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SECTION 5

CONCRETE STRUCTURES

5.1—SCOPE

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

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

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

5.2—DEFINITIONS

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

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

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

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

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

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

Blanketed Strand—See *Partially Debonded Strand*.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Partially Prestressed Component—See *Partially Prestressed Concrete*.

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

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

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

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

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

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

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

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

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

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

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

Reinforcement—Reinforcing bars and/or prestressing steel.

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

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

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

Shielded Strand—See *Partially Debonded Strand*.

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

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

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

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

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

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

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

Structural Concrete—All concrete used for structural purposes.

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

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

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

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

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

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

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

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

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

Yield Strength—The specified yield strength of reinforcement.

5.3—NOTATION

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

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

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

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

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

s = average spacing of mild steel reinforcement in layer closest to tension face (in.); spacing of reinforcing bars (in.); spacing of rows of ties (in.); anchorage spacing (in.); center-to-center spacing of anchorages (in.); spacing of hanger reinforcing bars (in.) (5.7.3.4) (5.8.2.5) (5.8.4.1) (5.10.9.3.6) (5.10.9.6.2) (5.13.2.5.5)
s_{max} = maximum permitted spacing of transverse reinforcement (in.) (5.8.2.7)
s_w = spacing of wires to be developed or spliced (in.) (5.11.2.5.1)
s_x = crack spacing parameter (in.) (C5.8.3.4.2)
s_{xe} = equivalent value of s_x which allows for influence of aggregate size (in.) (5.8.3.4.2)
T_{burst} = tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis (kip) (5.10.9.6.3)
T_{cr} = torsional cracking resistance (kip-in.) (5.8.2.1)
T_{ia} = tie-back tension force at the intermediate anchorage (kip) (5.10.9.3.4b)
T_n = nominal torsion resistance (kip-in.) (5.8.2.1)
T_r = factored torsional resistance provided by circulatory shear flow (kip-in.) (5.8.2.1)
T_u = factored torsional moment (kip-in.) (C5.6.3.1)
T_1 = edge tension force (kip) (5.10.9.3.6)
T_2 = bursting force (kip) (5.10.9.3.6)
t = time (day); thickness of wall (in.); thickness of the section (in.); average thickness of bearing plate (in.) (5.4.2.3.2) (5.7.4.7.1) (5.10.9.6.2) (5.10.9.7.2)
t_d = age at deck placement (day) (5.9.5.4.2b)
t_f = final age (day) (5.9.5.4.2a)
t_i = age of concrete when load is initially applied (day) (5.4.2.3.2)
U = for segmental construction: segment unbalance (kip) (5.14.2.3.2)
V_c = nominal shear resistance provided by tensile stresses in the concrete (kip) (5.8.2.4)
V_n = nominal shear resistance of the section considered (kip) (5.8.2.1)
V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip) (C5.8.2.3)
V_r = factored shear resistance (kip) (5.8.2.1)
V/S = volume-to-surface ratio (5.4.2.3.2)
V_s = shear resistance provided by shear reinforcement (kip) (5.8.3.3)
V_u = factored shear force at section (kip) (C5.6.3.1)
v_u = average factored shear stress on the concrete (ksi) (5.8.2.7) (5.8.2.9)
W = width of bearing plate measured along the length of a corbel, bracket, or beam ledge (in.) (C5.13.2.5.1)
W/C = water–cement ratio (5.12.3)
WE = for segmental construction: horizontal wind load on equipment (kip) (5.14.2.3.2)
WUP = for segmental construction: wind uplift on cantilever (ksf) (5.14.2.3.2)
w_c = unit weight of concrete (kcf) (5.4.2.4)
X_u = clear length of the constant thickness portion of a wall between other walls or fillers between walls (in.) (5.7.4.7.1)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft) (5.9.5.2.2b)
y_t = distance from the neutral axis to the extreme tension fiber (in.) (5.7.3.6.2)
α = angle of inclination of transverse reinforcement to longitudinal axis (degrees); total angular change of prestressing steel path from jacking end to a point under investigation (rad.); the angle of inclination of a tendon force with respect to the centerline of the member (degrees) (5.8.3.3) (5.9.5.2.2b) (5.10.9.6.3)
α_h = total horizontal angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.5.2.2b)
α_s = angle between compressive strut and adjoining tension tie (degrees) (5.6.3.3.3)
α_v = total vertical angular change of prestressing steel path from jacking end to a point under investigation (rad.) (5.9.5.2.2b)
β = factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension; ratio of long side to short side of footing (5.8.3.3) (5.13.3.5)
β_b = ratio of the area of reinforcement cut off to the total area of tension reinforcement at the section (5.11.1.2.1)
β_c = ratio of the long side to the short side of the concentrated load or reaction area (5.13.3.6.3)
β_d = ratio of maximum factored dead load moments to maximum factored total load moment; always positive (5.7.4.3)
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone (5.7.2.2)

