lengths, splitting cracks have been observed to occur along the bars being developed as illustrated in Figure C5.10.8.2.1c-1. When the center-to-center spacing of the bars is greater than about twice the distance from the center of the bar to the concrete surface, splitting cracks occur between the bars and the concrete surface. When the center-to-center spacing of the bars is less than about twice the distance from the center of the bar to the concrete surface, splitting cracks occur between the bars along the plane of the bars being developed. The presence of bars crossing the plane of splitting, as denoted by A_{tr}, controls these splitting cracks and results in shorter development lengths.

In any member, A_{tr} may be taken conservatively as zero in Eq. 5.10.8.2.1c-3. When $c_b > 2.5$ in. and there are no bars crossing the plain of splitting, $A_{tr} = 0$ and $\lambda_{rc} = 0.4$ for bar sizes of No. 8 and smaller. Otherwise, λ_{rc} is calculated using Eq. 5.10.8.2.1c-1.

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A_{tr} = total cross-sectional area of all transverse reinforcement which is within the spacing s and which crosses the potential plane of splitting through the reinforcement being developed (in.^{2})

s = maximum center-to-center spacing of transverse reinforcement within ℓ_d (in.)

n = number of bars or wires developed along plane of splitting

- Where anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis:

$$\lambda_{er} = \frac{\text{Required } A_s}{\text{Provided } A_s} \quad (5.10.8.2.1c\text{-}4)$$

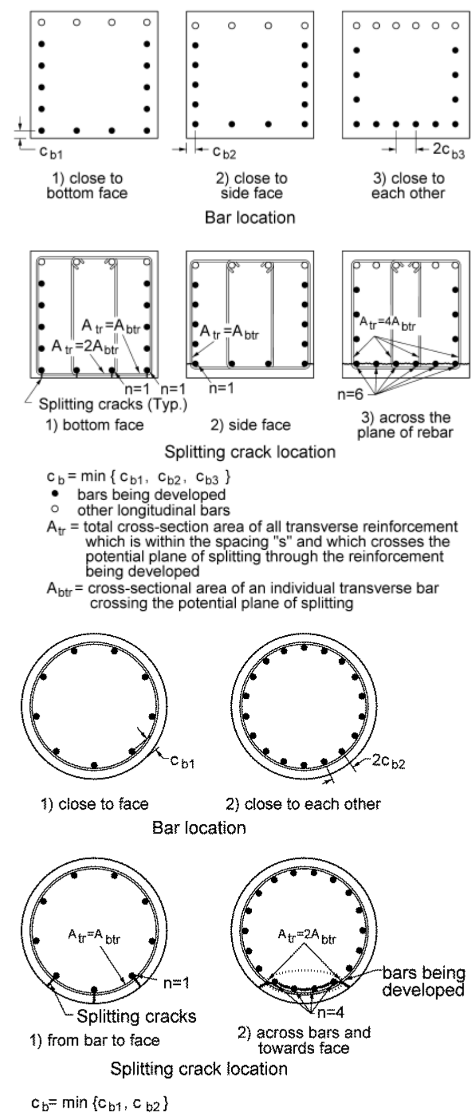

Figure C5.10.8.2.1c-1—Parameters for Determining Development Length Modifier, λ_{rc}

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5.10.8.2.2—Deformed Bars in Compression

5.10.8.2.2a—Compressive Development Length

The modified development length, ℓ_d, for deformed bars in compression shall not be less than either the basic development length, ℓ_{db}, specified herein modified by the applicable modification factors specified in Article 5.10.8.2.2b, or 8.0 in.

The modified compressive development length, ℓ_{db}, for deformed bars shall be taken as:

$$\ell_d = \ell_{db}\lambda_{er}\lambda_{rc} \qquad (5.10.8.2.2a\text{-}1)$$

The basic development length shall be larger than the greater of the following:

$$\ell_{db} \geq \frac{0.63\, d_b f_y}{\sqrt{f'_c}} \qquad (5.10.8.2.2a\text{-}2)$$

$$\ell_{db} \geq 0.3\, d_b f_y \qquad (5.10.8.2.2a\text{-}3)$$

where:

d_b = nominal diameter of reinforcing bar (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
f'_c = compressive strength of concrete for use in design (ksi)

5.10.8.2.2b—Modification Factors

The basic development length, ℓ_{db}, may be multiplied by applicable factors, where:

- Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement is provided in excess of that required by analysis, $\lambda_{er} = \dfrac{(\text{Required } A_s)}{(\text{Provided } A_s)}$
- Reinforcement is enclosed within a spiral composed of a bar of not less than 0.25 in. in diameter and spaced at not more than a 4.0 in. pitch, $\lambda_{rc} = 0.75$

5.10.8.2.3—Bundled Bars

The development length of individual bars within a bundle, in tension or compression shall be that for the individual bar, increased by 20 percent for a three-bar bundle and by 33 percent for a four-bar bundle.

For determining the factors specified in Articles 5.10.8.2.1b and 5.10.8.2.1c, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

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5.10.8.2.4—Standard Hooks in Tension

The provisions herein may be used for No. 11 bars or smaller in normal weight concrete with a compressive strength of concrete for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.

For hooks in reinforcing bars having a specified minimum yield strength greater than 75.0 ksi, ties satisfying the requirements of Article 5.10.8.2.4c shall be provided. For hooks not located at the discontinuous end of a member, the modification factors of Article 5.10.8.2.4b may be applied.

C5.10.8.2.4

Article 5.10.8.2.4 was verified for compressive strengths of concrete used in design up to 15.0 ksi in NCHRP Report 603 with the exception of the lightweight concrete factor. The previous limit of 10.0 ksi has been retained for lightweight concrete. Based on the analysis of NCHRP Report 603 and of tests of additional specimens reported in the literature, the approach in ACI 318-14 for anchorage of bars terminated with standard hooks, black and epoxy-coated, can be extended to normal weight concrete with compressive strengths of up to 15.0 ksi. NCHRP Report 603 recommends a minimum amount of transverse reinforcement consisting of at least No. 3 U bars at $3d_b$ spacing to improve the bond strength of No. 11 and larger bars in tension anchored by means of standard hooks. A modification factor of 0.8 instead of the previous factor of 0.7 was found to be adequate for No. 11 and smaller hooks with side cover not less than 2.5 in., and for 90 degree hooks with cover on bar extension beyond the hook not less than 2.0 in. Similar to the provisions of ACI 318-14, hooks are not considered effective in developing bars in compression.

5.10.8.2.4a—Basic Hook Development Length

The modified development length, ℓ_{dh}, in in., for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall be determined as the basic development length of standard hook in tension, ℓ_{hb}, adjusted by the applicable modification factors specified in Article 5.10.8.2.4b, but shall not be taken less than the greater of the following:

- 8.0 bar diameters; and
- 6.0 in.

The modified development length, ℓ_{dh}, of a standard hook in tension shall be taken as:

$$\ell_{dh} = \ell_{hb} \times \left(\frac{\lambda_{rc}\lambda_{cw}\lambda_{er}}{\lambda} \right) \tag{5.10.8.2.4a-1}$$

in which:

$$\ell_{hb} = \frac{38.0 d_b}{60.0} \left(\frac{f_y}{\sqrt{f'_c}} \right) \tag{5.10.8.2.4a-2}$$

where:

ℓ_{hb} = basic development length (in.)
λ_{rc} = reinforcement confinement factor
λ_{cw} = coating factor
λ_{er} = excess reinforcement factor
λ = concrete density modification factor as specified in Article 5.4.2.8

C5.10.8.2.4a

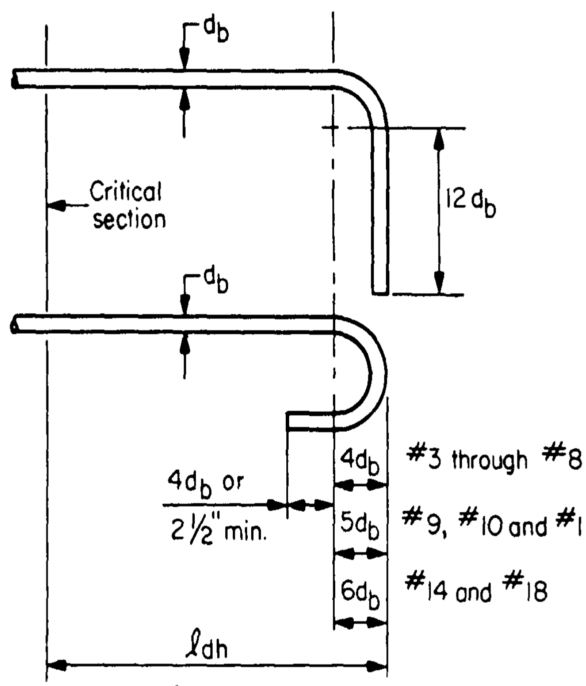

Figure C5.10.8.2.4a-1—Hooked Bar Details for Development of Standard Hooks (ACI Committee 318 2011)

d_b = nominal diameter of reinforcing bar or wire (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
f'_c = compressive strength of concrete for use in design not to be taken greater than 15.0 ksi for normal weight concrete and 10.0 ksi for lightweight concrete (ksi)

5.10.8.2.4b—Modification Factors

Basic development length of a standard hook in tension, ℓ_{hb}, shall be modified by the following factor or factors, as applicable:

- For lightweight concrete, with a design compressive strength not exceeding 10.0 ksi, λ as specified in Article 5.4.2.8.
- For epoxy-coated reinforcement, $\lambda_{cw} = 1.2$
- For No. 11 bar and smaller, hooks with side cover normal to plane of the hook not less than 2.5 in., and for 90-degree hook with cover on the bar extension beyond hook not less than 2.0 in., $\lambda_{rc} = 0.8$
- For 90-degree hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length, ℓ_{dh}, of the hook; or enclosed within ties or stirrups parallel to the bar being developed spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend, and in both cases the first tie or stirrup enclosing the bent portion of the hook is within $2d_b$ of the outside of the bend, $\lambda_{rc} = 0.8$
- For 180 deg hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length, ℓ_{dh}, of the hook, and the first tie or stirrup enclosing the bent portion of the hook is within $2d_b$ of the outside of the bend, $\lambda_{rc} = 0.8$
- For anchorage or development where full yield strength is not required, or where reinforcement is provided in excess of that required by analysis, $\lambda_{er} = \dfrac{(\text{Required } A_s)}{(\text{Provided } A_s)}$

5.10.8.2.4c—Hooked-Bar Tie Requirements

For bars being developed by a standard hook at discontinuous ends of members with both side cover and top or bottom cover less than 2.5 in., the hooked-bar shall be enclosed within ties or stirrups spaced along the full development length, ℓ_{dh}, not greater than $3d_b$ as shown in Figure 5.10.8.2.4c-1. The factor for transverse

C5.10.8.2.4b

The provisions in this article are adapted from ACI 318-14.

Confinement of hooked bars by stirrups perpendicular and parallel to the bar being developed is illustrated in Figure C5.10.8.2.4b-1.

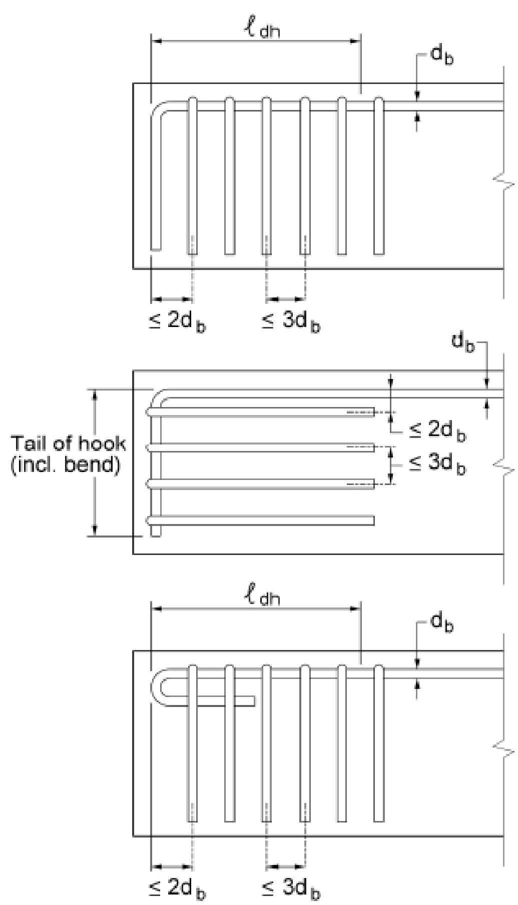

Figure C5.10.8.2.4b-1—Confinement of Hooked Bars by Stirrups

reinforcement, as specified in Article 5.10.8.2.4b, shall not apply.

For normal weight concrete with design concrete compressive strengths between 10.0 and 15.0 ksi, the development length of the hooked bars shall be enclosed with No. 3 bars or larger ties or stirrups along the full development, ℓ_{dh}, at a spacing not greater than $3d_b$. A minimum of three ties or stirrups shall be provided.

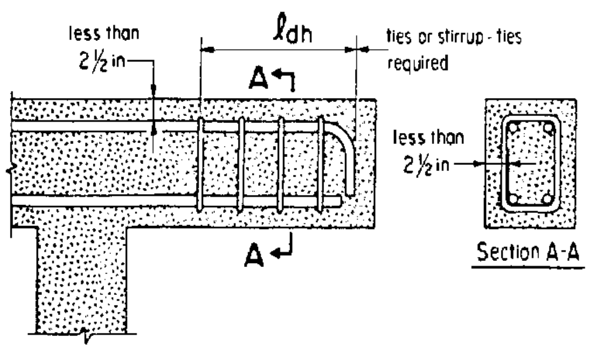

Figure 5.10.8.2.4c-1—Hooked-Bar Tie Requirements

5.10.8.2.5—Welded Wire Reinforcement

For applications other than shear reinforcement, the modified development length, ℓ_d, in inches, for deformed or plain welded wire reinforcement shall be determined as the basic development length, ℓ_{db}, adjusted by the modification factors specified in Eq. 5.10.8.2.5-1, but shall not be taken as less than the greatest of the following:

Deformed welded wire reinforcement:

- The distance that will include embedment of one cross wire with the cross wire not less than 2.0 in. from the point of critical section; or
- 8.0 in.

Plain welded wire reinforcement:

- The distance between two cross wires plus the distance from the critical section to the closer cross wire, where the closer cross wire is not less than 2.0 in. from the point of critical section; or
- 6.0 in.

The modified development length, ℓ_d, in inches, of deformed or plain welded wire reinforcement, measured from the point of critical section to the end of wire, shall be taken as:

$$\ell_d = \ell_{db} \times \left(\frac{\lambda_{er}}{\lambda}\right) \quad (5.10.8.2.5\text{-}1)$$

C5.10.8.2.5

Figure C5.10.8.2.5-1 shows the development requirements for deformed welded wire reinforcement within one cross wire within the development length. ASTM A1064 for deformed welded wire reinforcement requires the same strength of the weld as required for plain welded wire reinforcement. Some of the development is assigned to welds and some assigned to the length of deformed wire. For deformed welded wire reinforcement made with smaller wires, an embedment of at least one cross wire 2.0 in. or more beyond the point of critical section is adequate to develop the full yield strength of the anchored wires. However, for deformed welded wire reinforcement made with larger closely spaced wires, a longer embedment is required and a minimum development length is provided for this reinforcement.

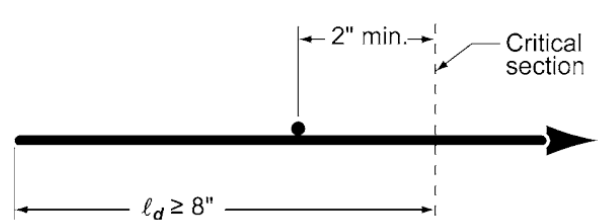

Figure C5.10.8.2.5-1—Development of Deformed Welded Wire Reinforcement (adapted from ACI 318-14, courtesy of ACI)

Figure C5.10.8.2.5-2 shows the development requirements for plain welded wire reinforcement with development primarily dependent on the location of cross wires. For plain welded wire reinforcement made

The basic development length, ℓ_{db}, for deformed welded wire reinforcement, shall be taken as the greater of:

$$\ell_{db} \geq 0.95 d_b \frac{f_y - 20.0}{\sqrt{f'_c}} \qquad (5.10.8.2.5\text{-}2)$$

$$\ell_{db} \geq 6.30 \frac{A_w f_y}{s_w \sqrt{f'_c}} \qquad (5.10.8.2.5\text{-}3)$$

The basic development length, ℓ_{db}, for plain welded wire reinforcement, shall be taken as:

$$\ell_{db} = 8.50 \frac{A_w f_y}{s_w \sqrt{f'_c}} \qquad (5.10.8.2.5\text{-}4)$$

where:

λ_{er} = excess reinforcement factor as specified in Article 5.10.8.2.1c
λ = concrete density modification factor as specified in Article 5.4.2.8
d_b = nominal diameter of reinforcing bar or wire (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
f'_c = compressive strength of concrete for use in design not to be taken greater than 15.0 ksi for normal weight concrete and 10.0 ksi for lightweight concrete (ksi)
A_w = area of individual wire to be developed or spliced (in.^{2})
s_w = spacing of wires to be developed or spliced (in.)

The basic development length of deformed welded wire reinforcement, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 5.10.8.2.1a.

For plain welded wire reinforcement development, use of two cross wires is required.

The development of shear reinforcement shall be taken as specified in Article 5.10.8.2.6.

5.10.8.2.6—Shear Reinforcement

5.10.8.2.6a—General

Stirrup reinforcement in concrete pipe shall satisfy the provisions of Article 12.10.4.2.7 and shall not be required to satisfy the provisions herein.

Shear reinforcement shall be located as close to the surfaces of members as cover requirements and proximity of other reinforcement permit.

Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

with the smaller wires, an embedment of at least two cross wires, 2.0 in. or more beyond the point of critical section is adequate to develop the full yield strength of the anchored wires. However, for plain welded wire reinforcement made with larger closely spaced wires, a longer embedment is required and a minimum development length is provided for this reinforcement.

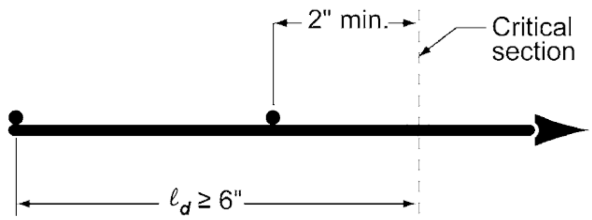

Figure C5.10.8.2.5-2—Development of Plain Welded Wire Reinforcement (adapted from ACI 318-14, courtesy of ACI)

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Longitudinal bars bent to act as transverse reinforcement, if extended into a region of tension, shall be continuous with the longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond the mid-depth, $h/2$, as specified for development length for that part of the stress in the reinforcement required to satisfy Eq. 5.7.3.3-5.

5.10.8.2.6b—Anchorage of Deformed Reinforcement

Ends of single-leg, simple U-, or multiple U-stirrups shall be anchored as follows:

- For No. 5 bar and D31 wire, and smaller—A standard hook around longitudinal reinforcement, and
- For No. 6, No. 7 and No. 8—A standard stirrup hook around a longitudinal bar, plus one embedment length between midheight of the member and the outside end of the hook, ℓ_e shall satisfy:

$$\ell_e \geq \frac{0.44\, d_b\, f_y}{\lambda\sqrt{f'_c}} \qquad (5.10.8.2.6b\text{-}1)$$

where:

d_b = nominal diameter of reinforcing bar or wire (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
λ = concrete density modification factor as specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design not to be taken greater than 15.0 ksi for normal weight concrete and 10.0 ksi for lightweight concrete (ksi)

5.10.8.2.6c—Anchorage of Wire Fabric Reinforcement

C5.10.8.2.6c

Each leg of welded plain wire reinforcement forming simple U-stirrups shall be anchored by:

- Two longitudinal wires spaced at 2.0 in. along the member at the top of the U; or
- One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2.0 in. from the first wire. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of bend not less than $8d_b$.

For each end of a single-leg stirrup of welded plain or deformed wire reinforcement, two longitudinal wires at a minimum spacing of 2.0 in. and with the inner wire at not less than $d/4$ or 2.0 in. from mid-depth of member shall be provided. The outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

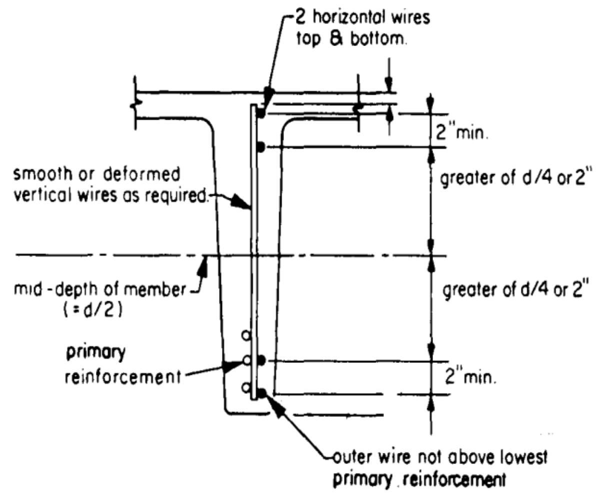

Figure C5.10.8.2.6c-1—Anchorage of Single-Leg Welded Wire Reinforcement Shear Reinforcement, ACI 318-14

5.10.8.2.6d—Closed Stirrups

Pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where length of laps are not less than 1.3 ℓ_d, where ℓ_d in this case is the development length for bars in tension.

In members not less than 18.0 in. deep, closed stirrup splices in stirrup legs extending the full available depth of the member, and with the tension force resulting from factored loads, $A_b f_y$, not exceeding 9.0 kips per leg, may be considered adequate.

Transverse torsion reinforcement shall be made fully continuous and shall be anchored by 135-degree standard hooks around longitudinal reinforcement.

5.10.8.3—Development by Mechanical Anchorages

Any mechanical device capable of developing the strength of reinforcement without damage to concrete may be used as an anchorage. Performance of mechanical anchorages shall be verified by laboratory tests.

Development of reinforcement may consist of a combination of mechanical anchorage and the additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

If mechanical anchorages are to be used, complete details shall be shown in the contract documents.

C5.10.8.3

Standard details for such devices have not been developed.

5.10.8.4—Splices of Bar Reinforcement

Reinforcement with specified minimum yield strengths up to 100 ksi may be used in elements and connections specified in Article 5.4.3.3. For spliced bars having a specified minimum yield strength greater than 75.0 ksi, transverse reinforcement satisfying the

C5.10.8.4

Confining reinforcement is not required in slabs or decks.

Research by Shahrooz et al. (2011) verified the use of these provisions for tensile splices for reinforcement with specified minimum yield strengths up to 100 ksi in requirements of Article 5.7.2.5 for beams and Article 5.10.4.3 for columns shall be provided over the required splice length.

applications in Seismic Zone 1. See Article C5.4.3.3 for further information.

5.10.8.4.1—Detailing

Permissible locations, types, and dimensions of splices, including staggers, for reinforcing bars shall be shown in the contract documents.

5.10.8.4.2—General Requirements

5.10.8.4.2a—Lap Splices

This provision of this article shall apply only to the grades of reinforcement noted.

The lengths of lap for lap splices of individual bars shall be as specified in Articles 5.10.8.4.3a and 5.10.8.4.5a.

Lap splices within bundles shall be as specified in Article 5.10.8.2.3. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

For reinforcement in tension, lap splices shall not be used for bars larger than No. 11.

Bars spliced by noncontact lap splices in flexural members shall not be spaced farther apart transversely than the lesser of the following:

- one-fifth the required lap splice length; or
- 6.0 in.

For columns with longitudinal reinforcement that anchors into oversized shafts, where bars are spliced by noncontact lap splices, and longitudinal column and shaft reinforcement are spaced farther apart transversely than the greater of the following:

- one-fifth the required lap splice length; or
- 6.0 in.,

the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation:

$$S_{\max} = \frac{2\pi A_{sp} f_{ytr} \ell_s}{k A_\ell f_{u\ell}} \qquad (5.10.8.4.2a\text{-}1)$$

where:

$S_{\max}$ = spacing of transverse shaft reinforcement (in.)
A_{sp} = area of shaft spiral or transverse reinforcement (in.^{2})
f_{ytr} = specified minimum yield strength of shaft transverse reinforcement (ksi)
ℓ_s = required tension lap splice length of the column longitudinal reinforcement (in.)
k = factor representing the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance

C5.10.8.4.2a

This ratio, k, could be determined from the column moment-curvature analysis using appropriate computer programs. For simplification, k = 0.5 could safely be used in most applications.

The development length of column longitudinal reinforcement in drilled shafts is from WSDOT-TRAC Report WA-RD 417.1 titled *Noncontact Lap Splices in Bridge Column-Shaft Connections*. Eq. 5.10.8.4.2a-1 is based upon a strut-and-tie analogy of the noncontact splice with an assumed strut angle of 45 degrees.

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A_ℓ = area of longitudinal column reinforcement (in.^{2})

$f_{u\ell}$ = specified minimum tensile strength of column longitudinal reinforcement (ksi), 90.0 ksi for ASTM A615 and 80.0 ksi for ASTM A706

5.10.8.4.2b—Mechanical Connections

The resistance of a full-mechanical connection shall not be less than 125 percent of the specified yield strength of the bar in tension or compression, as required. The total slip of the bar within the splice sleeve of the connector after loading in tension to 30.0 ksi and relaxing to 3.0 ksi shall not exceed the following measured displacements between gauge points clear of the splice sleeve:

- For bar sizes up to No. 14 .... 0.01 in.
- For No. 18 bars .... 0.03 in.

5.10.8.4.2c—Welded Splices

Welding for welded splices shall conform to the current edition of *Structural Welding Code—Reinforcing Steel of AWS* (D1.4).

A full-welded splice shall be required to develop, in tension, at least 125 percent of the specified yield strength of the bar.

No welded splices shall be used in decks.

5.10.8.4.3—Splices of Reinforcement in Tension

The provisions herein may be used for No. 11 bars or smaller in normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi. Transverse reinforcement consisting of at least No. 3 bars at 12.0-in. centers shall be provided along the required splice length where the design concrete compressive strength is greater than 10.0 ksi. A minimum of three bars shall be provided.

C5.10.8.4.3

The tension development length, ℓ_d, used as a basis for calculating splice lengths should include all of the modification factors specified in Article 5.10.8.2.

The extension of this article to design concrete compressive strengths between 10.0 and 15.0 ksi is limited to No. 11 bars and smaller based on the work presented in NCHRP Report 603 (Ramirez and Russell, 2008). The requirement for minimum transverse reinforcement along the splice length is based on research by Azizinamini et al. (1999). Transverse reinforcement used to satisfy the shear requirements may simultaneously satisfy this provision.

5.10.8.4.3a—Lap Splices in Tension

The minimum length of lap for tension lap splices shall be as required for Class A or B lap splice, but not less than 12.0 in., where:

Class A splice……………………………….1.0ℓ_d

Class B splice……………………………… 1.3ℓ_d

The tension development length, ℓ_d, for the specified yield strength shall be taken in accordance with Article 5.10.8.2.1a.

C5.10.8.4.3a

Research by Shahrooz et al. (2011) verified these provisions for applications in Seismic Zone I for reinforcement with specified minimum yield strengths up to 100 ksi combined with design concrete compressive strengths up to 15.0 ksi. See Article C5.4.3.3 for further information.

Tension lap splices were evaluated under NCHRP Report 603. Splices of bars in compression were not part of the experimental component of the research. Class C lap splices were eliminated based on the modifications to development length provisions.

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Except as specified herein, lap splices of deformed bars and deformed wire in tension shall be Class B lap splices. Class A lap splices may be used where:

- the area of reinforcement provided is at least twice that required by analysis over the entire length of the lap splice; and
- one-half or less of the total reinforcement is spliced within the required lap splice length.

For splices having f_y > 75.0 ksi, transverse reinforcement satisfying the requirements of Article 5.7.2.5 in beams and Article 5.10.4.3 in columns shall be provided over the required lap splice length.

5.10.8.4.3b—Mechanical Connections or Welded Splices in Tension

Mechanical connections or welded tension splices, used where the area of reinforcement provided is less than twice that required, shall meet the requirements for full-mechanical connections or full-welded splices.

Mechanical connections or welded splices, used where the area of reinforcement provided is at least twice that required by analysis and where the splices are staggered at least 24.0 in., may be designed to develop not less than either twice the tensile force effect in the bar at the section or half the minimum specified yield strength of the reinforcement.

C5.10.8.4.3b

In determining the tensile force effect developed at each section, spliced reinforcement may be considered to resist the specified splice strength. Unspliced reinforcement may be considered to resist the fraction of f_y defined by the ratio of the shorter actual development length to the development length, ℓ_d, required to develop the specified yield strength f_y.

5.10.8.4.4—Splices in Tie Members

Splices of reinforcement in tie members shall be made only with either full-welded splices or full-mechanical connections. Splices in adjacent bars shall be staggered not less than 30.0 in. apart.

C5.10.8.4.4

A tie member is assumed to have:

- an axial tensile force sufficient to create tension over the cross section, and
- a level of stress in the reinforcement such that every bar is fully effective.

Examples of members that may be classified as tension ties are arch ties, hangers carrying load to an overhead supporting structure, and main tension components in a truss.

5.10.8.4.5—Splices of Bars in Compression

5.10.8.4.5a—Lap Splices in Compression

C5.10.8.4.5a

The length of lap, ℓ_c, for compression lap splices shall not be less than both 12.0 in. and whichever of the following is appropriate:

- If $f_y \leq 60.0$ ksi then:

$$\ell_c = 0.5 m f_y d_b \qquad (5.10.8.4.5a\text{-}1)$$

- If $f_y > 60.0$ ksi then:

$$\ell_c = m\left(0.9 f_y - 24.0\right) d_b \qquad (5.10.8.4.5a\text{-}2)$$

in which:

- Where the compressive strength of concrete used in design, f'_c, is less than 3.0 ksi .... m = 1.33
- Where ties along the splice have an effective area not less than 0.15 percent of the product of the thickness of the compression component times the tie spacing .... m = 0.83
- With spirals .... m = 0.75
- In all other cases... m = 1.0

where:

f_y = specified yield strength of reinforcing bars (ksi)
d_b = nominal diameter of reinforcing bar (in.)

Where bars of different size are lap spliced in compression, the splice length shall not be less than the development length of the larger bar or the splice length of smaller bar. Bar sizes No. 14 and No. 18 may be lap spliced to No. 11 and smaller bars.

5.10.8.4.5b—Mechanical Connections or Welded Splices in Compression

Mechanical connections or welded splices used in compression shall satisfy the requirements for full-mechanical connections or full-welded splices as specified in Articles 5.10.8.4.2b and 5.10.8.4.2c, respectively.

5.10.8.4.5c—End-Bearing Splices

In bars required for compression only, the compressive force may be transmitted by bearing on square-cut ends held in concentric contact by a suitable device. End-bearing splices shall be used only in members confined by closed ties, closed stirrups, or spirals.

The end-bearing splices shall be staggered, or continuing bars shall be provided at splice locations. The continuing bars in each face of the member shall have a factored tensile resistance not less than $0.25f_y$ times the area of the reinforcement in that face.

The effective area of the ties is the area of the legs perpendicular to the thickness of the component, as seen in cross section.

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5.10.8.5—Splices of Welded Wire Reinforcement

5.10.8.5.1—Splices of Deformed Welded Wire Reinforcement in Tension

When measured between the ends of each reinforcement sheet, the length of lap for lap splices of deformed welded wire reinforcement with cross wires within the lap length shall not be less than the greater of the following:

- $1.3\ell_d$; or
- 8.0 in.

The overlap measured between the outermost cross wires of each reinforcement sheet shall not be less than 2.0 in.

Lap splices of deformed welded wire reinforcement with no cross wires within the lap splice length shall be determined as for deformed wire in accordance with the provisions of Article 5.10.8.4.3a.

C5.10.8.5.1

Splice provisions for deformed welded wire reinforcement are based on available tests (ACI 318-14). Lap splices for deformed welded wire reinforcement meeting the requirements of this provision are illustrated in Figure C5.10.8.5.1-1. If no cross wires are within the lap length, the provision for deformed wire apply.

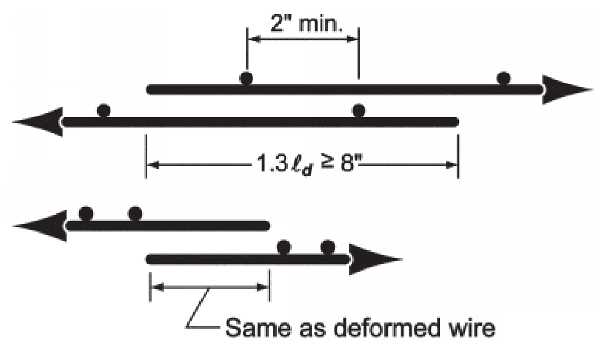

Figure C5.10.8.5.1-1—Lap Splices of Deformed Welded Wire Reinforcement (adapted from ACI 318-14, courtesy of ACI)

5.10.8.5.2—Splices of Plain Welded Wire Reinforcement in Tension

Where the area of reinforcement provided is less than twice that required at the splice location, the length of overlap measured between the outermost cross wires of each reinforcement sheet shall not be less than the greatest of the following:

- The sum of one spacing of cross wires plus 2.0 in.;
- $1.5\ell_d$; or
- 6.0 in.

Where the area of reinforcement provided is at least twice that required at the splice location, the length of overlap measured between the outermost cross wires of each reinforcement sheet shall not be less than the greater of the following:

- $1.5\ell_d$; or
- 2.0 in.

where:

C5.10.8.5.2

The strength of lap splices of plain welded wire reinforcement is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason, the lap is specified in terms of overlap of cross wires (in inches) rather than in wire diameters or length. The 2.0 in. additional lap required is to provide adequate overlap of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research (ACI 318-14) has shown an increased splice length is required when welded wire reinforcement of large, closely spaced wires is lapped and, as a consequence, additional splice length requirements are provided for this reinforcement in addition to an absolute minimum of 6.0 in. Splice requirements are illustrated in Figure C5.10.8.5.2-1.

Where the area of reinforcement provided is at least twice that required at the splice location, the lap splice for plain welded wire reinforcement is illustrated in Figure C5.10.8.5.2-2.

ℓ_d = development length computed in Eq. 5.10.8.2.5-1 (in.)

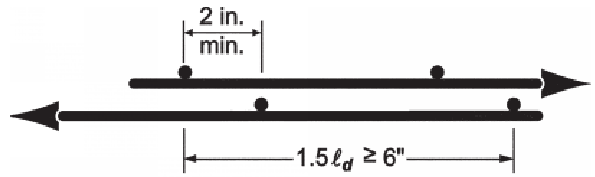

Figure C5.10.8.5.2-1—Lap Splices of Plain Welded Wire Reinforcement where (Provided A_s)/(Required A_s) < 2 (adapted from ACI 318-14, Courtesy of ACI)

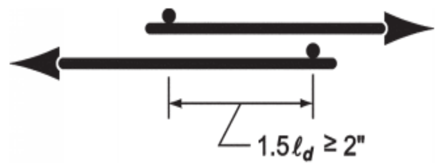

Figure C5.10.8.5.2-2—Lap Splices of Plain Welded Wire Reinforcement where (Provided A_s)/(Required A_s) ≥ 2 (adapted from ACI 318-14, Courtesy of ACI)

5.11—SEISMIC DESIGN AND DETAILS

5.11.1—General

The provisions of these articles shall apply only to the extreme event limit state.

In addition to the other requirements specified in Article 5.10, reinforcement shall also conform to the seismic resistance provisions specified herein.

Displacement requirements specified in Article 4.7.4.4 or longitudinal restrainers specified in Article 3.10.9.5 shall apply.

The provisions for piles in Article 5.12.9 shall apply unless they conflict with the provisions for Seismic Zones 2, 3, and 4 specified herein.

The use of reinforcement with specified minimum yield strengths of less than or equal to 100 ksi may be used in elements and connections specified in Article 5.4.3.3, where permitted by specific articles.

Bridges located in Seismic Zone 2 shall satisfy the requirements in Article 5.11.3. Bridges located in Seismic Zones 3 and 4 shall satisfy the requirements specified in Article 5.11.4.

C5.11.1

These specifications have their origins in the work by the Applied Technology Council in 1979–1980. As such they prescribe force based design. Insights gained from the response of bridges to earthquakes since that time have provided new insights into the behavior of concrete details under seismic loads. The California Department of Transportation initiated a number of research projects that have produced information that is useful for both the design of new structures and the retrofitting of existing structures. Much of this information has formed the basis of recent provisions published by NCHRP (2002, 2006), MCEER/ATC (2003), FHWA (2006) and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2011).

This new information relates to all facets of seismic engineering, including design spectra, analytical techniques, and design details. Bridge designers working in Seismic Zones 2, 3, and 4 are encouraged to avail themselves of current research reports and other literature to augment these specifications, including AASHTO (2011) which is displacement based rather than force based and has more explicit requirements for ductility and displacement capacity in higher seismic regions than is provided by these specifications.

5.11.2—Seismic Zone 1

For bridges in Seismic Zone 1 where the response acceleration coefficient, S_{D1}, specified in Article 3.10.4.2, is less than 0.10, no consideration of seismic forces shall be required for the design of structural components, except that the design of the connection of the superstructure to the substructure shall be as specified in Article 3.10.9.2.

C5.11.2

These requirements for Zone 1 are a departure from those in the previous edition of these specifications. These changes are necessary because the return period of the design event has been increased from 500 to 1000 years, and the Zone Boundaries (Table 3.10.6-1) have been increased accordingly. The high end of the new Zone 1 ($0.10 < S_{D1} < 0.15$) overlaps with the low

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For bridges in Seismic Zone 1 where the response acceleration coefficient, S_{D1}, is greater than or equal to 0.10 but less than or equal to 0.15, no consideration of seismic forces shall be required for the design of structural components, including concrete piles, except that:

- The design of the connection of the superstructure to the substructure shall be as specified in Article 3.10.9.2.
- The transverse reinforcement requirements at the top and bottom of a column shall be as specified in Articles 5.11.4.1.3, 5.11.4.1.4, and 5.11.4.1.5.

end of Zone 2 as it was specified before the fifth edition of these specifications. Since performance expectations have not changed with increasing return period, the minimum requirements for bridges in the high end of Zone 1 should therefore be the same as those for the previous Zone 2. Requirements for the remainder of Zone 1 ($S_{D1} < 0.10$) are unchanged.

Lap splices of reinforcement may be used in Seismic Zone 1.

Lap splices are permitted in Seismic Zone 1 on the basis that they were permitted in Zone 2 as it was specified before the Fifth Edition.

5.11.3—Seismic Zone 2

5.11.3.1—General

The requirements of Article 5.11.4 shall be taken to apply to bridges in Seismic Zone 2 except that the area of longitudinal reinforcement shall not be less than 0.01 or more than 0.06 times the gross cross-section area, A_g.

C5.11.3.1

Bridges in Seismic Zone 2 have a reasonable probability of being subjected to seismic forces that will cause yielding of the columns. Thus, it is deemed necessary that columns have some ductility capacity, although it is recognized that the ductility demand will not be as great as for columns of bridges in Seismic Zones 3 and 4. Nevertheless, all of the requirements for Zones 3 and 4 shall apply to bridges in Zone 2, with exception of the upper limit on reinforcement. This is a departure from the requirements in the previous edition of these specifications, in which selected requirements in Zones 3 and 4 were required for Zone 2. Satisfying all of the requirements, with one exception, is deemed necessary because the upper boundary for Zone 2 in the current edition is significantly higher than in the previous edition due to the increase in the return period for the design earthquake from 500 to 1,000 years.

5.11.3.2—Concrete Piles

5.11.3.2.1—General

Piles for structures in Zone 2 may be used to resist both axial and lateral loads. The minimum depth of embedment and axial and lateral pile resistances required for seismic loads shall be determined by means of design criteria established by site-specific geological and geotechnical investigations.

Concrete piles shall be anchored to the pile footing or cap by either embedment of reinforcement or anchorages to develop uplift forces. The embedment length shall not be less than the development length required for the reinforcement specified in Article 5.10.8.2.

Concrete-filled pipe piles shall be anchored with steel dowels as specified in Article 5.12.9.1, with a minimum steel ratio of 0.01. Dowels shall be embedded as required for concrete piles. Timber and steel piles, including unfilled pipe piles, shall be provided with anchoring devices to develop any uplift forces. The uplift force shall not be taken to be less than 10 percent of the factored axial compressive resistance of the pile.

5.11.3.2.2—Cast-in-Place Piles

For cast-in-place piles, longitudinal steel shall be provided in the upper end of the pile for a length not less than the greater of the following:

- one third of the pile length; or
- 8.0 ft,

with a minimum steel ratio of 0.005 provided by at least four bars. For piles less than 24.0 in. in diameter, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than greater of the following:

- 2.0 ft; or
- 1.5 pile diameters.

C5.11.3.2.2

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the state of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased, which results in larger openings between the parallel longitudinal and transverse reinforcement.

5.11.3.2.3—Precast Reinforced Piles

For precast reinforced piles, the longitudinal steel shall not be less than 1 percent of the cross-sectional area and provided by not less than four bars. Spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 in., except that a 3.0 in. pitch shall be used within a confinement length not less than 2.0 ft or 1.5 pile diameters below the pile cap reinforcement.

5.11.3.2.4—Precast Prestressed Piles

For precast prestressed piles, the ties shall conform to the requirements of precast piles, as specified in Article 5.11.3.2.3.

5.11.4—Seismic Zones 3 and 4

5.11.4.1—Column Requirements

For the purpose of this article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is not less than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Article 5.11.4.2 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

C5.11.4.1

The definition of a column in this article is provided as a guideline to differentiate between the additional design requirements for a wall-type pier and the requirements for a column. If a column or pier is above or below the recommended criterion, it may be considered to be a column or a pier, provided that the appropriate R-factor of Article 3.10.7.1 and the appropriate requirements of either Articles 5.11.4.1 or 5.11.4.2 are used. For columns with an aspect ratio less than 2.5, the forces resulting from plastic hinging will generally exceed the elastic design forces; consequently, the forces of Article 5.11.4.2 would not be applicable.

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5.11.4.1.1—Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.04 times the gross cross-section area, A_g.

C5.11.4.1.1

This requirement is intended to apply to the full section of the columns. The lower limit on the column reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments. Columns with less than 1 percent steel have also not exhibited good ductility (Halvorsen, 1987). The 4 percent maximum ratio is to avoid congestion and extensive shrinkage cracking and to permit anchorage of the longitudinal steel. The previous edition of these specifications limited this ratio to 6 percent but this cap is lowered in the current edition because the boundaries for Zones 3 and 4 are significantly higher than in the previous edition, due to the increase in the return period for the design earthquake from 500 to 1,000 years. The 4 percent figure is consistent with that recommended in recent publications by NCHRP (2002, 2006) and MCEER/ATC (2003).

5.11.4.1.2—Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4. The column shall be investigated for both directional combinations of force effects specified in Article 3.10.8, at the extreme event limit state. The resistance factors of Article 5.5.4.2 shall be replaced for columns with either spiral or tie reinforcement by the value of 0.9.

C5.11.4.1.2

Columns are required to be designed biaxially and to be investigated for both the minimum and maximum axial forces. In the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, the recommended flexural resistance factor is 1.0. However, since these Specifications are force-based and do not explicitly calculate the ductility demand as in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, limiting the factor to 0.9 is considered justified in lieu of a more rigorous analysis.

5.11.4.1.3—Column Shear and Transverse Reinforcement

The factored shear force V_u on each principal axis of each column and pile bent shall be as specified in Article 3.10.9.4.

The amount of transverse reinforcement shall not be less than that specified in Article 5.7.3.

The following provisions apply to the end regions of the top and bottom of the column and pile bents:

- In the end regions, V_c shall be taken as that specified in Article 5.7.3, provided that the minimum factored axial compression force exceeds $0.10f'_cA_g$. For compression forces less than $0.10f'_cA_g$, V_c shall be taken to decrease linearly from the value given in Article 5.7.3 to zero at zero compression force.
- The end region shall be assumed to extend from integral connections to soffits of girders or cap beams at the top of columns or from the top of foundations at the bottom of columns, a distance taken as the greater of:

C5.11.4.1.3

Seismic hoops may offer the following advantages over spirals:

- Improved constructability when the transverse reinforcement cage must extend up into a bent cap or down into a footing. Seismic hoops can be used at the top and bottom of the column in combination with spirals, or full height of the column in place of spirals.
- Ability to sample and perform destructive testing of in-situ splices prior to assembly.
- Breakage at a single location vs. potential unwinding and plastic hinge failure.

The requirements of this article are intended to minimize the potential for a column shear failure. The design shear force is specified as that capable of being developed by either flexural yielding of the columns or the elastic design shear force. This requirement was

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- the maximum cross-sectional dimension of the column;
- one sixth of the clear height of the column; or
- 18.0 in.

- The end region at the top of the pile bent shall be taken as that specified for columns. At the bottom of the pile bent, the end region shall be considered to extend from three pile diameters below the calculated point of maximum moment to one pile diameter below it, but shall not extend less than 18.0 in. above the mud line.

added because of the potential for superstructure collapse if a column fails in shear.

A column may yield in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

The concrete contribution to shear resistance is undependable within the plastic hinge zone, particularly at low axial load levels, because of full-section cracking under load reversals. As a result, the concrete shear contribution should be reduced for axial load levels less than $0.10 f'_c A_g$.

For a noncircular pile, this provision may be applied by substituting the larger cross-sectional dimension for the diameter.

5.11.4.1.4—Transverse Reinforcement for Confinement at Plastic Hinges

Seismic hooks consisting of a 135-degree bend, plus an extension of not less than the larger of $6.0d_b$ or 3.0 in., 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. Where seismic hooks are used, the specified minimum yield strength of the reinforcement shall not exceed 75.0 ksi.

The cores of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement, and the spacing shall be taken as specified in Article 5.11.4.1.5.

For a circular column, the volumetric ratio of spiral or seismic hoop reinforcement, ρ_s, shall be determined as:

$$\rho_s = \frac{4A_{sp}}{(d_c s)} \geq 0.12\frac{f'_c}{f_{yh}} \qquad (5.11.4.1.4\text{-}1)$$

and shall satisfy the requirements of Article 5.6.4.6.

where:

A_{sp} = cross-sectional area of spiral or hoop (in.^{2})
d_c = core diameter of column measured to the outside diameter of spiral or hoop (in.)
s = pitch of spiral or vertical spacing of hoops (in.)
A_c = area of core measured to the outside diameter of the spiral (in.^{2})
f'_c = compressive strength of concrete for use in design (ksi)

C5.11.4.1.4

Plastic hinge regions are generally located at the top and bottom of columns and pile bents. The largest of either these requirements or those of Article 5.11.4.1.3 should govern; these requirements are not in addition to those of Article 5.11.4.1.3. Detailing of seismic hooks has not been verified for reinforcement with yield strengths exceeding 75.0 ksi.

The main function of the transverse reinforcement specified in this article is to ensure that the axial load carried by the column after spalling of the concrete cover will at least equal the load carried before spalling and to ensure that buckling of the longitudinal reinforcement is prevented. Thus, the spacing of the confining reinforcement is also important.

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f_{yh} = specified minimum yield strength of spiral reinforcement (ksi) ≤100 ksi for elements and connections specified in Article 5.4.3.3; ≤75.0 ksi otherwise

Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall satisfy both of the following:

$$A_{sh} \geq 0.30\, sh_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1 \right] \quad (5.11.4.1.4\text{-}2)$$

$$A_{sh} \geq 0.12 sh_c \frac{f'_c}{f_{yh}} \quad (5.11.4.1.4\text{-}3)$$

where:

A_{sh} = total cross-sectional area of tie reinforcement, including supplementary cross-ties having a vertical spacing of s and crossing a section having a core dimension of h_c (in.^{2})
s = vertical spacing of hoops, not exceeding 4.0 in. (in.)
h_c = core dimension of tied column in the direction under consideration, measured to the outside of the hoop (in.)
A_g = gross area of section (in.^{2})
A_c = area of column core measured to outside diameter of the spiral (in.^{2})
f_{yh} = yield strength of tie or spiral reinforcement (ksi) ≤ 75.0 ksi

A_{sh} shall be determined for both principal axes of a rectangular column.

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- A continuous bar having a seismic hook at one end and a hook of not less than 90 degrees with an extension of not less than $6.0d_b$ at the other end.
- The 90-degree hooks of adjacent cross-ties are alternated so that hooks of the same bend angle are not adjacent to each other both vertically and horizontally.

Careful detailing of the confining steel in the plastic hinge zone is required because of spalling and loss of concrete cover. With deformation associated with plastic hinging, the strains in the transverse reinforcement increase. Ultimate-level splices are required. Similarly, rectangular hoops should be anchored by bending ends back into the core.

Figures C5.11.4.1.4-1 and C5.11.4.1.4-3 illustrate the use of Eqs. 5.11.4.1.4-2 and 5.11.4.1.4-3. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column. Figure C5.11.4.1.4-3 shows the distance to be utilized for h_c and the direction of the corresponding reinforcement for both principal directions of a rectangular column.

Where ties are used for transverse column reinforcement, the maximum clear spacing of unrestrained longitudinal reinforcement is recommended to be 6.0 in. to reduce buckling after cover spalling. A maximum tie spacing of 14.0 in. is recommended for the laterally supported longitudinal bars to provide confinement.

Where a spiral cage is used, a maximum spacing of longitudinal bars of 8.0 in. center-to-center is recommended to help confine the column core.

While these specifications allow the use of either spirals or ties for transverse column reinforcement, the use of spirals is recommended as the more effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C5.11.4.1.4-2.

Examples of transverse column reinforcement are shown herein.

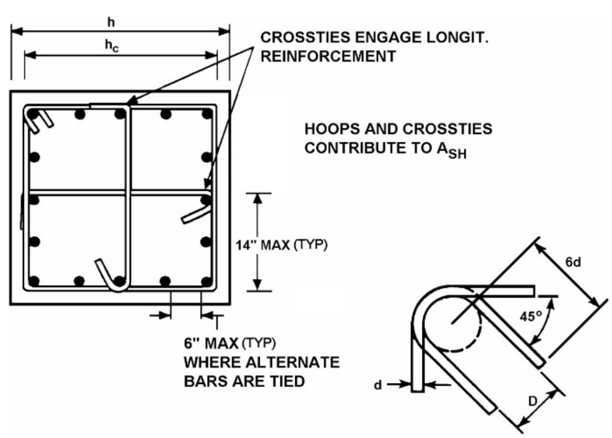

Figure C5.11.4.1.4-1—Column Tie Details

- A discontinuous bar with a Class A splice as specified in Article 5.10.8.4.3a on one end and a 180 degree hook with an extension of $6.0d_b$ on the other end.
- The hooks engage peripheral longitudinal bars.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar is a closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcement elements with a seismic hook at each end.
- A continuously wound tie with a seismic hook at each end that engages the longitudinal reinforcing bars.

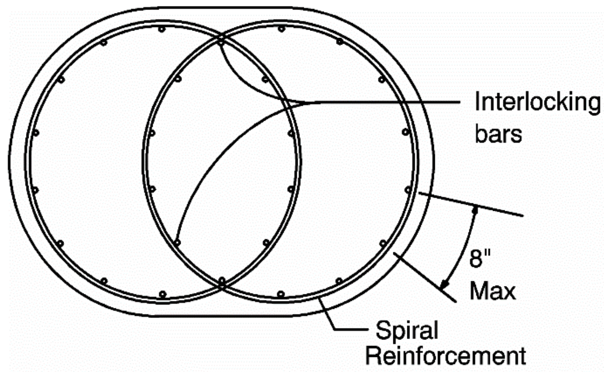

Figure C5.11.4.1.4-2—Column Interlocking Spiral Details

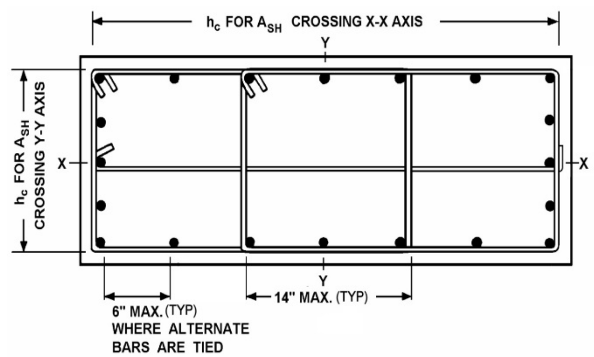

Figure C5.11.4.1.4-3—Column Tie Details

5.11.4.1.5—Spacing of Transverse Reinforcement for Confinement

Transverse reinforcement for confinement shall be:

- Provided at integral connections to soffits of girders or cap beams at the tops of columns and at the bottoms of the columns over a length not less than the greatest of the maximum cross-sectional column dimensions, one sixth of the clear height of the column, or 18.0 in.;
- Extended into the top and bottom connections as specified in Article 5.11.4.3;
- Provided at the top of piles in pile bents over the same length as specified for columns;
- Provided within piles in pile bents over a length extending from 3.0 times the maximum cross-sectional dimension below the calculated point of moment fixity to a distance larger than greater of the following:
  - o the maximum cross-sectional dimension or
  - o 18.0 in. above the mud line; and
- Spaced not to exceed the lesser of the following:
  - o one quarter of the minimum member dimension
  - o 4.0 in. center-to-center.

5.11.4.1.6—Splices

The provisions of Article 5.10.8.4 shall apply for the design of splices.

Lap splices in longitudinal reinforcement shall not be used.

The spacing of the transverse reinforcement over the length of the splice shall not exceed the lesser of the following:

- one quarter of the minimum member dimension; or
- 4.0 in

Full-welded or full-mechanical connection splices conforming to Article 5.10.8.4 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 24.0 in. measured along the longitudinal axis of the column.

C5.11.4.1.6

In the past, it was often desirable to lap longitudinal reinforcement with dowels at the column base. This is undesirable for seismic performance because:

- The splice occurs in a potential plastic hinge region where requirements for bond is critical, and
- Lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in a severe local curvature demand.

The prohibition on the use of lap splices in Seismic Zones 3 and 4 is extended to Seismic Zone 2 by virtue of Article 5.11.3.1.

Splices in seismic-critical elements should be designed for ultimate behavior under seismic deformation demands. Recommendations for acceptable strains are provided in Table C5.11.4.1.6-1. The strain demand at a cross section is obtained from the deformation demand at that cross section and the corresponding moment-curvature relationship. Traditional service level splices are only appropriate in components such as bent caps, girders, and footings, when not subjected to or protected from seismic damage by careful location and detailing of plastic hinge regions.

Table C5.11.4.1.6-1—Recommended Strain Limits in A706/A706M Bars, and Bars with Splices for Seismic Zones 3 and 4

	Minimum Required Resisting Strain, ε Bar only	Minimum Required Resisting Strain, ε Bar with Splice	Maximum Allowable Load Strain, ε	Resulting Factor of Safety
Ultimate	6% for #11 and larger 9% for #10 and smaller	6% for #11 and larger 9% for #10 and smaller	<2%	3 to 4.5
Service	(same as above)	>2%	<0.2%	>10
Lap (or welded / mechanical lap in lieu of lap splice)	(same as above)	>0.2%	<0.15% (unfactored loads) <0.2% (factored loads)	1.33

Limits are based on tests done by the California Department of Transportation and University of California-Berkeley. The demonstrated strain at ultimate resistance of butt-welded details was divided by the typical demand strain in order to document the factor of safety. Although current experimental limitations of other splice details performing at the service level preclude strain measurements, known values are shown in Table C5.11.4.1.6-1 for comparison. The variability of strain along the potential plastic hinge justifies the much higher factor of safety. Use of traditional splice details to resist extreme loading conditions where nonlinear behavior is desired and analyzed as such, are shown to be inefficient. ASTM

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A615/A615M steel is generally not permitted by Caltrans because of weldability and ductility concerns, and was not investigated.

5.11.4.2—Requirements for Wall-Type Piers

The provisions herein specified shall apply to the design for the strong direction of a pier. The weak direction of a pier may be designed as a column conforming to the provisions of Article 5.11.4.1, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in its weak direction, the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18.0 in. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r, in the pier shall be taken as the lesser of the following:

$$V_r = 0.253\lambda\sqrt{f'_c}bd, \tag{5.11.4.2-1}$$

$$V_r = \phi V_n \tag{5.11.4.2-2}$$

in which:

$$V_n = \left[0.063\lambda\sqrt{f'_c} + \rho_h f_y\right]bd \tag{5.11.4.2-3}$$

where:

λ = concrete density modification factor specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design (ksi)
b = width of pier (in.)
d = depth of pier (in.)
f_y = specified minimum yield strength of reinforcement (ksi)
ρ_h = ratio of area of horizontal shear reinforcement to area of gross concrete area of a vertical section

Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered, and splices in the two layers shall not occur at the same location.

C5.11.4.2

The requirements of this article are based on limited data available on the behavior of piers in the inelastic range. Consequently, the R-factor of 2.0 for piers is based on the assumption of minimal inelastic behavior.

The requirement that $\rho_v \geq \rho_h$ is intended to avoid the possibility of inadequate web reinforcements in piers, which are short in comparison to their height. Splices should be staggered in an effort to avoid weak sections.

The requirement for a minimum of two layers of reinforcement in walls carrying substantial design shears is based on the premise that two layers of reinforcement will tend to "basket" the concrete and retain the integrity of the wall after cracking of the concrete.

5.11.4.3—Column Connections

The design force for the connection between the column and the cap beam superstructure, pile cap, or spread footing shall be as specified in Article 3.10.9.4.3.

C5.11.4.3

A column connection, as referred to in this article, is the vertical extension of the column area into the adjoining member.

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The development length for all longitudinal steel shall be 1.25 times that required for the full yield strength of reinforcement as specified in Article 5.10.8.

Column transverse reinforcement, as specified in Article 5.11.4.1.4, shall be continued for a distance not less than one-half the maximum column dimension or 15.0 in. from the face of the column connection into the adjoining member.

The nominal shear resistance, V_n, provided by the concrete in the joint of a frame or bent in the direction under consideration, shall satisfy:

$$V_n \leq 0.380\, bd\lambda\sqrt{f'_c} \qquad (5.11.4.3\text{-}1)$$

where:

b = width of column (in.)
d = depth of column (in.)
λ = concrete density modification factor specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design (ksi)

The integrity of the column connection is important if the columns are to develop their flexural capacity. The longitudinal reinforcement should be capable of developing its overstrength capacity of $1.25f_y$. The transverse confining reinforcement of the column should be continued a distance into the joint to avoid a plane of weakness at the interface.

The strength of the column connections in a column cap is relatively insensitive to the amount of transverse reinforcement, provided that there is a minimum amount and that shear resistance is limited to the values specified.

5.11.4.4—Construction Joints in Piers and Columns

Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the nominal shear resistance across the joint, V_n, shall be taken as:

$$V_n = \left(A_{vf} f_y + 0.75P_u\right) \qquad (5.11.4.4\text{-}1)$$

where:

A_{vf} = the total area of reinforcement, including flexural reinforcement (in.^{2})
P_u = minimum factored axial load as specified in Article 3.10.9.4 for columns and piers (kip)

C5.11.4.4

Eq. 5.11.4.4-1 is based on Eq. 22.9.4.2 of ACI 318-14 but is restated to reflect dowel action and frictional resistance.

5.11.4.5—Concrete Piles

5.11.4.5.1—General

In addition to the requirements specified for Zone 2, piles in Zones 3 and 4 shall conform to the provisions specified herein.

5.11.4.5.2—Confinement Length

The upper end of every pile shall be reinforced and confined as a potential plastic hinge region, except where it can be established that there is no possibility of any significant lateral deflection in the pile. The potential plastic hinge region shall extend from the underside of the pile cap over a length of not less than either 2.0 pile diameters or 24.0 in. If an analysis of

C5.11.4.5.2

Note the special requirements for pile bents given in Article 5.11.4.1.

the bridge and pile system interacting with the surrounding soil indicates that a plastic hinge can form at a lower level, the confinement length with the specified transverse reinforcement and closer pitch, as specified in Article 5.11.3.2, shall extend thereto.

5.11.4.5.3—Volumetric Ratio for Confinement

The volumetric ratio of transverse reinforcement within the confinement length shall be that for columns, as specified in Article 5.11.4.1.4.

5.11.4.5.4—Cast-in-Place Piles

For cast-in-place piles, longitudinal steel shall be provided for the full length of the pile. In the upper two thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, shall not be less than 0.75 percent. For piles less than 24.0 in. in diameter, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 4.0 ft and where the volumetric ratio and splice details shall conform to Articles 5.11.4.1.4, 5.11.4.1.5, and 5.11.4.1.6.

C5.11.4.5.4

See Article C5.11.3.2.2.

5.11.4.5.5—Precast Piles

For precast piles, spiral ties shall not be less than No. 3 bars at a pitch not exceeding 9.0 in., except for the top 4.0 ft, where the pitch shall be 3.0 in. and the volumetric ratio and splice details shall conform to Article 5.11.4.1.4.

5.12—PROVISIONS FOR STRUCTURE COMPONENTS AND TYPES

5.12.1—Deck Slabs

Requirements for deck slabs in addition to those specified in Section 5 shall be as specified in Section 9.

5.12.2—Slab Superstructures

5.12.2.1—Cast-in-Place Solid Slab Superstructures

Cast-in-place, longitudinally reinforced slabs may be either conventionally reinforced or prestressed and may be used as slab-type bridges.

The distribution of live load may be determined by a refined analysis or as specified in Article 4.6.2.3. Slabs and slab bridges designed for moment in conformance with Article 4.6.2.3 may be considered satisfactory for shear.

Edge beams shall be provided as specified in Article 9.7.1.4.

C5.12.2.1

In this simple bridge superstructure, the deck slab also serves as the principal load-carrying component. The concrete slab, which may be solid, voided, or ribbed, is supported directly on the substructures.

The provisions are based on the performance of the relatively small span structures constructed to date. Any significant deviation from successful past practice for larger units that may become both structurally and economically feasible under these specifications should be reviewed carefully.

Transverse distribution reinforcement shall be placed in the bottoms of all slabs, except culvert tops or bridge slabs, where the depth of fill over the slab exceeds 2.0 ft. The amount of the bottom transverse reinforcement may be determined by two-dimensional analysis, or the amount of distribution reinforcement may be taken as the percentage of the main reinforcement required for positive moment taken as:

- For longitudinal reinforced concrete construction:

$$\frac{100}{\sqrt{L}} \leq 50\% \qquad (5.12.2.1\text{-}1)$$

- For longitudinal prestressed construction:

$$\frac{100}{\sqrt{L}} \frac{f_{pe}}{60} \leq 50\% \qquad (5.12.2.1\text{-}2)$$

where:

L = span length (ft)
f_{pe} = effective stress in the prestressing steel after losses (ksi)

Transverse shrinkage and temperature reinforcement in the tops of slabs shall conform to the requirements of Article 5.10.6.

5.12.2.2—Cast-in-Place Voided Slab Superstructures

5.12.2.2.1—Cross Section Dimensions

Cast-in-place voided slab superstructures may be post-tensioned both longitudinally and transversely.

For circular voids, the center-to-center spacing of the voids should not be less than the total depth of the slab, and the minimum thickness of concrete taken at the centerline of the void perpendicular to the outside surface shall not be less than 5.5 in.

For rectangular voids, the transverse width of the void should not exceed 1.5 times the depth of the void, the thickness of the web between voids should not be less than 20 percent of the total depth of the deck, and the minimum thickness of concrete above the voids shall not be less than 7.0 in.

The bottom flange depth shall satisfy the requirements specified in Article 5.12.3.5.1b.

Where the voids conform to the dimensional requirements herein and where the void ratio, based on cross-sectional area, does not exceed 40 percent, the superstructure may be analyzed as a slab, using either the provisions of Article 4.6.2.3 or a two-dimensional analysis for isotropic plates.

If the void ratio exceeds 40 percent, the superstructure shall be treated as cellular construction and analyzed as:

- A monolithic multicell box, as specified in Article 4.6.2.2.1, Type *d*;
- an orthotropic plate; or
- a three-dimensional continuum.

5.12.2.2.2—Minimum Number of Bearings

Columns may be framed into the superstructure, or a single bearing may be used for the internal supports of continuous structures. A minimum of two bearings shall be employed at end supports.

The transverse rotation of the superstructure shall not exceed 0.5 percent at service limit states.

C5.12.2.2.2

The high torsional stiffness of voided concrete decks and the inherent stability of horizontally-curved continuous structures permits the use of a single support at internal piers. A minimum of two bearings are required at the abutments to ensure torsional stability in the end zones. If the torsional rotation requirement is not satisfied, pairs of bearings may be used at some internal piers.

5.12.2.2.3—Solid End Sections

A solid section at least 3.0 ft long but not less than five percent of the length of the span shall be provided at either end of a span. Post-tensioned anchorage zones shall satisfy the requirements specified in Article 5.9.5.6. In the absence of more refined analysis, the solid sections of the deck may be analyzed as a transverse beam distributing forces to bridge bearings and to post-tensioning anchorages.

C5.12.2.2.3

The intent is to provide for the distribution of concentrated post-tensioning and bearing forces to the voided sections. For relatively wide decks, the analysis of the solid sections as beams is an acceptable approximation. For deep and narrow decks, a three-dimensional analysis or use of the strut-and-tie method is advisable.

5.12.2.2.4—General Design Requirements

For voided slabs conforming to the provisions of Article 5.12.2.2.1, global and local force effects due to wheel loads need not be combined. The top flange of deck with rectangular voids may be analyzed and designed as a framed slab or designed with the provisions of the empirical process, as specified in Article 9.7.2.

The top part of the slab over circular voids made with steel void-formers shall be post-tensioned transversely. At the minimum thickness of concrete, the average precompression after all losses, as specified in Article 5.9.3, shall not be less than 0.5 ksi. When transversely post-tensioned, no additional reinforcing steel need be provided above the circular voids.

Transverse shrinkage and temperature steel at the bottom of the voided slab shall satisfy the requirements specified in Article 5.10.6.

C5.12.2.2.4

Continuous voided decks should be longitudinally post-tensioned. Unless specified otherwise in this article, or required for construction purposes, additional global longitudinal reinforcement may be deemed to be unnecessary if longitudinal post-tensioning is used. The preference for longitudinal post-tensioning of continuous decks reflects the limited experience with this system in North America.

Experience indicates that due to a combination of transverse bending moment, shrinkage of concrete around the steel void-former and Poisson's effect, where steel void-formers are used, high transverse tensile stresses tend to develop at the top of the deck, resulting in excessive cracking at the centerline of the void. The minimum transverse prestress specified to counteract this tension is a conservative value. The intent of transverse temperature steel at the bottom of voided deck is also for control of cracks resulting from transverse positive moments due to post-tensioning.

The hidden solid transverse beam over an internal pier may be post-tensioned.

5.12.2.2.5—Compressive Zones in Negative Moment Area

At internal piers, the part of the cross section under compressive stresses may be considered as a horizontal column and reinforced accordingly.

C5.12.2.2.5

Tests on two-span, continuous, post-tensioned structures indicate that first failure occurs in the bottom compressive zones adjacent to the bearing at the internal pier. The failure is thought to be caused by a

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combination of shear and compression at those points in the bottom flange. The phenomenon is not yet clearly understood, and no specific design provisions have been developed. At this time, the best that can be done is to treat the bottom chord as a column with a reinforcement ratio of one percent and column-ties as specified in Article 5.10.4.

5.12.2.2.6—Drainage of Voids

Adequate drainage of the voids shall be provided in accordance with the provisions of Article 2.6.6.5.

C5.12.2.2.6

Occasional cracks large enough to permit entry of water into the voids may develop in these deck systems. The accumulating water adds to gravitational loads and may cause structural damage when it freezes.

5.12.2.3—Precast Deck Bridges

5.12.2.3.1—General

Precast concrete units placed adjacent to each other in the longitudinal direction may be joined together transversely to form a deck system. Precast concrete units may be continuous either for transient loads only or for both permanent and transient loads. Span-to-span continuity, where provided, shall be in accordance with the provisions of Article 5.12.3.4.2.

Where structural concrete overlay is not provided, the minimum thickness of concrete shall be 3.5 in. at the top of round voided components and 5.5 in. for all other components.

C5.12.2.3.1

Precast units may have solid, voided, box, T- and double-T cross sections.

Differential creep and shrinkage due to differences in age, concrete mix, environmental, and support conditions have been observed to cause internal force effects that are difficult to predict at the design phase. These force effects are often relieved by separation of the joints, causing maintenance problems and negatively affecting structural performance.

Standard AASHTO-PCI prestressed concrete voided slab and box-beam sections, which are commonly used to construct precast deck bridges, have been used successfully for many years in bridges with and without a structural concrete overlay. The standard prestressed concrete overlay slab sections have 3.5 in., 4.0 in. and 4.5 in. of concrete over 8.0 in., 10.0 in. and 12.0 in. diameter voids respectively. All standard box beams including both 3.0 and 4.0 ft wide sections, are detailed with 5.5 in. of concrete over rectangular voids with corner fillets.

5.12.2.3.2—Shear Transfer Joints

Precast longitudinal components may be joined together transversely by a shear key not less than 7.0 in. in depth. For the purpose of analysis, the longitudinal shear transfer joints shall be modeled as hinges.

The joint shall be filled with nonshrinking grout with a minimum compressive strength of 5.0 ksi at 24 hours.

C5.12.2.3.2

Many bridges have indications of joint distress where load transfer among the components relies entirely on shear keys because the grout is subject to extensive cracking. Long-term performance of the key joint should be investigated for cracking and separation.

5.12.2.3.3—Shear-Flexure Transfer Joints

5.12.2.3.3a—General

Precast longitudinal components may be joined together by transverse post-tensioning, cast-in-place closure joints, a structural overlay, or a combination thereof.

C5.12.2.3.3a

These joints are intended to provide full continuity and monolithic behavior of the deck.

5.12.2.3.3b—Design

Decks with shear-flexure transfer joints should be modeled as continuous plates, except that the empirical design procedure of Article 9.7.2 shall not be used. The joints shall be designed as flexural components, satisfying the provisions of Article 5.12.2.3.3d.

C5.12.2.3.3b

From the modeling point of view, these precast concrete deck systems are not different from cast-in-place ones of the same geometry.

5.12.2.3.3c—Post-Tensioning

Transverse post-tensioning shall be uniformly distributed in the longitudinal direction. Block-outs may be used to facilitate splicing of the post-tensioning ducts. The compressed depth of the joint shall not be less than 7.0 in., and the prestress after all losses shall not be less than 0.25 ksi therein.

C5.12.2.3.3c

When tensioning narrow decks, losses due to anchorage setting should be kept to a minimum. Ducts should preferably be straight and grouted.

The post-tensioning force is known to spread at an angle of 45 degrees or larger and to attain a uniform distribution within a short distance from the anchorage. The economy of prestressing is also known to increase with the spacing of ducts. For these reasons, the spacing of the ducts need not be smaller than about 4.0 ft or the width of the component housing the anchorages, whichever is larger.

5.12.2.3.3d—Longitudinal Construction Joints

Longitudinal construction joints between precast concrete flexural components shall consist of a key filled with a nonshrinkage mortar attaining a compressive strength of 5.0 ksi within 24 hours. The depth of the key should not be less than 5.0 in.

If the components are post-tensioned together transversely, the top flanges may be assumed to act as a monolithic slab. However, the empirical slab design specified in Article 9.7.2 is not applicable.

The amount of transverse prestress may be determined by either the strip method or two-dimensional analysis. The transverse prestress, after all losses, shall not be less than 0.25 ksi through the key. In the last 3.0 ft at a free end, the required transverse prestress shall be doubled.

C5.12.2.3.3d

This Article relates to deck systems composed entirely of precast beams of box, T-, and double-T sections, laid side-by-side and, preferably, joined together by transverse post-tensioning. The transverse post-tensioning tendons should be located at the centerline of the key.

Grinding of grout and concrete in the vicinity of the joint may be expected and specified for construction.

5.12.2.3.3e—Cast-in-Place Closure Joints

Concrete in the closure joint should have strength comparable to that of the precast components. The width of the longitudinal joint shall be large enough to accommodate development of reinforcement in the joint, but in no case shall the width of the joint be less than 12.0 in.

5.12.2.3.3f—Structural Overlay

Where a structural overlay is used to qualify for improved load distribution as provided in Articles 4.6.2.2.2 and 4.6.2.2.3, the thickness of structural concrete overlay shall not be less than 4.5 in. An isotropic layer of reinforcement shall be provided in accordance with the requirements of Article 5.10.6. The top surface of the precast components shall be roughened.

C5.12.2.3.3f

The composite overlay should be regarded as a structural component and should be designed and detailed accordingly.

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5.12.3—Beams and Girders

5.12.3.1—General

The provisions specified herein shall be applied to the design of cast-in-place and precast beams as well as girders with rectangular, I, T, bulb-T, double-T, and open- and closed-box sections.

Precast beams may resist transient loads with or without a superimposed deck. Where a structurally separate concrete deck is applied, it shall be made composite with the precast beams in accordance with the provisions of Article 5.7.4.

The flange width considered to be effective in flexure shall be that specified in Article 4.6.2.6 or Article 5.7.3.4.

C5.12.3.1

This Article applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.12.5. There is a large variety of possible concrete superstructure systems, some of which may fall into either category. Precast deck bridges, which utilize girder sections with integral decks, are covered in Article 5.12.2.3.

Components that directly carry live loads, i.e., incorporated elements of the deck, should be designed for the applicable provisions of Section 9 and with particular reference to minimum dimension requirements and the way the components are to be joined to provide a continuous deck.

5.12.3.2—Precast Beams

5.12.3.2.1—Preservice Conditions

The preservice conditions of prestressed girders for shipping and erection shall be the responsibility of the contractor.

C5.12.3.2.1

AASHTO LRFD Bridge Construction Specifications place the responsibility on the Contractor to provide adequate devices and methods for the safe storage, handling, erection, and temporary bracing of precast members. However, these preservice conditions may govern and should be considered in the design, as discussed in Article 2.5.3.

5.12.3.2.2—Extreme Dimensions

The thickness of any part of precast concrete beams shall not be less than:

Top flange	2.0 in.
Web, nonpost-tensioned	5.0 in.
Web, post-tensioned	6.5 in.
Bottom flange	5.0 in.

The maximum dimensions and weight of precast members manufactured at an offsite casting yard shall conform to local hauling restrictions.

C5.12.3.2.2

The 2.0-in. minimum dimension relates to bulb-T and double-T types of girders on which cast-in-place decks are used. The 5.0-in. and 6.5-in. web thicknesses have been successfully used by contractors experienced in working to close tolerances. The 5.0-in. limit for bottom flange thickness normally relates to box-type sections.

For highway transportation, the permissible load size and weight limits are constantly being revised. For large members, an investigation should be made prior to design to ensure transportability. Investigations may include driving the route or surveying route portions with known vertical or horizontal clearance problems. Contract documents should alert the contractor to weight and permitting complications as well as the possibility of law enforcement escort requirements.

When the weight or dimensions of a precast beam exceed local hauling restrictions, field splices conforming to the requirements of Article 5.12.3.4.2 may be used.

5.12.3.2.3—Lifting Devices

If it is anticipated that anchorages for lifting devices will be cast into the face of a member that will be exposed to view or to corrosive materials in the completed structure, any restriction on locations of embedded lifting devices, the depth of removal, and the method of filling the cavities after removal shall be shown in the contract documents. The depth of removal shall be not less than the depth of cover required for the reinforcement.

C5.12.3.2.3

AASHTO LRFD Bridge Construction Specifications allows the Contractor to select the type of lifting device for precast members provided that the Contractor accepts responsibility for their performance. Anchorages for lifting devices generally consist of loops of prestressing strand or nonprestressed reinforcement, with their tails embedded in the concrete or threaded anchorage devices that are cast into the concrete.

5.12.3.2.4—Detail Design

All details of reinforcement, connections, bearing seats, inserts, or anchors for diaphragms, concrete cover, openings, and fabrication and erection tolerances shall be shown in the contract documents. For any details left to the Contractor's choice, such as prestressing materials or methods, the submittal and review of working drawings shall be required.

C5.12.3.2.4

AASHTO LRFD Bridge Construction Specifications includes general requirements pertaining to the preparation and review of working drawings, but the contract documents should specifically indicate when they are required.

5.12.3.2.5—Concrete Strength

For slow curing concretes, the 90-day compressive strength may be used for all stress combinations that occur after 90 days, provided that the gain in strength is verified by prior tests for the concrete mix utilized.

For normal weight concrete, the 90-day strength of slow curing concretes may be estimated as 115 percent of the 28-day concrete strength.

C5.12.3.2.5

This Article recognizes the behavior of slow-curing concretes, such as those containing fly-ash. It is not often that a bridge is opened to traffic before the precast components are 90 days old. The Designer may now take advantage of this, provided that the gain in strength has previously been verified by testing of the utilized concrete mix.

5.12.3.3—Bridges Composed of Simple Span Precast Girders Made Continuous

5.12.3.3.1—General

The provisions of this article shall apply at the service and strength limit states as applicable.

When the requirements herein are satisfied, multi-span bridges composed of simple-span precast girders with continuity diaphragms cast between ends of girders at interior supports may be considered continuous for loads placed on the bridge after the continuity diaphragms are installed and have cured.

The connection between girders at the continuity diaphragm shall be designed for all effects that cause moment at the connection, including restraint moments from time-dependent effects, except as allowed herein.

These requirements supplement the requirements of other sections of these specifications for prestressed concrete components that are not segmentally constructed.

Multi-span bridges composed of precast girders with continuity diaphragms at interior supports that are designed as a series of simple spans are not required to satisfy these requirements.

C5.12.3.3.1

This type of bridge is generally constructed with a composite deck slab. However, with proper design and detailing, precast members used without a composite deck may also be made continuous for loads applied after continuity is established. Details of this type of construction are discussed in Miller et al. (2004).

The designer may choose to design a multi-span bridge as a series of simple spans but detail it as continuous with continuity diaphragms to eliminate expansion joints in the deck slab. This approach has been used successfully in several parts of the country.

Where this approach is used, the designer should consider adding reinforcement in the deck adjacent to the interior supports to control cracking that may occur from the continuous action of the structure.

Positive moment connections improve the structural integrity of a bridge, increasing its ability to resist extreme event and unanticipated loadings. These connections also control cracking that may occur in the continuity diaphragm. Therefore, it is recommended

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that positive moment connections be provided in all bridges detailed as continuous for live load.

5.12.3.3.2—Restraint Moments

The bridge shall be designed for restraint moments that may develop because of time-dependent or other deformations, except as allowed in Article 5.12.3.3.4.

Restraint moments shall not be included in any combination when the effect of the restraint moment is to reduce the total moment.

C5.12.3.3.2

Deformations that occur after continuity is established from time-dependent effects such as creep, shrinkage and temperature variation cause restraint moments.

Restraint moments are computed at interior supports of continuous bridges but affect the design moments at all locations on the bridge. Studies show that restraint moments can be positive or negative. The magnitude and direction of the moments depend on girder age at the time continuity is established, properties of the girder and slab concrete, and bridge and girder geometry. (Mirmiran et al., 2001). The data show that the later continuity is formed, the lower the predicted values of positive restraint moment which will form. Since positive restraint moments are generally not desirable except where the superstructure is intended to behave as though it was integral with the substructure, waiting as long as possible after the girders are cast to establish continuity and cast the deck appears to be beneficial.

Several methods have been published for computing restraint moments (Mirmiran et al., 2001). While these methods may be useful in estimating restraint moments, designers should be aware that these methods may overestimate the restraint moments—both positive and negative. Existing structures do not show the distress that would be expected from the moments computed by some analysis methods.

Most analysis methods indicate that differential shrinkage between the girder and deck mitigates positive moment formation. Data from various projects (Miller et al., 2004; Russell et al., 2003) does not show the effects of differential shrinkage. Therefore, it is questionable whether negative moments due to differential shrinkage form to the extent predicted by analysis. Since field observations of significant negative moment distress have not been reported, negative moments caused by differential shrinkage are often ignored in design.

Estimated restraint moments are highly dependent on actual material properties and project schedules and the computed restraint moments may never develop. Therefore, a critical design moment must not be reduced by a restraint moment in case the restraint moment does not develop.

5.12.3.3.3—Material Properties

Creep and shrinkage properties of the girder concrete and the shrinkage properties of the deck slab concrete shall be determined from either of the following:

C5.12.3.3.3

The development of restraint moments is highly dependent on the creep and shrinkage properties of the girder and deck concrete. Since these properties can vary widely, measured properties should be used when available to obtain the most accurate analysis. However,

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- Tests of concrete using the same proportions and materials that will be used in the girders and deck slab. Measurements shall include the time-dependent rate of change of these properties.
- The provisions of Article 5.4.2.3.

The restraining effect of reinforcement on concrete shrinkage may be considered.

these properties are rarely available during design. Therefore, the provisions of Article 5.4.2.3 may be used to estimate these properties.

Because longitudinal reinforcement in the deck slab restrains the shrinkage of the deck concrete, the apparent shrinkage is less than the free shrinkage of the deck concrete. This effect may be estimated using an effective concrete shrinkage strain, $\varepsilon_{effective}$, which may be taken as:

$$\varepsilon_{effective} = \varepsilon_{sh}\left(\frac{A_c}{A_{tr}}\right) \quad \text{(C5.12.3.3.3-1)}$$

where:

ε_{sh} = unrestrained shrinkage strain for deck concrete (in./in.)
A_c = gross area of concrete deck slab (in.^{2})
A_{tr} = area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2})
= $A_c + A_s(n - 1)$
A_s = total area of longitudinal deck reinforcement (in.^{2})
n = modular ratio between deck concrete and reinforcement
= $E_s / E_{c\,deck}$
$E_{c\,deck}$ = modulus of elasticity of deck concrete (ksi)

Eq. C5.12.3.3.3-1 is based on simple mechanics (Abdalla et al., 1993). If the amount of longitudinal reinforcement varies along the length of the slab, the average area of longitudinal reinforcement may be used to calculate the transformed area.

5.12.3.3.4—Age of Girder When Continuity Is Established

The minimum age of the precast girder when continuity is established should be specified in the contract documents. This age shall be used for calculating restraint moments due to creep and shrinkage. If no age is specified, a reasonable, but conservative estimate of the time continuity is established shall be used for all calculations of restraint moments.

The following simplification may be applied if acceptable to the Owner and if the contract documents require a minimum girder age of at least 90 days when continuity is established:

- Positive restraint moments caused by girder creep and shrinkage and negative restraint moments at piers caused by deck slab shrinkage may be taken to be zero.
- Computation of restraint moments shall not be required.

C5.12.3.3.4

Analytical studies show that the age of the precast girder when continuity is established is an important factor in the development of restraint moments (Mirmiran et al., 2001). According to analysis, establishing continuity when girders are young causes larger positive moments to develop. Therefore, if no minimum girder age for continuity is specified, the earliest reasonable age must be used. Results from surveys of practice (Miller et al., 2004) show a wide variation in girder ages at which continuity is established. An age of seven days was reported to be a realistic minimum. However, the use of seven days as the age of girders when continuity is established results in a large positive restraint moment. Therefore, a specified minimum girder age at continuity of at least 28 days is strongly recommended.

If girders are 90 days or older when continuity is established, the provisions of Article 5.4.2.3 predict that approximately 60 percent of the creep and 70 percent of

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A positive moment connection shall be provided with a factored resistance, ϕM_n, not less than $1.2M_{cr}$, as specified in Article 5.12.3.3.9.

For other ages at continuity, the age-related design parameters should be determined from the literature, approved by the Owner, and documented in the contract documents.

the shrinkage in the girders, which could cause positive moments, has already occurred prior to establishing continuity. The Owner may allow the use of k_{td} in Eq. 5.4.2.3.2-5 set at 0.7 to determine the time at which continuity can be established and, therefore, utilize the 90-day provisions of this article. Since most of the creep and shrinkage in the girder has already occurred before continuity is established, the potential development of time-dependent positive moments is limited. Differential shrinkage between the deck and the girders, to the extent to which it actually occurs (refer to Article C5.12.3.3.2) would also tend to limit positive moment development.

Even if the girders are 90 days old or older when continuity is established, some positive moment may develop at the connection and some cracking may occur. Research (Miller et al., 2004) has shown that if the connection is designed with a capacity of $1.2M_{cr}$, the connection can tolerate this cracking without appreciable loss of continuity.

This provision provides a simplified approach to design of precast girder bridges made continuous that eliminates the need to evaluate restraint moments. Some states allow design methods where restraint moments are not evaluated when continuity is established when girders are older than a specified age. These design methods have been used for many years with good success. However, an Owner may require the computation of restraint moments for all girder ages.

5.12.3.3.5—Degree of Continuity at Various Limit States

Both a positive and negative moment connection, as specified in Articles 5.12.3.3.8 and 5.12.3.3.9, are required for all continuity diaphragms, regardless of the degree of continuity as defined in this article.

The connection between precast girders at a continuity diaphragm shall be considered fully effective if either of the following are satisfied:

- The calculated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent loads, settlement, creep, shrinkage, 50 percent live load and temperature gradient, if applicable, is compressive.
- The contract documents require that the age of the precast girders shall be at least 90 days when continuity is established and the design simplifications of Article 5.12.3.3.4 are used.

If the connection between precast girders at a continuity diaphragm does not satisfy these requirements, the joint shall be considered partially effective.

Superstructures with fully effective connections at interior supports may be designed as fully continuous structures for loads applied after continuity is established.

C5.12.3.3.5

A fully effective joint at a continuity diaphragm is a joint that is capable of full moment transfer between spans, resulting in the structure behaving as a continuous structure.

In some cases, especially when continuity is established at an early girder age, continuing upward cambering of the girders due to creep may cause cracking at the bottom of the continuity diaphragm (Mirmiran et al., 2001). Analysis and tests indicate that such cracking may cause the structure to act as a series of simply supported spans when resisting some portion of the permanent or live loads applied after continuity is established, however, this condition only occurs when the cracking is severe and the positive moment connection is near failure (Miller et al., 2004). Where this occurs, the connections at the continuity diaphragm are partially effective.

Theoretically, the portion of the permanent or live loads required to close the cracks would be applied to a simply supported span, neglecting continuity. The remainder of the load would then be applied to the continuous span, assuming full continuity. However, in cases where the portion of the live load required to close the crack is less than 50 percent of the live load, placing part of the load on simple spans and placing the remainder on the continuous bridge results in only a

Superstructures with partially effective connections at interior supports shall be designed as continuous structures for loads applied after continuity is established for strength limit states only.

Gross composite girder section properties, ignoring any deck cracking, may be used for analysis as specified in Article 4.5.2.2.

If the negative moment resistance of the section at an interior support is less than the total amount required, the positive design moments in the adjacent spans shall be increased appropriately for each limit state investigated.

small change in total stresses at critical sections due to all loads. Tests have shown that the connections can tolerate some positive moment cracking and remain continuous (Miller et al., 2004). Therefore, if the conditions of the first bullet point are satisfied, it is reasonable to design the member as continuous for the entire load placed on the structure after continuity is established.

The second bullet follows from the requirements of Article 5.12.3.3.4 where restraint moments may be neglected if continuity is established when the age of the precast girder is at least 90 days. Without positive moment, the potential cracks in the continuity diaphragm would not form and the connection would be fully effective.

Partially effective construction joints are designed by applying the portion of the permanent and live loads applied after continuity is established to a simple span (neglecting continuity). Only the portion of the loads required to close the assumed cracks is applied. The remainder of the permanent and live loads would then be applied to the continuous span. The load required to close the crack can be taken as the load causing zero tension at the bottom of the continuity diaphragm. Such analysis may be avoided if the contract documents require the age of the girder at continuity to be at least 90 days.

5.12.3.3.6—Service Limit State

Simple-span precast girders made continuous shall be designed to satisfy service limit state stress limits given in Article 5.9.2.3. For service load combinations that involve traffic loading, tensile stresses in prestressed members shall be investigated using the Service III load combination specified in Table 3.4.1-1.

At the service limit state after losses, when tensile stresses develop at the top of the girders near interior supports, the tensile stress limits specified in Table 5.9.2.3.1b-1 for other than segmentally constructed bridges shall apply. The design concrete compressive strength of the girder concrete, f'_c, shall be substituted for f'_{ci} in the stress limit equations. The Service III load combination shall be used to compute tensile stresses for these locations.

Alternatively, the top of the precast girders at interior supports may be designed as reinforced concrete members at the strength limit state. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

A cast-in-place composite deck slab shall not be subject to the tensile stress limits for the service limit state after losses specified in Table 5.9.2.3.2b-1.

C5.12.3.3.6

Tensile stresses under service limit state loadings may occur at the top of the girder near interior supports. This region of the girder is not a precompressed tensile zone, so there is not an applicable tensile stress limit in Table 5.9.2.3.2b-1. Furthermore, the tensile zone is close to the end of the girder, so adding or debonding pretensioned strands has little effect in reducing the tensile stresses. Therefore, the limits specified for temporary stresses before losses have been used to address this condition, with modification to use the design concrete compressive strength. This provision provides some relief for the potentially high tensile stresses that may develop at the ends of girders because of negative service load moments.

This option allows the top of the girder at the interior support to be designed as a reinforced concrete element using the strength limit state rather than a prestressed concrete element using the service limit state.

The deck slab is not a prestressed element. Therefore, the tensile stress limits do not apply. It has been customary to apply the compressive stress limits to the deck slab.

5.12.3.3.7—Strength Limit State

The connections between precast girders and a continuity diaphragm shall be designed for the strength limit state.

C5.12.3.3.7

The continuity diaphragm is not prestressed concrete so the stress limits for the service limit state do

The reinforcement in the deck slab shall be proportioned to resist negative design moments at the strength limit state.

not apply. Connections to it are therefore designed using provisions for reinforced concrete elements.

5.12.3.3.8—Negative Moment Connections

The reinforcement in a cast-in-place, composite deck slab in a multi-span precast girder bridge made continuous shall be proportioned to resist negative design moments at the strength limit state.

Longitudinal reinforcement used for the negative moment connection over an interior pier shall be anchored in regions of the slab that are in compression at strength limit states and shall satisfy the requirements of Article 5.10.8.1.2c. The termination of this reinforcement shall be staggered. All longitudinal reinforcement in the deck slab may be used for the negative moment connection.

Negative moment connections between precast girders into or across the continuity diaphragm shall satisfy the requirements of Article 5.10.8.4. These connections shall be permitted where the bridge is designed with a composite deck slab and shall be required where the bridge is designed without a composite deck slab. Additional connection details shall be permitted if the strength and performance of these connections is verified by analysis or testing.

The requirements of Article 5.6.3 shall apply to the reinforcement in the deck slab and at negative moment connections at continuity diaphragms.

C5.12.3.3.8

Research at PCA (Kaar et al., 1961) and years of experience show that the reinforcement in a composite deck slab can be proportioned to resist negative design moments in a continuous bridge.

Limited tests on continuous model and full size structural components indicate that, unless the reinforcement is anchored in a compressive zone, the effectiveness becomes questionable at the strength limit state (Priestly et al, 1993). The termination of the longitudinal deck slab reinforcement is staggered to minimize potential deck cracking by distributing local force effects.

A negative moment connection between precast girders and the continuity diaphragm is not typically provided, because the deck slab reinforcement is usually proportioned to resist the negative design moments. However, research (Ma et al., 1998) suggests that mechanical connections between the tops of girders may also be used for negative moment connections, especially when continuity is established prior to placement of the deck slab. If a composite deck slab is not used on the bridge, a negative moment connection between girders is required to obtain continuity. Mechanical reinforcement splices have been successfully used to provide a negative moment connection between box beam bridges that do not have a composite deck slab.

5.12.3.3.9—Positive Moment Connections

5.12.3.3.9a—General

Positive moment connections at continuity diaphragms shall be made with reinforcement developed into both the girder and continuity diaphragm. Three types of connections shall be permitted:

- Nonprestressed reinforcement embedded in the precast girders and developed into the continuity diaphragm.
- Pretensioning strands extended beyond the end of the girder and anchored into the continuity diaphragm. These strands shall not be debonded at the end of the girder.
- Any connection detail shown by analysis, testing or as approved by the Bridge Owner to provide adequate positive moment resistance.