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

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

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

5.4—MATERIAL PROPERTIES

5.4.1—General

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

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

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

C5.4.1

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

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

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

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

5.4.2—Normal Weight and Structural Lightweight Concrete

5.4.2.1—Compressive Strength

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

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

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

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

C5.4.2.1

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

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

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

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

These classes are intended for use as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

5.4.2.2—Coefficient of Thermal Expansion

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

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

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

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 $6.0 \times 10^{-6}/°F$ and depend on the amount of natural sand used.

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

5.4.2.3—Shrinkage and Creep

5.4.2.3.1—General

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

C5.4.2.3.1

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

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

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

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

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

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

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

5.4.2.3.2—Creep

The creep coefficient may be taken as:

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

in which:

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

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

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

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

where:

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

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

k_f = factor for the effect of concrete strength

k_{hc} = humidity factor for creep

k_{td} = time development factor

C5.4.2.3.2

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

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

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

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

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

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

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

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

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

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

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

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

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

For creep:

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

For shrinkage:

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

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:

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

in which:

C5.4.2.3.3

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

Shrinkage is affected by:

- Aggregate characteristics and proportions,

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

where:

k_{hs} = humidity factor for shrinkage

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

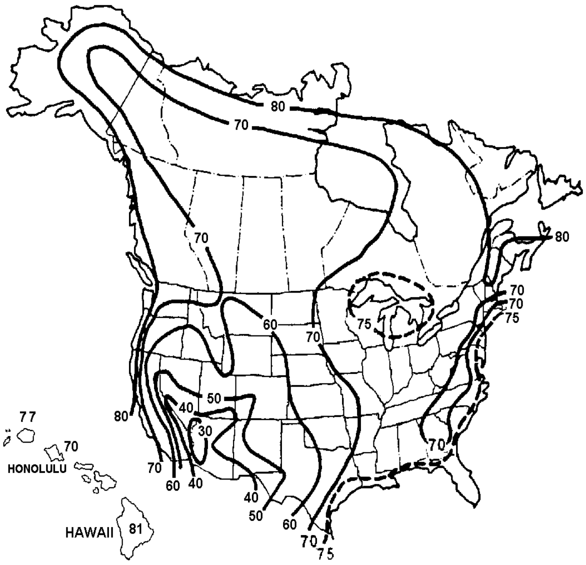


Figure 5.4.2.3.3-1—Annual Average Ambient Relative Humidity in Percent

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

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

5.4.2.4—Modulus of Elasticity

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

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

where:

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

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

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

C5.4.2.4

See commentary for specified strength in Article 5.4.2.1.