Additional requirements for connections made using each type of reinforcement are given in subsequent Articles.

The critical section for the development of positive moment reinforcement into the continuity diaphragm shall be taken at the face of the girder. The critical

C5.12.3.3.9a

Positive moment connections improve the structural integrity of a bridge, increasing its ability to resist extreme event and unanticipated loadings. Therefore, it is recommended that positive moment connections be provided in all bridges detailed as continuous for live load.

Both embedded bar and extended strand connections have been used successfully to provide positive moment resistance. Test results (Miller et al., 2004) indicate that connections using the two types of reinforcement perform similarly under both static and fatigue loads and both have adequate strength to resist the applied moments.

Analytical studies (Mirmiran et al., 2001) suggest that a minimum amount of reinforcement, corresponding to a capacity of $0.6M_{cr}$ is needed to develop adequate resistance to positive restraint moments. These same studies show that a positive moment connection with a capacity greater than $1.2M_{cr}$ provides only minor improvement in continuity behavior over a connection with a capacity of $1.2M_{cr}$. Therefore, it is recommended that the section for the development of positive moment reinforcement into the precast girder shall consider conditions in the girder as specified in this article for the type of reinforcement used.

The requirements of Article 5.6.3, except Article 5.6.3.3, shall apply to the reinforcement at positive moment connections at continuity diaphragms. This reinforcement shall be proportioned to resist the larger of the following, except when using the design simplifications of Article 5.12.3.3.4:

- Factored positive restraint moment; or
- $0.6M_{cr}$

The cracking moment M_{cr} shall be computed using Eq. 5.6.3.5.2-2 with the gross composite section properties for the girder and the effective width of composite deck slab, if any, and the material properties of the concrete in the continuity diaphragm.

The precast girders shall be designed for any positive restraint moments that are used in design. Near the ends of girders, the reduced effect of prestress within the transfer length shall be considered.

positive moment capacity of the connection not exceed $1.2M_{cr}$. If the computed positive moment exceeds $1.2M_{cr}$, the section should be modified or steps should be taken to reduce the positive moment.

The cracking moment M_{cr} is the moment that causes cracking in the continuity diaphragm. Since the continuity diaphragm is not a prestressed concrete section, the equation for computing the cracking moment for a reinforced section is used. The diaphragm is generally cast with the deck concrete, so the section properties are computed using uniform concrete properties, so the deck width is not transformed.

Article 5.6.3.3 specifies a minimum capacity for all flexural sections. This is to prevent sudden collapse at the formation of the first crack. However, the positive moment connection that is being discussed here is not intended to resist applied live loads. Even if the positive moment connection were to fail completely, the system may, at worst, become a series of simple spans. Therefore, the minimum reinforcement requirement of Article 5.6.3.3 does not apply. Allowing positive moment connections with lower quantities of reinforcement will relieve congestion in continuity diaphragms.

5.12.3.3.9b—Positive Moment Connection Using Nonprestressed Reinforcement

The anchorage of nonprestressed reinforcement used for positive moment connections shall satisfy the requirements of Article 5.10.8 and the additional requirements of this article. Where positive moment reinforcement is added between pretensioned strands, consolidation of concrete and bond of reinforcement shall be considered.

The critical section for the development of positive moment reinforcement into the precast girder shall consider conditions in the girder. The reinforcement shall be developed beyond the inside edge of the bearing area. The reinforcement shall also be detailed so that, for strands considered in resisting positive moments within the end of the girder, debonding of strands does not terminate within the development length.

Where multiple bars are used for a positive moment connection, the termination of the reinforcement shall be staggered in pairs symmetrical about the centerline of the precast girder.

C5.12.3.3.9b

The positive moment connection is designed to utilize the yield strength of the reinforcement. Therefore, the connection must be detailed to provide full development of the reinforcement. If the reinforcement cannot be detailed for full development, the connection may be designed using a reduced stress in the reinforcement.

Potential cracks are more likely to form in the precast girder at the inside edge of the bearing area and locations of termination of debonding. Since cracking within the development length reduces the effectiveness of the development, the reinforcement should be detailed to avoid this condition. It is recommended that reinforcement be developed beyond the location where a crack radiating from the inside edge of the bearing may cross the reinforcement.

The termination of the positive moment reinforcement is staggered to reduce the potential for cracking at the ends of the bars.

5.12.3.3.9c—Positive Moment Connection Using Prestressing Strand

Pretensioning strands that are not debonded at the end of the girder may be extended into the continuity diaphragm as positive moment reinforcement. The extended strands shall be anchored into the diaphragm by bending the strands into a 90-degree hook or by

C5.12.3.3.9c

Strands that are debonded at the end of a member may not be used as reinforcement for the positive moment connection. There are no requirements for development of the strand into the girder because the strands run continuously through the precast girder.

providing a development length as specified in Article 5.9.4.3.

The stress in the strands used for design, as a function of the total length of the strand, shall not exceed:

$$f_{psl} = \frac{(\ell_{dsh} - 8)}{0.228} \quad (5.12.3.3.9c\text{-}1)$$

$$f_{pul} = \frac{(\ell_{dsh} - 8)}{0.163} \quad (5.12.3.3.9c\text{-}2)$$

where:

f_{psl} = stress in the strand at the service limit state cracked section shall be assumed (ksi)
ℓ_{dsh} = total length of extended strand (in.)
f_{pul} = stress in the strand at the strength limit state (ksi)

Strands shall project at least 8.0 in. from the face of the girder before they are bent.

Eqs. 5.12.3.3.9c-1 and 5.12.3.3.9c-2 were developed for 0.5-in. strand by Salmons et al. (1980).

These are for prestressing strand extended from the end of the girder and given 90-degree hooks. Other equations are also available to estimate stress in bent strands (Noppakunwijai et al., 2002).

5.12.3.3.9d—Details of Positive Moment Connection

Positive moment reinforcement shall be placed in a pattern that is symmetrical, or as nearly symmetrical as possible, about the centerline of the cross section.

Fabrication and erection issues shall be considered in the detailing of positive moment reinforcement in the continuity diaphragm. Reinforcement from opposing girders shall be detailed to mesh during erection without significant conflicts. Reinforcement shall be detailed to enable placement of anchor bars and other reinforcement in the continuity diaphragm.

C5.12.3.3.9d

Tests (Miller et al., 2004) suggest that reinforcement patterns that have significant asymmetry may result in unequal bar stresses that can be detrimental to the performance of the positive moment connection.

With some girder shapes, it may not be possible to install prebent hooked bars without the hook tails interfering with the formwork. In such cases, a straight bar may be embedded and then bent after the girder is fabricated. Such bending is generally accomplished without heating and the bend must be smooth with a minimum bend diameter conforming to the requirements of Table 5.10.2.3-1. If the Engineer allows the reinforcement to be bent after the girder is fabricated, the contract documents shall indicate that field bending is permissible and shall provide requirements for such bending. Since requirements regarding field bending may vary, the preferences of the Owner should be considered.

Hairpin bars (a bar with a 180-degree bend with both legs developed into the precast girder) have been used for positive moment connections to eliminate the need for post-fabrication bending of the reinforcement and reduce congestion in the continuity diaphragm.

5.12.3.3.10—Continuity Diaphragms

The design of continuity diaphragms at interior supports may be based on the strength of the concrete in the precast girders.

Precast girders may be embedded into continuity diaphragms.

C5.12.3.3.10

The use of the increased concrete strength is permitted because the continuity diaphragm concrete between girder ends is confined by the girders and by the continuity diaphragm extending beyond the girders. It is recommended that this provision be applied only to

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If horizontal diaphragm reinforcement is passed through holes in the precast beam or is attached to the precast element using mechanical connectors, the end precast element shall be designed to resist positive moments caused by superimposed dead loads, live loads, creep and shrinkage of the girders, shrinkage of the deck slab, and temperature effects. Design of the end of the girder shall account for the reduced effect of prestress within the transfer length.

Where ends of girders are not directly opposite each other across a continuity diaphragm, the diaphragm must be designed to transfer forces between girders. Continuity diaphragms shall also be designed for situations where an angle change occurs between opposing girders.

conditions where the portion of the continuity diaphragm that is in compression is confined between ends of precast girders.

The width of the continuity diaphragm must be large enough to provide the required embedment for the development of the positive moment reinforcement into the diaphragm. An anchor bar with a diameter equal to or greater than the diameter of the positive moment reinforcement may be placed in the corner of a 90-degree hook or inside the loop of a 180-degree hook bar to improve the effectiveness of the anchorage of the reinforcement.

Several construction sequences have been successfully used for the construction of bridges with precast girders made continuous. When determining the construction sequence, the Engineer should consider the effect of girder rotations and restraint as the deck slab concrete is being placed.

Test results (Miller et al., 2004) have shown that embedding precast girders 6.0 in. into continuity diaphragms improves the performance of positive moment connections. The observed stresses in the positive moment reinforcement in the continuity diaphragm were reduced compared to connections without girder embedment.

The connection between precast girders and the continuity diaphragm may be enhanced by passing horizontal reinforcement through holes in the precast beam or attaching the reinforcement to the beam by embedded connectors. Test results (Miller et al., 2004; Salmons, 1980) show that such reinforcement stiffens the connection. The use of such mechanical connections requires that the end of the girder be embedded into the continuity diaphragm. Tests of continuity diaphragms without mechanical connections between the girder and diaphragm show the failure of connection occurs by the beam end pulling out of the diaphragm with all of the damage occurring in the diaphragm. Tests of connections with horizontal bars show that cracks may form in the end of the precast girder outside the continuity diaphragm if the connection is subjected to a significant positive moment. Such cracking in the end region of the girder may not be desirable.

A method such as given in Article 5.8.2 may be used to design a continuity diaphragm for these conditions.

5.12.3.4—Spliced Precast Girders

5.12.3.4.1—General

The provisions herein apply to precast girders fabricated in segments that are joined or spliced longitudinally to form the girders in the final structure.

The requirements specified herein shall supplement the requirements of other Sections of these Specifications for other than segmentally constructed bridges. There-fore, spliced precast girder bridges shall not be considered as segmental construction for the purposes of design. For special design cases, additional provisions for segmental construction found in

C5.12.3.4.1

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Article 5.12.5 and other Articles in these Specifications may be used where appropriate.

The method of construction assumed for the design shall be shown in the contract documents. All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor to the construction method or to the design shall comply with the requirements of Article 5.12.5.5.

Stresses due to changes in the statical system, in particular, the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude.

Spliced girder superstructures which satisfy all service limit state requirements of this article may be designed as fully continuous at all limit states for loads applied after the girder segments are joined.

Prestress losses in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.9.3. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered.

Where required, the effects of creep and shrinkage in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.4.2.3.

Precast deck girder bridges, for which some or all of the deck is cast integrally with a girder, may be spliced. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of Article 5.12.2.3.

Spliced precast girders may be made continuous for some permanent loads using details for simple span precast girders made continuous. In such cases, design shall conform to the applicable requirements of Article 5.12.3.3.

Spliced precast girder bridges may be distinguished from what is referred to as "segmental construction" elsewhere in these specifications by several features which typically include:

- The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span. In some cases, the segment may be the full span length.
- Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.
- Cast-in-place closure joints are usually used to join girder segments rather than match-cast joints.
- The bridge cross section is comprised of several individual girders with a cast-in-place concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be divided into pieces that are integrally cast with each girder. A bridge of this type is completed by connecting the girders across the longitudinal joints.
- Girder sections are used, such as bulb tee or open-topped trapezoidal boxes, rather than closed cell boxes with wide monolithic flanges.

Provisional ducts are required for segmental construction (Article 5.12.5.3.9a) to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced precast girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

The method of construction and any required temporary support is of paramount importance in the design of spliced precast girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

5.12.3.4.2—Joints Between Spliced Girders

5.12.3.4.2a—General

Joints between girder segments shall be either cast-in-place closure joints or match-cast joints. Match-cast joints shall satisfy the requirements of Article 5.12.5.4.2.

The sequence of placing concrete for the closure joints and deck shall be specified in the contract documents.

C5.12.3.4.2a

This Article codifies current best practice, which allows the Designer considerable latitude to formulate new structural systems. The great majority of in-span construction joints have been post-tensioned. Conventionally reinforced joints have been used in a limited number of bridges.

Cast-in-place closure joints are typically used in spliced girder construction. Machined bulkheads have been used successfully to emulate match-cast epoxy joints for spliced girders. Prestress, dead load, and creep effects may cause rotation of the faces of the match-cast epoxy joints prior to splicing. Procedures for splicing the girder segments that overcome this rotation to close the match-cast joint should be shown on the contract plans.

5.12.3.4.2b—Details of Closure Joints

Precast concrete girder segments, with or without a cast-in-place slab, may be made longitudinally continuous for both permanent and transient loads with combinations of post-tensioning and/or reinforcement crossing the closure joints.

The width of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of post-tensioning ducts. The width of a closure joint shall not be less than 12.0 in., except for joints located within a diaphragm, for which the width shall not be less than 4.0 in.

If the width of the closure joint exceeds 6.0 in., its compressive chord section shall be reinforced for confinement.

If the joint is located in the span, its web reinforcement, A_s/s, shall be the larger of that in the adjacent girders.

The face of the precast segments at closure joints shall be specified as either intentionally roughened to expose coarse aggregate or having shear keys in accordance with Article 5.12.5.4.2.

C5.12.3.4.2b

Where diaphragms are provided at closure joint locations, designers should consider extending the closure joint at the exterior girder beyond the outside face of the girder. Extending the closure joint beyond the face of the exterior girder also provides improved development of diaphragm reinforcement for bridges subject to extreme events.

The intent of the joint width requirement is to allow proper compaction of concrete in the cast-in-place closure joint. In some cases, narrower joints have been used successfully. Consolidation of concrete in a closure joint is enhanced when the joint is contained within a diaphragm. A wider closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

The bottom flange near an interior support acts nearly as a column, hence the requirement for confinement steel.

The *AASHTO LRFD Bridge Construction Specifications* requires vertical joints to be keyed. However, proper attention to roughened joint preparation is expected to ensure bond between the segments, providing better shear strength than shear keys.

5.12.3.4.2c—Details of Match-Cast Joints

Match-cast joints for spliced precast girder bridges shall be detailed in accordance with Article 5.12.5.4.2.

C5.12.3.4.2c

One or more large shear keys may be used with spliced girders rather than the multiple small amplitude shear keys indicated in Article 5.12.5.4.2. The shear key proportions specified in Article 5.12.5.4.2 should be used.

5.12.3.4.2d—Joint Design

Stress limits for temporary concrete stresses in joints before losses specified in Article 5.9.2.3.1 for segmentally constructed bridges shall apply at each stage of prestressing (pretensioning or post-tensioning). The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in Article 5.9.2.3.2 for segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

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Resistance factors for joints specified in Article 5.5.4.2 for segmental construction shall apply.

The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations.

5.12.3.4.3—Girder Segment Design

Stress limits for temporary concrete stresses in girder segments before losses specified in Article 5.9.2.3.1 for other than segmentally constructed bridges shall apply at each stage of prestressing (pretensioning or post-tensioning) with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in girder segments at the service limit state after losses specified in Article 5.9.2.3.2 for other than segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Where girder segments are precast without prestressed reinforcement, the provisions of Article 5.6.7 shall apply until post-tensioning is applied.

Where variable depth girder segments are used, the effect of inclined compression shall be considered.

The potential for buckling of tall thin web sections shall be considered.

C5.12.3.4.3

Segments of spliced precast girders shall preferably be pretensioned for dead load and all applicable construction loadings to satisfy temporary stress limits in the concrete.

Temporary construction loads must be considered where these loads may contribute to critical stresses in girder segments at an intermediate stage of construction, such as when the deck slab is placed when only a portion of the total prestress has been applied. Temporary construction loads are specified in the *AASHTO Guide Design Specifications for Bridge Temporary Works* (1995).

Because gravity loads induce compression in the bottom flange of girders at support locations, the vertical force component from inclined flexural stresses in a haunched girder segment generally acts to reduce the applied shear. Its effect can be accounted for in the same manner as the vertical component of the longitudinal prestressing force, V_p. However, the reduction of the vertical shear force from this effect is usually neglected.

5.12.3.4.4—Post-Tensioning

Post-tensioning may be applied either before and/or after placement of deck concrete. Part of the post-tensioning may be applied to provide girder continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement.

The contract documents shall require that all post-tensioning tendons shall be fully grouted after stressing.

Prior to grouting of post-tensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers.

Post-tensioning shall be shown on the contract documents according to the requirements of Article 5.12.5.3.10.

Where tendons terminate at the top of a girder segment, the contract documents shall require that duct openings be protected during construction to prevent debris accumulation and that drains be provided at tendon low points.

In the case of multistage post-tensioning, draped ducts for tendons to be tensioned before the slab concrete is placed and attains the minimum design concrete compressive strength f'_{ci} shall not be located in the slab.

Where some or all post-tensioning tendons are stressed after the deck concrete is placed, provisions shall be shown on the contract plans satisfying the provisions of Article 2.5.2.3 on maintainability of the deck.

C5.12.3.4.4

Where some or all post-tensioning is applied after the deck concrete is placed, fewer post-tensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the post-tensioning is applied before the deck concrete is placed, a greater number of post-tensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary. See Castrodale and White (2004).

See Article 5.9.5.3 for post-tensioning coupler requirements.

Where tendons terminate at the top of the girder, blockouts and pourbacks in the deck slab are required for access to the tendons and anchorages. While this arrangement has been used, it is preferable to anchor all tendons at the ends of girders. Minimizing or eliminating deck slab blockouts by placing anchorages at ends of girders reduces the potential for water seepage and corrosion at the post-tensioning tendon anchors.

This provision is to ensure that ducts as yet unsecured by concrete will not be used for active post-tensioning.

See Article 5.14.5 for deck protection systems.

5.12.3.5—Cast-in-Place Box Girders and T-Beams

5.12.3.5.1—Flange and Web Thickness

5.12.3.5.1a—Top Flange

The thickness of top flanges serving as deck slabs shall not be less than the greatest of the following:

- that determined in Section 9;
- that required for anchorage and cover for transverse prestressing, if used; and
- the clear span between fillets, haunches, or webs divided by 20, unless transverse ribs at a spacing equal to the clear span are used or transverse prestressing is provided.

5.12.3.5.1b—Bottom Flange

The bottom flange thickness shall be not less than the greatest of the following:

- 5.5 in.;
- the distance between fillets or webs of nonprestressed girders and beams divided by 16;
- the clear span between fillets, haunches, or webs for prestressed girders divided by 30, unless transverse ribs at a spacing equal to the clear span are used.

5.12.3.5.1c—Web

The thickness of webs shall be determined by requirements for shear, torsion, concrete cover, and placement of concrete.

Changes in girder web thickness shall be tapered for a minimum distance of 12.0 times the difference in web thickness.

C5.12.3.5.1c

For adequate field placement and consolidation of concrete, the minimum web thickness specified in Article 5.12.5.3.11b should be considered. For girders over about 8.0 ft in depth, these dimensions should be increased to compensate for the increased difficulty of concrete placement.

5.12.3.5.2—Reinforcement

5.12.3.5.2a—Deck Slab Reinforcement Cast-in-Place in T-Beams and Box Girders

The reinforcement in the deck slab of cast-in-place T-beams and box girders may be determined by either the traditional or the empirical design methods specified in Section 9.

Where the deck slab does not extend beyond the exterior web, at least one third of the bottom layer of the transverse reinforcement in the deck slab shall be extended into the exterior face of the outside web and anchored by a standard 90-degree hook. If the slab extends beyond the exterior web, at least one third of the bottom layer of the transverse reinforcement shall be extended into the slab overhang and shall have an anchorage beyond the exterior face of the web not less in resistance than that provided by a standard hook.

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5.12.3.5.2b—Bottom Slab Reinforcement in Cast-in-Place Box Girders

A uniformly distributed reinforcement of 0.4 percent of the flange area shall be placed in the bottom slab parallel to the girder span, either in single or double layers. The spacing of such reinforcement shall not exceed 18.0 in.

A uniformly distributed reinforcement of 0.5 percent of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 18.0 in. All transverse reinforcement in the bottom slab shall be extended to the exterior face of the outside web in each group and shall be anchored by a standard 90-degree hook.

C5.12.3.5.2b

This provision is intended to apply to both reinforced and prestressed boxes.

5.12.4—Diaphragms

Diaphragms, subjected primarily to shear and torsion and whose depth is large relative to their span shall be analyzed and designed by either the strut-and-tie method specified in Article 5.8.2 or legacy methods in Article 5.8.4.

Unless otherwise specified, diaphragms shall be provided at abutments, piers, and hinge joints to resist applied forces and transmit them to points of support.

Intermediate diaphragms may be used between beams in curved systems or where necessary to provide torsional resistance and to support the deck at points of discontinuity or at right angle points of discontinuity or at angle points in girders.

For spread box beams having an inside radius less than 800 ft, intermediate diaphragms shall be used.

In certain types of construction, end diaphragms may be replaced by an edge beam or a strengthened strip of slab made to act as a vertical frame with the beam ends. Such types are low I-beams and double-T beams. These frames should be designed for wheel loads.

The diaphragms should be essentially solid, except for access openings and utility holes, where required.

For curved bridges, the need for and the required spacing of diaphragms depends on the radius of curvature and the proportions of the webs and flanges. Some references have found that interior diaphragms contributed very little to the global behavior of concrete box girder bridges.

5.12.5—Segmental Concrete Bridges

5.12.5.1—General

The requirements specified herein shall supplement the requirements of other sections of these specifications for concrete structures designed to be constructed by the segmental method.

C5.12.5.1

For segmental construction, superstructures of single or multiple box sections are generally used. Segmental construction includes construction by free cantilever, span-by-span, or incremental launching methods using either precast or cast-in-place concrete segments which are connected to produce either continuous or simple spans.

Bridges utilizing beam-type sections may also be constructed using segmental construction techniques. Such bridges, which are referred to as spliced precast girder bridges in these specifications, are considered to be a special case of conventional concrete bridges. The design of such bridges is covered in Article 5.12.3.4.

The span length of bridges considered by these specifications ranges to 800 ft. Bridges supported by stay cables are not specifically covered in this article,

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although many of the specification provisions are also applicable to them.

The method of construction assumed for the design shall be shown in the contract documents. Temporary supports required prior to the time the structure, or component thereof, is capable of supporting itself and subsequently applied loads, shall also be shown in the contract documents.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor in the construction method or in the design shall comply with the requirements of Article 5.12.5.5.

The method of construction and any required temporary support is of paramount importance in the design of segmental concrete bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

For segmentally constructed bridges, designs should and generally do allow the Contractor some latitude in choice of construction methods. To ensure that the design features and details to be used are compatible with the proposed construction method, it is essential that the Contractor is required to prepare working drawings and calculations based on his choice of methods for review and approval by the Engineer before work begins.

5.12.5.2—Analysis of Segmental Bridges

5.12.5.2.1—General

The analysis of segmentally constructed bridges shall conform to the requirements of Section 4 and those specified herein.

5.12.5.2.2—Construction Analysis

For the analysis of the structure during the construction stage, the construction load combinations, stresses, and stability considerations shall be as specified in Article 5.12.5.3.

5.12.5.2.3—Analysis of the Final Structural System

The final structural system shall be analyzed for redistribution of construction-stage force effects due to internal deformations and changes in support and restraint conditions, including accumulated locked-in force effects resulting from the construction process.

C5.12.5.2.3

Results of analyses of a segmental concrete superstructure that has values of creep coefficient of 1, 2, and 3 and that uses both the ACI 209 and CEB-FIP (CEB 1990) creep models, have been published (AASHTO, 1999). Final stresses were essentially unchanged for creep coefficients of 1, 2, and 3 using the ACI 209 creep provisions. Although the analyses with the CEB-FIP creep model show somewhat more variation in final stresses, the range of stresses is still small for a large variation in creep coefficients. The selection of the ACI 209 or CEB-FIP creep model has a larger impact on the final stress values than the creep coefficients. However, it is doubtful that the full range of stresses reflected in the six analyses described would be of practical significance with respect to the performance of the structure.

Because the creep coefficient will be known or determined with reasonable accuracy under the requirements of these specifications, analysis using a single value of the creep coefficient is considered satisfactory, and use of low and high values of the creep coefficient in analysis is generally considered unnecessary. This is not intended to imply that creep values should not be determined accurately because these values do have a significant impact on the prestress losses, deflections, and axial shortening of the structure.

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Joints in segmental girders made continuous by unbonded tendons shall be investigated for the simultaneous effect of axial force, moment, and shear that may occur at a joint. These force effects, the opening of the joint, and the remaining contact surface between the components shall be determined by global consideration of strain and deformation. Shear shall be assumed to be transmitted through the contact area only.

Joining components with unbonded tendons may permit the opening of unreinforced joints at or close to strength limit states. The Designer should review the structural consequences of such joint openings.

5.12.5.3—Design

5.12.5.3.1—Loads

In addition to the loads specified in Section 3, the construction loads specified in Articles 5.12.5.3.2 through 5.12.5.3.4 shall be considered.

5.12.5.3.2—Construction Loads

Construction loads and conditions that are assumed in the design and that determine section dimensions, camber, and reinforcing and/or prestressing requirements shall be shown as maximum allowed in the contract documents. In addition to erection loads, any required temporary supports or restraints shall be defined as to magnitude or included as part of the design. The acceptable closure forces due to misalignment corrections shall be stated. Due allowance shall be made for all effects of any changes of the statical structural scheme during construction and the application, changes, or removal of the assumed temporary supports of special equipment, taking into account residual force effects, deformations, and any strain-induced effects.

The following construction loads shall be considered:

DC = weight of the supported structure (kip)

DIFF = differential load: applicable only to balanced cantilever construction taken as two percent of the dead load applied to one cantilever (kip)

DW = superimposed dead load if present at the stage of erection under consideration (kip or klf)

CLL = distributed construction live load: an allowance for miscellaneous items of plant, machinery, and other equipment, apart from the major specialized erection equipment; taken as 0.010 ksf of deck area; in cantilever construction, this load is taken as 0.010 ksf on one cantilever and 0.005 ksf on the other; for bridges built by incremental launching, this load may be neglected (ksf)

CEQ = specialized construction equipment: the load from segment or material delivery trucks, or both, and any special equipment, including a formtraveler launching gantry, beam and winch, truss, or similar major auxiliary structure and the maximum loads

C5.12.5.3.2

Construction loads comprise all loadings arising from the Designer's anticipated system of temporary supporting works and/or special erection equipment to be used in accordance with the assumed construction sequence and schedule.

Construction loads and conditions frequently determine section dimensions and reinforcing and/or prestressing requirements in segmentally constructed bridges. It is important that the Designer shows these assumed conditions in the contract documents.

These provisions are not meant to be limitations on the Contractor as to the means that may be used for construction. Controls are essential to prevent damage to the structure during construction and to ensure adequacy of the completed structure. It is also essential for the bidders to be able to determine if their equipment and proposed construction methods can be used without modifying the design or the equipment.

The contract documents should require the Engineer's approval of any changes in the assumed erection loadings or conditions.

Construction loads may be imposed on opposing cantilever ends by use of the formtraveler, diagonal alignment bars, a jacking tower, or external weights. Cooling of one cantilever with water has also been used to provide adjustment of misalignment. Any misalignment of interior cantilevers should be corrected at both ends before constructing either closure. The frame connecting cantilever ends at closure pours should be detailed to prevent differential rotation between cantilevers until the final structural connection is complete. The magnitude of closure forces should not induce stresses in the structure in excess of those tabulated in Table 5.12.5.3.3-1.

The load *DIFF* allows for possible variations in cross-section weight due to construction irregularities.

For very gradual lifting of segments, where the load involves small dynamic effects, the dynamic load *IE* may be taken as ten percent of the lifted weight.

The following information is based on some past experience and should be considered very preliminary.

applied to the structure by the equipment during the lifting of segments (kip)

IE = dynamic load from equipment: determined according to the type of machinery anticipated (kip)

CLE = longitudinal construction equipment load: the longitudinal load from the construction equipment (kip)

U = segment unbalance: the effect of any out-of-balance segments or other unusual conditions as applicable; applies primarily to balanced cantilever construction but may be extended to include any unusual lifting sequence that may not be a primary feature of the generic construction system (kip)

WS = horizontal wind load on structures in accordance with the provisions of Section 3 The wind speed associated with Service I load combination for load combinations e and f in Table 5.12.5.3.3-1 and the wind speed associated with Service IV load combination for load combinations c and d in Table 5.12.5.3.3-1 shall be as determined by the Owner (ksf)

WE = horizontal wind load on equipment; taken as 0.1 ksf of exposed surface (ksf)

WUP = wind uplift on cantilever: 0.005 ksf of deck area for balanced cantilever construction applied to one side only, unless an analysis of site conditions or structure configuration indicates otherwise (ksf)

A = static weight of precast segment being handled (kip)

AI = dynamic response due to accidental release or application of a precast segment load or other sudden application of an otherwise static load to be added to the dead load; taken as 100 percent of load *A* (kip)

CR = creep effects in accordance with Article 5.12.5.3.6

SH = shrinkage in accordance with Article 5.12.5.3.6

T = thermal: the sum of the effects due to uniform temperature variation (*TU*) and temperature gradients (*TG*) (°F)

5.12.5.3.3—Construction Load Combinations at the Service Limit State

Flexural tension and principal tension stresses shall be determined at service limit states as specified in Table 5.12.5.3.3-1, for which the following notes apply:

- Note 1: equipment not working;
- Note 2: normal erection; and
- Note 3: moving equipment.

For the construction load cases specified herein the stress limits shall conform to Table 5.12.5.3.3-1.

Formtravelers for cast-in-place segmental construction for a typical two-lane bridge with 15.0 to 16.0 ft segments may be estimated to weigh 160 to 180 kips. Weight of formtravelers for wider double-celled box sections may range up to approximately 280 kips. Consultation with contractors or subcontractors experienced in free cantilever construction, with respect to the specific bridge geometry under consideration, is recommended to obtain a design value for formtraveler weight.

The distribution and application of the individual erection loads appropriate to a construction phase shall be selected to produce the most unfavorable effects. The construction load compressive stress in concrete shall not exceed $0.50f'_c$, where f'_c is the design concrete compressive strength at the time of load application.

Tensile stresses in concrete due to construction loads shall not exceed the values specified in Table 5.12.5.3.3-1, except for structures with less than 60 percent of their tendon capacity provided by internal tendons, in which case the tensile stresses shall not exceed $0.095\sqrt{f'_c}$. The requirements of Table 5.12.5.3.3-1 shall apply to vertically post-tensioned substructures. The requirements of Table 5.12.5.3.3-1 shall not be applied to construction of cast-in-place substructures supporting segmental superstructures.

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Table 5.12.5.3.3-1—Load Factors and Tensile Stress Limits for Construction Load Combinations

Load Combination	LOAD FACTORS															STRESS LIMITS				See Note
	Dead Load			Live Load			Wind Load			Other Loads					Earth Loads	Flexural Tension		Principal Tension		
	DC *DW*	*DIFF*	*U*	*CEQ* *CLL*	*IE*	*CLE*	*WS*	*WUP*	*WE*	*CR*	*SH*	*TU*	*TG*	A AI *WA*	*EH* *EV* *ES*	Excluding "Other Loads"	Including "Other Loads"	Excluding "Other Loads"	Including "Other Loads"	
a	1.0	1.0	0.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	—
b	1.0	0.0	1.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	—
c	1.0	1.0	0.0	0.0	0.0	0.0	1.0	0.7	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	—
d	1.0	1.0	0.0	1.0	0.0	0.0	1.0	1.0	0.7	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	1
e	1.0	0.0	1.0	1.0	1.0	0.0	1.0	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	2
f	1.0	0.0	0.0	1.0	1.0	1.0	1.0	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.190\sqrt{f'_c}$	$0.220\sqrt{f'_c}$	$0.110\sqrt{f'_c}$	$0.126\sqrt{f'_c}$	3

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5.12.5.3.4—Construction Load Combinations at Strength Limit States

The minimum factored resistance of a component shall be determined using resistance factors specified in Article 5.5.4.2 and the load combinations specified in Articles 5.12.5.3.4a and 5.12.5.3.4b.

5.12.5.3.4a—Superstructure Load Effects and Structural Stability

- For maximum force effects:

$$\Sigma\gamma Q = 1.1(DC + DIFF) + 1.3(CEQ + CLL) + A + AI \quad (5.12.5.3.4\text{a-}1)$$

- For minimum force effects:

$$\Sigma\gamma Q = DC + CEQ + A + AI \quad (5.12.5.3.4\text{a-}2)$$

C5.12.5.3.4a

Eqs. 5.12.5.3.4a-1 and 5.12.5.3.4a-2 are strength checks for accident conditions only, and are not intended as alternative strength criteria in lieu of the service stress checks in Table 5.12.5.3.3-1.

5.12.5.3.4b—Substructures

The Strength I, III, and V load combinations from Table 3.4.1-1 shall apply. The loads *DIFF* and *CEQ* shall be included and factored with γ_{DC}. The load *WUP* shall be included and factored with γ_{WS}. The loads CLL and WE shall be included and used in place of LL and WL, respectively. Load factors for DC, DW, and WS shall be taken as specified in Article 3.4.2.

Construction strength load combinations shall also include load combinations from Eqs. 5.12.5.3.4a-1 and 5.12.5.3.4a-2. The dynamic response or dynamic allowance (AI) shall be applied to substructure elements above the drilled shaft or footing including the column to foundation connection.

C5.12.5.3.4b

Substructures for post-tensioned segmental superstructures should be reviewed for construction stage demands using the design basis for the strength limit state consistent with reinforced concrete design. Conventionally reinforced segmental superstructures, such as arches, should be similarly reviewed. A reduced load factor may be appropriate for the loads CLL and WE if the construction equipment is well defined during design.

5.12.5.3.5—Thermal Effects During Construction

Thermal effects that may occur during the construction of the bridge shall be considered.

The temperature setting variations for bearings and expansion joints shall be stated in the contract documents.

C5.12.5.3.5

The provisions of Article 3.12 relate to annual temperature variations and should be adjusted for the actual duration of superstructure construction as well as for local conditions.

Transverse analysis for the effects of differential temperature outside and inside box girder sections is not generally considered necessary. However, such an analysis may be necessary for relatively shallow bridges with thick webs. In that case, a ±10.0°F temperature differential is recommended.

5.12.5.3.6—Creep and Shrinkage

Creep coefficient $\Psi(t, t_i)$ shall be determined in accordance with Article 5.4.2.3 or by comprehensive tests. Stresses shall be determined for redistribution of restraint stresses developed by creep and shrinkage that are based on the assumed construction schedule as stated in the contract documents.

For determining the final post-tensioning forces, prestress losses shall be calculated for the construction schedule stated in the contract documents.

C5.12.5.3.6

A variety of computer programs and analytical procedures have been published to determine creep and shrinkage effects in segmental concrete bridges.

Creep strains and prestress losses that occur after closure of the structure cause a redistribution of the force effects.

For permanent loads, the behavior of segmental bridges after closure may be approximated by use of an effective modulus of elasticity, E_{eff}, which may be

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calculated as:

$$E_{eff} = \frac{E_c}{\psi(t,t_i)+1} \tag{C5.12.5.3.6-1}$$

where:

$\Psi(t, t_i)$ = creep coefficient at time t for loading applied at time t_i

A comprehensive series of equations for evaluating the time-related effects of creep and shrinkage is presented in the ACI Committee 209 report, *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures* (ACI 209, 1982). A procedure based on graphical values for creep and shrinkage parameters is presented in the *CEB-FIP Model Code* (CEB, 1990). Comparisons of the effects of application of the ACI and CEB provisions are presented in the Appendix, the first edition of the *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges* (AASHTO, 1999; Ketchum, 1986).

Bryant and Vadhanavikkit (1987) suggest that the ACI 209 predictions underestimate the creep and shrinkage strains for the large-scale specimens used in segmental bridges. The ACI 209 creep predictions were consistently about 65 percent of the experimental results in these tests. The report suggests modifications of the ACI 209 equations based on the size or thickness of the members.

5.12.5.3.7—Prestress Losses

The applicable provisions of Article 5.9.3 shall apply.

C5.12.5.3.7

The friction and wobble coefficients in Article 5.9.3.2.2 for galvanized duct were developed for conventional cast-in-place box girder bridges based on job-site tests of various sizes and lengths of tendons. The values are reasonably accurate for tendons comprised of 12 strands of 0.5-in. diameter in a 2.625-in. diameter galvanized metal sheathing. Tests and experience indicate that the values are conservative for larger tendons and duct diameters. However, experience with segmental concrete bridges to date has often indicated higher friction and wobble losses due to movement of ducts during concrete placement and misalignment at segment joints. For this reason, in-place friction tests are recommended at an early stage in major projects as a basis for modifying friction and wobble loss values. No reasonable values for friction and wobble coefficients can be recommended to account for gross duct misalignment problems. As a means of compensating for high friction and wobble losses or provisional post-tensioning tendons as well as for other contingencies, additional ducts are required in accordance with Article 5.12.5.3.9.

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5.12.5.3.8—Alternative Shear Design Procedure

5.12.5.3.8a—General

Where it is reasonable to assume that plane sections remain plane after loading, the provisions of Article 5.7.3 or the alternative provisions of the Articles herein may be used for the design of segmental post-tensioned concrete box girder bridges for shear and torsion.

The applicable provisions of Articles 5.7.1, 5.7.2, and 5.7.4 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 herein and the strut-and-tie method of Article 5.8.2. The provisions of Article 5.8.4 shall apply to special discontinuity regions such as deep beams, brackets and corbels, as appropriate.

The values of compressive strength of concrete for use in design using these provisions shall be in accordance with Article 5.7.2.1.

The design yield strength of transverse shear or torsion reinforcement shall be in accordance with Article 5.7.2.7.

C5.12.5.3.8a

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.

5.12.5.3.8b—Loading

Where the alternative provisions for design for shear and torsion permitted herein are invoked, 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 effects of applied factored torsional moments, T_u, shall be considered in the design when their magnitude exceeds the value specified in Article 5.7.2.1.

C5.12.5.3.8b

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.

5.12.5.3.8c—Nominal Shear Resistance

In lieu of the provisions of Article 5.7.3, the provisions herein may be used to determine the nominal shear and torsion 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. 5.12.5.3.8c-3.

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

C5.12.5.3.8c

The expression for V_c has been checked against a wide range of test data and has been found to be a conservative expression. Additional background on the shear and torsion provisions in this article can be found in Breen and Ramirez, 1991.

The maximum nominal shear resistance in Eq. 5.7.3.3-2 of Article 5.7.3 is higher than that in Article 5.12.5.3.8c. This difference increases as the compressive strength of the concrete increases. The limit in Eq. 5.12.5.3.8c-2 is similar to that in ACI 318-14 building code provisions and has been 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.

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

$$V_n = V_c + V_s \quad (5.12.5.3.8c\text{-}1)$$

$$V_n = 0.379\lambda\sqrt{f'_c}\, b_v d \quad (5.12.5.3.8c\text{-}2)$$

in which:

$$V_c = 0.0632K\lambda\sqrt{f'_c}\, b_v d \quad (5.12.5.3.8c\text{-}3)$$

$$V_s = \frac{A_v f_y d}{s} \quad (5.12.5.3.8c\text{-}4)$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \leq 2.0 \quad (5.12.5.3.8c\text{-}5)$$

Where the effects of torsion are required to be considered by Article 5.7.2.1, the cross-sectional dimensions shall be such that:

$$\left(\frac{V_u}{b_v\, d}\right) + \left(\frac{T_u}{2\, A_o\, b_e}\right) \leq 0.474\lambda\sqrt{f'_c} \quad (5.12.5.3.8c\text{-}6)$$

where:

λ = concrete density modification factor as specified in Article 5.4.2.8
f'_c = compressive strength of concrete for use in design (ksi)
b_v = effective web width taken as the total minimum width of all webs within the depth d adjusted for the effect of openings or ducts as specified in Article 5.7.2.8.
d = $0.8h$ or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.)
A_v = total area of transverse reinforcing in all webs in the cross section within a distance s (in.^{2})
s = spacing of stirrups (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)
V_u = factored design shear including any normal component from the primary prestressing force (kip)
T_u = applied factored torsional moment (kip-in.)

found to be unnecessarily conservative (Rangan, 1991) (Tantipidok et al., 2011).

Eq. 5.12.5.3.8c-4 is based on an assumed 45-degree truss model.

Eqs. 5.12.5.3.8c-3 and 5.12.5.3.8c-6 are only used to establish appropriate concrete section dimensions.

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A_o = area enclosed by shear flow path, including any area of holes therein (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 5.7.2.1 (in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

5.12.5.3.8d—Torsional Reinforcement

Where consideration of torsional effects is required by Article 5.7.2.1 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 5.12.5.3.8c, flexure and axial forces that may act concurrently with the torsion.

The longitudinal and transverse reinforcement required for torsion shall satisfy:

$$T_u \leq \phi T_n \quad (5.12.5.3.8d\text{-}1)$$

The nominal torsional resistance from transverse reinforcement shall be based on a truss model with 45-degree diagonals and shall be computed as:

$$T_n = \frac{2A_o A_t f_y}{s} \quad (5.12.5.3.8d\text{-}2)$$

The minimum additional longitudinal reinforcement for torsion, A_ℓ, shall satisfy:

$$A_\ell \geq \frac{T_u \, p_h}{2\phi A_o f_y} \quad (5.12.5.3.8d\text{-}3)$$

where:

T_u = applied factored torsional moment (kip-in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

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

A_t = total area of transverse torsion reinforcing in the exterior web and flange (in.^{2})

f_y = yield strength of additional longitudinal reinforcement (ksi)

A_ℓ = total area of longitudinal torsion reinforcement in a box girder (in.^{2})

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.)

C5.12.5.3.8d

Use of reinforcement with f_y greater than 75.0 ksi has not been verified by tests.

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing is taken into account by considering the longitudinal prestressing force in excess of that required for concurrent flexure and shear as an equivalent area of reinforcement.

The total area of transverse reinforcing, A_t, must be placed in each exterior web and flange that forms the closed section.

Unlike solid sections, when designing the webs of segmental bridges, the shear and torsion reinforcing should be directly added together. Reinforcing for transverse bending in the webs and other box girder elements should be accounted for in the total reinforcing demand.

A_ℓ shall be distributed around the outer-most webs and top and bottom slabs of the box girder in accordance with Article 5.12.5.3.8e. At least one bar or tendon shall be placed in the corners of the stirrups. Where corner bars are used, they shall not be less than 0.625 in.

Subject to the minimum reinforcement requirements of Article 5.12.5.3.8e, the area of additional longitudinal torsion reinforcement in the flexural compression zone may be reduced by an amount, A_{red}, taken as:

$$A_{red} = \frac{M_u}{(0.9 d_e f_y)} \quad (5.12.5.3.8d\text{-}4)$$

where:

M_u = factored moment acting at the 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)

5.12.5.3.8e—Reinforcement Details

In addition to the provisions herein, the provisions of Article 5.10 shall 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.

The spacing of the transverse reinforcement shall comply with the provisions of Articles 5.7.2.5 and 5.7.2.6.

5.12.5.3.9—Provisional Post-Tensioning Ducts and Anchorages

5.12.5.3.9a—General

Provisions for adjustments of prestressing force to compensate for unexpected losses during construction or at a later time, future dead loads, and control of cracking and deflections shall be considered. Where such adjustments are deemed necessary, the requirements specified herein shall be satisfied.

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5.12.5.3.9b—Bridges with Internal Ducts

For bridges with internal ducts, provisional anchorage and duct capacity for negative and positive moment tendons located symmetrically about the bridge centerline shall provide for an increase in the post-tensioning force during original construction. The total provisional force potential of both positive and negative moment anchorages and ducts shall not be less than five percent of the total positive and negative moment post-tensioning forces, respectively. Anchorages for the provisional prestressing force shall be distributed uniformly at three segment intervals along the length of the bridge.

At least one empty duct per web shall be provided. For continuous bridges, provisional positive moment ducts and anchorage capacity need not be used for 25 percent of the span length on either side of the pier supports.

Any provisional ducts not used for adjustment of the post-tensioning force shall be grouted at the same time as other ducts in the span.

C5.12.5.3.9b

Provisional post-tensioning ducts and anchorages permit the introduction of additional prestressing force to compensate for installation or stressing problems that might arise during construction.

Excess capacity may be provided by use of oversize ducts and oversize anchorage hardware at selected anchorage locations.

The purpose of grouting unused ducts is to prevent entrapment of water in the ducts.

5.12.5.3.9c—Provision for Future Dead Load or Deflection Adjustment

Provision shall be made for access and for anchorage attachments, pass-through openings, and deviation block attachments to permit future addition of corrosion-protected unbonded external tendons located inside the box section symmetrically about the bridge centerline for a post-tensioning force of not less than ten percent of the primary positive moment and negative moment post-tensioning forces.

C5.12.5.3.9c

This provides for future addition of internal unbonded post-tensioning tendons draped from the top of the diaphragm at piers to the intersection of the web and flange at midspan. Tendons from adjacent spans should be lapped at opposite faces of the diaphragm to provide negative moment capacity. The requirement of a force of ten percent of the primary positive moment and negative moment post-tensioning forces is an arbitrary but reasonable value. Provision for larger amounts of post-tensioning might be developed, as necessary, to carry specific amounts of additional dead load as considered appropriate for the structure.

5.12.5.3.10—Plan Presentation

Contract documents shall include description of one construction method upon which the design is based. Contract drawings shall be detailed according to the provisions of *AASHTO LRFD Bridge Construction Specifications*, Section 10, "Prestressing."

The concrete cross section shall be proportioned to accommodate an assumed post-tensioning system, reinforcement, and all other embedded items. The concrete cross section should also accommodate comparable anchorage sizes of competitive post-tensioning systems, unless noted otherwise on the plans.

C5.12.5.3.10

Integrated drawings utilizing the assumed system should be defined to a scale and quality required to confirm elimination of interferences by all items embedded in the concrete.

Congested areas of post-tensioned concrete structures can easily be identified on integrated drawings using an assumed post-tensioning system. Such areas should include, but are not necessarily limited to, anchorage zones, diaphragms, deviators, blisters and other areas containing embedded items for the assumed post-tensioning system, and areas where post-tensioning ducts deviate both in the vertical and transverse directions. For curved structures, conflicts between webs and external tendons are possible. A check should be made to identify conflicts between future post-tensioning tendons and permanent tendons, and to provide for the necessary clearances in the design details to accommodate the post-tensioning jacks.

5.12.5.3.11—Box Girder Cross section Dimensions and Details

5.12.5.3.11a—Minimum Flange Thickness

Top and bottom flange thickness shall not be less than all of the following:

- Unless transverse ribs spaced equal to the clear span between webs or haunches are provided:
  - $^{1}/_{30}$ of the clear span between webs where no haunches are present in the flange span, or:
  - $^{1}/_{30}$ of the span between the thinner ends of the haunches where haunches are present in the flange span.

- Top flange thickness shall not be less than 9.0 in. in anchorage zones where transverse post-tensioning is used and 8.0 in. beyond anchorage zones or for pretensioned slabs.

C5.12.5.3.11a

Top and bottom flanges for box girder sections may need to be thickened for structural reasons, such as transverse flexure, longitudinal flexure and longitudinal interface shear considerations. They may also need to be thickened to accommodate ducts, anchorages or other post-tensioning hardware. To save concrete and reduce weight, linear haunches at the webs may be utilized in lieu of thickening the entire slab. The length and amount of thickening for the haunches shall be determined by analysis and detailing considerations.

Typical haunch lengths are 20–25 percent of the total span between the webs, or longer, as required to accommodate post-tensioning hardware. The minimum amount of thickening that is typically utilized for haunches is 3.0 in. for the bottom flange, 5.0 in. for the top flange.

Where the clear span between the faces of webs is 15.0 ft or larger, transverse prestressing of the top deck is typically utilized. However, it may also be utilized to improve deck durability (regardless of span length) at the Owner's discretion. To ensure that these benefits are realized, Owners need to ensure that project specifications explicitly state when transverse prestressing of the deck is required for other than structural purposes.

Transverse deck prestressing is typically accomplished through post-tensioning, although pretensioning has been utilized in the past. Strands for transverse pretensioning are limited to 0.6 in. diameter or less, consistent with other articles in these specifications.

5.12.5.3.11b—Minimum Web Thickness

The following minimum values shall apply, except as specified herein:

- Webs with no longitudinal or vertical post-tensioning tendons—8.0 in.
- Webs with only longitudinal (or vertical) post-tensioning tendons—12.0 in.
- Webs with both longitudinal and vertical tendons—15.0 in.

The minimum thickness of ribbed webs may be taken as 7.0 in.

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5.12.5.3.11c—Length of Top Flange Cantilever

The cantilever length of the top flange measured from the centerline of the web should preferably not exceed 0.45 the interior span of the top flange measured between the centerline of the webs.

5.12.5.3.11d—Overall Cross Section Dimensions

Overall dimensions of the box girder cross section should preferably not be less than that required to limit live load plus impact deflection calculated using the gross section moment of inertia and the secant modulus of elasticity to $^{1}/_{1,000}$ of the span. The live loading shall consist of all traffic lanes fully loaded and adjusted for the number of loaded lanes as specified in Article 3.6.1.1.2. The live loading shall be considered to be uniformly distributed to all longitudinal flexural members.

C5.12.5.3.11d

With four lanes of live load and using applicable reduction factors, the live load deflection of the model of the Corpus Christi Bridge was approximately L/3,200 in the main span. The deflection limit of L/1,000 was arbitrarily chosen to provide guidance concerning the maximum live load deflections anticipated for segmental concrete bridges with normal dimensions of the box girder cross section.

Girder depth and web spacing determined in accordance with the following dimensional ranges will generally provide satisfactory deflection behavior:

- Constant depth girder

  $1/5 > d_o/L > 1/30$

  optimum 1/18 to 1/20

  where:

  d_o = girder depth (ft)
  L = span length between supports (ft)

  In case of incrementally launched girders, the girder depth should preferably be between the following limits:

  For L = 100 ft, $1/15 < d_o/L < 1/12$

  For L = 200 ft, $1/13.5 < d_o/L < 1/11.5$

  For L = 300 ft, $1/12 < d_o/L < 1/11$

- Variable depth girder with straight haunches at pier $1/16 > d_o/L > 1/20$ optimum 1/18

  at center of span $1/22 > d_o/L > 1/28$ optimum 1/24

  A diaphragm will be required at the point where the bottom flange changes direction.

- Variable depth girder with circular or parabolic haunches at pier $1/16 > d_o/L > 1/20$

  optimum 1/18

  at center of span $1/30 > d_o/L > 1/50$
  Depth width ratio
  A single cell box should preferably be used when $d_o/b \geq 1/6$

A two cell box should preferably be used when $d_o/b < 1/6$

where:

b = width of the top flange

If in a single cell box the limit of depth to width ratio given above is exceeded, a more rigorous analysis is required and longitudinal edge beams at the tip of the cantilever may be required to distribute loads acting on the cantilevers. An analysis for shear lag should be made in such cases. Transverse load distribution is not substantially increased by the use of three or more cells.

5.12.5.3.12—Seismic Design

Segmental superstructure design with moment resisting column to superstructure connections shall consider the inelastic hinging forces from columns in accordance with Article 3.10.9.4.3. Bridge superstructures in Seismic Zones 3 and 4 with moment resisting column to superstructure connections shall be reinforced with ductile details to resist longitudinal and transverse flexural demands produced by column plastic hinging.

Segment joints shall provide capacity to transfer seismic demands.

Superstructure prestressing tendons shall be designed to remain below yield for the combined dead load plus seismic demands. The stress in the tendon may be computed by detailed moment curvature analysis, with the stress in tendon computed by strain compatibility with the section and the stress in unbonded tendon computed using global displacement compatibility between bonded sections of tendons located within the span.

C5.12.5.3.12

The distinction between bonded tendons and unbonded tendons with respect to seismic behavior reflects the general condition that bonded tendons are effectively bonded at all sections along the span, whereas unbonded tendons are effectively bonded at only their anchorages and intermediate bonded sections, such as deviators. Hence, the overall section strength achieved with bonded tendons is typically larger than that achieved with unbonded tendons. However, both bonded and unbonded tendons have been shown to provide significant displacement ductility (Megally et al., 2003).

The *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2011) determines the resistance of concrete structures using nonlinear "push-over" analysis. Various peer review teams urged this methodology following the Loma Prieta and Northridge earthquakes, in order to better access global behavior, and to achieve more economically-justifiable designs. Superstructures are designed for forces to resist plastic-hinging of the column(s). Frames are modeled using soil springs on the substructure, and stress–strain relationships for the concrete and steel. The frame is pushed, to incur plastic hinges in the columns, and reaches a point of collapse. The resulting displacement must be greater than that from a three-dimensional linear dynamic analysis. The acceleration response spectrum (ARS) may be generic for the soil type and anticipated acceleration, or be developed for the specific bridge site.

5.12.5.4—Types of Segmental Bridges

5.12.5.4.1—General

Bridges designed for segmentally placed superstructures shall conform to the requirements specified herein, based on the concrete placement method and the erection methods to be used.

C5.12.5.4.1

Precast segmental bridges are normally erected by balanced cantilever, use of erection trusses, or progressive placement.

Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons.

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Bridges built with erection trusses may utilize internal tendons, external tendons, or combinations thereof. Due to considerations of segment weight, span lengths for precast segmental box girder bridges, except for cable-stayed bridges, rarely exceed 400 ft.

5.12.5.4.2—Details for Precast Construction

The compressive strength of precast concrete segments shall not be less than 2.5 ksi prior to removal from the forms and shall have a maturity equivalent to 14 days at 70°F prior to assembly into the structure.

Multiple small-amplitude shear keys at match-cast joints in webs of precast segmental bridges shall extend over as much of the web as is compatible with other details. Details of shear keys in webs should be similar to those shown in Figure 5.12.5.4.2-1. Shear keys shall also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single-element keys.

C5.12.5.4.2

This provision intends to limit the magnitude of construction deflections and to prevent erratic construction deflections and creep.

Small-amplitude shear keys in the webs are less susceptible to construction damage, which will result in loss of geometry control, than larger single-element keys. Shear keys in the top and bottom flanges are less susceptible to such damage.

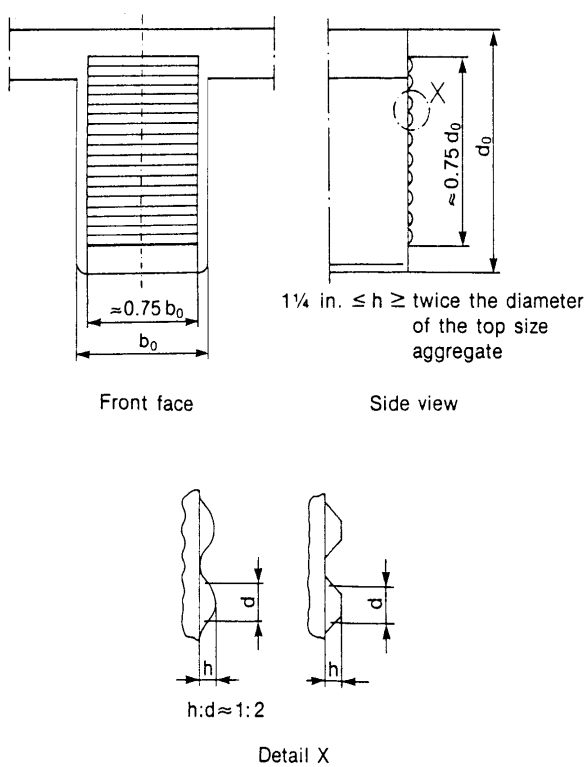

Figure 5.12.5.4.2-1—Example of Fine Indentation Shear Keys

Joints in precast segmental bridges shall be either cast-in-place closures or match cast epoxied joints.

Precast segmental bridges using internal post-tensioning tendons and bridges located in areas subject to freezing temperatures or deicing chemicals shall employ bonded joints.

Match casting is necessary to ensure control of the geometry upon reassembly of the segments.

Epoxy on both faces serves as a lubricant during placement of the segments, prevents water intrusion, provides a seal to prevent cross-over during grouting, and provides some tensile strength across the joint.

The use of dry joints (identified as Type B in past versions of these specifications) was eliminated with the adoption of the 2003 revision due to the critical nature of post-tensioning reinforcement and the need for a multiple layer protection system. Failures of some post-tensioning reinforcement in Florida and Europe due to corrosion have resulted in a review of the effectiveness of previous multiple layer protection systems. The most rigorous review was performed by the British Concrete Society and the recommendations are contained in the report titled "Durable Post-Tensioned Concrete Bridges." This European report codifies the need for a three-level protection system and suggested details to achieve the required results. Improved grout and duct materials and methods are also discussed. As a result of this European Report and studies by Dr. John Breen of the University of Texas, Austin, the multiple level protection system for post-tensioning has been universally accepted.

A temporary prestressing system shall provide a minimum compressive stress of 0.030 ksi and an average stress of 0.040 ksi across the joint until the epoxy has cured.

The *AASHTO LRFD Bridge Construction Specifications* require this temporary stress to ensure full bond and to prevent uneven epoxy thickness. Such variations could lead to a systematic accumulation of geometric error. Large stress changes on epoxy joints should be avoided during the initial curing period.

5.12.5.4.3—Details for Cast-in-Place Construction

Joints between cast-in-place segments shall be specified as either intentionally roughened to expose coarse aggregate or keyed.

The width of closure joints shall permit the coupling of the tendon ducts.

Diaphragms shall be provided at abutments, piers, hinge joints, and bottom flange angle points in structures with straight haunches. Diaphragms shall be substantially solid at piers and abutments, except for access openings and utility holes. Diaphragms shall be sufficiently wide as required by design, with a minimum overhang over bearings of not less than 6.0 in.

C5.12.5.4.3

The *AASHTO LRFD Bridge Construction Specifications* require vertical joints to be keyed. However, proper attention to roughened joint preparation is expected to ensure bond between the segments, providing better shear strength than shear keys.

5.12.5.4.4—Cantilever Construction

The provisions specified herein shall apply to both precast and cast-in-place cantilever construction.

Longitudinal tendons may be anchored in the webs, in the slab, or in blisters built out from the web or slab. A minimum of two longitudinal tendons shall be anchored in each segment.

The cantilevered portion of the structure shall be investigated for overturning during erection. The factor of safety against overturning shall not be less than 1.5 under any combination of loads, as specified in Article 5.12.5.3.3. Minimum wind velocity for erection stability analyses shall be 55 mph, unless a better estimate of probable wind velocity is obtained by analysis or meteorological records.

C5.12.5.4.4

Stability during erection may be provided by moment resisting column/superstructure connections, falsework bents, or a launching girder. Loads to be considered include construction equipment, forms, stored material, and wind.

The 70 mph 3-second gust wind speed, when applied with a load factor of 1.0, is equivalent to the 100 mph fastest-mile wind speed applied with a load factor of 0.30 shown in Table 3.4.1-1 in earlier specifications. The 100 mph fastest-mile wind speed applied with a load factor of 0.30 was meant to be equivalent to 55 mph fastest-mile wind speed applied with a load factor of 1.0.

Continuity tendons shall be anchored at least one segment beyond the point where they are theoretically required for stresses.

The segment lengths assumed in the design shall be shown on the plans. Any changes proposed by the Contractor shall be supported by reanalysis of the construction and computation of the final stresses.

The formtraveler weight assumed in stress and camber calculations shall be stated on the plans.

Tendon force requires an "induction length" due to shear lag before it may be assumed to be effective over the whole section.

Lengths of segments for free cantilever construction usually range between 10.0 and 18.0 ft. Lengths may vary with the construction method, the span length and the location within the span.

Formtravelers for a typical 40.0-ft wide, two-lane bridge with 15.0- to 16.0-ft segments may be estimated to weigh 160 to 180 kips. Weight of formtravelers for wider two-cell box sections may range up to 280 kips. Segment length is adjusted for deeper and heavier segments to control segment weight. Consultation with contractors experienced in free cantilever construction is recommended to obtain a design value for formtraveler weight for a specific bridge cross section.

5.12.5.4.5—Span-by-Span Construction

Provisions shall be made in design of span-by-span construction for accumulated construction stresses due to the change in the structural system as construction progresses.

Stresses due to the changes in the structural system, in particular the effects of the application of a load to one system and its removal from a different system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance made for possible variations in the creep rate and magnitude.

C5.12.5.4.5

Span-by-span construction is defined as construction where the segments, either precast or cast-in-place, are assembled or cast on falsework supporting one entire span between permanent piers. The falsework is removed after application of post-tensioning to make the span capable of supporting its own weight and any construction loads. Additional stressing may be utilized after adjacent spans are in place to develop continuity over piers.

5.12.5.4.6—Incrementally Launched Construction

5.12.5.4.6a—General

Stresses under all stages of launching shall not exceed the limits specified in Article 5.9.2.3 for members with bonded reinforcement through the joint and internal tendons.

Provision shall be made to resist the frictional forces on the substructure during launching and to restrain the superstructure if the structure is launched down a gradient. For determining the critical frictional forces, the friction on launching bearings shall be assumed to vary between 0 and 4 percent, whichever is critical. The upper value may be reduced to 3.5 percent if pier deflections and launching jack forces are monitored during construction.

C5.12.5.4.6a

Incrementally launched girders are subject to reversal of moments during launching. Temporary piers and/or a launching nose may be used to reduce launching stresses.

These friction coefficients are only applicable to bearings employing a combination of virgin Teflon and stainless steel with a roughness of less than 1.0×10^{-4} in.

5.12.5.4.6b—Force Effects Due to Construction Tolerances

Force effects due to the following permissible construction tolerances shall be superimposed upon those resulting from gravity loads:

- In the longitudinal direction between two adjacent bearings…… ....0.2 in.

- In the transverse direction between two adjacent bearings .... 0.1 in.
- Between the fabrication area and the launching equipment in the longitudinal and transverse direction .... 0.1 in.
- Lateral deviation at the outside of the webs ..... 0.1 in.

The horizontal force acting on the lateral guides of the launching bearings shall not be taken to be less than one percent of the vertical support reaction.

For stresses during construction, one-half of the force effects due to construction tolerances and one-half of the force effects due to temperature in accordance with Article 5.12.5.3 shall be superimposed upon those from gravity loads. Concrete tensile stresses due to the combined moments shall not exceed $0.221\sqrt{f'_c}$.

5.12.5.4.6c—Design Details

Piers and superstructure diaphragms at piers shall be designed to permit jacking of the superstructure during all launching stages and for the installation of permanent bearings. Frictional forces during launching shall be considered.

Local stresses that may develop at the underside of the web during launching shall be investigated. The following requirements shall be satisfied:

- Launching pads shall be placed not closer than 3.0 in. to the outside of the web,
- Concrete cover between the soffit and post-tensioning ducts shall not be less than 6.0 in., and
- Bearing pressures at the web/soffit corner shall be investigated and the effects of ungrouted ducts and any eccentricity between the intersection of the centerlines of the web and the bottom slab and the centerline of the bearing shall be considered.

C5.12.5.4.6c

The dimensional restrictions on placement of launching bearings are shown in Figure C5.12.5.4.6c-1. Eccentricity between the intersection of the centerlines of the web and the bottom slab and the centerline of the bearing is illustrated in Figure C5.12.5.4.6c-2.

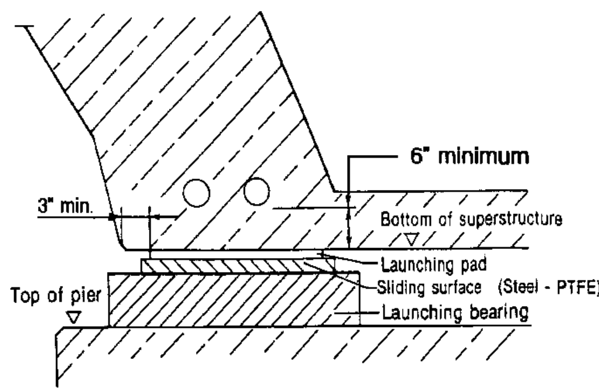

Figure C5.12.5.4.6c-1—Location of Launching Pads

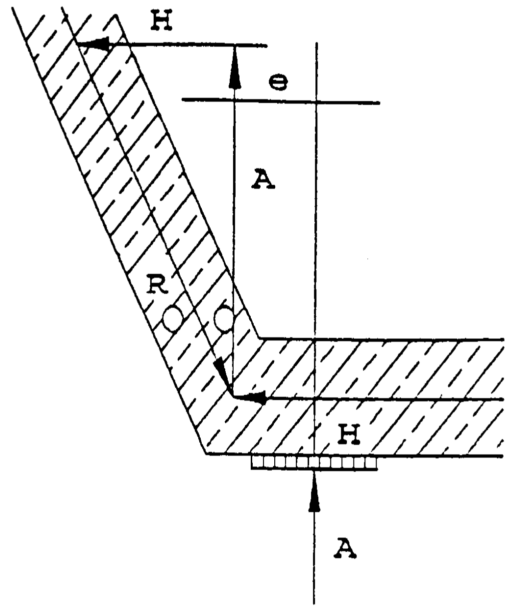

Figure C5.12.5.4.6c-2—Eccentric Reaction at Launching Pads

The straight tendons required for launching shall be placed in the top and bottom slabs for box girders and in the lower third of the web for T-sections. Not more than 50 percent of the tendons shall be coupled at one construction joint. Anchorages and locations for the straight tendons shall be designed for the concrete strength at the time of tensioning.

The faces of construction joints shall be provided with shear keys or a roughened surface with a minimum roughness amplitude of 0.25 in. Bonded nonprestressed reinforcement shall be provided longitudinally and transversely at all concrete surfaces crossing the joint and over a distance of 7.0 ft on either side of the joint. Minimum reinforcement shall be equivalent to No. 4 bars spaced at 5.0 in.

The stresses in each cross-section change from tension to compression during launching. These tensile stresses during launching are counteracted by the straight tendons. The straight tendons are stressed at an early concrete age (e.g., 3 days).

The inclined launching bearings, as opposed to horizontal permanent bearings, create forces at the launching jacks and at the pier tops.

5.12.5.4.6d—Design of Construction Equipment

Where construction equipment for incremental launching is shown on the contract documents, the design of such equipment shall include, but not be limited to the following features:

- The construction tolerances in the sliding surface at the bottom of the launching nose shall be limited to those of the superstructure, as specified in Article 5.12.5.4.6b.
- The introduction of the support reactions in the launching nose shall be investigated with respect to strength, stability, and deformation.
- Launching bearings shall be designed in such a way that they can compensate for local deviations of the sliding surface of up to 0.08 in. by elastic deformation.

- The launching equipment shall be sized for friction in accordance with Article 5.12.5.4.6a and the actual superstructure gradient.
- The launching equipment shall be designed to ensure that a power failure will not lead to uncontrolled sliding of the superstructure.
- The friction coefficient between concrete and the hardened profiled steel surfaces of the launching equipment shall be taken as 60 percent at the service limit state and the friction shall exceed the driving forces by 30 percent.

The forms for the sliding surfaces underneath and outside the web shall be wear-resistant and sufficiently stiff so that their deflection during casting does not exceed 0.08 in.

5.12.5.5—Use of Alternative Construction Methods

When permitted by contract documents that do not require value engineering, the Contractor may be allowed to choose alternative construction methods and a modified post-tensioning layout suitable for the selected construction method. In such a case, the Contractor shall supply a structural analysis, documenting that the post-tensioning forces and eccentricities shown on the construction plans meet all requirements of the design specifications. If additional post-tensioning is required for construction stages or other reasons, it shall be demonstrated that the stresses at critical sections in the final structure meet the allowable stress provisions of the design specifications. Removal of temporary post-tensioning to achieve such conditions shall be permissible. Use of additional nonprestressed reinforcement for construction stages shall be permitted. All extra materials required for construction stages shall be provided by the Contractor at no cost to the Owner.

Value engineering provisions may be included in the contract special provisions permitting alternative construction methods that require a complete redesign of the final structure. The Contractor's engineering expenses for preparing the value engineering design and the Owner's engineering expenses for checking the design shall be considered as part of the cost of the redesign structure.

Pier spacing, alignment, outside concrete, appearance, and dimensions shall not be changed under value engineering proposals, except when contract documents define such changes as being permitted.

For the value engineering, the Contractor shall provide a complete set of design computations and revised contract documents. The value engineering redesign shall be prepared by a Professional Engineer experienced in segmental bridge design. Upon acceptance of a value engineering redesign, the

C5.12.5.5

Opinions vary among state bridge engineers and consultants about the desirability of permitting alternate construction methods. Some state transportation departments do not permit any deviation from the details and construction methods shown on the plans and specified in the contract special provisions. Other states permit great latitude for contractor submission of alternate construction methods. An example of the latter is presented below, which is taken verbatim from the contract documents for a recent California bridge project.

"**Alternative Proposals**—Continuous cast-in-place prestressed box girder bridges have been designed to be fully supported during construction. Except as provided herein, such bridges shall be constructed on falsework and in accordance with the provisions in Section 51, "Concrete Structures," of the Standard Specifications.

The Contractor may submit proposals for such bridges which modify the original design assumptions for dead load support or the requirements in Section 51, 'Concrete Structures,' of the Standard Specifications. Such proposals are subject to the following requirements and limitation.

The structure shall, after completion, have a capacity to carry or resist loads at least equal to those used in the design of the bridge shown on the plans. When necessary, strengthening of the superstructure and the substructure will be required to provide such capacity and to support construction loads at each stage of construction.

All proposed modifications shall be designed in accordance with the bridge design specifications currently employed by the Department.

Modifications may be proposed in the thickness of girders and deck slabs, the thickness and length of overhang, the structure depth, the number of girders, Professional Engineer responsible for the redesign shall become the Engineer of Record.

and the amount and location of reinforcing steel or prestressing force. The strength of the concrete used may be increased, but the strength employed for design or analysis shall not exceed 6,000 psi.

Modifications may also be proposed in the requirements in `Prestressing Concrete' of these special provisions which pertain to the minimum amount of prestressing force which must be provided by full length draped tendons.

No modifications will be permitted in the width of the bridge. Fixed connections at the tops and bottoms of columns shown on the plans shall not be eliminated.

Temporary prestressing tendons, if used, shall be detensioned and any temporary ducts shall be filled with grout before completion of the work. Temporary tendons shall be either removed or fully encased in grout before completion of the work.

The Contractor shall be responsible for determining construction camber and obtaining the final profile grade as shown on the plans. The Contractor shall provide the Engineer with diagrams showing the predicted deck profile at each construction stage for all portions of the completed bridge. Any remedial measures necessary to correct deviation from the predicted camber will be the responsibility of the Contractor.

The Contractor shall furnish to the Engineer complete working drawings and checked calculations for all changes proposed, including revisions in camber and falsework requirements, in accordance with the provisions of Section 5-1.02, 'Drawings,' of the Standard Specifications. The calculations must verify that all requirements are satisfied. Such drawings and calculations shall be signed by an Engineer who is registered as a Civil Engineer in the State of California.

Working drawings and calculations shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and correction by the Contractor of the drawings without delaying the work. Such time shall be proportional to the complexity of the work, but in no case shall such time be less than eight weeks.

The Contractor shall reimburse the State for the cost of investigating the proposal. The Department may deduct such amount from any monies due, or that may become due, the Contractor under contract.

The Engineer shall be the sole judge as to the acceptability of any proposal and may disapprove any proposal which in his judgment may not produce a structure which is at least equivalent in all respects to the planned structure.

Any additional materials required or increased costs resulting from the use of such proposal will be considered to be for the convenience of the Contractor and no additional payment will be made therefore."

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5.12.5.6—Segmentally Constructed Bridge Substructures

5.12.5.6.1—General

Pier and abutment design shall conform to Section 11 and to the provisions of this section. Consideration shall be given to erection loads, moments, and shears imposed on piers and abutments by the construction method shown in the contract documents. Auxiliary supports and bracing shall be shown as required. Hollow, rectangular precast segmental piers shall be designed in accordance with Article 5.6.4.7. The area of discontinuous longitudinal nonprestressed reinforcement may be as specified in Article 5.12.5.6.3.

C5.12.5.6.1

Nonsegmentally constructed substructures are addressed in Sections 10 and 11 and in Article 5.12.5.3.4b.

5.12.5.6.2—Construction Load Combinations

Tensile stresses in vertically prestressed substructures during construction shall be computed for applicable load combinations of Table 5.12.5.3.3-1.

5.12.5.6.3—Longitudinal Reinforcement of Hollow, Rectangular Precast Segmental Piers

The minimum area of discontinuous longitudinal nonprestressed reinforcement in hollow, rectangular precast segmental piers shall satisfy the shrinkage and temperature reinforcement provisions specified in Article 5.10.7.

C5.12.5.6.3

Minimum longitudinal reinforcement of hollow, rectangular precast segmental piers is based on Article 5.10.6 for shrinkage and temperature reinforcement. This provision reflects the satisfactory performance of several segmental piers constructed between 1982 and 1995, with longitudinal reinforcement ratios ranging from 0.0014 to 0.0028. The discontinuous longitudinal bars in precast segmental piers do not carry significant loads. Tensile reinforcement of precast segmental piers is provided by post-tensioning tendons.

5.12.6—Arches

5.12.6.1—General

The shape of an arch shall be selected with the objective of minimizing flexure under the effect of combined permanent and transient loads.

5.12.6.2—Arch Ribs

The in-plane stability of the arch rib(s) shall be investigated using a modulus of elasticity and moment of inertia appropriate for the combination of loads and moment in the rib(s).

In lieu of a rigorous analysis, the effective length for buckling may be estimated as the product of the arch half span length and the factor specified in Table 4.5.3.2.2c-1.

For the analysis of arch ribs, the provisions of Article 4.5.3.2.2 may be applied. When using the approximate second-order correction for moment

C5.12.6.2

Stability under long-term loads with a reduced modulus of elasticity may govern the stability. In this condition, there would typically be little flexural moment in the rib, the appropriate modulus of elasticity would be the long-term tangent modulus, and the appropriate moment of inertia would be the transformed section inertia. Under transient load conditions, the appropriate modulus of elasticity would be the short-term tangent modulus, and the appropriate moment of inertia would be the cracked section inertia, including the effects of the factored axial load.

specified in Article 4.5.3.2.2c, an estimate of the short-term secant modulus of elasticity may be calculated, as specified in Article 5.4.2.4, based on a strength of $0.40f'_c$.

Arch ribs shall be reinforced as compression members. The minimum reinforcement of one percent of the gross concrete area shall be evenly distributed about the section of the rib. Confinement reinforcement shall be provided as required for columns.

Unfilled spandrel walls greater than 25.0 ft in height shall be braced by counterforts or diaphragms.

The value indicated may be used in stability calculations because the scatter in predicted versus actual modulus of elasticity is greater than the difference between the tangent modulus and the secant modulus at stress ranges normally encountered.

The long-term modulus may be found by dividing the short-term modulus by the creep coefficient.

Under certain conditions the moment of inertia may be taken as the sum of the moment of inertia of the deck and the arch ribs at the quarter point. A large deflection analysis may be used to predict the in-plane buckling load. A preliminary estimate of second-order moments may be made by adding to the first-order moments the product of the thrust and the vertical deflection of the arch rib at the point under consideration.

Spandrel walls shall be provided with expansion joints. Temperature reinforcement shall be provided corresponding to the joint spacing.

The spandrel wall shall be jointed at the springline.

The spandrel fill shall be provided with effective drainage. Filters shall be provided to prevent clogging of drains with fine material.

The ACI 207.2R (2007) contains a discussion of joint spacing and temperature reinforcement of restrained walls.

Drainage of the spandrel fill is important to ensure durability of the concrete in the rib and the spandrel walls and to control the unit weight of the spandrel fill. Drainage details should keep the drainage water from running down the ribs.

5.12.7—Culverts

5.12.7.1—General

The soil structure aspects of culvert design are specified in Section 12.

5.12.7.2—Design for Flexure

The provisions of Article 5.6 shall apply.

5.12.7.3—Design for Shear in Slabs of Box Culverts

The provisions of Article 5.7 apply unless modified herein. For slabs of box culverts under 2.0 ft or more fill, nominal shear resistance V_c may be determined as the lesser of the following:

$$V_c = \left(0.0676\,\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \qquad (5.12.7.3\text{-}1)$$

$$V_c \le 0.126\lambda\sqrt{f'_c}\,bd_e \qquad (5.12.7.3\text{-}2)$$

where:

λ = concrete density modification factor as specified in Article 5.4.2.8
A_s = area of reinforcement in the design width (in.^{2})
b = design width (in.)
d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

C5.12.7.3

Eq. 5.12.7.3-1, as originally proposed, included an additional multiplier to account for axial compression. Because the effect was considered relatively small, it was deleted from Eq. 5.12.7.3-1. However, if the Designer wishes, effect of axial compression may be included by multiplying the results of Eq. 5.12.7.3-1 by the quantity $(1+0.04\,N_u/V_u)$.

The lower limits of $0.0948\lambda\sqrt{f'_c}$ and $0.0791\lambda\sqrt{f'_c}$ are compared with test results in Figure C5.12.7.3-1 with $\lambda = 1.0$.

V_u = shear from factored loads (kip)
M_u = factored moment at the section (kip-in.)

For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than $0.0948\lambda\sqrt{f'_c}bd_e$, and V_c for slabs simply supported need not be taken to be less than $0.0791\lambda\sqrt{f'_c}bd$. The quantity V_ud_e/M_u shall not be taken to be greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. The provisions of Articles 5.7 and 5.12.8.6 shall apply to slabs of box culverts under less than 2.0 ft of fill and to sidewalls.

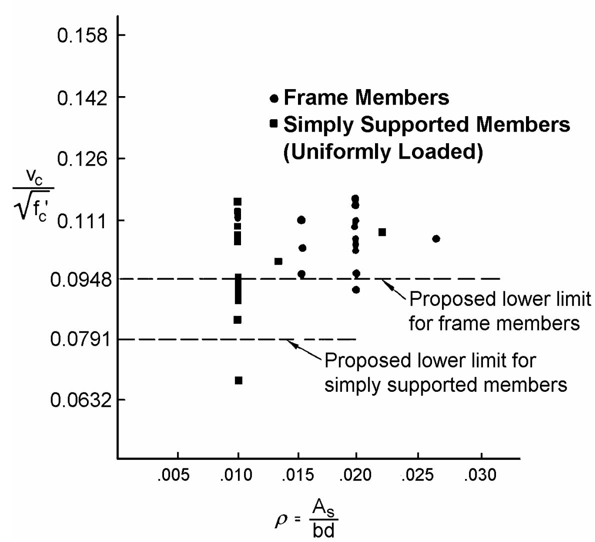

Figure C5.12.7.3-1—Culvert Test Results

5.12.8—Footings

5.12.8.1—General

Provisions herein shall apply to the design of isolated footings, combined footings, and foundation mats.

In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section.

Circular or regular polygon-shaped concrete columns or piers may be treated as square members with the same area for the location of critical sections for moment, shear, and development of reinforcement in footings.

C5.12.8.1

Although the provisions of Article 5.12.8 apply to isolated footings supporting a single column or wall, most of the provisions are generally applicable to combined footings and mats supporting several columns or walls or a combination thereof.

5.12.8.2—Loads and Reactions

The resistance of foundation material for piles shall be as specified in Section 10, "Foundations."

Where an isolated footing supports a column, pier, or wall, the footing shall be assumed to act as a cantilever. Where a footing supports more than one column, pier, or wall, the footing shall be designed for the actual conditions of continuity and restraint.

For the design of footings, unless the use of special equipment is specified to ensure precision driving of piles, it shall be assumed that individual driven piles may be out of planned position in a footing by either 6.0 in. or one quarter of the pile diameter and that the center of a group of piles may be 3.0 in. from its planned position. For pile bents, the contract documents may require a 2.0 in. tolerance for pile position, in which case that value should be accounted for in the design.

C5.12.8.2

The assumption that the as-built location of piles may differ from the planned location recognizes the construction variations sometimes encountered and is consistent with the tolerances allowed by *AASHTO LRFD Bridge Construction Specifications*. Lesser variations may be assumed if the contract documents require the use of special equipment, such as templates, for more precise driving.

For noncircular piles, the larger cross-sectional dimension should be used as the "diameter."

5.12.8.3—Resistance Factors

For determination of geotechnical requirements for footing size and number of piles, the resistance factors, ϕ, for soil-bearing pressure and for pile resistance as a function of the soil shall be as specified in Section 10. Resistance factors for structural design shall be as specified herein.

5.12.8.4—Moment in Footings

The critical section for flexure shall be taken at the face of the column, pier, or wall. In the case of columns that are not rectangular, the critical section shall be taken at the side of the concentric rectangle of equivalent area. For footings under masonry walls, the critical section shall be taken as halfway between the center and edge of the wall. For footings under metallic column bases, the critical section shall be taken as halfway between the column face and the edge of the metallic base.

C5.12.8.4

Moment at any section of a footing may be determined by passing a vertical plane through the footing and computing the moment of the forces acting on one side of that vertical plane.

5.12.8.5—Distribution of Moment Reinforcement

In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across the entire width of the footing.

The following guidelines apply to the distribution of reinforcement in two-way rectangular footings:

- In the long direction, reinforcement shall be distributed uniformly across the entire width of footing.
- In the short direction, a portion of the total reinforcement as specified by Eq. 5.12.8.5-1, shall be distributed uniformly over a band width equal to the length of the short side of footing and centered on the centerline of column or pier. The remainder of reinforcement required in the short direction shall be distributed uniformly outside of the center band width of footing. The area of steel in the band width shall satisfy Eq. 5.12.8.5-1.

$$A_{s\text{-}BW} = A_{s\text{-}SD}\left(\frac{2}{\beta + 1}\right) \tag{5.12.8.5-1}$$

where:

$A_{s\text{-}BW}$ = area of steel in the band width (in.^{2})
$A_{s\text{-}SD}$ = total area of steel in short direction (in.^{2})
β = ratio of the long side to the short side of footing

5.12.8.6—Shear in Slabs and Footings

5.12.8.6.1—Critical Sections for Shear

In determining the shear resistance of slabs and footings in the vicinity of concentrated loads or reaction forces, the more critical of the following conditions shall govern:

- One-way action, with a critical section extending in a plane across the entire width and located at a distance taken as specified in Article 5.7.3.2.
- Two-way action, with a critical section perpendicular to the plane of the slab and located so that its perimeter, b_o, is a minimum but not closer than $0.5d_v$ to the perimeter of the concentrated load or reaction area.
- Where the slab thickness is not constant, critical sections located at a distance not closer than $0.5d_v$ from the face of any change in the slab thickness and located such that the perimeter, b_o, is a minimum.

Where a portion of a pile lies inside the critical section, the pile load shall be considered to be uniformly distributed across the width or diameter of the pile, and the portion of the load outside the critical section shall be included in the calculation of shear on the critical section.

C5.12.8.6.1

In the general case of a cantilever retaining wall, where the downward load on the heel is larger than the upward reaction of the soil under the heel, the critical section for shear is taken at the back face of the stem, as illustrated in Figure C5.12.8.6.1-1, in which d_v is the effective depth for shear.

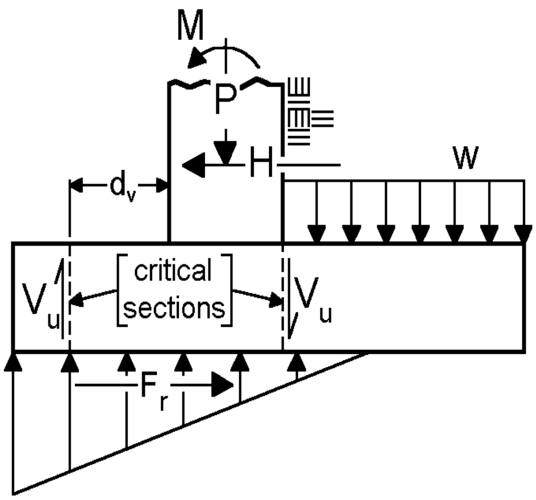

Figure C5.12.8.6.1-1—Example of Critical Section for Shear in Footings

If a haunch has a rise-to-span ratio of 1:1 or more where the rise is in the direction of the shear force under investigation, it may be considered an abrupt change in section, and the design section may be taken as d_v into the span with d_v taken as the effective depth for shear past the haunch.

If a large-diameter pile is subjected to significant flexural moments, the load on the critical section may be adjusted by considering the pile reaction on the footing to be idealized as the stress distribution resulting from the axial load and moment.

5.12.8.6.2—One-Way Action

For one-way action, the shear resistance of the footing or slab shall satisfy the requirements specified in Article 5.7.3, except for culverts with 2.0 ft or more of fill, for which the provisions of Article 5.12.7.3 shall apply.

5.12.8.6.3—Two-Way Action

For two-way action for sections without transverse reinforcement, the nominal shear resistance, V_n in kips, of the concrete shall be taken as:

C5.12.8.6.3

If shear perimeters for individual loads overlap or project beyond the edge of the member, the critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter that will

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$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\lambda\sqrt{f'_c}\,b_o d_v \le 0.126\,\lambda\sqrt{f'_c}\,b_o d_v \quad (5.12.8.6.3\text{-}1)$$

where:

β_c = ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted

λ = concrete density modification factor as specified in Article 5.4.2.8

b_o = the perimeter of the critical section for shear (in.)

d_v = effective shear depth (in.)

Where $V_u > \phi V_n$, shear reinforcement shall be added in compliance with Article 5.7.3.3, with angle θ taken as 45 degrees.

For two-way action for sections with transverse reinforcement, the nominal shear resistance, in kips, shall be taken as:

$$V_n = V_c + V_s \le 0.192\lambda\sqrt{f'_c}\;b_o d_v \quad (5.12.8.6.3\text{-}2)$$

in which:

$$V_c = 0.0632\lambda\sqrt{f'_c}\,b_o d_v\text{, and} \quad (5.12.8.6.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v}{s} \quad (5.12.8.6.3\text{-}4)$$

5.12.8.7—Development of Reinforcement

For the development of reinforcement in slabs and footings, the provisions of Article 5.10.8 shall apply.

Critical sections for development of reinforcement shall be assumed to be at the locations specified in Article 5.12.8.4 and at all other vertical planes where changes of section or reinforcement occur.

5.12.8.8—Transfer of Force at Base of Column

All forces and moments applied at the base of a column or pier shall be transferred to the top of footing by bearing on concrete and by reinforcement. Bearing on concrete at the contact surface between the supporting and supported member shall not exceed the concrete-bearing strength, as specified in Article 5.6.5, for either surface.

Lateral forces shall be transferred from the pier to the footing in accordance with shear-transfer provisions specified in Article 5.7.4 on the basis of the appropriate bulleted item in Article 5.7.4.4.

Reinforcement shall be provided across the interface between supporting and supported member, either by extending the main longitudinal column or

actually resist the critical shear for the group under consideration. One such situation is illustrated in Figure C5.12.8.6.3-1.

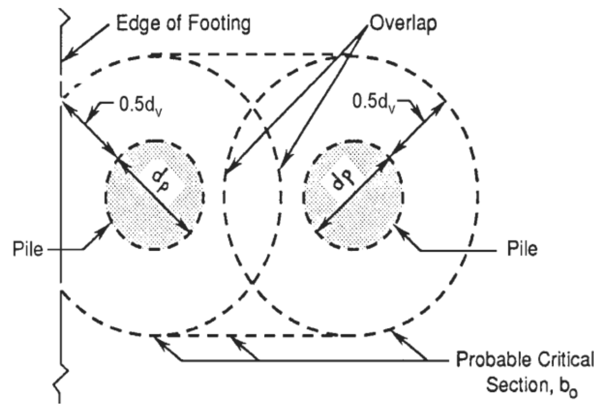

Figure C5.12.8.6.3-1—Modified Critical Section for Shear with Overlapping Critical Perimeters

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wall reinforcement into footings or by using dowels or anchor bolts.

Reinforcement across the interface shall satisfy the following requirements:

- All force effects that exceed the concrete bearing strength in the supporting or supported member shall be transferred by reinforcement;
- If load combinations result in uplift, the total tensile force shall be resisted by the reinforcement; and
- The area of reinforcement shall not be less than 0.5 percent of the gross area of the supported member, and the number of bars shall not be less than four.

The diameter of dowels, if used, shall not exceed the diameter of longitudinal reinforcement by more than 0.15 in.

At footings, the No. 14 and No. 18 main column longitudinal reinforcement that is in compression only may be lap spliced with footing dowels to provide the required area. Dowels shall be no larger than No. 11 and shall extend into the column a distance not less than either the development length of the No. 14 or No. 18 bars or the splice length of the dowels and into the footing a distance not less than the development length of the dowels.

5.12.9—Concrete Piles

5.12.9.1—General

All loads resisted by the footing and the weight of the footing itself shall be assumed to be transmitted to the piles. Piles installed by driving shall be designed to resist driving and handling forces. For transportation and erection, a precast pile should be designed for not less than 1.5 times its self-weight.

Any portion of a pile where lateral support adequate to prevent buckling may not exist at all times shall be designed as a column.

The points or zones of fixity for resistance to lateral loads and moments shall be determined by an analysis of the soil properties, as specified in Article 10.7.3.13.4.

Concrete piles shall be embedded into footings or pile caps, as specified in Article 10.7.1.1. Anchorage reinforcement shall consist of either an extension of the pile reinforcement or the use of dowels. Uplift forces or stresses induced by flexure shall be resisted by the reinforcement. The steel ratio for anchorage reinforcement shall not be less than 0.005, and the number of bars shall not be less than four. The reinforcement shall be developed sufficiently to resist a force of $1.25f_yA_s$.

In addition to the requirements specified in Articles 5.12.9.1 through 5.12.9.5, piles used in the seismic zones shall conform to the requirements specified in Articles 5.11.3.2 or 5.11.4.5, whichever is applicable.

C5.12.9.1

The material directly under a pile-supported footing is not assumed to carry any of the applied loads.

Locations where such lateral support does not exist include any portion of a pile above the anticipated level of scour or future excavation as well as portions that extend above ground, as in pile bents.

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5.12.9.2—Splices

Splices in concrete of piles shall develop the axial, flexural, shear, and torsional resistance of the pile. Details of splices shall be shown in the contract documents.

C5.12.9.2

AASHTO LRFD Bridge Construction Specifications has provisions for short extensions or "buildups" for the tops of concrete piles. This allows for field corrections due to unanticipated events, such as breakage of heads or driving slightly past the cutoff elevation.

5.12.9.3—Precast Reinforced Piles

5.12.9.3.1—Pile Dimensions

Precast concrete piles may be of uniform section or tapered. Tapered piling shall not be used for trestle construction, except for that portion of the pile that lies below the ground line, or in any location where the piles are to act as columns.

Where concrete piles are not exposed to salt water, they shall have a cross-sectional area measured above the taper of not less than 140 in.^{2} Concrete piles used in salt water shall have a cross-sectional area of not less than 220 in.^{2} The corners of a rectangular section shall be chamfered.

The diameter of tapered piles measured 2.0 ft from the point shall be not less than 8.0 in. where, for all pile cross sections, the diameter shall be considered as the least dimension through the center of cross section.

C5.12.9.3.1

A 1.0-in. connection chamfer is desirable, but smaller chamfers have been used successfully. Local experience should be considered.

5.12.9.3.2—Reinforcement

Longitudinal reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile. The area of reinforcement shall not be less than 1.5 percent of the gross concrete cross-sectional area measured above the taper.

The full length of longitudinal steel shall be enclosed with spiral reinforcement or equivalent hoops. The spiral reinforcement shall be as specified in Article 5.12.9.4.3.

5.12.9.4—Precast Prestressed Piles

5.12.9.4.1—Pile Dimensions

Prestressed concrete piles may be octagonal, square, or circular and shall conform to the minimum dimensions specified in Article 5.12.9.3.1.