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

$$E_c = 1{,}820\sqrt{f'_c} \tag{C5.4.2.4-1}$$

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

5.4.2.5—Poisson's Ratio

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

C5.4.2.5

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

5.4.2.6—Modulus of Rupture

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

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

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

C5.4.2.6

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

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

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

5.4.2.7—Tensile Strength

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

C5.4.2.7

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

5.4.3—Reinforcing Steel

5.4.3.1—General

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

C5.4.3.1

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

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

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

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

5.4.3.2—Modulus of Elasticity

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

5.4.3.3—Special Applications

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

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

C5.4.3.3

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

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

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

5.4.4—Prestressing Steel

5.4.4.1—General

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

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

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

C5.4.4.1

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

Table 5.4.4.1-1—Properties of Prestressing Strand and Bar

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

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

5.4.4.2—Modulus of Elasticity

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

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

C5.4.4.2

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

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

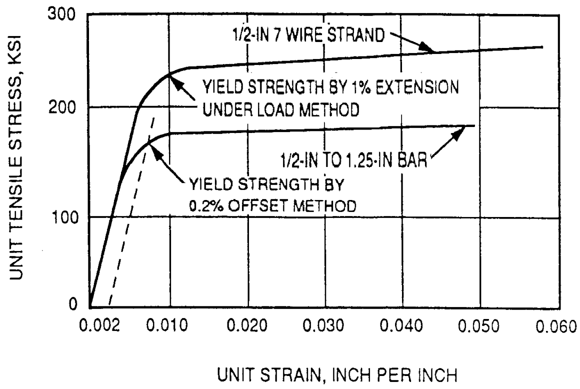


Figure C5.4.4.2-1—Typical Stress-Strain Curve for Prestressing Steels

5.4.5—Post-Tensioning Anchorages and Couplers

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

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

C5.4.5

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

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

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

5.4.6—Ducts

5.4.6.1—General

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

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

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

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

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

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

C5.4.6.1

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

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

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

5.4.6.2—Size of Ducts

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

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

C5.4.6.2

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

5.4.6.3—Ducts at Deviation Saddles

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

5.5—LIMIT STATES

5.5.1—General

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

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

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

5.5.2—Service Limit State

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

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

5.5.3—Fatigue Limit State

5.5.3.1—General

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

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

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

For fatigue considerations, concrete members shall satisfy:

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

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)

C5.5.3.1

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

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

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

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

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

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

5.5.3.2—Reinforcing Bars

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

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

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

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

where:

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

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

C5.5.3.2

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

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

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

5.5.3.3—Prestressing Tendons

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

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

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

C5.5.3.3

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

5.5.3.4—Welded or Mechanical Splices of Reinforcement

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

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

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

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

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

C5.5.3.4

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

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

5.5.4—Strength Limit State

5.5.4.1—General

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

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

C5.5.4.1

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

5.5.4.2—Resistance Factors

5.5.4.2.1—Conventional Construction

Resistance factor ϕ shall be taken as:

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

C5.5.4.2.1

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

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

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

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

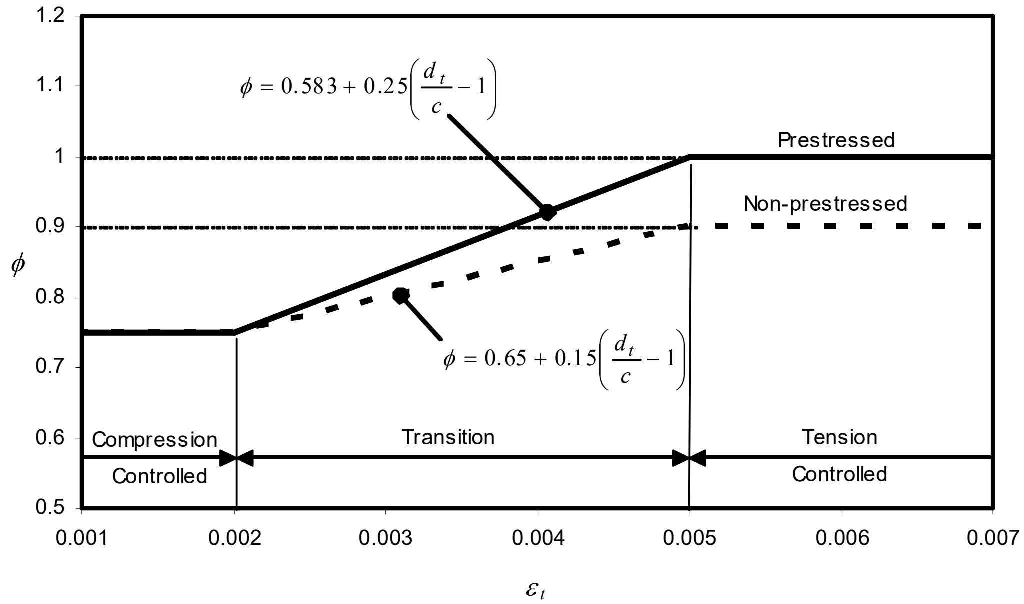


Figure C5.5.4.2.1-1—Variation of ϕ with Net Tensile Strain ε_t and d_t/c for Grade 60 Reinforcement and for Prestressing Steel

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

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

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

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

and for nonprestressed members such that:

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

where:

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

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

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

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

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

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

in which:

$$PPR = \frac{A_{ps} f_{py}}{A_{ps} f_{py} + A_s f_y} \tag{5.5.4.2.1-4}$$

where:

PPR = partial prestress ratio

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

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

f_y = specified yield strength of reinforcing bars (ksi)

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

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

5.5.4.2.2—Segmental Construction

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

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

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

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

C5.5.4.2.2

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

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

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

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

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

5.5.4.3—Stability

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

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

5.5.5—Extreme Event Limit State

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

5.6—DESIGN CONSIDERATIONS

5.6.1—General

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

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

C5.6.1

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

5.6.2—Effects of Imposed Deformation

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

C5.6.2

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

5.6.3—Strut-and-Tie Model

5.6.3.1—General

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

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

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

C5.6.3.1

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

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

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

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

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

5.6.3.2—Structural Modeling

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

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

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

where:

C5.6.3.2

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

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

P_n = nominal resistance of strut or tie (kip)

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

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

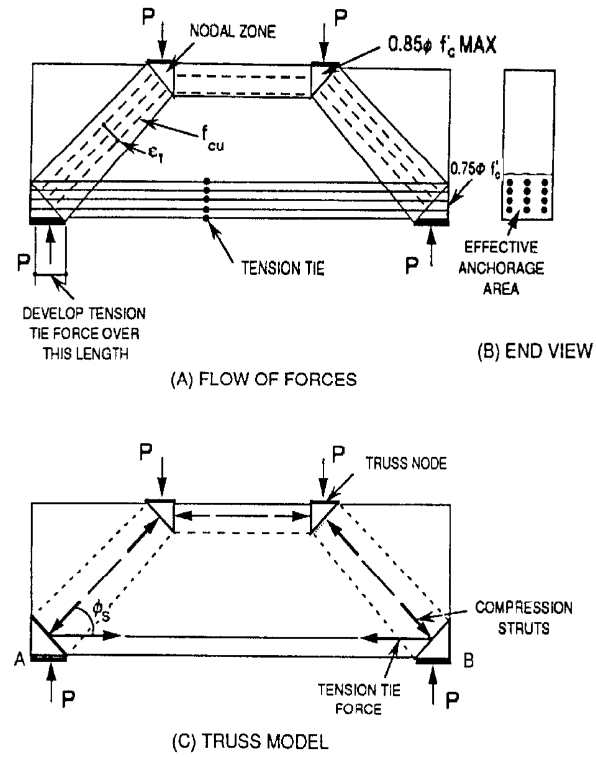


Figure C5.6.3.2-1—Strut-and-Tie Model for a Deep Beam

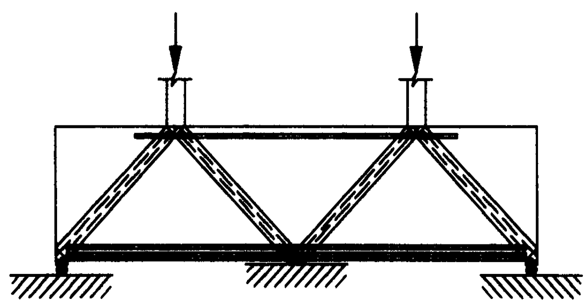


Figure C5.6.3.2-2—Strut-and-Tie Model for Continuous Deep Beam

5.6.3.3—Proportioning of Compressive Struts

5.6.3.3.1—Strength of Unreinforced Strut

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

$$P_n = f_{cu} A_{cs} \tag{5.6.3.3.1-1}$$

where:

P_n = nominal resistance of a compressive strut (kip)