Prestressed concrete piles may be solid or hollow. For hollow piles, precautionary measures, such as venting, shall be taken to prevent breakage due to internal water pressure during driving, ice pressure in trestle piles, or gas pressure due to decomposition of material used to form the void.

The wall thickness of cylinder piles shall not be less than 5.0 in.

5.12.9.4.2—Concrete Quality

The compressive strength of the pile at the time of driving shall not be less than 5.0 ksi. Air-entrained concrete shall be used in piles that are subject to freezing and thawing or wetting and drying.

5.12.9.4.3—Reinforcement

Unless otherwise specified by the Owner, the prestressing strands should be spaced and stressed to provide a uniform compressive stress on the cross section of the pile after losses of not less than 0.7 ksi.

The full length of the prestressing strands shall be enclosed with spiral reinforcement as follows:

For piles not greater than 24.0 in. in diameter:

- spiral wire not less than W3.9;
- spiral reinforcement at the ends of piles having a pitch of 3.0 in. for approximately 16 turns;
- the top 6.0 in. of pile having five turns of additional spiral winding at 1.0-in. pitch; and
- for the remainder of the pile, the strands enclosed with spiral reinforcement with not more than 6.0-in. pitch.

For piles greater than 24.0 in. in diameter:

- spiral wire not less than W4.0;
- spiral reinforcement at the end of the piles having a pitch of 2.0 in. for approximately 16 turns;
- the top 6.0 in. having four additional turns of spiral winding at 1.5-in. pitch; and
- for the remainder of the pile, the strands enclosed with spiral reinforcement with not more than 4.0-in. pitch.

C5.12.9.4.3

The purpose of the 0.7 ksi compression is to prevent cracking during handling and installation. A lower compression may be used if approved by the Owner.

For noncircular piles, use the least dimension through the cross section in place of the "diameter."

5.12.9.5—Cast-in-Place Piles

Piles cast in drilled holes may be used only where soil conditions permit.

Shells for cast-in-place piles shall be of sufficient thickness and strength to hold their form and to show no harmful distortion during driving or after adjacent shells have been driven and the driving core, if any, has been withdrawn. The contract documents shall stipulate that alternative designs of the shell be approved by the Engineer before any driving is done.

C5.12.9.5

Cast-in-place concrete piles include piles cast in driven steel shells that remain in place and piles cast in unlined drilled holes or shafts.

The construction of piles in drilled holes should generally be avoided in sloughing soils, where large cobblestones exist or where uncontrollable groundwater is expected. The special construction methods required under these conditions increase both the cost and the probability of defects in the piles.

The thickness of shells should be shown in the contract documents as "minimum". This minimum thickness should be that needed for pile reinforcement or for strength required for usual driving conditions: e.g., 0.134 in. minimum for 14.0-in. pile shells driven without a mandrel. *AASHTO LRFD Bridge Construction Specifications* requires the Contractor to furnish shells of greater thickness, if necessary, to permit his choice of driving equipment.

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5.12.9.5.1—Pile Dimensions

Cast-in-place concrete piles may have a uniform section or may be tapered over any portion if cast in shells or may be bell-bottomed if cast in drilled holes or shafts.

The area at the butt of the pile shall be at least 100 in.^{2} The cross-sectional area at the tip of the pile shall be at least 50.0 in.^{2}. For pile extensions above the butt, the minimum size shall be as specified for precast piles in Article 5.12.9.3.

5.12.9.5.2—Reinforcement

The area of longitudinal reinforcement shall not be less than 0.8 percent of A_g, with spiral reinforcement not less than W3.9 at a pitch of 6.0 in. The reinforcement shall be extended 10.0 ft below the plane where the soil provides adequate lateral restraint.

Shells that are more than 0.12 in. in thickness may be considered as part of the reinforcement. In corrosive environments, a minimum of 0.06 in. shall be deducted from the shell thickness in determining resistance.

Except where more restrictive requirements are specified in Article 5.11.3.2 for bridges in Seismic Zone 2 or Article 5.11.4.5 for bridges in Seismic Zones 3 or 4, the clear distance between parallel longitudinal, and parallel transverse reinforcing bars in cast-in-place concrete piling, shall not be less than the greater of the following:

- Five times the maximum aggregate size;
- 5.0 in.

C5.12.9.5.2

If the shell is circular and considered as reinforcing, the provisions of Articles 6.9.6, 6.12.2.3 and 6.12.3.2.2 can be utilized to determine capacities.

5.13—ANCHORS

5.13.1—General

Anchors intended to comply with the provisions of this article shall be designed, detailed and installed using the provisions of ACI 318-14, Chapter 17 which is incorporated by reference, unless those provisions are specifically amended herein.

The following types of cast-in-place and post installed anchors as described in ACI 318-14 Article 17.1.3 may be used:

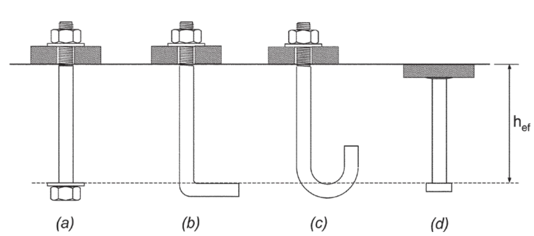
- Headed studs and headed bolts;
- Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding 1.4*Np*, where *Np* is given in ACI 318-14 Eq. 17.4.3.5;
- Adhesive anchors meeting the assessment criteria of ACI 355.4 (2011); and,
- Post-installed expansion and undercut anchors meeting the assessment criteria of ACI 355.2 (2007).

C5.13.1

ACI 318 uses pound units instead of the kip units used in the *AASHTO LRFD Bridge Design Specifications*.

Excerpts from ACI 318-14, Chapter 17 have been provided in this article for emphasis. The absence herein of any provisions from the ACI 318-14, Chapter 17 does not negate their validity as part of that specification.

The various types of anchors covered by ACI 318-14, Chapter 17 are shown below.



Cast-in-place anchors: (a) hex head bolt with washer; (b) L-bolt; (c) J-bolt; and (d) welded headed stud

Figure C5.13.1-1—Anchor Types Included in ACI 318-14, Chapter 17 (Courtesy of ACI)

These specifications shall not be used for the design of grouted anchors.

The concrete compressive strength used for design shall not exceed 10.0 ksi for cast-in-place anchors or 8.0 ksi for post-installed anchors.

All design, material, testing, anchor, geometry and depth limitations, anchor spacing, edge distances, and acceptance requirements specified in ACI 318-14, Chapter 17 shall be satisfied for anchors designed or supplied under these provisions. Group effects, eccentricity of loading, presence or lack of anchor reinforcement, and the possibility of concrete cracking or splitting shall be considered in the design.

Lightweight concrete factors for anchor design shall comply with ACI 318-14, Chapter 17.

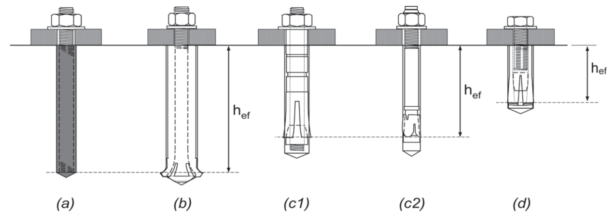
Safety levels associated with ACI 318-14, Chapter 17 are intended only for in-service conditions rather than short-term handling and construction condition, or for impact or cyclic loads other than those associated with seismic events, and should not be used for those purposes except as noted herein.

For the purpose of these specifications, anchors attaching pedestrian or bicycle rails or fences separated from the roadway by a traffic railing meeting the requirements of Section 13 need not be considered subject to the impact exclusion in ACI 318-14, Chapter 17.

The exclusion for impact loads need not apply to other attachments using post-installed anchors shown to have impact strength at least equal to their static strength as documented by testing or a combination of analysis and testing deemed appropriate by the Owner.

Design for seismic events shall comply with the appropriate provisions of ACI 318-14, Chapter 17.

Corrosion control shall be considered in any anchor application exposed to the elements.



Post-installed anchors: (a) adhesive anchor; (b) undercut anchor; (c) Torque-controlled expansion anchors (c-1) sleeve-type and (c2) stud type; and (d) drop-in type displacement controlled expansion anchor

Figure C5.13.1-1 (continued)—Anchor Types Included in ACI 318-14, Chapter 17 (Courtesy of ACI)

Further background and sample calculations for specific cases involving cast-in-place anchors and adhesive anchors can be found in PCA (2013) and Cook et al. (2013), respectively, and for both types in ACI 355.3R (2011). ACI 355.3R contains design examples for cast-in-place and post installed anchors albeit with resistance factors that are not current with those specified ACI 318-14, Chapter 17.

The decision to not require the impact exclusion for the situation indicated is based on the references cited below. Additional references are available in the literature. The use of any documented increase in resistance due to impact behavior is not required but is an Owner option.

ACI 349-13 no longer requires the impact exclusion. Appendix F contains Dynamic Increase Factors (DIF) for undercut and expansion anchors. Shirvani et al. (2004) has documentation of similar DIFs for expansion and undercut anchors in tension in both cracked and uncracked concrete.

Dickey et al. documents the impact behavior of epoxy adhesive anchors and recommends DIFs. The amount of improved strength under impact loads is thought to be dependent on the adhesive used. Similar results were reported by Braimah et al. (2009).

Typical corrosion protection consists of the use of coatings or corrosion resistant materials. For adhesive anchors, the manufacturer's literature must document that the adhesive used is compatible with the type and extent of any coating used.

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5.13.2—General Strength Requirements

5.13.2.1—Failure Modes to be Considered

The following failure modes shall be considered as applicable:

- Tension
  - Tensile strength of anchor steel
  - Pullout—cast-in-place, post-installed expansion and undercut anchors
  - Concrete breakout—all types
  - Concrete splitting
  - Side-face blowout—headed anchors
  - Bond failure—adhesive anchors
- Shear
  - Shear strength of anchor steel
  - Pryout—all types
  - Concrete breakout—all types
- Tension-shear interaction

Design of steel components, including reinforcing bars, plain bars, or threaded bars, against failure in tension, shear, or combined shear and tension shall comply with the appropriate provisions of ACI 318-14, Chapter 17.

C5.13.2.1

The cited failure modes are illustrated schematically below.

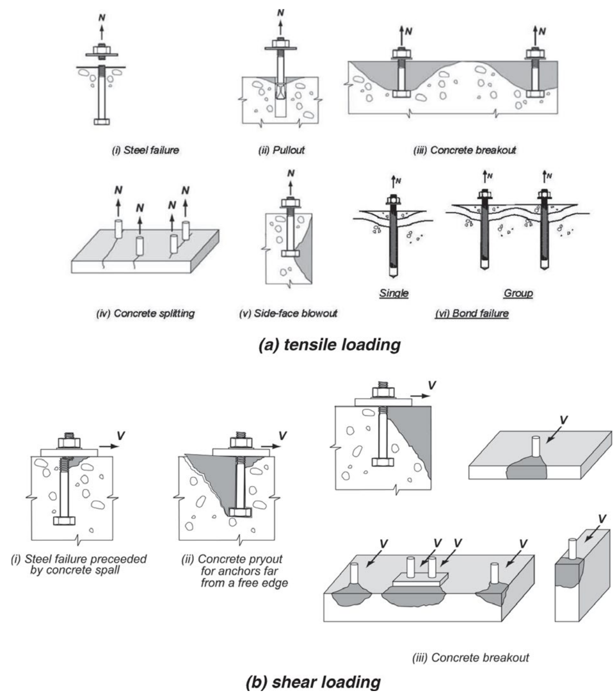

Figure C5.13.2.1-1—Failure Modes Considered in ACI 318-14, Chapter 17 (Courtesy of ACI)

5.13.2.2—Resistance Factors

Resistance factors for various applications shall be taken as the corresponding strength reduction factors specified in ACI 318-14 Article 17.3.3 and shall be used in conjunction with the loads, load factors and load combinations specified in Section 3 of these specifications.

Where adhesive anchors are subjected to significant sustained tensile loads the factored bond resistance shall be further reduced to avoid creep-rupture or creep-displacement failure. In lieu of Owner-supplied criteria this reduction may be accomplished by using the provisions of ACI 318-14, Chapter 17 with a factor of 0.50 in place of 0.55 in ACI 318-14 Eq. 17.3.1.2 for the sustained load combination.

C5.13.2.2

Based on a comparison of load factors and resistance factors for concrete design in the ACI and AASHTO specifications it was concluded that the strength reduction factors in ACI 318-14, Chapter 17 are reasonable and possibly somewhat conservative for this first introduction of anchor design into the *AASHTO LRFD Bridge Design Specifications.* Further discussion related to adhesive anchors in particular can be found in Cook et al. (2013).

The reduction for sustained tensile loading is based on the experience in the 2006 tunnel ceiling collapse in Boston. ACI 318-14, Chapter 17 recommends that less than 55 percent of the resistance be used for a design life greater than 50 years but no specific value is suggested. The 50 percent factor specified herein is based solely on recommendations for 100-year life at 70°F or 20 years at 110°F in Cook et al. (2013).

In lieu of Owner-supplied criteria significant sustained tensile loads may be considered to be those with an unfactored magnitude exceeding 10 percent of the ultimate capacity of the anchor or anchor group. This is based on years of successful past practice by one state.

5.13.2.3—Determination of Anchor Resistance

The nominal resistance of cast-in-place anchors shall be determined either by calculation procedures provided in ACI 318-14, Chapter 17 or by use of five percent fractile strength as described in ACI 318-14 Article 17.3.2. The nominal resistance of post-installed anchors shall be determined by tests using the criteria of ACI 355.2 (2007) for mechanical anchors or ACI 355.4 for adhesive anchors. For adhesive anchors for which test data is not available at the time of design, the minimum characteristic bond stresses given in ACI 318-14 Table 17.4.5.2, with adjustments for sustained tensile loading and seismic situations may be used.

The factored resistance shall be determined by applying the resistance factors specified in Article 5.13.2.2 to the nominal resistance.

C5.13.2.3

Qualification by testing is required for post-installed anchors because their resistance, particularly their bond strength from their constituent components cannot currently be predicted with sufficient reliability. The values given in ACI 318-14 Table 17.4.5.2 are lower bound bond stresses permitted by ACI 355.4 (2011). The characteristic bond strength of a qualified adhesive in a specific situation may be much higher.

The resistance of post-installed anchors is sensitive to installation procedures. ACI 355.4 has special requirements for horizontal or upwardly inclined adhesive anchors.

5.13.3—Seismic Design Requirements

The requirements of ACI 318-14 Article 17.2.3 shall apply to anchors used in the seismic load path on structures in Seismic Zones 2, 3, and 4 specified in Article 3.10.6 of these specifications.

C5.13.3

ACI 318-14, Chapter 17 uses the Seismic Design Categories (SDC) A through F in ASCE/SEI 7 2010. ACI Chapter 17 requires seismic design for anchors used in structures in SDC C through F. SDC C applies to situations with moderate to intermediate seismic risk. SDC D and higher categories apply to high-risk situations.

A direct correlation between SDC and AASHTO Seismic Zones (SZ) is not straight forward. The issues can be understood by comparing ASCE Figure 11.4-1 to AASHTO Figure 3.10.4.1-1, the design response spectrum. The generalized shape of the spectrum in both documents is quite similar for periods below the ASCE long term period, T_L, for which AASHTO makes no distinction. The basic information required to go from a generalized figure to a site specific elastic seismic response coefficient, C_{sm}, are map data and site factors. The ASCE maps present data for a probability of structural failure of approximately once in 5,000 years while the AASHTO maps present data for a return period of seismic event exceedance of approximately 1,000 years. This difference is partly and nonuniformly assuaged by multiplying by 2/3 in the case of the ASCE data before constructing the design response spectrum. The site factors are tabulated by site class and short period acceleration coefficient. The values in the tables are identical, but the selection of which value to use is affected by which set of maps is used to determine the short period acceleration coefficient.

Nevertheless, a comparison of the ASCE SDCs in Table 11.6-2 and the AASHTO seismic zones in Table 3.10.6-1 suggests that Seismic Zone 1 is a reasonable choice to represent SDCs A and B.

There is no distinction in the requirements in ACI 318-14, Chapter 17 for structures in SDC C, D, E, or F and therefore no distinction is made herein among Seismic Zones 2, 3, and 4.

Anchors intended to resist seismic forces shall be suitable for use in cracked concrete.

ACI 355.2 (2007) and 355.4 (2011) have provisions for Simulated Seismic Tests for expansion and undercut anchors and for adhesive anchors, respectively.

ACI 318-14, Chapter 17 contains conceptual details for anchor restraining reinforcement.

Material for anchor restraining reinforcement in Seismic Zones 2, 3, and 4 shall comply with the applicable provisions of Articles 5.4.3 and 5.11 of these specifications.

The provisions of ACI 318-14, Chapter 17 do not apply to the design of anchors in plastic hinge zones of concrete structures subject to earthquake forces. Anchors located in plastic hinge zones shall be designed to transfer loads directly to anchor reinforcement.

The more extensive cracking and spalling expected in plastic hinge zones are behaviors beyond that anticipated by the concrete-governed resistance equations in ACI 318-14, Chapter 17. Where anchors must be located in regions of plastic hinging, they should be designed to transfer load directly to anchor reinforcement that is specifically designed to carry the anchor forces into the body of the member beyond the anchorage region. Configurations that rely on concrete tensile strength should not be used.

5.13.4—Installation

Unless Owner-supplied installation requirements are more stringent, the contract documents shall require compliance with the provisions of ACI 318-14 Article 17.8 as applicable to the type of anchor being installed.

C5.13.4

Anchor performance is dependent on proper installation which, as a minimum requires:

- qualified, and in some cases certified, installers and inspectors;
- compliance with written supplier instructions;
- adequate support of cast-in-place anchors prior to and during concrete placement;
- continuous monitoring of installation of adhesive anchors;
- compliance with the specified type, size, and procedures for hole drilling; and
- adequate cleaning and any other specified post-drilling hole preparation.

ACI 318-14 Article 17.8 contains special installation guidance for horizontal or upwardly inclined adhesive anchors.

5.14—DURABILITY

5.14.1—Design Concepts

Protective measures for durability shall satisfy the requirements specified in Article 2.5.2.1.

Concrete structures shall be designed to provide protection of the reinforcing and prestressing steel against corrosion throughout the life of the structure.

Special requirements that may be needed to provide durability shall be indicated in the contract documents. Portions of the structure shall be identified where any of the following are required:

- air-entrainment of the concrete;
- epoxy-coated or galvanized reinforcement;
- use of corrosion resistant bars complying with AASHTO M 334;
- sealing or coating;
- special concrete additives;

C5.14.1

NCHRP (2013) points out that "durability" is not a single property of concrete but rather it is a series of properties required for the particular environment to which the concrete will be exposed during its service life. While material aspects of concrete design are a major factor in producing durable concrete structures, attention to design, detailing and construction QA/QC are also vital for a successful outcome.

The literature contains numerous reports and papers on concrete durability. This commentary contains information from several sources but should not be considered exhaustive, nor should it be interpreted as conflicting with documented good local experience with other durability enhancing policies or protocols.

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- special curing procedures; and
- low permeability concrete,

or where the concrete is expected to be exposed to saltwater or to sulfate soils or water.

Decisions regarding the use of durability enhancing materials and strategies should be based on life cycle cost analysis. Factors to be considered may include, but are not limited to:

- associated savings from reduced cover, if any, versus potential problems;
- initial cost of durability enhancing materials and strategies versus long-term benefits;
- negative impacts from any work over navigable waterways, major or congested roadways, environmentally sensitive areas, or active railroads;
- possible future issues when project requirements dictate a deck with less depth and/or cover than standard practice; and
- the impact of longer service life on user costs and worker safety.

Freyermuth (2009) lists the following options for achieving extended service life of concrete bridges, although the extension is not quantified:

- Use of high-performance concrete to decrease permeability.
- Use of prestressing to reduce or control cracking.
- Use of jointless bridges, or bridge segments, and integral bridges.
- Use of integral deck overlays on precast concrete segmental bridges in aggressive environments.
- Selective use of stainless steel reinforcing.

Structures intended to provide extended service lives must have the following attributes:

- Conceived, sited, and designed to provide an acceptable level of reliability with respect to the natural environment and man-made loads.
- Properly constructed with suitable materials and details.
- Provided with adequate control of deck drainage, especially in areas where deicing salt is applied or environmental salt is present.
- Treated with timely preventative maintenance of protective coatings, drainage systems, joints, and bearings.

Design considerations for durability include concrete quality, protective coatings, minimum cover, distribution and size of reinforcement, details, and crack widths or prestressing. Further guidance can be found in ACI Committee Report 222 (2001) and Poston et al. (1987).

The principal aim of these specifications, with regard to durability is the prevention of corrosion of the reinforcing steel. There are provisions in *AASHTO LRFD Bridge Construction Specifications* for air-entrainment of concrete and some special construction procedures for concrete exposed to sulfates or salt water. For unusual conditions, the contract documents should augment the provisions for durability.

The critical factors contributing to the durability of concrete structures are:

- adequate cover over reinforcement;
- nonreactive aggregate-cement combinations;
- thorough consolidation of concrete;
- adequate cementitious material;
- low *W/CM* ratio; and
- thorough curing, preferably with water.

5.14.2—Major Chemical and Mechanical Factors Affecting Durability

5.14.2.1—General

Design for durability should consider detrimental regional and site specific chemical and mechanical agents that can reduce durability.

C5.14.2.1

Table C5.14.2.1-1 summarizes defects which can reduce concrete durability, their manifestations, causes, time of appearance and some preventative actions, taken

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from NCHRP (2013) which cited VanDam et al (1998). Additional information can be found in PCA (2011).

Table C5.14.2.1-1—Factors in Concrete Durability

Type of Materials-Related Defect	Surface Distress Manifestations and Locations	Cause or Mechanisms	Time of Appearance	Prevention or Reduction
Due to Physical Mechanisms				
Mechanical wear of decks and wearing surfaces decks and wearing surfaces	Abrasion and polishing polishing, rutting	Tire contact, improper curing, water floating to surface	Varies	Proper curing, sealants
Freezing and thawing deterioration of hardened cement paste	Scaling or map cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated cycles of freezing and thawing.	1–5 years	Addition of air-entraining agent to establish protective air-void system.
Deicer scaling and deterioration	Scaling or crazing of the slab surface.	Deicing chemicals can amplify deterioration due to freezing and thawing and may interact chemically with cement hydration products.	1–5 years	Limiting W/C ratio to no more than 0.45, and providing a minimum 30-day drying period after curing before allowing the use of deicers.
Deterioration of aggregate due to freezing and thawing	Cracking parallel to joints and cracks and later spalling; maybe accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing or excessive dilation of aggregate.	10–15 years	Use of nonsusceptible aggregates or reduction in maximum coarse aggregate size.
Early age cracking	Map cracking	Shrinkage of concrete	<28 days	Shrinkage limits, fibers continuous wet cure
Due to Chemical Mechanisms				
Alkali–silica reaction (ASR)	Map cracking (rarely more than 2.0 in. deep) over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Reaction between alkalis in cement and reactive silica in aggregate, resulting in an expansive gel and the degradation of the aggregate particle.	5–15 years	Use of nonsusceptible aggregates, addition of pozzolans, limiting of alkalis in concrete, addition of lithium salts.
Alkali–carbonate reaction	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in cement and carbonates in certain Aggregates containing clay fractions.	5–15 years	Avoiding susceptible aggregates, or blending susceptible aggregate with nonreactive aggregate.
External sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Expansive formation of ettringite or gypsum that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with aluminates in cement or fly ash.	1–5 years	Minimizing tricalcium aluminate content in cement or using blended cements, Class F fly ash, or GGBFS.

(continued on next page)

Table C5.14.2.1-1 (continued)—Factors in Concrete Durability

Internal sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption on the paste phase or fills available air voids.	1–5 years	Minimizing tricalcium aluminate content in cement, using low sulfate cement, eliminating source of slowly soluble sulfate, and use cements conforming to ASTM C150, C595, or C1157, and avoiding high curing temperatures.
Corrosion of embedded steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete and corrode embedded steel.	3–10 years	Reducing the permeability of the concrete, providing adequate concrete cover, and coating steel.

5.14.2.2—Corrosion Resistance

Reinforcement that is susceptible to corrosion and is used in concrete exposed to deicing salts or salt water shall be protected by the use of low-permeability concrete and concrete cover to the reinforcement in accordance with Article 5.14.3 (or 5.10.1).

C5.14.2.2

Salt intrusion is slowed by drainage control, providing suitable cover, use of dense, low permeability concrete such as high performance concrete (HPC) and ultra-high performance concrete (UHPC), and control of cracking.

The effects of salt intrusion and depassivation due to carbonation can be mitigated by using corrosion inhibitors, coated reinforcing, bimetallic reinforcement, stainless steel reinforcing, or nonmetallic reinforcement such as fiber-reinforced polymer (FRP) composites. Without citing specifics of cost/benefit, it will generally be found that cost increases with each step in the reinforcing path above.

Prestressing contributes to control of salt intrusion by reducing in-service cracking. While some cracking may result from overloads, thermal gradients, and shrinkage, the cracks will generally close when the causative effect is reduced or eliminated. Beam ends exposed to salt-laden deck drainage have been found to be susceptible to corrosion damage resulting from water entering the beam via the strand ends. Tabatabai et al. (2004) documented the benefit of coating the ends (end 2 feet in this study) with sealing materials and concluded that of four tested materials, a polymer resin coating was most effective and easiest to apply.

5.14.2.3—Freeze–Thaw Resistance

Air-entrained concrete, designated "AE" in Table 8.2.2-1 of the *AASHTO LRFD Bridge Construction Specifications*, 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.

C5.14.2.3

Freeze–thaw cycles can lead to scaling of the concrete surface due to pressure caused by the expansion of water in the concrete. Use of air-entrainment, high-strength mixes, and low permeability are effective countermeasures, although air-entrainment can result in reduced strength and may not be compatible with HPC or high-strength concrete (HSC). Use of fly ash can be counterproductive if delayed strength gain exposes the

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concrete to freezing before sufficient strength has been developed. The use of air-entrainment is generally recommended when 20 or more cycles of freezing and thawing per year are expected at the location and exposure. Decks and rails are most vulnerable, whereas buried footings are seldom damaged by freeze–thaw action.

5.14.2.4—External Sulfate Attack

The provisions of the *AASHTO LRFD Bridge Construction Specifications* Article 8.6.7 shall apply.

C5.14.2.4

Sulfate soils or water, sometimes called alkali, contain high levels of sulfates of sodium, potassium, calcium, or magnesia. Sulfate attack is a result of the growth of minerals caused by reaction of chemicals in the cement with sulfates that were in the mix, usually in the water but possibly in the aggregate. It debonds the aggregate, creates expansive pressure leading to crack or delamination. The causes and effects are similar to ASR. Use of Type II, Type V, or a blended cement are often indicated as well as use of approved material sources. Factors which reduce permeability are also usually helpful. Detwiler (2008) states that “Maximum limits on the water–cementitious materials ratio, combined with good concreting practices—especially good curing—are even more important to sulfate resistance than the right cement.” Salt water, water soluble sulfate in soil above 0.1 percent, or sulfates in water above 150 ppm justify use of the special construction procedures called for in *AASHTO LRFD Bridge Construction Specifications*. These include avoidance of construction joints between the levels of low water and the upper limit of wave action. For sulfate contents above 0.2 percent in soil or 1,500 ppm in water, special concrete mixes may be justified. Further guidance may be found in ACI 201 (2008), which provides recommended mix practices for various sulfate concentrations, or PCA (2011).

5.14.2.5—Delayed Ettringite Formation

Concrete temperatures during curing shall not exceed 160°F to minimize delayed ettringite formation.

5.14.2.6—Alkali–Silica Reactive Aggregates

The provisions of *AASHTO LRFD Bridge Construction Specifications* Article 8.3.4 shall apply.

C5.14.2.6

Aggregate reactivity issues such as ASR are usually handled by prescreening possible sources using laboratory tests to identify susceptibility. Most states have approved sources that largely eliminate aggregate reactivity. Use of low-alkali cement can also reduce susceptibility of a concrete mix.

5.14.2.7—Alkali–Carbonate Reactive Aggregates

Aggregates susceptible to alkali–carbonate reactivity shall not be used unless blended according to the appendix in ASTM C1105.

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5.14.3—Concrete Cover

The provisions of Article 5.10.1 shall apply unless superseded by the contract documents.

5.14.4—Corrosion-Resistant Reinforcement

Protection of steel against chloride-induced corrosion may be provided by epoxy coating or galvanizing of reinforcing steel, post-tensioning duct, and anchorage hardware and by epoxy coating of prestressing strand or by using corrosion-resistant reinforcement bars complying with AASHTO M 334.

Protection of concrete may include coatings such as silane sealer, methacrylate sealers and bituminous coatings for underground concrete.

C5.14.4

Specifications for acceptable epoxy coatings are included in the materials section of the *AASHTO LRFD Bridge Construction Specifications*. Other corrosion-resistant reinforcing bars meeting the requirements of AASHTO M 334 have been increasingly used in bridge structures. Refer to Berke (2012), Darwin and Browning (2009), Hartt et al. (2009), Salomon and Moen (2014), and Zhang et al. (2009).

5.14.5—Deck Protection Systems

Deck protection systems shall be considered for all bridge decks exposed to freeze–thaw cycles and application of deicing chemicals. The Owner should consider providing additional protection against penetration of chlorides. For segmental bridges the Owner should consider additional concrete cover acting as an integral wearing surface or a minimum 1.5 in. thickness overlay. If an integral overlay is selected, the Owner should consider an additional 0.5 in. of cover as a grinding allowance for rideability. Alternatively, a waterproof membrane with bituminous overlay or a noncementitious overlay may be used. The Owner may require specific materials and placement techniques stipulated by local practices.

C5.14.5

Deck protection systems are encouraged because they will add durability. The thickness of an integral overlay should be decided upon by the Owner considering local experience including the permeability of the concrete being used. Delamination of overlays is generally due to poor installation practices or material selection.

Careful attention to detail is required when using overlays to assure the proper railing heights are obtained. All railings next to deck areas to be overlayed should be detailed from the top of the overlay. See Article 13.7.3.2 for further information.

The need to remove and replace the overlay can be based on measurement of chloride penetration into the overlay. Use of high performance concrete is an effective means of minimizing chloride penetration into concrete.

Bridges located in other corrosive environments, such as coastal bridges over salt water, should be evaluated for the need for additional protection.

5.14.6—Protection for Prestressing Tendons

Ducts for internal post-tensioned tendons, designed to provide bonded resistance, shall be grouted after stressing. Other tendons shall be permanently protected against corrosion and the details of protection shall be indicated in the contract documents.

C5.14.6

In certain cases, such as longitudinal precast elements transversely post-tensioned together, the integrity of the structure does not depend on the bonded resistance of the tendons, but rather on the confinement provided by the prestressing elements. The unbonded tendons can be more readily inspected and replaced, one at a time, if so required.

External tendons have been successfully protected by cement grout in polyethylene ducts and metal pipes. Tendons have also been protected by heavy grease or other anticorrosion medium where future replacement is envisioned. Tendon anchorage regions should be protected by encapsulation or other effective means. This is critical in unbonded tendons because any failure of the anchorage can release the entire tendon.

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APPENDIX A5—BASIC STEPS FOR CONCRETE BRIDGES

A5.1—GENERAL

This outline is intended to be a generic overview of the design process using the simplified methods for illustration. It should not be regarded as complete, nor should it be used as a substitute for a working knowledge of the provisions of this section.

A5.2—GENERAL CONSIDERATIONS

A. Design Philosophy (1.3.1)
B. Limit States (1.3.2)
C. Design Objectives and Location Features (2.3) (2.5)

A5.3—BEAM AND GIRDER SUPERSTRUCTURE DESIGN

A. Develop General Section
  1. Roadway Width (Highway-Specified)
  2. Span Arrangements (2.3.2) (2.5.4) (2.5.5) (2.6)
  3. Select Bridge Type
B. Develop Typical Section
  1. Precast P/S Beams
    a. Top Flange (5.12.3.2.2)
    b. Bottom Flange (5.12.3.2.2)
    c. Webs (5.12.3.2.2)
    d. Structure Depth (2.5.2.6.3)
    e. Minimum Reinforcement (5.6.7) (5.6.3.3)
    f. Lifting Devices (5.12.3.2.3)
    g. Joints (5.12.3.4.2)
  2. CIP T-Beams and Multiweb Box Girders (5.12.3.5)
    a. Top Flange (5.12.3.5.1a)
    b. Bottom Flange (5.12.3.5.1b)
    c. Webs (5.12.3.5.1c)
    d. Structure Depth (2.5.2.6.3)
    e. Reinforcement (5.12.3.5.2)
      (1) Minimum Reinforcement (5.6.3.3) (5.6.7)
      (2) Temperature and Shrinkage Reinforcement (5.10.6)
    f. Effective Flange Widths (4.6.2.6)
    g. Strut-and-Tie Areas, if Any (5.8.2)
C. Design Conventionally Reinforced Concrete Deck
  1. Deck Slabs (4.6.2.1)
  2. Minimum Depth (9.7.1.1)
  3. Empirical Design (9.7.2)
  4. Traditional Design (9.7.3)
  5. Strip Method (4.6.2.1)
  6. Live Load Application (3.6.1.3.3) (4.6.2.1.5)
  7. Distribution Reinforcement (9.7.3.2)
  8. Overhang Design (A13.4) (3.6.1.3.4)
D. Select Resistance Factors
  Strength Limit State (Conventional) (5.5.4.2)
E. Select Load Modifiers
  1. Ductility (1.3.3)
  2. Redundancy (1.3.4)
  3. Operational Importance (1.3.5)
F. Select Applicable Load Combinations and Load Factors (3.4.1, Table 3.4.1-1)
G. Calculate Live Load Force Effects
  1. Live Loads (3.6.1) and Number of Lanes (3.6.1.1.1)
  2. Multiple Presence (3.6.1.1.2)
  3. Dynamic Load Allowance (3.6.2)
  4. Distribution Factor for Moment (4.6.2.2.2)

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a. Interior Beams with Concrete Decks (4.6.2.2.2b)
b. Exterior Beams (4.6.2.2.2d)
c. Skewed Bridges (4.6.2.2.2e)
5. Distribution Factor for Shear (4.6.2.2.3)
a. Interior Beams (4.6.2.2.3a)
b. Exterior Beams (4.6.2.2.3b)
c. Skewed Bridges (4.6.2.2.3c, Table 4.6.2.2.3c-1)
6. Reactions to Substructure (3.6)
H. Calculate Force Effects from Other Loads as Required
I. Investigate Service Limit State
1. P/S Losses (5.9.3)
2. Stress Limitations for P/S Tendons (5.9.2.2)
3. Stress Limitations for P/S Concrete (5.9.2.3)
a. Before Losses (5.9.2.3.1)
b. After Losses (5.9.2.3.2)
4. Durability (5.14)
5. Crack Control (5.6.7)
6. Fatigue, if Applicable (5.5.3)
7. Deflection and Camber (2.5.2.6.2) (3.6.1.3.2) (5.6.3.5.2)
J. Investigate Strength Limit State
1. Flexure
a. Stress in P/S Steel—Bonded Tendons (5.6.3.1.1)
b. Stress in P/S Steel—Unbonded Tendons (5.6.3.1.2)
c. Flexural Resistance (5.6.3.2)
d. Limits for Reinforcement (5.6.3.3)
2. Shear (Assuming No Torsional Moment)
a. General Requirements (5.7.2)
b. Sectional Design Model (5.7.3)
(1) Nominal Shear Resistance (5.7.3.3)
(2) Determination of β and θ (5.7.3.4)
(3) Longitudinal Reinforcement (5.7.3.5)
(4) Transverse Reinforcement (5.7.2.3) (5.7.2.5) (5.7.2.4) (5.7.2.6)
(5) Horizontal Shear (5.7.4)
K. Check Details
1. Cover Requirements (5.10.1)
2. Development Length—Reinforcement (5.10.8.1) (5.10.8.2)
3. Development Length—Prestressing (5.9.4.3)
4. Splices (5.10.8.4) (5.10.8.5)
5. Anchorage Zones
a. Post-Tensioned (5.9.5.6)
b. Pretensioned (5.9.4.4)
6. Ducts (5.4.6)
7. Tendon Profile Limitation
a. Tendon Confinement (5.9.5.4)
b. Curved Tendons (5.9.5.4)
c. Spacing Limits (5.9.5.1)
8. Reinforcement Spacing Limits (5.10.3)
9. Transverse Reinforcement (5.7.2.4) (5.7.2.6) (5.7.2.7)
10. Beam Ledges (5.8.4.3)

A5.4—SLAB BRIDGES

Generally, the design approach for slab bridges is similar to beam and girder bridges with some exceptions, as noted below.

A. Check Minimum Recommended Depth (2.5.2.6.3)
B. Determine Live Load Strip Width (4.6.2.3)
C. Determine Applicability of Live Load for Decks and Deck Systems (3.6.1.3.3)
D. Design Edge Beam (9.7.1.4)
E. Investigate Shear (5.12.2.1)
F. Investigate Distribution Reinforcement (5.12.2.1)

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G. If Not Solid
   1. Check if Voided Slab or Cellular Construction (5.12.2.2.1)
   2. Check Minimum and Maximum Dimensions (5.12.2.2.1)
   3. Design Diaphragms (5.12.2.2.3)
   4. Check Design Requirements (5.12.2.2.4)

A5.5—SUBSTRUCTURE DESIGN

A. Establish Minimum Seat Width (4.7.4.4)
B. Compile Force Effects Not Compiled for Superstructure
   1. Wind (3.8)
   2. Water (3.7)
   3. Effect of Scour (2.6.4.4.2)
   4. Ice (3.9)
   5. Earthquake (3.10) (4.7.4)
   6. Temperature (3.12.2) (3.12.3) (4.6.6)
   7. Superimposed Deformation (3.12)
   8. Ship Collision (3.14) (4.7.5)
   9. Vehicular Collision (3.6.5)
   10. Braking Force (3.6.4)
   11. Centrifugal Force (3.6.3)
   12. Earth Pressure (3.11)
C. Analyze Structure and Compile Load Combinations
   1. Table 3.4.1-1
   2. Special Earthquake Load Combinations (3.10.8)
D. Design Compression Members (5.6.4)
   1. Factored Axial Resistance (5.6.4.4)
   2. Biaxial Flexure (5.6.4.5)
   3. Slenderness Effects (4.5.3.2.2) (5.6.4.3)
   4. Transverse Reinforcement (5.6.4.6)
   5. Shear (Usually EQ and Ship Collision Induced) (3.10.9.4.3)
   6. Reinforcement Limits (5.6.4.2)
   7. Bearing (5.6.5)
   8. Durability (5.14)
   9. Detailing (as in Step A5.3K) and Seismic (5.11)
E. Design Foundations (Structural Considerations)
   1. Scour
   2. Footings (5.12.8)
   3. Abutments (Section 11) (5.12.9)
   4. Pile Detailing (5.12.9)

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APPENDIX B5—GENERAL PROCEDURE FOR SHEAR DESIGN WITH TABLES

B5.1—BACKGROUND

The general procedure herein is an acceptable alternative to the procedure specified in Article 5.7.3.4.2. The procedure in this Appendix utilizes tabularized values of β and θ instead of Eqs. 5.7.3.4.2-1, 5.7.3.4.2-2, and 5.7.3.4.2-3. Appendix B5 is a complete presentation of the general procedures in LRFD Design (AASHTO 2007) without any interim changes.

B5.2—SECTIONAL DESIGN MODEL—GENERAL PROCEDURE

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-1. In using this table, ε_x shall be taken as the calculated longitudinal strain at the mid-depth of the member when the section is subjected to M_u, N_u, and V_u as shown in Figure B5.2-1.

For sections containing less transverse reinforcement than specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-2. In using this table, ε_x shall be taken as the largest calculated longitudinal strain which occurs within the web of the member when the section is subjected to N_u, M_u, and V_u as shown in Figure B5.2-2.

Where consideration of torsion is required by the provisions of Article 5.7.2, V_u in Eqs. B5.2-3 through B5.2-5 shall be replaced by V_{eff}.

For solid sections:

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \quad \text{(B5.2-1)}$$

For hollow sections:

$$V_{eff} = V_u + \frac{T_u d_s}{2A_o} \quad \text{(B5.2-2)}$$

Unless more accurate calculations are made, ε_x shall be determined as:

- If the section contains at least the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps} f_{po}\right)}{2(E_s A_s + E_p A_{ps})} \quad \text{(B5.2-3)}$$

CB5.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.7.3.1. Such an analysis (see Figure CB5.2-1) 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) 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 CB5.2-2.

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. B5.2-1. The 0.9 p_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 hollow 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. B5.2-2 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.

Members containing at least the minimum amount of transverse reinforcement have a considerable capacity to redistribute shear stresses from the most highly strained portion of the cross section to the less highly strained portions. Because

The initial value of ε_x should not be taken greater than 0.001.

- If the section contains less than the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}}$$

(B5.2-4)

The initial value of ε_x should not be taken greater than 0.002.

- If the value of ε_x from Eqs. B5.2-3 or B5.2-4 is negative, the strain shall be taken as:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_cA_{ct} + E_sA_s + E_pA_{ps}\right)}$$

(B5.2-5)

where:

A_{ct} = area of concrete on the flexural tension side of the member as shown in Figure B5.2-1 (in.^{2})

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure B5.2-1 (in.^{2})

A_o = area enclosed by the shear flow path, including any area of holes therein (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 B5.2-1. In calculating A_s for use in this equation, bars which are terminated at a distance less than their development length from the section under consideration shall be ignored (in.^{2})

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

f_{po} = a parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete. For the usual levels of prestressing, a value of $0.7f_{pu}$ will be appropriate for both pretensioned and post-tensioned members (ksi)

of this capacity to redistribute, it is appropriate to use the mid-depth of the member as the location at which the biaxial stress conditions are determined. Members that contain no transverse reinforcement, or contain less than the minimum amount of transverse reinforcement, have less capacity for shear stress redistribution. Hence, for such members, it is appropriate to perform the biaxial stress calculations at the location in the web subject to the highest longitudinal tensile strain; see Figure B5.2-2.

The longitudinal strain at the mid-depth of the member, ε_x, can be determined by the procedure illustrated in Figure CB5.2-3. 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_{ct}. 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 $(V_u - V_p)\cot\theta$. 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 $0.5(V_u - V_p)\cot\theta$, being taken by each flange. To avoid a trial and error iteration process, it is a convenient simplification to take this flange force due to shear as $V_u - V_p$. This amounts to taking $0.5\cot\theta = 1.0$ in the numerator of Eqs. B5.2-3, B5.2-4, and B5.2-5. This simplification is not expected to cause a 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 CB5.2-4.

For members containing at least the minimum amount of transverse reinforcement, ε_x can be taken as:

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2}$$

(CB5.2-1)

where ε_t and ε_c are positive for tensile strains and negative for compressive strains. If, for a member subject to flexure, the strain ε_c is assumed to be negligibly small, then ε_x becomes one half of ε_t. This is the basis for the expression for ε_x given in Eq. B5.2-3. For members containing less than the minimum amount of transverse reinforcement, Eq. B5.2-4 makes the conservative simplification that ε_x is equal to ε_t.

In some situations, it will be more appropriate to determine ε_x using the more accurate procedure of Eq. CB5.2-1 rather than the simpler Eqs. B5.2-3 through B5.2-5. For example, the shear capacity of sections near the ends of precast, pretensioned simple beams made continuous for live load will be estimated in a very conservative manner by

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M_u = factored moment at the section, not to be taken less than $V_u d_v$ (kip-in.)
N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)
p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.)
T_u = applied factored torsional moment on the girder (kip-in.)
V_u = factored shear force for the girder in Eq. B5.2-1 and for the web under consideration in Eq. B5.2-2 (kip)

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 B5.2-1.

The crack spacing parameter as influenced by aggregate size, s_{xe}, used in Table B5.2-2, shall be determined as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ in.} \tag{B5.2-6}$$

where:

a_g = maximum aggregate size (in.)
s_x = crack spacing parameter, taken as the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003 b_v s_x$, as shown in Figure B5.2-3 (in.)

In the evaluation of ε_x, β and θ, the following should be considered:

- M_u shall be taken as positive quantities and M_u shall not be taken less than $(V_u - V_p)d_v$.
- In calculating A_s and A_{ps} the area of bars or tendons which are terminated less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.
- For sections closer than d_v to the face of the support, the value of ε_x 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 resulting increase in ε_x shall be taken into account. In lieu of more accurate calculations, the value calculated from Eq. B5.2-4 should be doubled.
- It is permissible to determine β and θ from Tables B5.2-1 and B5.2-2 using a value of ε_x that is greater than that calculated from Eqs. B5.2-4 and B5.2-5.

Eqs. B5.2-3 through B5.2-5 because, at these locations, the prestressing strands are located on the flexural compression side and, therefore, will not be included in A_{ps}. This will result in the benefits of prestressing not being accounted for by Eqs. B5.2-3 through B5.2-5.

Absolute value signs were added to Eqs. B5.2-3 through B5.2-5 in 2004. This notation replaced direction in the nomenclature to take M_u and V_u as positive values. For shear, absolute value signs in Eqs. B5.2-3 through B5.2-5 are needed to properly consider the effects due to V_u and V_p in sections containing a parabolic tendon path which may not change signs at the same location as shear demand, particularly at midspan.

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 posttensioning is completed.

Note that in both Table B5.2-1 and Table B5.2-2, the values of β and θ given in a particular cell of the table can be applied over a range of values. Thus from Table B5.2-1, θ = 34.4 degrees and β = 2.26 can be used provided that ε_x is not greater than 0.75×10^{-3} and v_u / f'_c is not greater than 0.125. Linear interpolation between the values given in the tables may be used, but is not recommended for hand calculations. Assuming a value of ε_x larger than the value calculated using Eqs. B5.2-3, B5.2-4, or B5.2-5, as appropriate, is permissible and will result in a higher value of θ and a lower value of β. Higher values of θ will typically require more transverse shear reinforcement, but will decrease the tension force required to be resisted by the longitudinal reinforcement. Figure CB5.2-5 illustrates the shear design process by means of a flow chart. This figure is based on the simplified assumption that 0.5 cot θ = 1.0.

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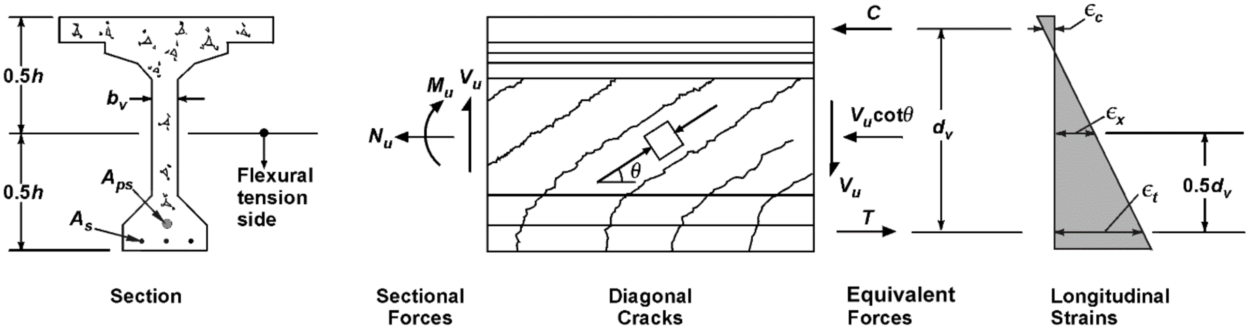

Figure B5.2-1—Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, $V_p = 0$

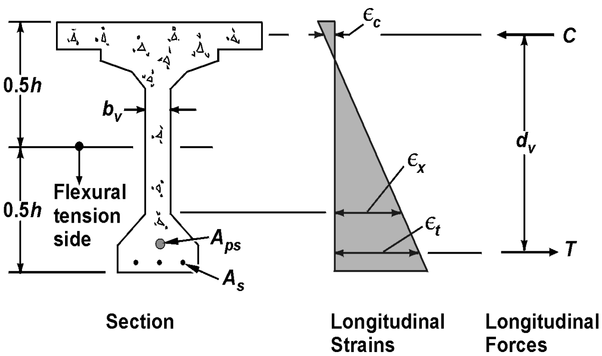

Figure B5.2-2—Longitudinal Strain, ε_x, for Sections Containing Less than the Minimum Amount of Transverse Reinforcement

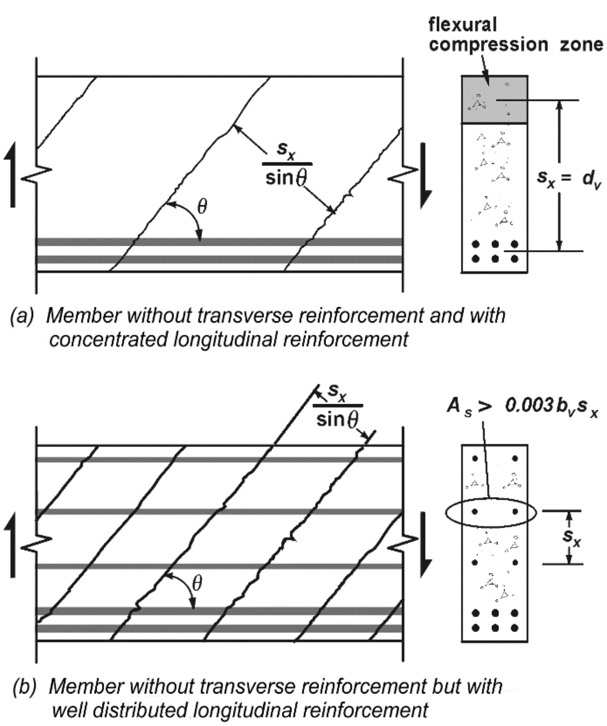

Figure B5.2-3—Definition of Crack Spacing Parameter, s_x

For sections containing a specified amount of transverse reinforcement, a shear-moment interaction diagram, see Figure CB5.2-6, can be calculated directly from the procedures in this article. For a known concrete strength and a certain value of ε_x, each cell of Table B5.2-1 corresponds to a certain value of v_u/f'_c, i.e., a certain value of V_n. This value of V_n requires an amount of transverse reinforcement expressed in terms of the parameter $A_v f_y/(b_v s)$. The shear capacity corresponding to the provided shear reinforcement can be found by linearly interpolating between the values of V_n corresponding to two consecutive cells where one cell requires more transverse reinforcement than actually provided and the other cell requires less reinforcement than actually provided. After V_n and θ have been found in this manner, the corresponding nominal flexural resistance, M_n, can be found by calculating, from Eqs. B5.2-3 through B5.2-5, the moment required to cause this chosen value of ε_x, and calculating, from Eq. 5.7.3.5-1, the moment required to yield the reinforcement. The predicted moment capacity will be the lower of these two values. In using Eqs. 5.7.2.8-1, 5.7.3.5-1, and Eqs. B5.2-3 through B5.2-5 of the procedure to calculate a $V_n - M_n$ interaction diagram, it is appropriate to replace V_u by V_n, M_u by M_n, and N_u by N_n and to take the value of φ as 1.0. With an appropriate spreadsheet, the use of shear-moment interaction diagrams is a convenient way of performing shear design and evaluation.

The values of β and θ listed in Table B5.2-1 and Table B5.2-2 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 B5.2-3. 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.

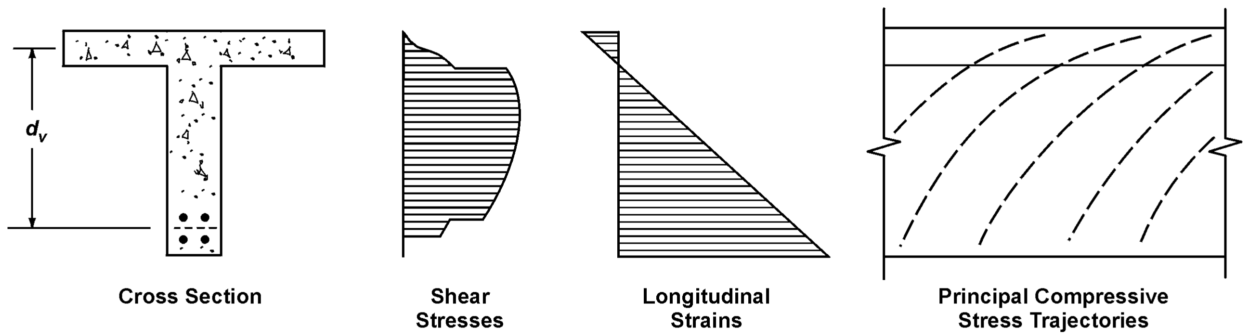

Figure CB5.2-1—Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.7.3.1

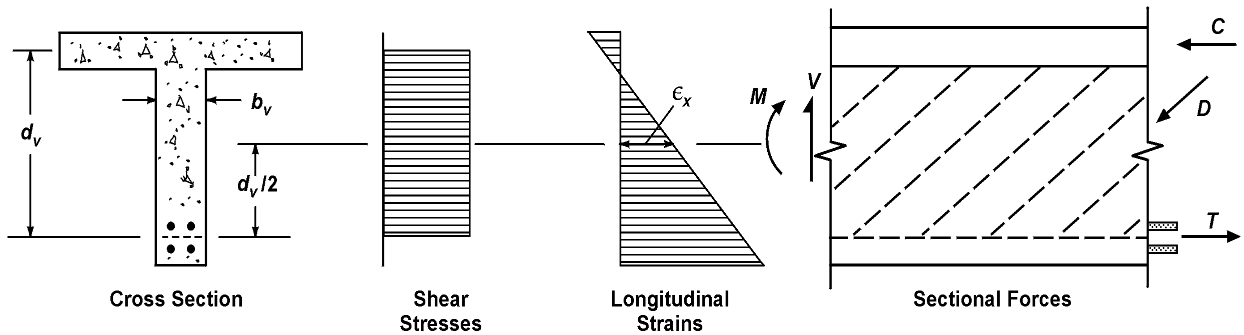

Figure CB5.2-2—More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.7.3.4.2

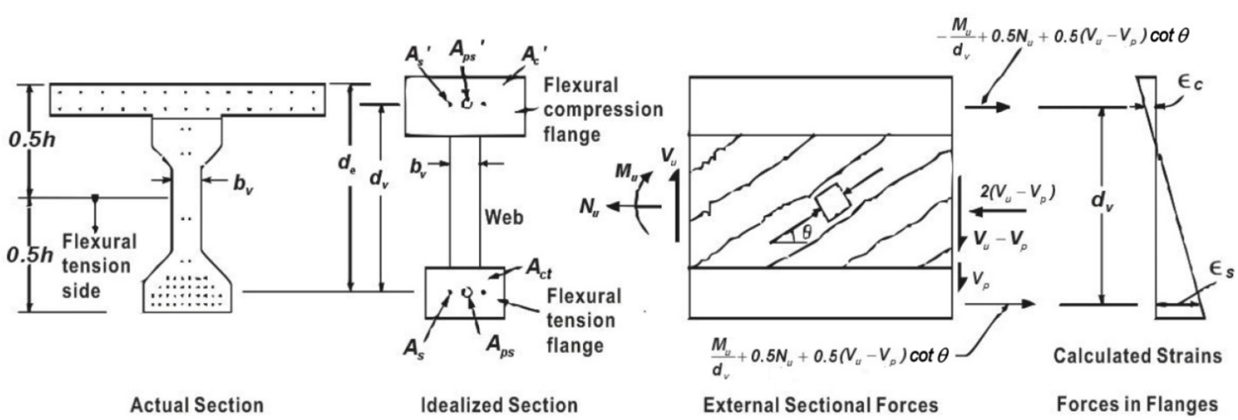

Figure CB5.2-3—More Accurate Calculation Procedure for Determining ε_x

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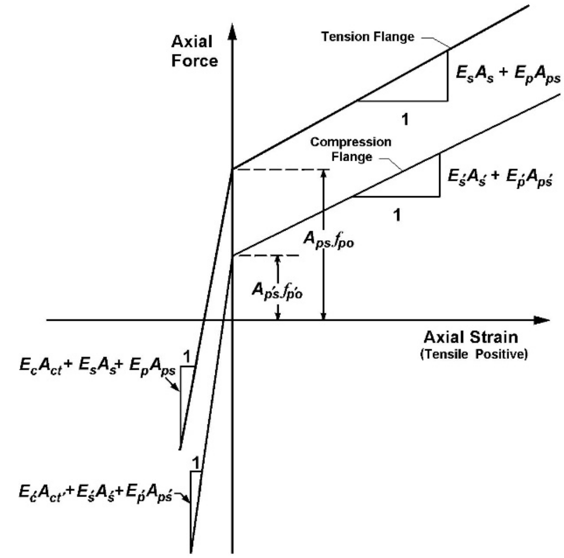

Figure CB5.2-4—Assumed Relationships between Axial Force in Flange and Axial Strain of Flange

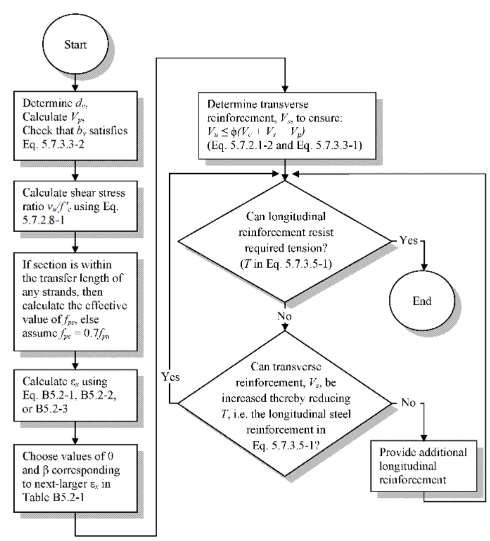

Figure CB5.2-5—Flow Chart for Shear Design of Section Containing at Least Minimum Transverse Reinforcement

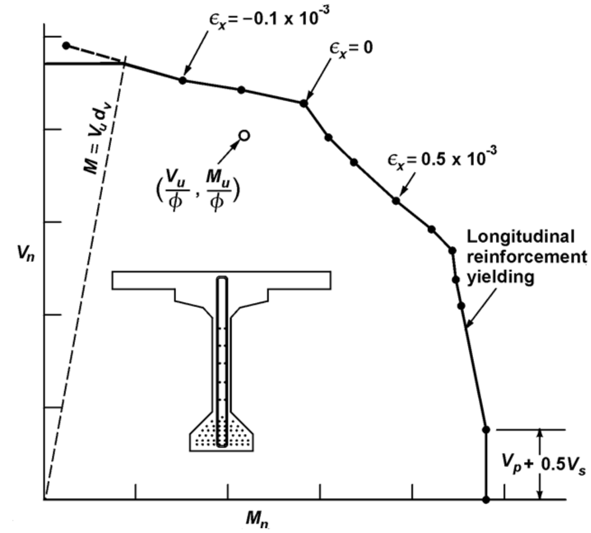

Figure CB5.2-6—Typical Shear-Moment Interaction Diagram

More details on the procedures used in deriving the tabulated values of θ and β are given in Collins and Mitchell (1991).

Table B5.2-1—Values of θ and β for Sections with Transverse Reinforcement

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤–0.20	≤–0.10	≤–0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

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Table B5.2-2—Values of θ and β for Sections with Less than Minimum Transverse Reinforcement

s_{xe}, in.	$\varepsilon_x \times 1{,}000$										
	≤–0.20	≤–0.10	≤–0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00	≤1.50	≤2.00
≤5	25.4 6.36	25.5 6.06	25.9 5.56	26.4 5.15	27.7 4.41	28.9 3.91	30.9 3.26	32.4 2.86	33.7 2.58	35.6 2.21	37.2 1.96
≤10	27.6 5.78	27.6 5.78	28.3 5.38	29.3 4.89	31.6 4.05	33.5 3.52	36.3 2.88	38.4 2.50	40.1 2.23	42.7 1.88	44.7 1.65
≤15	29.5 5.34	29.5 5.34	29.7 5.27	31.1 4.73	34.1 3.82	36.5 3.28	39.9 2.64	42.4 2.26	44.4 2.01	47.4 1.68	49.7 1.46
≤20	31.2 4.99	31.2 4.99	31.2 4.99	32.3 4.61	36.0 3.65	38.8 3.09	42.7 2.46	45.5 2.09	47.6 1.85	50.9 1.52	53.4 1.31
≤30	34.1 4.46	34.1 4.46	34.1 4.46	34.2 4.43	38.9 3.39	42.3 2.82	46.9 2.19	50.1 1.84	52.6 1.60	56.3 1.30	59.0 1.10
≤40	36.6 4.06	36.6 4.06	36.6 4.06	36.6 4.06	41.2 3.20	45.0 2.62	50.2 2.00	53.7 1.66	56.3 1.43	60.2 1.14	63.0 0.95
≤60	40.8 3.50	40.8 3.50	40.8 3.50	40.8 3.50	44.5 2.92	49.2 2.32	55.1 1.72	58.9 1.40	61.8 1.18	65.8 0.92	68.6 0.75
≤80	44.3 3.10	44.3 3.10	44.3 3.10	44.3 3.10	47.1 2.71	52.3 2.11	58.7 1.52	62.8 1.21	65.7 1.01	69.7 0.76	72.4 0.62

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APPENDIX C5—UPPER LIMITS FOR ARTICLES AFFECTED BY CONCRETE COMPRESSIVE STRENGTH

Article[a]	Upper Limit, ksi	
	10.0	15.0[b]
5.1—Scope		By exception
5.4.2.1—Compressive Strength		By exception
5.4.2.3—Creep and Shrinkage		X
5.4.2.4—Modulus of Elasticity		X
5.4.2.5—Poisson's Ratio		X
5.4.2.6—Modulus of Rupture		X
C5.4.2.7—Tensile Strength	X	
5.5.3.1—General	X	
5.5.4.2—Resistance Factors	X	
5.6.2—Assumptions for Strength and Extreme Event Limit States		X
5.6.3—Flexural Members		X
5.6.4.2—Limits for Reinforcement		X
5.6.4.3—Approximate Evaluation of Slenderness Effects	X	
5.6.4.4—Factored Axial Resistance		X
5.6.4.5—Biaxial Flexure		X
5.6.4.6—Spirals and Ties		X
5.6.4.7—Hollow Rectangular Compression Members	X	
5.6.5—Bearing	X	
5.7.2.1—General	X	
5.7.2.2—Transfer and Development Lengths	X	
5.7.2.6—Maximum Spacing of Transverse Reinforcement	X	
5.7.3—Sectional Design Model		X
5.7.4—Interface Shear Transfer—Shear Friction	X	
5.8.2.1—General		X
5.8.2.7—Application of the Strut-and-Tie Method to the Design of General Zone		X
5.8.4.2—Brackets and Corbels	X	
5.8.4.3—Beam Ledges	X	
5.8.4.4.2—Bearing Resistance	X	
5.9.1—General Design Considerations	X	
5.9.2.3.2a—Compressive Stresses		X
5.9.2.3.2b—Tensile Stresses		Partially
5.9.3—Loss of Prestress		X
5.9.4.3.1—General		X
5.9.4.3.2—Bonded Strand		X
5.9.4.3.3— Debonded Strands		X
5.9.5.4.3—Effects of Curved Tendons	X	
5.9.5.6.5a—Design Methods	X	
5.10.4.2—Spirals	X	
5.10.4.3—Ties		X
5.10.6—Shrinkage and Temperature Reinforcement	X	
5.10.8.2.1—Deformed Bars and Deformed Wire in Tension		X
5.10.8.2.2—Deformed Bars in Compression	X	
5.10.8.2.3—Bundled Bars	X	

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Article[a]	Upper Limit, ksi	
	10.0	15.0[b]
5.10.8.2.4—Standard Hooks in Tension		X
5.10.8.2.5—Welded Wire Reinforcement	X	
5.10.8.2.6—Shear Reinforcement	X	
5.10.8.4.3a—Lap Splices in Tension		X
5.10.8.4.5—Splices of Bars in Compression	X	
5.11.3.2 —Seismic Requirements	X	
5.11.4—Seismic Zones 3 and 4	X	
5.11.4.5—Seismic Requirements	X	
5.12.5.3—Design	X	
5.12.5.3.8—Shear and Torsion for Segmental Box Girder Bridges	X	
5.12.7—Additional Provisions for Culverts	X	
5.12.8.6—Shear in Slabs and Footings	X	

Notes:

a Applies to all subarticles of the listed Article
b Normal weight concrete only

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APPENDIX D5—ARTICLES MODIFIED TO ALLOW THE USE OF REINFORCEMENT WITH A SPECIFIED MINIMUM YIELD STRENGTH UP TO 100 KSI

Article	Brief Summary of Changes
5.2—DEFINITIONS	Modified the definition of tension-controlled section by changing "0.005" to "tension-controlled strain limit." Added definition of tension-controlled strain limit.
5.3—NOTATION	Modified the definition of f_y to allow higher yield strengths. Added definitions of ε_{cl} and ε_{tl}; compression- and tension-controlled strain limits, respectively.
5.4.3.1 and C5.4.3.1—General	Permits the use of reinforcement with specified minimum yield strengths up to 100 ksi where allowed by specific articles.
5.4.3.2—Modulus of Elasticity	E_s = 29,000 ksi may be used for specified minimum yield strengths up to 100 ksi.
5.4.3.3 and C5.4.3.3—Special Applications	Permits the use of reinforcement with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone 1.
5.5.3.2 and C5.5.3.2—Reinforcing Bars and Welded Wire Reinforcement	Modifies the fatigue equation for reinforcing bars to allow the equation to be used for specified minimum yield strengths up to 100 ksi.
5.5.4.2 and C5.5.4.2—Resistance Factors	Allows the use of reinforcement with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone 1. Modifies the equation, figure, and commentary for ϕ. These now use ε_{cl} and ε_{tl} (compression- and tension-controlled strain limits) in place of 0.002 and 0.005.
5.6 and C5.6—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS - B REGIONS	Allows the use of reinforcement with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone 1.
5.6.2.1 and C5.6.2.1—General	Keeps compression-controlled strain limit of 0.002 for reinforcement with specified minimum yield strengths up to 60.0 ksi and tension-controlled strain limit of 0.005 for reinforcement with specified minimum yield strengths up to 75.0 ksi. Provides compression- and tension-controlled strain limits of 0.004 and 0.008 for reinforcement with a specified minimum yield strength equal to 100 ksi. Linear interpolation is used for reinforcement with specified minimum yield strengths between 60.0 or 75.0 and 100 ksi. Equations are provided for where f_y may replace f_s or f_s' in Article 5.6.3.1 and Article 5.6.3.2.

Article	Brief Summary of Changes
5.6.3.2.5—Strain Compatibility Approach	Limits the steel stress in a strain compatibility calculation to the specified minimum yield strength.
5.6.3.4 and C5.6.3.4—Moment Redistribution	Adjusts strain limit to allow moment redistribution in structures using reinforcement with specified minimum yield strengths up to 100 ksi.
C5.6.4.4—Factored Axial Resistance	Warns that designs should consider that columns using higher strength reinforcing steel may be smaller and have lower axial stiffness.
5.6.4.6—Spirals, hoops and Ties	Permits spirals and ties made of reinforcement with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone 1.
5.6.7 and C5.6.7—Control of Cracking by Distribution of Reinforcement	Limits f_{ss} to 0.6 f_y (ksi) in Eq. 5.6.7-1. Only Class 1 requirement needs to be satisfied with greater corrosion resistant reinforcement.
5.7.2.4 and C5.7.2.4—Types of Transverse Reinforcement 5.7.2.5 and C5.7.2.5—Minimum Transverse Reinforcement	Permits transverse reinforcement with specified minimum yield strengths up to 100 ksi in applications with flexural shear without torsion.
C5.7.2.6—Maximum Spacing of Transverse Reinforcement	Indicates that spacing requirements have been verified for transverse reinforcement with specified minimum yield strengths up to 100 ksi in applications of shear without torsion.
5.7.2.7 and C5.7.2.7—Design and Detailing Requirements.	Permits transverse reinforcement with specified minimum yield strengths up to 100 ksi in applications with flexural shear without torsion.
C5.7.3.3—Nominal Shear Resistance	Identifies that transverse reinforcement with specified minimum yield strengths up to 100 ksi may be used in applications with flexural shear without torsion.
5.7.3.5 and C5.7.3.5—Longitudinal Reinforcement	Permits longitudinal reinforcing steel with specified minimum yield strengths up to 100 ksi.
5.7.4.2—Minimum Area of Interface Reinforcement	Clarifies that f_y is limited to 60.0 ksi in Eq. 5.7.4.2-1.
5.10.2 and C5.10.2 Hooks and Bends	Permits hooks with specified minimum yield strengths up to 100 ksi with transverse confining steel for elements in Seismic Zone 1.
5.10.4.1 and C5.10.4.1—Transverse Reinforcement for Compression Members, General	Permits spirals with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone1.
5.10.8.1.1—Basic Requirements 5.10.8.2 —Development of Reinforcement	Permits the development length equations to be used for reinforcement with specified minimum yield strengths up to 100 ksi.

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Article	Brief Summary of Changes
5.10.8.2.1 Deformed Bars and Deformed Wire in Tension	Requires transverse confining steel for development of reinforcement with specified minimum yield strengths greater than 75.0 ksi.
5.10.8.2.4—Standard Hooks in Tension	Requires the use of modification factors or ties for hooks of reinforcement with specified minimum yield strengths exceeding 75.0 ksi.
5.10.8.4 and add C5.10.8.4—Splices of Bar Reinforcement	Permits splices in reinforcement with specified minimum yield strengths up to 100 ksi and requires transverse confining steel.
5.10.8.4.3a and C5.10.8.4.3a—Lap Splices in Tension	Requires transverse confining steel in splices of reinforcement with specified minimum yield strengths exceeding 75.0 ksi.
5.11.1—General	Permits the use of reinforcement with specified minimum yield strengths up to 100 ksi for elements in Seismic Zone 1.

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APPENDIX E5—CROSSWALK BETWEEN 7TH AND 8TH EDITIONS

7th Ed. With 2016 Interim Revisions		8th Ed.		Modifications to the 8th Ed.				
Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.1	**Scope**	**5.1**	**Scope**			✓		
5.2	**Definitions**	**5.2**	**Definitions**			✓		
5.3	**Notation**	**5.3**	**Notation**			✓		
5.4	**Material Properties**	**5.4**	**Material Properties**	✓				
5.4.1	General	5.4.1	General	✓				
5.4.2	Normal Weight and Structural Lightweight Concrete	5.4.2	Normal Weight and Lightweight Concrete	✓				
5.4.2.1	Compressive Strength	5.4.2.1	Compressive Strength			✓		
5.4.2.2	Coefficient of Thermal Expansion	5.4.2.2	Coefficient of Thermal Expansion	✓				
5.4.2.3	Shrinkage and Creep	5.4.2.3	Creep and Shrinkage		✓			
5.4.2.3.1	General	5.4.2.3.1	General		✓			
5.4.2.3.2	Creep	5.4.2.3.2	Creep	✓				
5.4.2.3.2-1	$\psi(t,t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118}$	5.4.2.3.2-1	$\psi(t,t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118}$	✓				
5.4.2.3.2-2	$k_s = 1.45 - 0.13(V/S) \geq 1.0$	5.4.2.3.2-2	$k_s = 1.45 - 0.13(V/S) \geq 1.0$	✓				
5.4.2.3.2-3	$k_{hc} = 1.56 - 0.008H$	5.4.2.3.2-3	$k_{hc} = 1.56 - 0.008H$	✓				
5.4.2.3.2-4	$k_f = \dfrac{5}{1+f'_{ci}}$	5.4.2.3.2-4	$k_f = \dfrac{5}{1+f'_{ci}}$					
5.4.2.3.2-5	$k_{td} = \dfrac{t}{12\left(\dfrac{100-4f'_{ci}}{f'_{ci}+20}\right)+t}$	5.4.2.3.2-5	$k_{td} = \dfrac{t}{12\left(\dfrac{100-4f'_{ci}}{f'_{ci}+20}\right)+t}$					
C5.4.2.3.2-1	$k_c = \left[\dfrac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\dfrac{1.80+1.77e^{-0.54(V/S)}}{2.587}\right]$	C5.4.2.3.2-1	$k_c = \left[\dfrac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\dfrac{1.80+1.77e^{-0.54(V/S)}}{2.587}\right]$	✓				
C5.4.2.3.2-2	$k_s = \left[\dfrac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\dfrac{1064-94(V/S)}{923}\right]$	C5.4.2.3.2-2	$k_s = \left[\dfrac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\dfrac{1064-94(V/S)}{923}\right]$	✓				
5.4.2.3.3	Shrinkage	5.4.2.3.3	Shrinkage	✓				

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7th Ed. With 2016 Interim Revisions		8th Ed.		Modifications to the 8th Ed.				
Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.4.2.3.3-1	$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$	5.4.2.3.3-1	$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$	✓				
5.4.2.3.3-2	$k_{hs} = (2.00 - 0.014H)$	5.4.2.3.3-2	$k_{hs} = (2.00 - 0.014H)$	✓				
5.4.2.4	Modulus of Elasticity	5.4.2.4	Modulus of Elasticity		✓			
5.4.2.4-1	$E_c = 120{,}000\ K_1\ w_c^{2.0}\ f_c'^{0.33}$	5.4.2.4-1	$E_c = 120{,}000\ K_1\ w_c^{2.0}\ f_c'^{0.33}$	✓				
C5.4.2.4-1	$E_c = 2{,}500 f_c'^{0.33}$	C5.4.2.4-1	$E_c = 2{,}500 f_c'^{0.33}$	✓				
C5.4.2.4-2	$E_c = 33{,}000\ K_1\ w_c^{1.5} \sqrt{f_c'}$	C5.4.2.4-2	$E_c = 33{,}000\ K_1\ w_c^{1.5} \sqrt{f_c'}$	✓				
C5.4.2.4-3	$E_c = 1{,}820 \sqrt{f_c'}$	C5.4.2.4-3	$E_c = 1{,}820 \sqrt{f_c'}$	✓				
5.4.2.5	Poisson's Ratio	5.4.2.5	Poisson's Ratio		✓			
5.4.2.6	Modulus of Rupture	5.4.2.6	Modulus of Rupture		✓			
5.4.2.7	Tensile Strength	5.4.2.7	Tensile Strength		✓			
5.4.2.8	Concrete Density Modification Factors	5.4.2.8	Concrete Density Modification Factors		✓			
5.4.2.8-1	$\lambda = 4.7 f_{ct} / \sqrt{f'_c} \leq 1.0$	5.4.2.8-1	$\lambda = 4.7 f_{ct} / \sqrt{f'_c} \leq 1.0$	✓				
5.4.2.8-2	$0.75 \leq \lambda = 7.5\ w_c \leq 1.0$	5.4.2.8-2	$0.75 \leq \lambda = 7.5\ w_c \leq 1.0$	✓				
5.4.3	Reinforcing Steel	5.4.3	Reinforcing Steel		✓			
5.4.3.1	General	5.4.3.1	General		✓			
5.4.3.2	Modulus of Elasticity	5.4.3.2	Modulus of Elasticity	✓				
5.4.3.3	Special Applications	5.4.3.3	Special Applications		✓			
5.4.4	Prestressing Steel	5.4.4	Prestressing Steel	✓				
5.4.4.1	General	5.4.4.1	General		✓			
5.4.4.2	Modulus of Elasticity	5.4.4.2	Modulus of Elasticity	✓				
5.4.5	Post-Tensioning Anchorages and Couplers	5.4.5	Post-Tensioning Anchorages and Couplers	✓				
5.4.6	Ducts	5.4.6	Post-Tensioning Ducts		✓			
5.4.6.1	General	5.4.6.1	General			✓		
5.4.6.2	Size of Ducts	5.4.6.2	Size of Ducts			✓		
5.4.6.3	Ducts at Deviation Saddles	5.4.6.3	Ducts at Deviation Saddles	✓				

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7th Ed. With 2016 Interim Revisions		8th Ed.		Modifications to the 8th Ed.				
Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.5	**Limit States**	**5.5**	**Limit States and Design Methodologies**		✓			
5.5.1	General	5.5.1	General	✓				
5.5.1.1	Limit-State Applicability	5.5.1.1	Limit-State Applicability		✓			
5.5.1.2	Design Methodologies	5.5.1.2	Design Methodologies	✓				
5.5.1.2.1	General	5.5.1.2.1	General			✓		
5.5.1.2.2	B-Regions	5.5.1.2.2	B-Regions		✓			
5.5.1.2.3	D-Regions	5.5.1.2.3	D-Regions		✓			
5.5.2	Service Limit State	5.5.2	Service Limit State		✓			
5.5.3	Fatigue Limit State	5.5.3	Fatigue Limit State	✓				
5.5.3.1	General	5.5.3.1	General		✓			
5.5.3.1-1	$\gamma(\Delta f) \le (\Delta F)_{TH}$	5.5.3.1-1	$\gamma(\Delta f) \le (\Delta F)_{TH}$	✓				
5.5.3.2	Reinforcing Bars	5.5.3.2	Reinforcing Bars and Welded Wire Fabric		✓			
5.5.3.2-1	$(\Delta F)_{TH} = 24 - 20 f_{min} / f_y$	5.5.3.2-1	$(\Delta F)_{TH} = 26 - 22 f_{min} / f_y$			✓		
5.5.3.2-2	$(\Delta F)_{TH} = 16 - 0.33 f_{\min}$	5.5.3.2-2	$(\Delta F)_{TH} = 18 - 0.36 f_{\min}$			✓		
5.5.3.3	Prestressing Tendons	5.5.3.3	Prestressing Steel		✓			
5.5.3.4	Welded or Mechanical Splices of Reinforcement	5.5.3.4	Welded or Mechanical Splices of Reinforcement		✓			
5.5.4	Strength Limit State	5.5.4	Strength Limit State	✓				
5.5.4.1	General	5.5.4.1	General	✓				
5.5.4.2	Resistance Factors	5.5.4.2	Resistance Factors			✓		
5.5.4.2.1	Conventional Construction	5.5.4.2	Resistance Factors			✓		
5.5.4.2.1-1	$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0$	5.5.4.2-1	$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0$	✓				
5.5.4.2.1-2	$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9$	5.5.4.2-2	$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9$	✓				
5.5.4.2.2	Segmental Construction	5.5.4.2	Resistance Factors			✓		
5.5.4.2.3	Special Requirements for Seismic Zones 2, 3, and 4	5.5.5.2	Special Requirements for Seismic Zones 2, 3, and 4		✓			

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7th Ed. With 2016 Interim Revisions		8th Ed.		Modifications to the 8th Ed.				
Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.5.4.3	Stability	5.5.4.3	Stability	✓				
5.5.5	Extreme Event Limit State	5.5.5	Extreme Event Limit State	✓				
5.5.5	Extreme Event Limit State	5.5.5.1	General		✓			
5.6	**Design Considerations**	**5.8**	**Design of D-Regions**				✓	
5.6.1	General	5.5.1.1	Limit-State Applicability			✓		
5.6.2	Effects of Imposed Deformations	5.5.1.1	Limit-State Applicability		✓			
N/A		5.8.1	General				✓	
5.6.3	Strut-and-Tie Method	5.8.2	Strut-and-Tie Method (STM)		✓			
5.6.3.1	General	5.8.2.1	General		✓			
5.6.3.2	Structural Modeling	5.8.2.2	Structural Modeling		✓			
C5.6.3.2-1	$V_{cr} = \left[0.2 - 0.1\left(\frac{a}{d}\right)\right]\sqrt{f'_c}b_w d$	C5.8.2.2-1	$V_{cr} = \left[0.2 - 0.1\left(\frac{a}{d}\right)\right]\sqrt{f'_c}b_w d$	✓				
5.6.3.3	Factored Resistance	5.8.2.3	Factored Resistance	✓				
5.6.3.3-1	$P_r = \phi P_n$	5.8.2.3-1	$P_r = \phi P_n$	✓				
5.6.3.4	Proportioning of Ties	5.8.2.4	Proportioning of Ties	✓				
5.6.3.4.1	Strength of Tie	5.8.2.4.1	Strength of Tie		✓			
5.6.3.4.1-1	$P_n = f_y A_{st} + A_{ps}\left(f_{pe} + f_y\right)$	5.8.2.4.1-1	$P_n = f_y A_{st} + A_{ps}\left[f_{pe} + f_y\right]$		✓			
5.6.3.4.2	Anchorage of Tie	5.8.2.4.2	Anchorage of Tie		✓			
5.6.3.5	Proportioning of Node Regions	5.8.2.5	Proportioning of Node Regions	✓				
5.6.3.5.1	Strength of the Node Face	5.8.2.5	Proportioning of Node Regions		✓			
5.6.3.5.1-1	$P_n = f_{cu} A_{cn}$	5.8.2.5.1-1	$P_n = f_{cu} A_{cn}$	✓				
5.6.3.5.2	Effective Cross-Sectional Area of Node Face	5.8.2.5.2	Effective Cross-Sectional Area of Node Face		✓			
5.6.3.5.3	Limiting Compressive Stress at the Node Face	5.8.2.5.3	Limiting Compressive Stress at the Node Face	✓				
5.6.3.5.3a	General	5.8.2.5.3a	General		✓			
5.6.3.5.3a-1	$f_{cu} = m\nu f'_c$	5.8.2.5.3a-1	$f_{cu} = m\nu f'_c$	✓				
5.6.3.5.3b	Back Face of a CCT Node	5.8.2.5.3b	Back Face of a CCT Node		✓			

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7th Ed. With 2016 Interim Revisions		8th Ed.		Modifications to the 8th Ed.				
Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.6.3.6	Crack Control Reinforcement	5.8.2.6	Crack Control Reinforcement		✓			
5.6.3.6-1	$\frac{A_v}{b_w s_v} \geq 0.003$	5.8.2.6-1	$\frac{A_v}{b_w s_v} \geq 0.003$	✓				
5.6.3.6-2	$\frac{A_h}{b_w s_h} \geq 0.003$	5.8.2.6-2	$\frac{A_h}{b_w s_h} \geq 0.003$	✓				
5.7	**Design for Flexural and Axial Force Effects**	**5.6**	**Design for Flexural and Axial Force Effects – B Regions**		✓			
5.7.1	Assumptions for Service and Fatigue Limit States	5.6.1	Assumptions for Service and Fatigue Limit States		✓			
5.7.2	Assumptions for Strength and Extreme Event Limit States	5.6.2	Assumptions for Strength and Extreme Event Limit States	✓				
5.7.2.1	General	5.6.2.1	General			✓		
5.7.2.1-1	$\frac{c}{d_s} \leq \frac{0.003}{0.003 + \varepsilon_{cl}}$	5.6.2.1-1	$\frac{c}{d_s} \leq \frac{0.003}{0.003 + \varepsilon_{cl}}$	✓				
5.7.2.2	Rectangular Stress Distribution	5.6.2.2	Rectangular Stress Distribution		✓			
5.7.3	Flexural Members	5.6.3	Flexural Members			✓		
5.7.3.1	Stress in Prestressing Steel at Nominal Flexural Resistance	5.6.3.1	Stress in Prestressing Steel at Nominal Flexural Resistance	✓				
5.7.3.1.1	Components with Bonded Tendons	5.6.3.1.1	Components with Bonded Tendons	✓				
5.7.3.1.1-1	$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$	5.6.3.1.1-1	$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$	✓				
5.7.3.1.1-2	$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$	5.6.3.1.1-2	$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$	✓				
5.7.3.1.1-3	$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \frac{f_{pu}}{d_p}}$	5.6.3.1.1-3	$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \frac{f_{pu}}{d_p}}$	✓				
5.7.3.1.1-4	$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$	5.6.3.1.1-4	$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$	✓				

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Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.7.3.1.2	Components with Unbonded Tendons	5.6.3.1.2	Components with Unbonded Tendons	✓				
5.7.3.1.2-1	$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py}$	5.6.3.1.2-1	$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py}$	✓				
5.7.3.1.2-2	$\ell_e = \left(\frac{2\ell_i}{2 + N_s}\right)$	5.6.3.1.2-2	$\ell_e = \left(\frac{2\ell_i}{2 + N_s}\right)$	✓				
5.7.3.1.2-3	$c = \frac{A_{ps}f_{ps} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w}$	5.6.3.1.2-3	$c = \frac{A_{ps}f_{ps} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w}$	✓				
5.7.3.1.2-4	$c = \frac{A_{ps}f_{ps} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b}$	5.6.3.1.2-4	$c = \frac{A_{ps}f_{ps} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b}$	✓				
C5.7.3.1.2-1	$f_{ps} = f_{pe} + 15.0$ (ksi)	C5.6.3.1.2-1	$f_{ps} = f_{pe} + 15.0$ (ksi)	✓				
5.7.3.1.3	Components with Both Bonded and Unbonded Tendons	5.6.3.1.3	Components with Both Bonded and Unbonded Tendons	✓				
5.7.3.1.3a	Detailed Analysis	5.6.3.1.3a	Detailed Analysis	✓				
5.7.3.1.3b	Simplified Analysis	5.6.3.1.3b	Simplified Analysis	✓				
5.7.3.2	Flexural Resistance	5.6.3.2	Flexural Resistance	✓				
5.7.3.2.1	Factored Flexural Resistance	5.6.3.2.1	Factored Flexural Resistance	✓				
5.7.3.2.1-1	$M_r = \phi M_n$	5.6.3.2.1-1	$M_r = \phi M_n$	✓				
5.7.3.2.2	Flanged Sections	5.6.3.2.2	Flanged Sections	✓				
5.7.3.2.2-1	$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c (b - b_w) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$	5.6.3.2.2-1	$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c (b - b_w) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$	✓				
5.7.3.2.3	Rectangular Sections	5.6.3.2.3	Rectangular Sections	✓				
5.7.3.2.4	Other Cross-Sections	5.6.3.2.4	Other Cross-Sections	✓				
5.7.3.2.5	Strain Compatibility Approach	5.6.3.2.5	Strain Compatibility Approach		✓			
5.7.3.2.6	Composite Girder Section	5.6.3.2.6	Composite Girder Section	✓				
5.7.3.3	Limits for Reinforcement	N/A						

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5.7.3.3.1	Maximum Reinforcement--- deleted 2005	N/A						
5.7.3.3.2	Minimum Reinforcement	5.6.3.3	Minimum Reinforcement			✓		
5.7.3.3.2-1	$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$	5.6.3.3-1	$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$	✓				
5.7.3.4	Control Cracking by Distribution of Reinforcement	5.6.7	Control Cracking by Distribution of Reinforcement			✓		
5.7.3.4-1	$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$	5.6.7-1	$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$	✓				
N/A	$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$	5.6.7-2	$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$	✓				
5.7.3.4-2	$A_{sk} \geq 0.012\ (d_l - 30)$	5.6.7-3	$A_{sk} \geq 0.012\ (d_l - 30)$					
5.7.3.5	Moment Redistribution	5.6.3.4	Moment Redistribution	✓				
5.7.3.6	Deformations	5.6.3.5	Deformations	✓				
5.7.3.6.1	General	5.6.3.5.1	General	✓				
5.7.3.6.2	Deflection and Camber	5.6.3.5.2	Deflection and Camber		✓			
5.7.3.6.2-1	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$	5.6.3.5.2-1	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$	✓				
5.7.3.6.2-2	$M_{cr} = f_r \frac{I_g}{y_t}$	5.6.3.5.2-2	$M_{cr} = f_r \frac{I_g}{y_t}$	✓				
5.7.3.6.3	Axial Deformation	5.6.3.5.3	Axial Deformation	✓				
5.7.4	Compression Members	5.6.4	Compression Members	✓				
5.7.4.1	General	5.6.4.1	General	✓				
5.7.4.2	Limits for Reinforcement	5.6.4.2	Limits for Reinforcement		✓			
5.7.4.2-1	$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08$	5.6.4.2-1	$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08$	✓				
5.7.4.2-2	$\frac{A_{ps} f_{pe}}{A_g f'_c} \leq 0.30$	5.6.4.2-2	$\frac{A_{ps} f_{pe}}{A_g f'_c} \leq 0.30$	✓				

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5.7.4.2-3	$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \geq 0.135 \frac{f'_c}{f_y}$	5.6.4.2-3	$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \geq 0.135 \frac{f'_c}{f_y}$	✓				
N/A	but not greater than 0.015	5.6.4.2-4	$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \geq 0.015$		✓			
5.7.4.3	Approximate Evaluation of Slenderness Effects	5.6.4.3	Approximate Evaluation of Slenderness Effects			✓		
5.7.4.3-1	$EI = \frac{\frac{E_c I_g}{5} + E_s I_s}{1 + \beta_d}$	5.6.4.3-1	$EI = \frac{\frac{E_c I_g}{5} + E_s I_s}{1 + \beta_d}$	✓				
5.7.4.3-2	$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d}$	5.6.4.3-2	$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d}$	✓				
5.7.4.4	Factored Axial Resistance	5.6.4.4	Factored Axial Resistance			✓		
5.7.4.4-1	$P_r = \phi P_n$	5.6.4.4-1	$P_r = \phi P_n$	✓				
5.7.4.4-2	$P_n = 0.85 \begin{bmatrix} k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st} \\ -A_{ps} (f_{pe} - E_p \varepsilon_{cu}) \end{bmatrix}$	5.6.4.4-2	$P_n = 0.85 \begin{bmatrix} k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st} \\ -A_{ps} (f_{pe} - E_p \varepsilon_{cu}) \end{bmatrix}$	✓				
5.7.4.4-3	$P_n = 0.80 \begin{bmatrix} k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st} \\ -A_{ps} (f_{pe} - E_p \varepsilon_{cu}) \end{bmatrix}$	5.6.4.4-3	$P_n = 0.80 \begin{bmatrix} k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st} \\ -A_{ps} (f_{pe} - E_p \varepsilon_{cu}) \end{bmatrix}$	✓				
5.7.4.5	Biaxial Flexure	5.6.4.5	Biaxial Flexure	✓				
5.7.4.5-1	$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{\phi P_o}$	5.6.4.5-1	$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{\phi P_o}$	✓				
5.7.4.5-2	$P_o = k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st}$ $-A_{ps} (f_{pe} - E_p \varepsilon_{cu})$	5.6.4.5-2	$P_o = k_c f'_c (A_g - A_{st} - A_{ps}) + f_y A_{st}$ $-A_{ps} (f_{pe} - E_p \varepsilon_{cu})$	✓				
5.7.4.5-3	$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0$	5.6.4.5-3	$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0$	✓				
5.7.4.6	Spirals and Ties	5.6.4.6	Spirals, Hoops and Ties		✓			

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5.7.4.6-1	$\rho_s \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f'_c}{f_{yh}}$	5.6.4.6-1	$\rho_s = \frac{4A_{sp}}{(d_c s)} \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f'_c}{f_{yh}}$			✓		
5.7.4.7	Hollow Rectangular Compression Members	5.6.4.7	Hollow Rectangular Compression Members	✓				
5.7.4.7.1	Wall Slenderness Ratio	5.6.4.7.1	Wall Slenderness Ratio	✓				
5.7.4.7.1-1	$\lambda_w = \frac{X_u}{t}$	5.6.4.7.1-1	$\lambda_w = \frac{X_u}{t}$	✓				
5.7.4.7.2	Limitations on the Use of the Rectangular Stress Block Method	5.6.4.7.2	Limitations on the Use of the Rectangular Stress Block Method	✓				
5.7.4.7.2a	General	5.6.4.7.2a	General	✓				
5.7.4.7.2b	Refined Method for Adjusting Maximum Usable Strain Limit	5.6.4.7.2b	Refined Method for Adjusting Maximum Usable Strain Limit	✓				
5.7.4.7.2c	Approximate Method for Adjusting Factored Resistance	5.6.4.7.2c	Approximate Method for Adjusting Factored Resistance	✓				
5.7.4.7.2c-1	$\lambda_w \leq 15, \text{ then } \phi_w = 1.0$	5.6.4.7.2c-1	$\lambda_w \leq 15, \text{ then } \phi_w = 1.0$	✓				
5.7.4.7.2c-2	$15 < \lambda_w \leq 25, \text{ then } \phi_w = 1-0.025(\lambda_w - 15)$	5.6.4.7.2c-2	$15 < \lambda_w \leq 25, \text{ then } \phi_w = 1-0.025(\lambda_w - 15)$	✓				
5.7.4.7.2c-3	$25 < \lambda_w \leq 35, \text{ then } \phi_w = 0.75$	5.6.4.7.2c-3	$25 < \lambda_w \leq 35, \text{ then } \phi_w = 0.75$	✓				
5.7.5	Bearing	5.6.5	Bearing			✓		
5.7.5-1	$P_r = \phi P_n$	5.6.5-1	$P_r = \phi P_n$	✓				
5.7.5-2	$P_n = 0.85 f'_c A_1 m$	5.6.5-2	$P_n = 0.85 f'_c A_1 m$	✓				
5.7.5-3	$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0$	5.6.5-3	$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0$	✓				
5.7.5-4	$m = 0.75\sqrt{\frac{A_2}{A_1}} \leq 1.50$	5.6.5-4	$m = 0.75\sqrt{\frac{A_2}{A_1}} \leq 1.50$	✓				
5.7.6	Tension Members	5.6.6	Tension Members	✓				
5.7.6.1	Factored Tension Resistance	5.6.6.1	Resistance to Tension		✓			
5.7.6.1-1	$P_r = \phi P_n$	5.6.6.1-1	$P_r = \phi P_n$	✓				
5.7.6.2	Resistance to Combinations of Tension and Flexure	5.6.6.2	Resistance to Combined Tension and Flexure		✓			

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5.8	**Shear and Torsion**	**5.7**	**Design for Shear and Torsion – B Regions**		✓			
5.8.1	Design Procedures	5.7.1	Design Procedures	✓				
5.8.1.1	Flexural Regions	5.7.1.1	Flexural Regions		✓			
5.8.1.2	Regions Near Discontinuities	5.7.1.2	Regions Near Discontinuities		✓			
5.8.1.3	Interface Regions	5.7.1.3	Interface Regions	✓				
5.8.1.4	Slabs and Footings	5.7.1.4	Slabs and Footings	✓				
5.8.1.5	Webs of Curved Post-Tensioned Box Girder Bridges	5.7.1.5	Webs of Curved Post-Tensioned Box Girder Bridges			✓		
5.8.2	General Requirements	5.7.2	General Requirements	✓				
5.8.2.1	General	5.7.2.1	General			✓		
5.8.2.1-1	$T_r = \phi T_n$	5.7.2.1-2	$T_r = \phi T_n$	✓				
5.8.2.1-2	$V_r = \phi V_n$	5.7.2.1-1	$V_r = \phi V_n$	✓				
5.8.2.1-3	$T_u > 0.25\phi T_{cr}$	5.7.2.1-3	$T_u > 0.25\phi T_{cr}$	✓				
5.8.2.1-4	$T_{cr} = 0.125\lambda\sqrt{f'_c}\,\frac{A_{cp}^{\,2}}{p_c}\sqrt{1+\frac{f_{pc}}{0.125\lambda\sqrt{f'_c}}}$	5.7.2.1-4	$T_{cr} = 0.126K\lambda\sqrt{f'_c}\,\frac{A_{cp}^{\,2}}{p_c}$		✓			
5.8.2.1-5	$\frac{A_{cp}^{\,2}}{p_c} \leq 2A_o b_v$	N/A						✓
5.8.2.1-6	$\sqrt{V_u^2 + \left(\frac{0.9\,p_h T_u}{2A_o}\right)^2}$	5.7.3.4.2-5	$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9\,p_h T_u}{2A_o}\right)^2}$			✓		
5.8.2.1-7	$V_u + \frac{T_u d_s}{2A_o}$	5.7.3.4.2-6	$V_{eff} = V_u + \frac{T_u d_s}{2A_o}$			✓		
5.8.2.2	Modifications for Lightweight Concrete	5.4.2.8	Concrete Density Modification Factor					✓
5.8.2.3	Transfer and Development Lengths	5.7.2.2	Transfer and Development Lengths			✓		
5.8.2.4	Regions Requiring Transverse Reinforcement	5.7.2.3	Regions Requiring Transverse Reinforcement			✓		
5.8.2.4-1	$V_u > 0.5\phi\left(V_c + V_p\right)$	5.7.2.3-1	$V_u > 0.5\phi\left(V_c + V_p\right)$	✓				
5.8.2.5	Minimum Transverse Reinforcement	5.7.2.5	Minimum Transverse Reinforcement			✓		

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5.8.2.5-1	$A_v \geq 0.0316\,\lambda\sqrt{f'_c}\,\dfrac{b_v s}{f_y}$	5.7.2.5-1	$A_v \geq 0.0316\,\lambda\sqrt{f'_c}\,\dfrac{b_v s}{f_y}$	✓				
5.8.2.6	Types of Transverse Reinforcement	5.7.2.4	Types of Transverse Reinforcement			✓		
5.8.2.7	Maximum Spacing of Transverse Reinforcement	5.7.2.6	Maximum Spacing of Transverse Reinforcement		✓			
5.8.2.7-1	$s_{max} = 0.8 d_v \leq 24.0$	5.7.2.6-1	$s_{max} = 0.8 d_v \leq 24.0$	✓				
5.8.2.7-2	$s_{max} = 0.4 d_v \leq 12.0$	5.7.2.6-2	$s_{max} = 0.4 d_v \leq 12.0$	✓				
5.8.2.8	Design and Detailing Requirements	5.7.2.7	Design and Detailing Requirements			✓		
5.8.2.9	Shear Stress on Concrete	5.7.2.8	Shear Stress on Concrete			✓		
5.8.2.9-1	$v_u = \dfrac{\lvert V_u - \phi V_p \rvert}{\phi b_v d_v}$	5.7.2.8-1	$v_u = \dfrac{\lvert V_u - \phi V_p \rvert}{\phi b_v d_v}$	✓				
5.8.2.9-2	$d_e = \dfrac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$	5.7.2.8-2	$d_e = \dfrac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$	✓				
C5.8.2.9-1	$d_v = \dfrac{M_n}{A_s f_y + A_{ps} f_{ps}}$	C5.7.2.8-1	$d_v = \dfrac{M_n}{A_s f_y + A_{ps} f_{ps}}$	✓				
C5.8.2.9-2	$d_e = \dfrac{D}{2} + \dfrac{D_r}{\pi}$	C5.7.2.8-2	$d_e = \dfrac{D}{2} + \dfrac{D_r}{\pi}$	✓				
5.8.3	Sectional Design Model	5.7.3	Sectional Design Model	✓				
5.8.3.1	General	5.7.3.1	General			✓		
5.8.3.2	Sections Near Supports	5.7.3.2	Sections Near Supports			✓		
5.8.3.3	Nominal Shear Resistance	5.7.3.3	Nominal Shear Resistance			✓		
5.8.3.3-1	$V_n = V_c + V_s + V_p$	5.7.3.3-1	$V_n = V_c + V_s + V_p$	✓				
5.8.3.3-2	$V_n = 0.25 f'_c b_v d_v + V_p$	5.7.3.3-2	$V_n = 0.25 f'_c b_v d_v + V_p$	✓				
5.8.3.3-3	$V_c = 0.0316\,\beta\lambda\sqrt{f'_c}\; b_v d_v$	5.7.3.3-3	$V_c = 0.0316\,\beta\lambda\sqrt{f'_c}\; b_v d_v$	✓				
5.8.3.3-4	$V_s = \dfrac{A_v f_y d_v\,(\cot\theta + \cot\alpha)\sin\alpha}{s}$	5.7.3.3-4	$V_s = \dfrac{A_v f_y d_v\,(\cot\theta + \cot\alpha)\sin\alpha}{s}$	✓				

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5.8.3.3-5	$V_s = A_v f_y \sin\alpha \le 0.095\lambda\sqrt{f'_c}\, b_v d_v$	5.7.3.3-5	$V_s = A_v f_y \sin\alpha \le 0.095\lambda\sqrt{f'_c}\, b_v d_v$	✓				
C5.8.3.3-1	$V_s = \dfrac{A_v f_y d_v \cot\theta}{s}$	C5.7.3.3-1	$V_s = \dfrac{A_v f_y d_v \cot\theta}{s}$	✓				
5.8.3.4	Procedures for Determining Shear Resistance	5.7.3.4	Procedures for Determining Shear Resistance Parameters β and θ			✓		
5.8.3.4.1	Simplified Procedure for Nonprestressed Sections	5.7.3.4.1	Simplified Procedure for Nonprestressed Sections	✓				
5.8.3.4.2	General Procedure	5.7.3.4.2	General Procedure			✓		
5.8.3.4.2-1	$\beta = \dfrac{4.8}{(1+750\varepsilon_s)}$	5.7.3.4.2-1	$\beta = \dfrac{4.8}{(1+750\varepsilon_s)}$	✓				
5.8.3.4.2-2	$\beta = \dfrac{4.8}{(1+750\varepsilon_s)}\dfrac{51}{(39+s_{xe})}$	5.7.3.4.2-2	$\beta = \dfrac{4.8}{(1+750\varepsilon_s)}\dfrac{51}{(39+s_{xe})}$	✓				
5.8.3.4.2-3	$\theta = 29 + 3500\varepsilon_s$	5.7.3.4.2-3	$\theta = 29 + 3500\varepsilon_s$	✓				
5.8.3.4.2-4	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u\rvert}{d_v} + 0.5N_u + \lvert V_u - V_p\rvert - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$	5.7.3.4.2-4	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u\rvert}{d_v} + 0.5N_u + \lvert V_u - V_p\rvert - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$	✓				
5.8.3.4.2-5	$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63}$	5.7.3.4.2-7	$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63}$	✓				
5.8.3.4.3	Simplified Procedure for Prestressed and Nonprestressed Sections	N/A						✓
5.8.3.4.3-1	$V_{ci} = 0.02\lambda\sqrt{f'_c}\, b_v d_v + V_d + \dfrac{V_i M_{cre}}{M_{max}} \ge 0.06\lambda\sqrt{f'_c}\, b_v d_v$	N/A						✓
5.8.3.4.3-2	$M_{cre} = S_c\left(f_r + f_{cpe} - \dfrac{M_{dnc}}{S_{nc}}\right)$	N/A						✓
5.8.3.4.3-3	$V_{cw} = \left(0.06\lambda\sqrt{f'_c} + 0.30 f_{pc}\right) b_v d_v + V_p$	N/A						✓
5.8.3.4.3-4	$\cot\theta = 1.0 + 3\left(\dfrac{f_{pc}}{\sqrt{f'_c}}\right) \le 1.8$	N/A						✓

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5.8.3.5	Longitudinal Reinforcement	5.7.3.5	Longitudinal Reinforcement		✓			
5.8.3.5-1	$A_{ps}f_{ps}+A_sf_y \geq \frac{\lvert M_u \rvert}{d_v\phi_f}+0.5\frac{N_u}{\phi_c}+\left(\left\lvert\frac{V_u}{\phi_v}-V_p\right\rvert-0.5V_s\right)\cot\theta$	5.7.3.5-1	$A_{ps}f_{ps}+A_sf_y \geq \frac{\lvert M_u \rvert}{d_v\phi_f}+0.5\frac{N_u}{\phi_c}+\left(\left\lvert\frac{V_u}{\phi_v}-V_p\right\rvert-0.5V_s\right)\cot\theta$	✓				
5.8.3.5-2	$A_sf_y+A_{ps}f_{ps} \geq \left(\frac{V_u}{\phi_v}-0.5V_s-V_p\right)\cot\theta$	5.7.3.5-2	$A_sf_y+A_{ps}f_{ps} \geq \left(\frac{V_u}{\phi_v}-0.5V_s-V_p\right)\cot\theta$	✓				
5.8.3.6	Sections Subjected to Combined Shear and Torsion	5.7.3.6	Sections Subjected to Combined Shear and Torsion	✓				
5.8.3.6.1	Transverse Reinforcement	5.7.3.6.1	Transverse Reinforcement	✓				
5.8.3.6.2	Torsional Resistance	5.7.3.6.2	Torsional Resistance		✓			
5.8.3.6.2-1	$T_n = \frac{2A_oA_tf_y\cot\theta}{s}$	5.7.3.6.2-1	$T_n = \frac{2A_oA_tf_y\cot\theta}{s}$	✓				
5.8.3.6.3	Longitudinal Reinforcement	5.7.3.6.3	Longitudinal Reinforcement		✓			
5.8.3.6.3-1	$A_{ps}f_{ps}+A_sf_y \geq \frac{\lvert M_u \rvert}{\phi d_v}+\frac{0.5N_u}{\phi}+\cot\theta\sqrt{\left(\left\lvert\frac{V_u}{\phi}-V_p\right\rvert-0.5V_s\right)^2+\left(\frac{0.45p_hT_u}{2A_o\phi}\right)^2}$	5.7.3.6.3-1	$A_{ps}f_{ps}+A_sf_y \geq \frac{\lvert M_u \rvert}{\phi d_v}+\frac{0.5N_u}{\phi}+\cot\theta\sqrt{\left(\left\lvert\frac{V_u}{\phi}-V_p\right\rvert-0.5V_s\right)^2+\left(\frac{0.45p_hT_u}{2A_o\phi}\right)^2}$	✓				
5.8.3.6.3-2	$A_\ell = \frac{T_np_h}{2A_of_y}$	5.7.3.6.3-2	$A_\ell = \frac{T_np_h}{2A_of_y}$	✓				
5.8.4	Interface Shear Transfer—Shear Friction	5.7.4	Interface Shear Transfer—Shear Friction	✓				
5.8.4.1	General	5.7.4.1	General		✓			
5.8.4.1	General	5.7.4.3	Interface Shear Resistance		✓			
5.8.4.1-1	$V_{ri} = \phi V_{ni}$	5.7.4.3-1	$V_{ri} = \phi V_{ni}$	✓				
5.8.4.1-2	$V_{ri} \geq V_{ui}$	5.7.4.3-2	$V_{ri} \geq V_{ui}$	✓				
5.8.4.1-3	$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c)$	5.7.4.3-3	$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c)$	✓				
5.8.4.1-4	$V_{ni} \leq K_1 f'_c A_{cv}$	5.7.4.3-4	$V_{ni} \leq K_1 f'_c A_{cv}$	✓				
5.8.4.1-5	$V_{ni} \leq K_2 A_{cv}$	5.7.4.3-5	$V_{ni} \leq K_2 A_{cv}$	✓				
5.8.4.1-6	$A_{cv} = b_{vi} L_{vi}$	5.7.4.3-6	$A_{cv} = b_{vi} L_{vi}$	✓				

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5.8.4.1	General	5.7.4.6	Interface Shear in Box Girder Bridges				✓	
5.8.4.2	Computation of the Factored Interface Shear Force, Vui, for Girder/Slab Bridges	5.7.4.5	Computation of the Factored Interface Shear Force for Girder/Slab Bridges		✓			
5.8.4.2-1	$v_{ui} = \frac{V_{u1}}{b_{vi}d_v}$	5.7.4.5-1	$v_{ui} = \frac{V_{u1}}{b_{vi}d_v}$	✓				
5.8.4.2-2	$V_{ui} = v_{ui}A_{cv} = v_{ui}12b_{vi}$	5.7.4.5-2	$V_{ui} = v_{ui}A_{cv} = v_{ui}12b_{vi}$	✓				
5.8.4.2-3	$A_{vpc} = \frac{P_c}{\phi f_y}$	5.7.4.5-3	$A_{vpc} = \frac{P_c}{\phi f_y}$	✓				
C5.8.4.2-1	$M_{u2} = M_1 + V_1 \Delta\ell$	C5.7.4.5-1	$M_{u2} = M_1 + V_1 \Delta\ell$	✓				
C5.8.4.2-2	$C_{u2} = \frac{M_{u2}}{d_v}$	C5.7.4.5-2	$C_{u2} = \frac{M_{u2}}{d_v}$	✓				
C5.8.4.2-3	$C_{u2} = \frac{M_1}{d_v} + \frac{V_1\Delta\ell}{d_v}$	C5.7.4.5-3	$C_{u2} = \frac{M_1}{d_v} + \frac{V_1\Delta\ell}{d_v}$	✓				
C5.8.4.2-4	$C_1 = \frac{M_1}{d_v}$	C5.7.4.5-4	$C_1 = \frac{M_1}{d_v}$	✓				
C5.8.4.2-5	$V_h = C_{u2} - C_1$	C5.7.4.5-5	$V_h = C_{u2} - C_1$	✓				
C5.8.4.2-6	$V_h = \frac{V_1\Delta\ell}{d_v}$	C5.7.4.5-6	$V_h = \frac{V_1\Delta\ell}{d_v}$	✓				
C5.8.4.2-7	$V_{hi} = \frac{V_1}{d_v}$	C5.7.4.5-7	$V_{hi} = \frac{V_1}{d_v}$	✓				
5.8.4.3	Cohesion and Friction Factors	5.7.4.4	Cohesion and Friction Factors		✓			
5.8.4.4	Minimum Area of Interface Shear Reinforcement	5.7.4.2	Minimum Area of Interface Shear Reinforcement		✓			
5.8.4.4-1	$A_{vf} \ge \frac{0.05A_{cv}}{f_y}$	5.7.4.2-1	$A_{vf} \ge \frac{0.05A_{cv}}{f_y}$	✓				
5.8.5	Principal Tensile Stresses in Webs of Segmental Concrete Bridges	5.9.2.3.3	Principal Tensile Stresses in Webs			✓	✓	

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Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
N/A		5.9.2.3.3-1	$\tau = \frac{VQ_g}{I_g b_w}$				✓	
N/A		5.9.2.3.3-2	$\tau = \frac{VQ_g}{I_g b_w} + \frac{Tp_c}{A_{cp}^2}$				✓	
N/A		5.9.2.3.3-3	$\tau = \frac{VQ_g}{I_g B_w} + \frac{T}{2A_o b_w}$				✓	
N/A		5.9.2.3.3-4	$f_{min} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) - \sqrt{\left(f_{pcx} + f_{pcy}\right)^2 + (2\tau)^2}\right)$				✓	
N/A		C5.9.2.3.3-1	$f_{max} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) + \sqrt{\left(f_{pcx} + f_{pcy}\right)^2 + (2\tau)^2}\right)$				✓	
5.8.6	Shear and Torsion for Segmental Box Girder Bridges	5.12.5.3.8	Alternative Shear Design Procedure		✓			
5.8.6.1	General	5.12.5.3.8a	General			✓		
5.8.6.2	Loading	5.12.5.3.8b	Loading			✓		
5.8.6.3	Regions Requiring Consideration of Torsional Effects	5.7.2.1	General			✓		
5.8.6.3-1	$T_u > 1/3\ \phi T_{cr}$	N/A						✓
5.8.6.3-2	$T_{cr} = 0.0632K\lambda\sqrt{f'_c}2A_o b_e$	5.7.2.1-5	$T_{cr} = 0.126K\lambda\sqrt{f'_c}2A_o b_e$			✓		
5.8.6.3-3	$K = \sqrt{1 + \frac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \leq 2.0$	5.7.2.1-6	$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f'_c}}} \leq 2.0$			✓		
5.8.6.4	Torsional Reinforcement	5.12.5.3.8d	Torsional Reinforcement			✓		
5.8.6.4-1	$T_u \leq \phi T_n$	5.12.5.3.8d-1	$T_u \leq \phi T_n$	✓				
5.8.6.4-2	$T_n = \frac{2A_o A_v f_y}{s}$	5.12.5.3.8d-2	$T_n = \frac{2A_o A_v f_y}{s}$	✓				
5.8.6.4-3	$A_\ell \geq \frac{T_u\, p_h}{2\,\phi A_o\, f_y}$	5.12.5.3.8d-3	$A_\ell \geq \frac{T_u\, p_h}{2\,\phi A_o\, f_y}$	✓				
5.8.6.4-4	$\frac{M_u}{(0.9d_e f_y)}$	5.12.5.3.8d-4	$A_{req}\frac{M_u}{(0.9d_e f_y)}$		✓			

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5.8.6.5	Nominal Shear Resistance	5.12.5.3.8c	Nominal Shear Resistance			✓		
5.8.6.5-1	$V_n = V_c + V_s$	5.12.5.3.8c-1	$V_n = V_c + V_s$	✓				
5.8.6.5-2	$V_n = 0.379\lambda\sqrt{f'_c}\, b_v d_v$	5.12.5.3.8c-2	$V_n = 0.379\lambda\sqrt{f'_c}\, b_v d$		✓			
5.8.6.5-3	$V_c = 0.0632\ K\lambda\sqrt{f'_c}\ \ b_v d_v$	5.12.5.3.8c-3	$V_c = 0.0632\ K\lambda\sqrt{f'_c}\ \ b_v d$		✓			
5.8.6.5-4	$V_s = \dfrac{A_v f_y d_v}{s}$	5.12.5.3.8c-4	$V_s = \dfrac{A_v f_y d_v}{s}$	✓				
5.8.6.3-3	$K = \sqrt{1 + \dfrac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \ \leq\ 2.0$	5.12.5.3.8c-5	$K = \sqrt{1 + \dfrac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \ \leq\ 2.0$	✓				
5.8.6.5-5	$\left(\dfrac{V_u}{b_v d_v}\right) + \left(\dfrac{T_u}{2A_o b_e}\right) \leq 0.474\lambda\sqrt{f'_c}$	5.12.5.3.8c-6	$\left(\dfrac{V_u}{b_v d}\right) + \left(\dfrac{T_u}{2A_o b_e}\right) \leq 0.474\lambda\sqrt{f'_c}$		✓			
5.8.6.6	Reinforcement Details	5.12.5.3.8e	Reinforcement Details			✓		
5.8.6.6-1	$s_{max} = 0.8d_v \leq 36.0$ in.	5.7.2.6-1	$s_{max} = 0.8d_v \leq 24.0$			✓		
5.8.6.6-2	$s_{max} = 0.4d_v \leq 18.0$ in.	5.7.2.6-2	$s_{max} = 0.4d_v \leq 12.0$			✓		
5.9	**Prestressed Concrete**	**5.9**	**Prestressing**	✓				
5.9.1	General Design Considerations	5.9.1	General Design Considerations	✓				
5.9.1.1	General	5.9.1.1	General	✓				
5.9.1.2	Specified Concrete Strengths	5.9.1.2	Design Concrete Strengths		✓			
5.9.1.3	Buckling	5.9.1.5	Buckling	✓				
5.9.1.4	Section Properties	5.9.1.3	Section Properties			✓		
5.9.1.5	Crack Control	5.9.1.4	Crack Control		✓			
5.9.1.6	Tendons with Angle Points or Curves	5.9.1.6	Tendons with Angle Points or Curves	✓				
N/A		5.9.2	Stress Limitations				✓	
5.9.2	Stresses Due to Imposed Deformation	5.9.2.1	Stresses Due to Imposed Deformation	✓				
5.9.2-1	$F' = F\left(1 - e^{-\Psi(t,t_i)}\right)$	5.9.2.1-1	$F' = F\left(1 - e^{-\Psi(t,t_i)}\right)$	✓				

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5.9.2-2	$F' = F\left(1 - e^{-\psi(t,t_i)}\right)/\psi(t,t_i)$	5.9.2.1-2	$F' = F\left(1 - e^{-\psi(t,t_i)}\right)/\psi(t,t_i)$	✓				
5.9.3	Stress Limitations for Prestressing Tendons	5.9.2.2	Stress Limitations for Prestressing Steel		✓			
5.9.4	Stress Limits for Concrete	5.9.2.3	Stress Limits for Concrete	✓				
5.9.4.1	For Temporary Stresses before Losses	5.9.2.3.1	For Temporary Stresses before Losses	✓				
5.9.4.1.1	Compression Stresses	5.9.2.3.1a	Compressive Stresses		✓			
5.9.4.1.2	Tensile Stresses	5.9.2.3.1b	Tensile Stresses			✓		
5.9.4.2	For Stresses at Service Limit State after Losses	5.9.2.3.2	For Stresses at Service Limit State after Losses	✓				
5.9.4.2.1	Compression Stresses	5.9.2.3.2a	Compressive Stresses		✓			
5.9.4.2.2	Tension Stresses	5.9.2.3.2b	Tensile Stresses			✓		
5.9.5	Loss of Prestress	5.9.3	Prestress Loss		✓			
5.9.5.1	Total Loss of Prestress	5.9.3.1	Total Prestress Loss		✓			
5.9.5.1-1	$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$	5.9.3.1-1	$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$	✓				
5.9.5.1-2	$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT}$	5.9.3.1-2	$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT}$	✓				
5.9.5.2	Instantaneous Losses	5.9.3.2	Instantaneous Losses	✓				
5.9.5.2.1	Anchorage Set	5.9.3.2.1	Anchorage Set	✓				
5.9.5.2.2	Friction	5.9.3.2.2	Friction	✓				
5.9.5.2.2a	Pretensioned Construction	5.9.3.2.2a	Pretensioned Members		✓			
5.9.5.2.2b	Post-Tensioned Construction	5.9.3.2.2b	Post-Tensioned Members		✓			
5.9.5.2.2b-1	$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right)$	5.9.3.2.2b-1	$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right)$	✓				
5.9.5.2.2b-2	$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha + 0.04)}\right)$	5.9.3.2.2b-2	$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha + 0.04)}\right)$	✓				
C5.9.5.2.2b-1	$\alpha = \sqrt{\alpha_v^2 + \alpha_h^2}$	C5.9.3.2.2b-1	$\alpha = \sqrt{\alpha_v^2 + \alpha_h^2}$	✓				
C5.9.5.2.2b-2	$\alpha = \Sigma\Delta\alpha = \Sigma\sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2}$	C5.9.3.2.2b-2	$\alpha = \Sigma\Delta\alpha = \Sigma\sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2}$	✓				
5.9.5.2.3	Elastic Shortening	5.9.3.2.3	Elastic Shortening	✓				
5.9.5.2.3a	Pretensioned Members	5.9.3.2.3a	Pretensioned Members	✓				

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5.9.5.2.3a-1	$\Delta f_{pES} = \dfrac{E_p}{E_{ct}} f_{cgp}$	5.9.3.2.3a-1	$\Delta f_{pES} = \dfrac{E_p}{E_{ct}} f_{cgp}$	✓				
C5.9.5.2.3a-1	$\Delta f_{pES} = \dfrac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$	C5.9.3.2.3a-1	$\Delta f_{pES} = \dfrac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$	✓				
5.9.5.2.3b	Post-Tensioned Members	5.9.3.2.3b	Post-Tensioned Members	✓				
5.9.5.2.3b-1	$\Delta f_{pES} = \dfrac{N-1}{2N} \dfrac{E_p}{E_{ci}} f_{cgp}$	5.9.3.2.3b-1	$\Delta f_{pES} = \dfrac{N-1}{2N} \dfrac{E_p}{E_{ci}} f_{cgp}$	✓				
C5.9.5.2.3b-1	$\Delta f_{pES} = \dfrac{N-1}{2N} \dfrac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$	C5.9.3.2.3b-1	$\Delta f_{pES} = \dfrac{N-1}{2N} \dfrac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}}$	✓				
C5.9.5.2.3b-2	$N = N_1 + N_2 \dfrac{A_{sp2}}{A_{sp1}}$	C5.9.3.2.3b-2	$N = N_1 + N_2 \dfrac{A_{sp2}}{A_{sp1}}$	✓				
5.9.5.2.3c	Combined Pretensioning and Post-Tensioning	5.9.3.2.3c	Combined Pretensioning and Post-Tensioning	✓				
5.9.5.3	Approximate Estimate of Time Dependent Losses	5.9.3.3	Approximate Estimate of Time Dependent Losses	✓				
5.9.5.3-1	$\Delta f_{pLT} = 10.0 \dfrac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$	5.9.3.3-1	$\Delta f_{pLT} = 10.0 \dfrac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$	✓				
5.9.5.3-2	$\gamma_h = 1.7 - 0.01H$	5.9.3.3-2	$\gamma_h = 1.7 - 0.01H$	✓				
5.9.5.3-3	$\gamma_{st} = \dfrac{5}{(1 + f'_{ci})}$	5.9.3.3-3	$\gamma_{st} = \dfrac{5}{(1 + f'_{ci})}$	✓				
5.9.5.4	Refined Estimates of Time-Dependent Losses	5.9.3.4	Refined Estimates of Time-Dependent Losses	✓				
5.9.5.4.1	General	5.9.3.4.1	General			✓		
5.9.5.4.1-1	$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + \underline{(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}}$	5.9.3.4.1-1	$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + \underline{(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}}$	✓				
5.9.5.4.2	Losses: Time of Transfer to Time of Deck Placement	5.9.3.4.2	Losses: Time of Transfer to Time of Deck Placement	✓				
5.9.5.4.2a	Shrinkage of Girder Concrete	5.9.3.4.2a	Shrinkage of Girder Concrete	✓				
5.9.5.4.2a-1	$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id}$	5.9.3.4.2a-1	$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id}$	✓				

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5.9.5.4.2a-2	$K_{id} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1+\frac{A_g e_{pg}^2}{I_g}\right)\left[1+0.7\psi_b(t_f,t_i)\right]}$	5.9.3.4.2a-2	$K_{id} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1+\frac{A_g e_{pg}^2}{I_g}\right)\left[1+0.7\psi_b(t_f,t_i)\right]}$	✓				
5.9.5.4.2b	Creep of Girder Concrete	5.9.3.4.2b	Creep of Girder Concrete	✓				
5.9.5.4.2b-1	$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b(t_d,t_i)K_{id}$	5.9.3.4.2b-1	$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b(t_d,t_i)K_{id}$	✓				
5.9.5.4.2c	Relaxation of Prestressing Strands	5.9.3.4.2c	Relaxation of Prestressing Strands			✓		
5.9.5.4.2c-1	$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}}-0.55\right)$	5.9.3.4.2c-1	$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}}-0.55\right)$	✓				
C5.9.5.4.2c-1	$\Delta f_{pR1} = \left[\frac{f_{pt}}{K_L'}\frac{\log(24t)}{\log(24t_i)}\left(\frac{f_{pt}}{f_{py}}-0.55\right)\right]\left[1-\frac{3(\Delta f_{pSR}+\Delta f_{pCR})}{f_{pt}}\right]K_{id}$	C5.9.3.4.2c-1	$\Delta f_{pR1} = \left[\frac{f_{pt}}{K_L'}\frac{\log(t)}{\log(t_i)}\left(\frac{f_{pt}}{f_{py}}-0.55\right)\right]\left[1-\frac{3(\Delta f_{pSR}+\Delta f_{pCR})}{f_{pt}}\right]K_{id}$			✓		
5.9.5.4.3	Losses: Time of Deck Placement to Final Time	5.9.3.4.3	Losses: Time of Deck Placement to Final Time	✓				
5.9.5.4.3a	Shrinkage of Girder Concrete	5.9.3.4.3a	Shrinkage of Girder Concrete	✓				
5.9.5.4.3a-1	$\Delta f_{pSD} = \varepsilon_{bdf}E_pK_{df}$	5.9.3.4.3a-1	$\Delta f_{pSD} = \varepsilon_{bdf}E_pK_{df}$	✓				
5.9.5.4.3a-2	$K_{df} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_c}\left(1+\frac{A_c e_{pc}^2}{I_c}\right)\left[1+0.7\psi_b(t_f,t_i)\right]}$	5.9.3.4.3a-2	$K_{df} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_c}\left(1+\frac{A_c e_{pc}^2}{I_c}\right)\left[1+0.7\psi_b(t_f,t_i)\right]}$	✓				
5.9.5.4.3b	Creep of Girder Concrete	5.9.3.4.3b	Creep of Girder Concrete	✓				
5.9.5.4.3b-1	$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left[\psi_b(t_f,t_i)-\psi_b(t_d,t_i)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b(t_f,t_d)K_{df}$	5.9.3.4.3b-1	$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left[\psi_b(t_f,t_i)-\psi_b(t_d,t_i)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b(t_f,t_d)K_{df}$	✓				
5.9.5.4.3c	Relaxation of Prestressing Strands	5.9.3.4.3c	Relaxation of Prestressing Strands	✓				
5.9.5.4.3c-1	$\Delta f_{pR2} = \Delta f_{pR1}$	5.9.3.4.3c-1	$\Delta f_{pR2} = \Delta f_{pR1}$	✓				
5.9.5.4.3d	Shrinkage of Deck Concrete	5.9.3.4.3d	Shrinkage of Deck Concrete	✓				

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5.9.5.4.3d-1	$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1+0.7\psi_b\left(t_f,t_d\right)\right]$	5.9.3.4.3d-1	$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1+0.7\psi_b\left(t_f,t_d\right)\right]$	✓				
5.9.5.4.3d-2	$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{c\,deck}}{\left[1+0.7\psi_d\left(t_f,t_d\right)\right]}\left(\frac{1}{A_c}-\frac{e_{pc}e_d}{I_c}\right)$	5.9.3.4.3d-2	$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{c\,deck}}{\left[1+0.7\psi_d\left(t_f,t_d\right)\right]}\left(\frac{1}{A_c}-\frac{e_{pc}e_d}{I_c}\right)$	✓				
5.9.5.4.4	Precast Pretensioned Girders without Composite Topping	5.9.3.4.4	Precast Pretensioned Girders without Composite Topping	✓				
5.9.5.4.5	Post-Tensioned Nonsegmental Girders	5.9.3.4.5	Post-Tensioned Nonsegmental Girders	✓				
N/A		5.9.3.5	Losses in Multi-Stage Prestressing				✓	
5.9.5.5	Losses for Deflection Calculations	5.9.3.6	Losses for Deflection Calculations	✓				
N/A		5.9.4	Details for Pretensioning				✓	
5.10	**Details of Reinforcement**	**5.10**	**Reinforcement**		✓			
5.10.1	Concrete Cover	5.10.1	Concrete Cover			✓		
5.10.1	Concrete Cover	5.14.3	Concrete Cover			✓		
5.10.2	Hooks and Bends	5.10.2	Hooks and Bends	✓				
5.10.2.1	Standard Hooks	5.10.2.1	Standard Hooks	✓				
5.10.2.2	Seismic Hooks	5.10.2.2	Seismic Hooks		✓			
5.10.2.2	Seismic Hooks	5.11.4.1.4	Transverse Reinforcement for Confinement of Plastic Hinges			✓		
5.10.2.3	Minimum Bend Diameters	5.10.2.3	Minimum Bend Diameters	✓				
5.10.3	Spacing of Reinforcement	5.10.3	Spacing of Reinforcement	✓				
5.10.3.1	Minimum Spacing of Reinforcing Bars	5.10.3.1	Minimum Spacing of Reinforcing Bars	✓				
5.10.3.1.1	Cast-in-Place Concrete	5.10.3.1.1	Cast-in-Place Concrete		✓			
5.10.3.1.2	Precast Concrete	5.10.3.1.2	Precast Concrete		✓			
5.10.3.1.3	Multilayers	5.10.3.1.3	Multilayers	✓				
5.10.3.1.4	Splices	5.10.3.1.4	Splices	✓				
5.10.3.1.5	Bundled Bars	5.10.3.1.5	Bundled Bars	✓				

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5.10.3.2	Maximum Spacing of Reinforcing Bars	5.10.3.2	Maximum Spacing of Reinforcing Bars		✓			
5.10.3.3	Minimum Spacing of Prestressing Tendons and Ducts	5.9.4.1	Minimum Spacing of Pretensioning Strand		✓			
5.10.3.3.1	Prestensioning Strand	5.9.4.1	Minimum Spacing of Pretensioning Strand		✓			
N/A		5.9.5.1	Minimum Spacing of Post-Tensioning Tendons and Ducts				✓	
5.10.3.3.2	Post-Tensioning Ducts—Girders Straight in Plan	5.9.5.1.1	Post-Tensioning Ducts—Girders Straight in Plan			✓		
5.10.3.3.3	Post-Tensioning Ducts—Girders Curved in Plan	5.9.5.1.2	Post-Tensioning Ducts—Girders Curved in Plan	✓				
5.10.3.4	Maximum Spacing of Prestressing Tendons and Ducts in Slabs	5.9.4.2	Maximum Spacing of Pretensioning Strand in Slabs		✓			
5.10.3.4	Maximum Spacing of Prestressing Tendons and Ducts in Slabs	5.9.5.2	Maximum Spacing of Post-Tensioning Tendons and Ducts in Slabs		✓			
5.10.3.5	Couplers in Post-Tensioning Tendons	5.9.5.3	Couplers in Post-Tensioning Tendons		✓			
5.10.4	Tendon Confinement	5.9.5.4	Tendon Confinement	✓				
5.10.4.1	General	5.9.5.4.1	General			✓		
5.10.4.2	Wobble Effect in Slabs	5.9.5.4.2	Wobble Effect in Slabs		✓			
5.10.4.3	Effects of Curved Tendons	5.9.5.4.3	Effects of Curved Tendons		✓			
5.10.4.3.1	Design for In-Plane Force Effects	5.9.5.4.4	Design for In-Plane Force Effects	✓				
5.10.4.3.1a	In-Plane Force Effects	5.9.5.4.4a	In-Plane Force Effects		✓			
5.10.4.3.1a-1	$F_{u\text{-}in} = \frac{P_u}{R}$	5.9.5.4.4a-1	$F_{u\text{-}in} = \frac{P_u}{R}$	✓				
5.10.4.3.1b	Shear Resistance to Pull-out	5.9.5.4.4b	Shear Resistance to Pull-out			✓		
5.10.4.3.1b-1	$V_r = \phi V_n$	5.9.5.4.4b-1	$V_r = \phi V_n$	✓				
5.10.4.3.1b-2	$V_n = 0.15 d_{eff} \lambda \sqrt{f'_{ci}}$	5.9.5.4.4b-2	$V_n = 0.15 d_{eff} \lambda \sqrt{f'_{ci}}$	✓				
5.10.4.3.1b-3	$d_{eff} = d_c + \frac{d_{duct}}{4}$	5.9.5.4.4b-3	$d_{eff} = d_c + \frac{d_{duct}}{4}$	✓				
5.10.4.3.1b-4	$d_{eff} = t_w - \frac{d_{duct}}{2}$	5.9.5.4.4b-4	$d_{eff} = t_w - \frac{d_{duct}}{2}$	✓				
5.10.4.3.1b-5	$d_{eff} = d_c + \frac{d_{duct}}{4} + \frac{\Sigma s_{duct}}{2}$	5.9.5.4.4b-5	$d_{eff} = d_c + \frac{d_{duct}}{4} + \frac{\Sigma s_{duct}}{2}$	✓				

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5.10.4.3.1c	Cracking of Cover Concrete	5.9.5.4.4c	Cracking of Cover Concrete			✓		
5.10.4.3.1c-1	$M_{end} = \dfrac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{12}$	5.9.5.4.4c-1	$M_{end} = \dfrac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{12}$	✓				
5.10.4.3.1c-2	$M_{mid} = \dfrac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{24}$	5.9.5.4.4c-2	$M_{mid} = \dfrac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{24}$	✓				
5.10.4.3.1c-3	$f_{cr} = \phi f_r$	5.9.5.4.4c-3	$f_{cr} = \phi f_r$	✓				
5.10.4.3.1d	Regional Bending	5.9.5.4.4d	Regional Bending	✓				
5.10.4.3.1d-1	$M_u = \dfrac{\phi_{cont} \Sigma F_{u-in} h_c}{4}$	5.9.5.4.4d-1	$M_u = \dfrac{\phi_{cont} \Sigma F_{u-in} h_c}{4}$	✓				
5.10.4.3.2	Out-of-Plane Force Effects	5.9.5.4.5	Out-of-Plane Force Effects		✓			
5.10.4.3.2-1	$F_{u-out} = \dfrac{P_u}{\pi R}$	5.9.5.4.5-1	$F_{u-out} = \dfrac{P_u}{\pi R}$	✓				
5.10.5	External Tendon Supports	5.9.5.5	External Tendon Supports	✓				
5.10.6	Transverse Reinforcement for Compression Members	5.10.4	Transverse Reinforcement for Compression Members	✓				
5.10.6.1	General	5.10.4.1	General	✓				
5.10.6.2	Spirals	5.10.4.2	Spirals		✓			
5.10.6.3	Ties	5.10.4.3	Ties			✓		
5.10.7	Transverse Reinforcement for Flexural Members	5.10.5	Transverse Reinforcement for Flexural Members	✓				
5.10.8	Shrinkage and Temperature Reinforcement	5.10.6	Shrinkage and Temperature Reinforcement		✓			
5.10.8-1	$A_s \geq \dfrac{1.30bh}{2(b+h)f_y}$	5.10.6-1	$A_s \geq \dfrac{1.30bh}{2(b+h)f_y}$	✓				
5.10.8-2	$0.11 \leq A_s \leq 0.60$	5.10.6-2	$0.11 \leq A_s \leq 0.60$	✓				
C5.10.8-1	$A_s \geq \dfrac{1.3A_g}{Perimeter(f_y)}$	C5.10.6-1	$A_s \geq \dfrac{1.3A_g}{Perimeter(f_y)}$	✓				
5.10.9	Post-Tensioned Anchorage Zones	5.9.5.6	Post-Tensioned Anchorage Zones	✓				

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5.10.9.1	General	5.9.5.6.1	General	✓				
5.10.9.2	General Zone and Local Zone	5.9.5.6.1	General	✓				
5.10.9.2.1	General	5.9.5.6.1	General	✓				
5.10.9.2.2	General Zone	5.9.5.6.2	General Zone	✓				
5.10.9.2.3	Local Zone	5.9.5.6.3	Local Zone	✓				
5.10.9.2.4	Responsibilities	5.9.5.6.4	Responsibilities			✓		
5.10.9.3	Design of the General Zone	5.9.5.6.5	Design of the General Zone	✓				
5.10.9.3.1	Design Methods	5.9.5.6.5a	Design Methods	✓				
5.10.9.3.2	Design Principles	5.9.5.6.5b	Design Principles		✓			
5.10.9.3.3	Special Anchorage Devices	5.9.5.6.6	Special Anchorage Devices	✓				
5.10.9.3.4	Intermediate Anchorages	5.9.5.6.7	Intermediate Anchorages	✓				
5.10.9.3.4a	General	5.9.5.6.7a	General			✓		
5.10.9.3.4b	Crack Control Behind Intermediate Anchors	5.9.5.6.7b	Crack Control Behind Intermediate Anchors		✓			
5.10.9.3.4b-1	$T_{ia} = 0.25P_s - f_{cb}A_{cb}$	5.9.5.6.7b-1	$T_{ia} = 0.25P_s - f_{cb}A_{cb}$	✓				
5.10.9.3.4c	Blister and Rib Reinforcement	5.9.5.6.7c	Blister and Rib Reinforcement	✓				
5.10.9.3.5	Diaphragms	5.9.5.6.8	Diaphragms	✓				
5.10.9.3.6	Multiple Slab Anchorages	5.8.4.5.5	Multiple Slab Anchorages	✓				
5.10.9.3.6-1	$T_1 = 0.10\ P_u\left(1 - \frac{a}{s}\right)$	5.8.4.5.5-1	$T_1 = 0.10\ P_u\left(1 - \frac{a}{s}\right)$	✓				
5.10.9.3.6-2	$T_2 = 0.20\ P_u\left(1 - \frac{a}{s}\right)$	5.8.4.5.5-2	$T_2 = 0.20\ P_u\left(1 - \frac{a}{s}\right)$	✓				
5.10.9.3.7	Deviation Saddles	5.9.5.6.9	Deviation Saddles	✓				
5.10.9.4	Application of the Strut-and-Tie Model to the Design of the General Zone	5.8.2.7	Application to the Design of the General Zones of Post-Tensioning Anchorages		✓			
5.10.9.4.1	General	5.8.2.7.1	General	✓				
5.10.9.4.2	Nodes	5.8.2.7.2	Nodes	✓				
5.10.9.4.3	Struts	5.8.2.7.3	Struts		✓			

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5.10.9.4.4	Ties	5.8.2.7.4	Ties	✓				
5.10.9.5	Elastic Stress Analysis	5.8.3	Elastic Stress Analysis		✓			
5.10.9.5	Elastic Stress Analysis	5.8.3.1	General		✓			
5.10.9.5	Elastic Stress Analysis	5.8.3.2	General Zones of Post-Tensioning Anchorages		✓			
5.10.9.6	Approximate Stress Analyses and Design	5.8.4	Approximate Stress Analysis and Design	✓				
5.10.9.6	Approximate Stress Analyses and Design	5.8.4.5	General Zones of Post-Tensioning Anchorages		✓			
5.10.9.6.1	Limitations of Application	5.8.4.5.1	Limitations of Application	✓				
5.10.9.6.2	Compressive Stresses	5.8.4.5.2	Compressive Stresses			✓		
5.10.9.6.2-1	$f_{ca} = \dfrac{0.6P_u\kappa}{A_b\left(1+\ell_c\left(\dfrac{1}{b_{eff}}-\dfrac{1}{t}\right)\right)}$	5.8.4.5.2-1	$f_{ca} = \dfrac{0.6P_u\kappa}{A_b\left(1+\ell_c\left(\dfrac{1}{b_{eff}}-\dfrac{1}{t}\right)\right)}$	✓				
5.10.9.6.2-2	$\kappa = 1 + \left(2-\dfrac{s}{a_{eff}}\right)\left(0.3+\dfrac{n}{15}\right)$	5.8.4.5.2-2	$\kappa = 1 + \left(2-\dfrac{s}{a_{eff}}\right)\left(0.3+\dfrac{n}{15}\right)$	✓				
5.10.9.6.2-3	$\kappa = 1$	5.8.4.5.2-3	$\kappa = 1$	✓				
5.10.9.6.3	Bursting Forces	5.8.4.5.3	Bursting Forces		✓			
5.10.9.6.3-1	$T_{burst} = 0.25\Sigma P_u\left(1-\dfrac{a}{h}\right) + 0.5\ \lvert\Sigma(P_u \sin\alpha)\rvert$	5.8.4.5.3-1	$T_{burst} = 0.25\Sigma P_u\left(1-\dfrac{a}{h}\right) + 0.5\ \lvert\Sigma(P_u \sin\alpha)\rvert$	✓				
5.10.9.6.3-2	$d_{burst} = 0.5(h-2e)+5e\sin\alpha$	5.8.4.5.3-2	$d_{burst} = 0.5(h-2e)+5e\sin\alpha$	✓				
5.10.9.6.4	Edge Tension Forces	5.8.4.5.4	Edge Tension Forces	✓				
5.10.9.7	Design of Local Zones	5.8.4.4	Local Zones		✓			
5.10.9.7.1	Dimensions of Local Zone	5.8.4.4.1	Dimensions of Local Zone		✓			
5.10.9.7.2	Bearing Resistance	5.8.4.4.2	Bearing Resistance		✓			
5.10.9.7.2-1	$P_r = \phi f_n A_b$	5.8.4.4.2-1	$P_r = \phi f_n A_b$	✓				
5.10.9.7.2-2	$f_n = 0.7 f'_{ci}\sqrt{\dfrac{A}{A_g}}$	5.8.4.4.2-2	$f_n = 0.7 f'_{ci}\sqrt{\dfrac{A}{A_g}}$	✓				

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5.10.9.7.2-3	$f_n = 2.25\, f'_{ci}$	5.8.4.4.2-3	$f_n = 2.25\, f'_{ci}$	✓				
5.10.9.7.2-4	$n/t \le 0.08 \left(\frac{E_b}{f_b}\right)^{0.33}$	5.8.4.4.2-4	$n/t \le 0.08 \left(\frac{E_b}{f_b}\right)^{0.33}$	✓				
5.10.9.7.3	Special Anchorage Devices	5.8.4.4.3	Special Anchorage Devices		✓			
5.10.10	Pretensioned Anchorage Zones	5.9.4.4	Pretensioned Anchorage Zones	✓				
5.10.10.1	Splitting Resistance	5.9.4.4.1	Splitting Resistance		✓			
5.10.10.1-1	$P_r = f_s A_s$	5.9.4.4.1-1	$P_r = f_s A_s$	✓				
5.10.10.2	Confinement Reinforcement	5.9.4.4.2	Confinement Reinforcement	✓				
N/A		5.9.5	Details for Post-Tensioning				✓	
5.10.11	Provisions for Seismic Design	**5.11**	**Seismic Design and Details**		✓			
5.10.11.1	General	5.11.1	General			✓		
5.10.11.2	Seismic Zone 1	5.11.2	Seismic Zone 1		✓			
5.10.11.3	Seismic Zone 2	5.11.3	Seismic Zone 2	✓				
5.10.11.3	General	5.11.3.1	General	✓				
5.10.11.4	Seismic Zones 3 and 4	5.11.4	Seismic Zones 3 and 4	✓				
5.10.11.4.1	Column Requirements	5.11.4.1	Column Requirements	✓				
5.10.11.4.1a	Longitudinal Reinforcement	5.11.4.1.1	Longitudinal Reinforcement	✓				
5.10.11.4.1b	Flexural Resistance	5.11.4.1.2	Flexural Resistance		✓			
5.10.11.4.1c	Column Shear and Transverse Reinforcement	5.11.4.1.3	Column Shear and Transverse Reinforcement		✓			
5.10.11.4.1d	Transverse Reinforcement for Confinement at Plastic Hinges	5.11.4.1.4	Transverse Reinforcement for Confinement at Plastic Hinges			✓		
5.10.11.4.1d-1	$\rho_s \ge 0.12 \frac{f'_c}{f_y}$	5.11.4.1.4-1	$\rho_s = \frac{4A_{sp}}{(d_s s)} \ge 0.12 \frac{f'_c}{f_{yh}}$			✓		
5.10.11.4.1d-2	$A_{sh} \ge 0.30\, s h_c \frac{f'_c}{f_y} \left[\frac{A_g}{A_c} - 1\right]$	5.11.4.1.4-2	$A_{sh} \ge 0.30\, s h_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1\right]$			✓		
5.10.11.4.1d-3	$A_{sh} \ge 0.12\, s h_c \frac{f'_c}{f_y}$	5.11.4.1.4-3	$A_{sh} \ge 0.12 s h_c \frac{f'_c}{f_{yh}}$			✓		

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5.10.11.4.1e	Spacing of Transverse Reinforcement for Confinement	5.11.4.1.5	Spacing of Transverse Reinforcement for Confinement			✓		
5.10.11.4.1f	Splices	5.11.4.1.6	Splices		✓			
5.10.11.4.2	Requirements for Wall-Type Piers	5.11.4.2	Requirements for Wall-Type Piers		✓			
5.10.11.4.2-1	$V_r = 0.253\lambda\sqrt{f_c'}bd$, and	5.11.4.2-1	$V_r = 0.253\lambda\sqrt{f_c'}bd$, and	✓				
5.10.11.4.2-2	$V_r = \phi V_n$	5.11.4.2-2	$V_r = \phi V_n$	✓				
5.10.11.4.2-3	$V_n = \left[0.063\lambda\sqrt{f_c'} + \rho_h f_y\right]bd$	5.11.4.2-3	$V_n = \left[0.063\lambda\sqrt{f_c'} + \rho_h f_y\right]bd$	✓				
5.10.11.4.3	Column Connections	5.11.4.3	Column Connections		✓			
5.10.11.4.3-1	$V_n \leq 0.380\, bd\lambda\sqrt{f_c'}$	5.11.4.3-1	$V_n \leq 0.380\, bd\lambda\sqrt{f_c'}$	✓				
5.10.11.4.4	Construction Joints in Piers and Columns	5.11.4.4	Construction Joints in Piers and Columns	✓				
5.10.11.4.4-1	$V_n = \left(A_{vf} f_y + 0.75P_u\right)$	5.11.4.4-1	$V_n = \left(A_{vf} f_y + 0.75P_u\right)$	✓				
5.10.12	Reinforcement for Hollow Rectangular Compression Members	5.10.7	Reinforcement for Hollow Rectangular Compression Members	✓				
5.10.12.1	General	5.10.7.1	General	✓				
5.10.12.2	Spacing of Reinforcement	5.10.7.2	Spacing of Reinforcement	✓				
5.10.12.3	Ties	5.10.7.3	Ties			✓		
5.10.12.4	Splices	5.10.7.4	Splices	✓				
5.10.12.5	Hoops	5.10.7.5	Hoops	✓				
5.11	**Development and Splices of Reinforcement**	5.10.8	Development and Splices of Reinforcement	✓				
5.11.1	General	5.10.8.1	General		✓			
5.11.1.1	Basic Requirements	5.10.8.1.1	Basic Requirements		✓			
5.11.1.2	Flexural Reinforcement	5.10.8.1.2	Flexural Reinforcement	✓				
5.11.1.2.1	General	5.10.8.1.2a	General		✓			
5.11.1.2.2	Positive Moment Reinforcement	5.10.8.1.2b	Positive Moment Reinforcement	✓				
C5.11.1.2.2-1	$\ell_d \leq \frac{M_n}{V_u} + \ell_a$	C5.10.8.1.2b-1	$\ell_d \leq \frac{M_n}{V_u} + \ell_a$	✓				

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5.11.1.2.3	Negative Moment Reinforcement	5.10.8.1.2c	Negative Moment Reinforcement		✓			
5.11.1.2.4	Moment Resisting Joints	5.10.8.1.2d	Moment Resisting Joints	✓				
5.11.2	Development of Reinforcement	5.10.8.2	Development of Reinforcement		✓			
5.11.2.1	Deformed Bars and Deformed Wire in Tension	5.10.8.2.1	Deformed Bars and Deformed Wire in Tension			✓		
5.11.2.1.1	Tension Development Length	5.10.8.2.1a	Tension Development Length		✓			
5.11.2.1.1-1	$\ell_d = \ell_{db} \times \left(\frac{\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc} \times \lambda_{er}}{\lambda} \right)$	5.10.8.2.1a-1	$\ell_d = \ell_{db} \times \left(\frac{\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc} \times \lambda_{er}}{\lambda} \right)$	✓				
5.11.2.1.1-2	$\ell_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}}$	5.10.8.2.1a-2	$\ell_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}}$	✓				
5.11.2.1.2	Modification Factors which Increase ℓd	5.10.8.2.1b	Modification Factors which Increase ℓd		✓			
5.11.2.1.3	Modification Factors which Decrease ℓd	5.10.8.2.1c	Modification Factors which Decrease ℓd		✓			
5.11.2.1.3-1	$0.4 \le \lambda_{rc} = \frac{d_b}{c_b + k_{tr}} \le 1.0$	5.10.8.2.1c-1	$0.4 \le \lambda_{rc} \le 1.0$		✓			
5.11.2.1.3-1	$0.4 \le \lambda_{rc} = \frac{d_b}{c_b + k_{tr}} \le 1.0$	5.10.8.2.1c-2	$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}}$		✓			
5.11.2.1.3-2	$k_{tr} = 40A_{tr}/(sn)$	5.10.8.2.1c-3	$k_{tr} = 40A_{tr}/(sn)$	✓				
5.11.2.1.3-3	$\lambda_{er} = \frac{(A_s \text{required})}{(A_s \text{provided})}$	5.10.8.2.1c-4	$\lambda_{er} = \frac{(A_s \text{required})}{(A_s \text{provided})}$	✓				
5.11.2.2	Deformed Bars in Compression	5.10.8.2.2	Deformed Bars in Compression	✓				
5.11.2.2.1	Compressive Development Length	5.10.8.2.2a	Compressive Development Length		✓			
N/A		5.10.8.2.2a-1	$\ell_d = \ell_{db} \lambda_{er} \lambda_{rc}$				✓	
5.11.2.2.1-1	$\ell_{db} \ge \frac{0.63\, d_b f_y}{\sqrt{f'_c}}$	5.10.8.2.2a-2	$\ell_{db} \ge \frac{0.63\, d_b f_y}{\sqrt{f'_c}}$	✓				
5.11.2.2.1-2	$\ell_{db} \ge 0.3\, d_b f_y$	5.10.8.2.2a-3	$\ell_{db} \ge 0.3\, d_b f_y$	✓				
5.11.2.2.2	Modification Factors	5.10.8.2.2b	Modification Factors		✓			
5.11.2.3	Bundled Bars	5.10.8.2.3	Bundled Bars	✓				

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5.11.2.4	Standard Hooks in Tension	5.10.8.2.4	Standard Hooks in Tension		✓			
5.11.2.4.1	Basic Hook Development Length	5.10.8.2.4a	Basic Hook Development Length			✓		
N/A		5.10.8.2.4a-1	$\ell_{dh} = \ell_{hb} \times \left(\frac{\lambda_{rc}\lambda_{cw}\lambda_{er}}{\lambda} \right)$				✓	
5.11.2.4.1-1	$\ell_{hb} = \frac{38.0 d_b}{60.0} \left(\frac{f_y}{\lambda \sqrt{f'_c}} \right)$	5.10.8.2.4a-2	$\ell_{hb} = \frac{38.0 d_b}{60.0} \left(\frac{f_y}{\sqrt{f'_c}} \right)$			✓		
5.11.2.4.2	Modification Factors	5.10.8.2.4b	Modification Factors			✓		
5.11.2.4.3	Hooked-Bar Tie Requirements	5.10.8.2.4c	Hooked-Bar Tie Requirements	✓				
5.11.2.5	Welded Wire Fabric	5.10.8.2.5	Welded Wire Fabric	✓				
5.11.2.5.1	Deformed Wire Fabric	5.10.8.2.5a	Deformed Wire Fabric		✓			
5.11.2.5.1-1	$\ell_{hd} \geq 0.95 d_b \frac{f_y - 20.0}{\lambda \sqrt{f'_c}}$	5.10.8.2.5a-1	$\ell_{hd} \geq 0.95 d_b \frac{f_y - 20.0}{\lambda \sqrt{f'_c}}$	✓				
5.11.2.5.1-2	$\ell_{hd} \geq 6.30 \frac{A_w f_y}{s_w \lambda \sqrt{f'_c}}$	5.10.8.2.5a-2	$\ell_{hd} \geq 6.30 \frac{A_w f_y}{s_w \lambda \sqrt{f'_c}}$	✓				
5.11.2.5.2	Plain Wire Fabric	5.10.8.2.5b	Plain Wire Fabric		✓			
5.11.2.5.2-1	$\ell_d = 8.50 \frac{A_w f_y}{s_w \lambda \sqrt{f'_c}}$	5.10.8.2.5b-1	$\ell_d = 8.50 \frac{A_w f_y}{s_w \lambda \sqrt{f'_c}}$	✓				
5.11.2.6	Shear Reinforcement	5.10.8.2.6	Shear Reinforcement	✓				
5.11.2.6.1	General	5.10.8.2.6a	General	✓				
5.11.2.6.2	Anchorage of Deformed Reinforcement	5.10.8.2.6b	Anchorage of Deformed Reinforcement			✓		
5.11.2.6.2-1	$\ell_d \geq \frac{0.44 d_b f_y}{\lambda \sqrt{f'_c}}$	5.10.8.2.6b-1	$\ell_d \geq \frac{0.44 d_b f_y}{\lambda \sqrt{f'_c}}$	✓				
5.11.2.6.3	Anchorage of Wire Fabric Reinforcement	5.10.8.2.6c	Anchorage of Wire Fabric Reinforcement	✓				
5.11.2.6.4	Closed Stirrups	5.10.8.2.6d	Closed Stirrups			✓		
5.11.3	Development by Mechanical Anchorages	5.10.8.3	Development by Mechanical Anchorages	✓				
5.11.4	Development of Prestressing Strand	5.9.4.3	Development of Pretensioning Strand		✓			

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5.11.4.1	General	5.9.4.3.1	General	✓				
5.11.4.2	Bonded Strand	5.9.4.3.2	Bonded Strand	✓				
5.11.4.2-1	$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$	5.9.4.3.2-1	$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$	✓				
5.11.4.2-2	$f_{px} = \frac{f_{pe}\ell_{px}}{60d_b}$	5.9.4.3.2-2	$f_{px} = \frac{f_{pe}\ell_{px}}{60d_b}$	✓				
5.11.4.2-3	$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{(\ell_d - 60d_b)}(f_{ps} - f_{pe})$	5.9.4.3.2-3	$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{(\ell_d - 60d_b)}(f_{ps} - f_{pe})$	✓				
5.11.4.3	Partially Debonded Strands	5.9.4.3.3	Debonded Strands		✓			
5.11.5	Splices of Bar Reinforcement	5.10.8.4	Splices of Bar Reinforcement		✓			
5.11.5.1	Detailing	5.10.8.4.1	Detailing	✓				
5.11.5.2	General Requirements	5.10.8.4.2	General Requirements	✓				
5.11.5.2.1	Lap Splices	5.10.8.4.2a	Lap Splices		✓			
5.11.5.2.1-1	$S_{max} = \frac{2\pi A_{sp} f_{ytr} \ell_s}{kA_s f_{ut}}$	5.10.8.4.2a-1	$S_{max} = \frac{2\pi A_{sp} f_{ytr} \ell_s}{kA_s f_{ut}}$	✓				
5.11.5.2.2	Mechanical Connections	5.10.8.4.2b	Mechanical Connections	✓				
5.11.5.2.3	Welded Splices	5.10.8.4.2c	Welded Splices	✓				
5.11.5.3	Splices of Reinforcement in Tension	5.10.8.4.3	Splices of Reinforcement in Tension		✓			
5.11.5.3.1	Lap Splices in Tension	5.10.8.4.3a	Lap Splices in Tension		✓			
5.11.5.3.2	Mechanical Connections or Welded Splices in Tension	5.10.8.4.3b	Mechanical Connections or Welded Splices in Tension	✓				
5.11.5.4	Splices in Tension Tie Members	5.10.8.4.4	Splices in Tie Members		✓			
5.11.5.5	Splices of Bars in Compression	5.10.8.4.5	Splices of Bars in Compression	✓				
5.11.5.5.1	Lap Splices in Compression	5.10.8.4.5a	Lap Splices in Compression		✓			
5.11.5.5.1-1	$\ell_c = 0.5mf_y d_b$	5.10.8.4.5a-1	$\ell_c = 0.5mf_y d_b$	✓				
5.11.5.5.1-2	$\ell_c = m(0.9f_y - 24.0)d_b$	5.10.8.4.5a-2	$\ell_c = m(0.9f_y - 24.0)d_b$	✓				
5.11.5.5.2	Mechanical Connections or Welded Splices in Compression	5.10.8.4.5b	Mechanical Connections or Welded Splices in Compression	✓				

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5.11.5.5.3	End-Bearing Splices	5.10.8.4.5c	End-Bearing Splices	✓				
5.11.6	Splices of Welded Wire Fabric	5.10.8.5	Splices of Welded Wire Fabric	✓				
5.11.6.1	Splices of Welded Deformed Wire Fabric in Tension	5.10.8.5.1	Splices of Welded Deformed Wire Fabric in Tension		✓			
5.11.6.2	Splices of Welded Smooth Wire Fabric in Tension	5.10.8.5.2	Splices of Welded Smooth Wire Fabric in Tension		✓			
5.12	**Durability**	**5.14**	**Durability**	✓				
5.12.1	General	5.14.1	Design Concepts			✓		
N/A		5.14.2	Major Chemical and Mechanical Factors Affecting Durability				✓	
N/A		5.14.2.1	General				✓	
N/A		5.14.2.2	Corrosion Resistance				✓	
N/A		5.14.2.3	Freeze-Thaw Resistance				✓	
N/A		5.14.2.4	External Sulfate Attack				✓	
N/A		5.14.2.5	Delayed Ettringite Formation				✓	
5.12.2	Alkali-Silica Reactive Aggregates	5.14.2.6	Alkali-Silica Reactive Aggregates	✓				
N/A		5.14.2.7	Alkali-Carbonate reactive Aggregates				✓	
5.10.1	Concrete Cover	5.14.3	Concrete Cover			✓		
5.12.3	Concrete Cover	5.10.1	Concrete Cover			✓		
5.12.4	Protective Coatings	5.14.4	Protective Coatings			✓		
5.12.5	Protection of Prestressing Tendons	5.14.6	Protection of Prestressing Tendons	✓				
5.13	**Specific Members**	**5.12**	**Provisions for Structure, Components and Types**		✓			
5.13.1	Deck Slabs	5.12.1	Deck Slabs	✓				
5.13.2	Diaphragms, Deep Beams, Brackets, Corbels, and Beam Ledges	N/A						✓
5.13.2.1	General	N/A						✓
5.13.2.2	Diaphragms	5.8.2.8	Application to design of Pier Diaphragms				✓	
5.13.2.2	Diaphragms	5.12.4	Diaphragms			✓		
5.13.2.3	Detailing Requirements for Deep Beams	5.8.4.1	Deep Beams			✓		

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5.13.2.3-1	$N_R = \phi f_y A_v \geq 0.12 b_v s$	N/A						✓
5.13.2.4	Brackets and Corbels	5.8.4.2	Brackets and Corbels	✓				
5.13.2.4.1	General	5.8.4.2.1	General		✓			
5.13.2.4.1-1	$M_u = V_u a_v + N_{uc}(h-d)$	5.8.4.2.1-1	$M_u = V_u a_v + N_{uc}(h-d)$	✓				
5.13.2.4.2	Alternative to Strut-and-Tie Model	5.8.2.9	Application to the Design of Brackets and Corbels			✓		
5.13.2.4.2	Alternative to Strut-and-Tie Model	5.8.4.2.2	Additional Requirements			✓		
5.13.2.4.2-1	$V_n = 0.2 f'_c b_w d_e$	5.8.4.2.2-1	$V_n = 0.2 f'_c b d_e$			✓		
5.13.2.4.2-2	$V_n = 0.8\ b_w\ d_e$	5.8.4.2.2-2	$V_n = 0.8\ b d_e$			✓		
5.13.2.4.2-3	$V_n = (0.2 - 0.07 a_v / d) f'_c b_w d_e$	5.8.4.2.2-3	$V_n = \left(\frac{0.2 - 0.07 a_v}{d_e}\right) f'_c b d_e$			✓		
5.13.2.4.2-4	$V_n = (0.8 - 0.28\ a_v / d_e) b_w d$	5.8.4.2.2-4	$V_n = \left(\frac{0.8 - 0.28\ a_v}{d_e}\right) b d_e$			✓		
5.13.2.4.2-5	$A_s \geq \frac{2A_{vf}}{3} + A_n$	5.8.4.2.2-5	$A_s \geq \frac{2A_{vf}}{3} + A_n$	✓				
5.13.2.4.2-6	$A_h \geq 0.5(A_s - A_n)$	5.8.4.2.2-6	$A_h \geq 0.5(A_s - A_n)$	✓				
5.13.2.4.2-7	$A_n \geq N_{uc} / \phi f_y$	5.8.4.2.2-7	$A_n \geq \frac{N_{uc}}{\phi f_y}$	✓				
5.13.2.5	Beam Ledges	5.8.4.3	Beam Ledges	✓				
5.13.2.5.1	General	5.8.4.3.1	General	✓				
5.13.2.5.2	Design for Shear	5.8.4.3.2	Design for Shear		✓			
5.13.2.5.3	Design for Flexure and Horizontal Force	5.8.4.3.3	Design for Flexure and Horizontal Force		✓			
5.13.2.5.4	Design for Punching Shear	5.8.4.3.4	Design for Punching Shear			✓		
5.13.2.5.4-1	$V_n = 0.125\lambda\sqrt{f'_c}(W + 2L + 2d_e) d_e$	5.8.4.3.4-3	$V_n = 0.125\lambda\sqrt{f'_c} b_o d_f$			✓		
5.13.2.5.4-2	$V_n = 0.125\lambda\sqrt{f'_c}(W + L + d_e) d_e$	5.8.4.3.4-3	$V_n = 0.125\lambda\sqrt{f'_c} b_o d_f$			✓		

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5.13.2.5.4-3	$V_n = 0.125\lambda\sqrt{f'_c}\left(0.5W + L + d_e + c\right)d_e$	5.8.4.3.4-3	$V_n = 0.125\lambda\sqrt{f'_c}b_o d_f$			✓		
N/A		5.8.4.3.4-1	$S > 2d_f + W$				✓	
N/A		5.8.4.3.4-2	$b_w + 2a_v > L + 2d_f$				✓	
N/A		5.8.4.3.4-4	$b_o = W + 2L + 2d_f$				✓	
N/A		5.8.4.3.4-5	$b_o = 0.5W + L + d_f + c \le W + 2L + 2d_f$				✓	
N/A		5.8.4.3.4-6	$b_o = \frac{\pi}{2}\left(D + d_f\right) + D$				✓	
N/A		5.8.4.3.4-7	$b_o = \frac{\pi}{4}\left(D + d_f\right) + \frac{D}{2} + c \le \frac{\pi}{2}\left(D + d_f\right) + D$				✓	
5.13.2.5.5	Design of Hanger Reinforcement	5.8.4.3.5	Design of Hanger Reinforcement			✓		
5.13.2.5.5-1	$V_n = \frac{A_{hr}\left(0.5f_y\right)}{s}\left(W + 3a_v\right)$	5.8.4.3.5-1	$V_n = \frac{0.5A_{hr}f_y\left(W + 3a_v\right)}{s}$		✓			
5.13.2.5.5-2	$V_n = \frac{A_{hr}f_y}{s}S$	5.8.4.3.5-2	$V_n = \frac{A_{hr}f_y}{s}S$	✓				
5.13.2.5.5-3	$V_n = \left(0.063\lambda\sqrt{f'_c}b_f d_f\right) + \frac{A_{hr}f_y}{s}\left(W + 2d_f\right)$	5.8.4.3.5-3	$V_n = \left(0.063\lambda\sqrt{f'_c}b_f d_f\right) + \frac{A_{hr}f_y}{s}\left(W + 2d_f\right)$	✓				
5.13.2.5.6	Design for Bearing	5.8.4.3.6	Design for Bearing	✓				
5.13.3	Footings	5.12.8	Footings	✓				
5.13.3.1	General	5.12.8.1	General	✓				
5.13.3.2	Loads and Reactions	5.12.8.2	Loads and Reactions	✓				
5.13.3.3	Resistance Factors	5.12.8.3	Resistance Factors			✓		
5.13.3.4	Moment in Footings	5.12.8.4	Moment in Footings	✓				
5.13.3.5	Distribution of Moment Reinforcement	5.12.8.5	Distribution of Moment Reinforcement	✓				
5.13.3.5-1	$A_{s\text{-}BW} = A_{s\text{-}SD}\left(\frac{2}{\beta + 1}\right)$	5.12.8.5-1	$A_{s\text{-}BW} = A_{s\text{-}SD}\left(\frac{2}{\beta + 1}\right)$	✓				

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Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
5.13.3.6	Shear in Slabs and Footings	5.12.8.6	Shear in Slabs and Footings	✓				
5.13.3.6.1	Critical Sections for Shear	5.12.8.6.1	Critical Sections for Shear	✓				
5.13.3.6.2	One-Way Action	5.12.8.6.2	One-Way Action	✓				
5.13.3.6.3	Two-Way Action	5.12.8.6.3	Two-Way Action		✓			
5.13.3.6.3-1	$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\lambda\sqrt{f'_c}\, b_o d_v \le 0.126\lambda\sqrt{f'_c}\, b_o d_v$	5.12.8.6.3-1	$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\lambda\sqrt{f'_c}\, b_o d_v \le 0.126\lambda\sqrt{f'_c}\, b_o d_v$	✓				
5.13.3.6.3-2	$V_n = V_c + V_s \le 0.192\lambda\sqrt{f'_c}\; b_o d_v$	5.12.8.6.3-2	$V_n = V_c + V_s \le 0.192\lambda\sqrt{f'_c}\; b_o d_v$	✓				
5.13.3.6.3-3	$V_c = 0.0632\,\lambda\sqrt{f'_c}\, b_o d_v$	5.12.8.6.3-3	$V_c = 0.0632\,\lambda\sqrt{f'_c}\, b_o d_v$	✓				
5.13.3.6.3-4	$V_s = \frac{A_v f_y d_v}{s}$	5.12.8.6.3-4	$V_s = \frac{A_v f_y d_v}{s}$	✓				
5.13.3.7	Development of Reinforcement	5.12.8.7	Development of Reinforcement	✓				
5.13.3.8	Transfer of Force at Base of Column	5.12.8.8	Transfer of Force at Base of Column	✓				
5.13.4	Concrete Piles	5.12.9	Concrete Piles	✓				
5.13.4.1	General	5.12.9.1	General	✓				
5.13.4.2	Splices	5.12.9.2	Splices	✓				
5.13.4.3	Precast Reinforced Piles	5.12.9.3	Precast Reinforced Piles	✓				
5.13.4.3.1	Pile Dimensions	5.12.9.3.1	Pile Dimensions	✓				
5.13.4.3.2	Reinforcing Steel	5.12.9.3.2	Reinforcement		✓			
5.13.4.4	Precast Prestressed Piles	5.12.9.4	Precast Prestressed Piles	✓				
5.13.4.4.1	Pile Dimensions	5.12.9.4.1	Pile Dimensions	✓				
5.13.4.4.2	Concrete Quality	5.12.9.4.2	Concrete Quality	✓				
5.13.4.4.3	Reinforcement	5.12.9.4.3	Reinforcement	✓				
5.13.4.5	Cast-in-Place Piles	5.12.9.5	Cast-in-Place Piles	✓				
5.13.4.5.1	Pile Dimensions	5.12.9.5.1	Pile Dimensions	✓				
5.13.4.5.2	Reinforcing Steel	5.12.9.5.2	Reinforcement		✓			
5.13.4.6	Seismic Requirements	5.11	Seismic Design and Details					

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5.13.4.6.1	Zone 1	N/A						
5.13.4.6.2	Zone 2	5.11.3.2	Concrete Piles					
5.13.4.6.2a	General	5.11.3.2.1	General	✓				
5.13.4.6.2b	Cast in Place Piles	5.11.3.2.2	Cast in Place Piles		✓			
5.13.4.6.2c	Precast Reinforced Piles	5.11.3.2.3	Precast Reinforced Piles	✓				
5.13.4.6.2d	Precast Prestressed Piles	5.11.3.2.4	Precast Prestressed Piles	✓				
5.13.4.6.3	Zones 3 and 4	5.11.4.5	Concrete Piles					
5.13.4.6.3a	General	5.11.4.5.1	General	✓				
5.13.4.6.3b	Confinement Length	5.11.4.5.2	Confinement Length	✓				
5.13.4.6.3c	Volumetric Ratio for Confinement	5.11.4.5.3	Volumetric Ratio for Confinement	✓				
5.13.4.6.3d	Cast in Place Piles	5.11.4.5.4	Cast in Place Piles	✓				
5.13.4.6.3e	Precast Piles	5.11.4.5.5	Precast Piles		✓			
5.14	**Provisions for Structure Types**	**5.12**	**Provisions for Structure Components and Types**		✓			
5.14.1	Beams and Girders	5.12.3	Beams and Girders	✓				
5.14.1.1	General	5.12.3.1	General	✓				
5.14.1.2	Precast Beams	5.12.3.2	Precast Beams	✓				
5.14.1.2.1	Preservice Conditions	5.12.3.2.1	Preservice Conditions	✓				
5.14.1.2.2	Extreme Dimensions	5.12.3.2.2	Extreme Dimensions	✓				
5.14.1.2.3	Lifting Devices	5.12.3.2.3	Lifting Devices		✓			
5.14.1.2.4	Detail Design	5.12.3.2.4	Detail Design	✓				
5.14.1.2.5	Concrete Strength	5.12.3.2.5	Concrete Strength		✓			
5.14.1.3	Spliced Precast Girders	5.12.3.4	Spliced Precast Girders	✓				
5.14.1.3.1	General	5.12.3.4.1	General	✓				
5.14.1.3.2	Joints between Segments	5.12.3.4.2	Joints between Spliced Girders		✓			
5.14.1.3.2a	General	5.12.3.4.2a	General	✓				
5.14.1.3.2b	Details of Closure Joints	5.12.3.4.2b	Details of Closure Joints	✓				

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5.14.1.3.2c	Details of Match-Cast Joints	5.12.3.4.2c	Details of Match-Cast Joints	✓				
5.14.1.3.2d	Joint Design	5.12.3.4.2d	Joint Design	✓				
5.14.1.3.3	Girder Segment Design	5.12.3.4.3	Girder Segment Design	✓				
5.14.1.3.4	Post-Tensioning	5.12.3.4.4	Post-Tensioning		✓			
5.14.1.4	Bridges Composed of Simple Span Precast Girders Made Continuous	5.12.3.3	Bridges Composed of Simple Span Precast Girders Made Continuous	✓				
5.14.1.4.1	General	5.12.3.3.1	General		✓			
5.14.1.4.2	Restraint Moments	5.12.3.3.2	Restraint Moments		✓			
5.14.1.4.3	Material Properties	5.12.3.3.3	Material Properties		✓			
C5.14.1.4.3-1	$\varepsilon_{effective} = \varepsilon_{sh}\left(\frac{A_c}{A_{tr}}\right)$	C5.12.3.3.3-1	$\varepsilon_{effective} = \varepsilon_{sh}\left(\frac{A_c}{A_{tr}}\right)$	✓				
5.14.1.4.4	Age of Girder When Continuity Is Established	5.12.3.3.4	Age of Girder When Continuity Is Established	✓				
5.14.1.4.5	Degree of Continuity at Various Limit States	5.12.3.3.5	Degree of Continuity at Various Limit States	✓				
5.14.1.4.6	Service Limit State	5.12.3.3.6	Service Limit State	✓				
5.14.1.4.7	Strength Limit State	5.12.3.3.7	Strength Limit State	✓				
5.14.1.4.8	Negative Moment Connections	5.12.3.3.8	Negative Moment Connections	✓				
5.14.1.4.9	Positive Moment Connections	5.12.3.3.9	Positive Moment Connections	✓				
5.14.1.4.9a	General	5.12.3.3.9a	General		✓			
5.14.1.4.9b	Positive Moment Connection Using Mild Reinforcement	5.12.3.3.9b	Positive Moment Connection Using Nonprestressed Reinforcement		✓			
5.14.1.4.9c	Positive Moment Connection Using Prestressing Strand	5.12.3.3.9c	Positive Moment Connection Using Prestressing Strand	✓				
5.14.1.4.9c-1	$f_{psl} = \frac{(\ell_{dsh} - 8)}{0.228}$	5.12.3.3.9c-1	$f_{psl} = \frac{(\ell_{dsh} - 8)}{0.228}$	✓				
5.14.1.4.9c-2	$f_{pul} = \frac{(\ell_{dsh} - 8)}{0.163}$	5.12.3.3.9c-2	$f_{pul} = \frac{(\ell_{dsh} - 8)}{0.163}$	✓				
5.14.1.4.9d	Details of Positive Moment Connection	5.12.3.3.9d	Details of Positive Moment Connection	✓				
5.14.1.4.10	Continuity Diaphragms	5.12.3.3.10	Continuity Diaphragms	✓				
5.14.1.5	Cast-in-Place Girders and Box and T-Beams	5.12.3.5	Cast-in-Place Girders and Box and T-Beams	✓				

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5.14.1.5.1	Flange and Web Thickness	5.12.3.5.1	Flange and Web Thickness	✓				
5.14.1.5.1a	Top Flange	5.12.3.5.1a	Top Flange		✓			
5.14.1.5.1b	Bottom Flange	5.12.3.5.1b	Bottom Flange		✓			
5.14.1.5.1c	Web	5.12.3.5.1c	Web		✓			
5.14.1.5.2	Reinforcement	5.12.3.5.2	Reinforcement	✓				
5.14.1.5.2a	Deck Slab Reinforcement Cast-in-Place in T-Beams and Box Girders	5.12.3.5.2a	Deck Slab Reinforcement Cast-in-Place in T-Beams and Box Girders	✓				
5.14.1.5.2b	Bottom Slab Reinforcement in Cast-in-Place Box Girders	5.12.3.5.2b	Bottom Slab Reinforcement in Cast-in-Place Box Girders	✓				
5.14.2	Segmental Construction	5.12.5	Segmental Concrete Bridges		✓			
5.14.2.1	General	5.12.5.1	General			✓		
5.14.2.2	Analysis of Segmental Bridges	5.12.5.2	Analysis of Segmental Bridges	✓				
5.14.2.2.1	General	5.12.5.2.1	General	✓				
5.14.2.2.2	Construction Analysis	5.12.5.2.2	Construction Analysis	✓				
5.14.2.2.3	Analysis of the Final Structural System	5.12.5.2.3	Analysis of the Final Structural System		✓			
5.14.2.3	Design	5.12.5.3	Design	✓				
5.14.2.3.1	Loads	5.12.5.3.1	Loads	✓				
5.14.2.3.2	Construction Loads	5.12.5.3.2	Construction Loads		✓			
5.14.2.3.3	Construction Load Combinations at the Service Limit State	5.12.5.3.3	Construction Load Combinations at the Service Limit State			✓		
5.14.2.3.4	Construction Load Combinations at Strength Limit States	5.12.5.3.4	Construction Load Combinations at Strength Limit States	✓				
5.14.2.3.4a	Superstructures	5.12.5.3.4a	Superstructure Load Effects and Structural Stability	✓				
5.14.2.3.4a-1	$\Sigma\gamma Q = 1.1(DC + DIFF) + 1.3(CEQ + CLL) + A + AI$	5.12.5.3.4a-1	$\Sigma\gamma Q = 1.1(DC + DIFF) + 1.3(CEQ + CLL) + A + AI$	✓				
5.14.2.3.4a-2	$\Sigma\gamma Q = DC + CEQ + A + AI$	5.12.5.3.4a-2	$\Sigma\gamma Q = DC + CEQ + A + AI$	✓				
5.14.2.3.4b	Substructures	5.12.5.3.4b	Substructures		✓			
5.14.2.3.5	Thermal Effects During Construction	5.12.5.3.5	Thermal Effects During Construction	✓				
5.14.2.3.6	Creep and Shrinkage	5.12.5.3.6	Creep and Shrinkage	✓				
C5.14.2.3.6-1	$E_{eff} = \dfrac{E_c}{\psi(t,t_i) + 1}$	C5.12.5.3.6-1	$E_{eff} = \dfrac{E_c}{\psi(t,t_i) + 1}$	✓				

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5.14.2.3.7	Prestress Losses	5.12.5.3.7	Prestress Losses	✓				
5.14.2.3.8	Provisional Post-Tensioning Ducts and Anchorages	5.12.5.3.9	Provisional Post-Tensioning Ducts and Anchorages	✓				
5.14.2.3.8a	General	5.12.5.3.9a	General	✓				
5.14.2.3.8b	Bridges with Internal Ducts	5.12.5.3.9b	Bridges with Internal Ducts	✓				
5.14.2.3.8c	Provision for Future Dead Load or Deflection Adjustment	5.12.5.3.9c	Provision for Future Dead Load or Deflection Adjustment	✓				
5.14.2.3.9	Plan Presentation	5.12.5.3.10	Plan Presentation	✓				
5.14.2.3.10	Box Girder Cross-Section Dimensions and Details	5.12.5.3.11	Box Girder Cross-Section Dimensions and Details	✓				
5.14.2.3.10a	Minimum Flange Thickness	5.12.5.3.11a	Minimum Flange Thickness			✓		
5.14.2.3.10b	Minimum Web Thickness	5.12.5.3.11b	Minimum Web Thickness	✓				
5.14.2.3.10c	Length of Top Flange Cantilever	5.12.5.3.11c	Length of Top Flange Cantilever	✓				
5.14.2.3.10d	Overall Cross-Section Dimensions	5.12.5.3.11d	Overall Cross-Section Dimensions	✓				
5.14.2.3.10e	Overlays	5.14.5	Deck Protection Systems			✓		
5.14.2.3.11	Seismic Design	5.12.5.3.12	Seismic Design	✓				
5.14.2.4	Types of Segmental Bridges	5.12.5.4	Types of Segmental Bridges	✓				
5.14.2.4.1	General	5.12.5.4.1	General	✓				
5.14.2.4.2	Details for Precast Construction	5.12.5.4.2	Details for Precast Construction	✓				
5.14.2.4.3	Details for Cast-in-Place Construction	5.12.5.4.3	Details for Cast-in-Place Construction	✓				
5.14.2.4.4	Cantilever Construction	5.12.5.4.4	Cantilever Construction			✓		
5.14.2.4.5	Span-by-Span Construction	5.12.5.4.5	Span-by-Span Construction	✓				
5.14.2.4.6	Incrementally Launched Construction	5.12.5.4.6	Incrementally Launched Construction	✓				
5.14.2.4.6a	General	5.12.5.4.6a	General	✓				
5.14.2.4.6b	Force Effects Due to Construction Tolerances	5.12.5.4.6b	Force Effects Due to Construction Tolerances	✓				
5.14.2.4.6c	Design Details	5.12.5.4.6c	Design Details	✓				
5.14.2.4.6d	Design of Construction Equipment	5.12.5.4.6d	Design of Construction Equipment	✓				
5.14.2.5	Use of Alternative Construction Methods	5.12.5.5	Use of Alternative Construction Methods	✓				
5.14.2.6	Segmentally Constructed Bridge Substructures	5.12.5.6	Segmentally Constructed Bridge Substructures	✓				
5.14.2.6.1	General	5.12.5.6.1	General	✓				

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5.14.2.6.2	Construction Load Combinations	5.12.5.6.2	Construction Load Combinations	✓				
5.14.2.6.3	Longitudinal Reinforcement of Hollow, Rectangular Precast Segmental Piers	5.12.5.6.3	Longitudinal Reinforcement of Hollow, Rectangular Precast Segmental Piers	✓				
5.14.3	Arches	5.12.6	Arches	✓				
5.14.3.1	General	5.12.6.1	General	✓				
5.14.3.2	Arch Ribs	5.12.6.2	Arch Ribs	✓				
5.14.4	Slab Superstructures	5.12.2	Slab Superstructures	✓				
5.14.4.1	Cast-in-Place Solid Slab Superstructures	5.12.2.1	Cast-in-Place Solid Slab Superstructures	✓				
5.14.4.1-1	$\frac{100}{\sqrt{L}} \leq 50\%$	5.12.2.1-1	$\frac{100}{\sqrt{L}} \leq 50\%$	✓				
5.14.4.1-2	$\frac{100}{\sqrt{L}} \frac{f_{pe}}{60} \leq 50\%$	5.12.2.1-2	$\frac{100}{\sqrt{L}} \frac{f_{pe}}{60} \leq 50\%$	✓				
5.14.4.2	Cast-in-Place Voided Slab Superstructures	5.12.2.2	Cast-in-Place Voided Slab Superstructures	✓				
5.14.4.2.1	Cross-Section Dimensions	5.12.2.2.1	Cross-Section Dimensions	✓				
5.14.4.2.2	Minimum Number of Bearings	5.12.2.2.2	Minimum Number of Bearings	✓				
5.14.4.2.3	Solid End Sections	5.12.2.2.3	Solid End Sections	✓				
5.14.4.2.4	General Design Requirements	5.12.2.2.4	General Design Requirements	✓				
5.14.4.2.5	Compressive Zones in Negative Moment Area	5.12.2.2.5	Compressive Zones in Negative Moment Area	✓				
5.14.4.2.6	Drainage of Voids	5.12.2.2.6	Drainage of Voids	✓				
5.14.4.3	Precast Deck Bridges	5.12.2.3	Precast Deck Bridges	✓				
5.14.4.3.1	General	5.12.2.3.1	General	✓				
5.14.4.3.2	Shear Transfer Joints	5.12.2.3.2	Shear Transfer Joints	✓				
5.14.4.3.3	Shear-Flexure Transfer Joints	5.12.2.3.3	Shear-Flexure Transfer Joints	✓				
5.14.4.3.3a	General	5.12.2.3.3a	General	✓				
5.14.4.3.3b	Design	5.12.2.3.3b	Design	✓				
5.14.4.3.3c	Post-Tensioning	5.12.2.3.3c	Post-Tensioning	✓				
5.14.4.3.3d	Longitudinal Construction Joints	5.12.2.3.3d	Longitudinal Construction Joints	✓				

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5.14.4.3.3e	Cast-in-Place Closure Joint	5.12.2.3.3e	Cast-in-Place Closure Joints	✓				
5.14.4.3.3f	Structural Overlay	5.12.2.3.3f	Structural Overlay	✓				
5.14.5	Additional Provisions for Culverts	5.12.7	Culverts		✓			
5.14.5.1	General	5.12.7.1	General	✓				
5.14.5.2	Design for Flexure	5.12.7.2	Design for Flexure	✓				
5.14.5.3	Design for Shear in Slabs of Box Culverts	5.12.7.3	Design for Shear in Slabs of Box Culverts		✓			
5.14.5.3-1	$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\dfrac{A_s}{bd_e}\dfrac{V_u d_e}{M_u}\right)bd_e$	5.12.7.3-1	$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\dfrac{A_s}{bd_e}\dfrac{V_u d_e}{M_u}\right)bd_e$	✓				
N/A		5.12.7.3-2	$V_c \leq 0.126\lambda\sqrt{f'_c}\,bd_e$				✓	
5.15	**References**	**5.15**	**References**			✓		
Appendix A5	**Basic Steps for Concrete Bridges**	**Appendix A5**	**Basic Steps for Concrete Bridges**	✓				
A5.1	General	A5.1	General	✓				
A5.2	General Considerations	A5.2	General Considerations	✓				
A5.3	Beam and Girder Superstructure Design	A5.3	Beam and Girder Superstructure Design	✓				
A5.4	Slab Bridges	A5.4	Slab Bridges	✓				
A5.5	Substructure Design	A5.5	Substructure Design	✓				
Appendix B5	**General Procedure for Shear Design with Tables**	**Appendix B5**	**General Procedure for Shear Design with Tables**	✓				
B5.1	Background	B5.1	Background	✓				
B5.2	Sectional Design Model—General Procedure	B5.2	Sectional Design Model—General Procedure			✓		
5.8.2.1-6	$\sqrt{V_u^2 + \left(\dfrac{0.9 p_h T_u}{2A_o}\right)^2}$	B5.2-1	$V_{eff} = \sqrt{V_u^2 + \left(\dfrac{0.9 p_h T_u}{2A_o}\right)^2}$				✓	
5.8.2.1-7	$V_u + \dfrac{T_u d_s}{2A_o}$	B5.2-2	$V_{eff} = V_u + \dfrac{T_u d_s}{2A_o}$				✓	

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B5.2-1	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{2(E_s A_s + E_p A_{ps})}$	B5.2-3	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{2(E_s A_s + E_p A_{ps})}$	✓				
B5.2-2	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$	B5.2-4	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$	✓				
B5.2-3	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{2\left(E_c A_c + E_s A_s + E_p A_{ps}\right)}$	B5.2-5	$\varepsilon_s = \dfrac{\left(\dfrac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert \cot\theta - A_{ps} f_{po}\right)}{2\left(E_c A_c + E_s A_s + E_p A_{ps}\right)}$	✓				
B5.2-4	$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} \leq 80 \text{ in.}$	B5.2-6	$s_{xe} = s_x \dfrac{1.38}{a_g + 0.63} \leq 80 \text{ in.}$	✓				
CB5.2-1	$\varepsilon_x = \dfrac{\varepsilon_t + \varepsilon_c}{2}$	CB5.2-1	$\varepsilon_x = \dfrac{\varepsilon_t + \varepsilon_c}{2}$	✓				
Appendix C5	**Upper Limits for Articles Affected by Concrete Compressive Strength**	**Appendix C5**	**Upper Limits for Articles Affected by Concrete Compressive Strength**					
Appendix D5	**Articles Modified to Allow the Use of Reinforcement with a Specified Minimum Yield Strength Up to 100 ksi**	**Appendix D5**	**Articles Modified to Allow the Use of Reinforcement with a Specified Minimum Yield Strength Up to 100 ksi**					
N/A		**5.13**	**Anchors**				✓	
N/A		5.13.1	General				✓	
N/A		5.13.2	General Strength Requirements				✓	
N/A		5.13.2.1	Failure Modes to be Considered				✓	
N/A		5.13.2.2	Resistance Factors				✓	
N/A		5.13.2.3	Determination of Anchor Resistance				✓	

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Article or Equation		Article or Equation		Unchanged	Editorial	Updated	New	Removed
N/A		5.13.3	Seismic Design Requirements				✓	
N/A		5.13.4	Installation				✓	

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