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

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

5.6.3.3.2—Effective Cross-Sectional Area of Strut

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

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

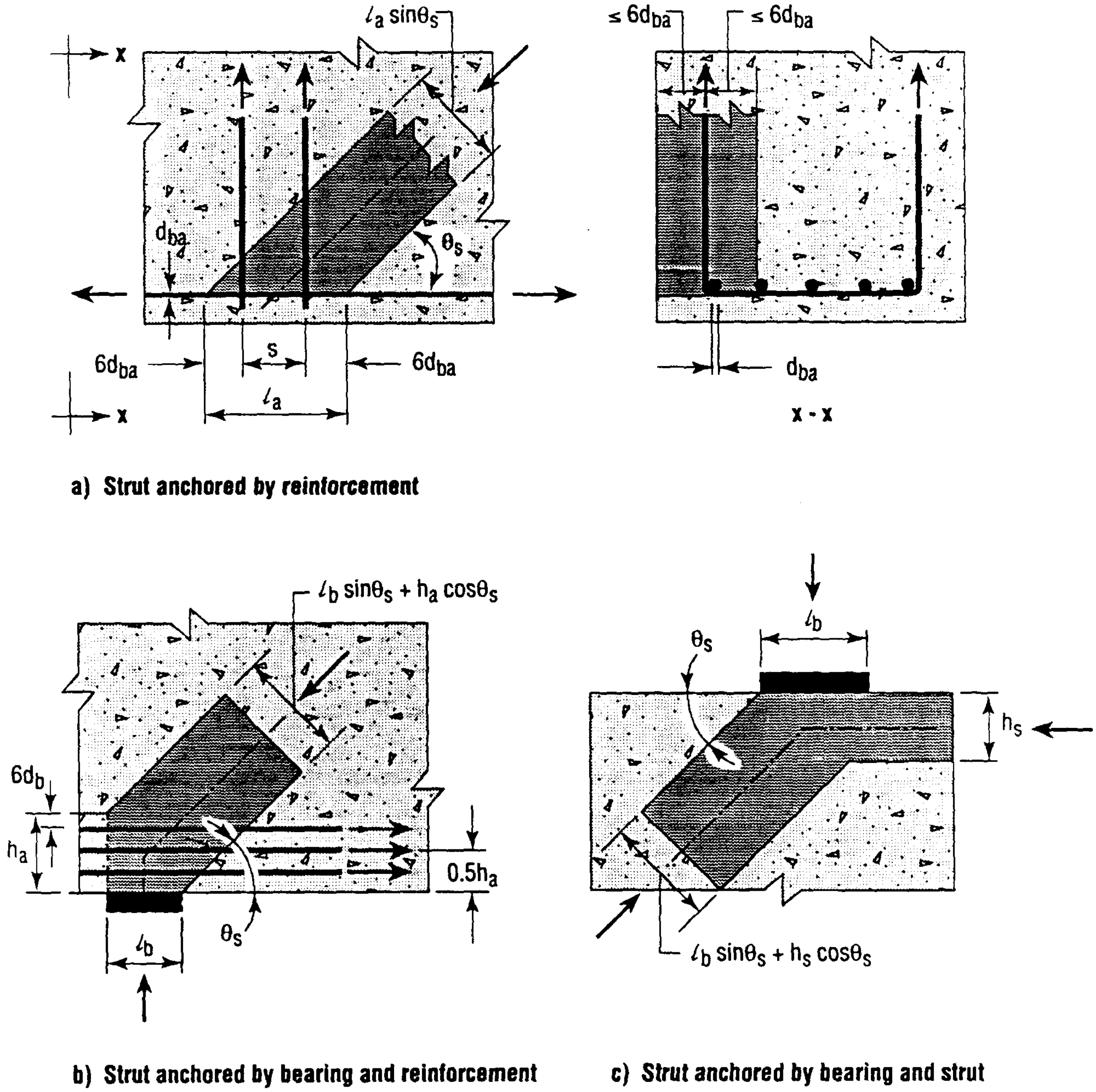


Figure 5.6.3.3.2-1—Influence of Anchorage Conditions on Effective Cross-Sectional Area of Strut

5.6.3.3.3—Limiting Compressive Stress in Strut

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

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

in which:

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

C5.6.3.3.3

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

where:

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

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

f'_c = specified compressive strength (ksi)

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

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

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

5.6.3.3.4—Reinforced Strut

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

$$P_n = f_{cu} A_{cs} + f_y A_{ss} \quad (5.6.3.3.4\text{-}1)$$

where:

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

5.6.3.4—Proportioning of Tension Ties

5.6.3.4.1—Strength of Tie

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

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

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

where:

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

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

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

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

C5.6.3.4.1

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

5.6.3.4.2—Anchorage of Tie

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

5.6.3.5—Proportioning of Node Regions

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

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

where:

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

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

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

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

C5.6.3.5

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

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

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

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

5.6.3.6—Crack Control Reinforcement

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

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

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

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

C5.6.3.6

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

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

where:

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

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

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

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

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

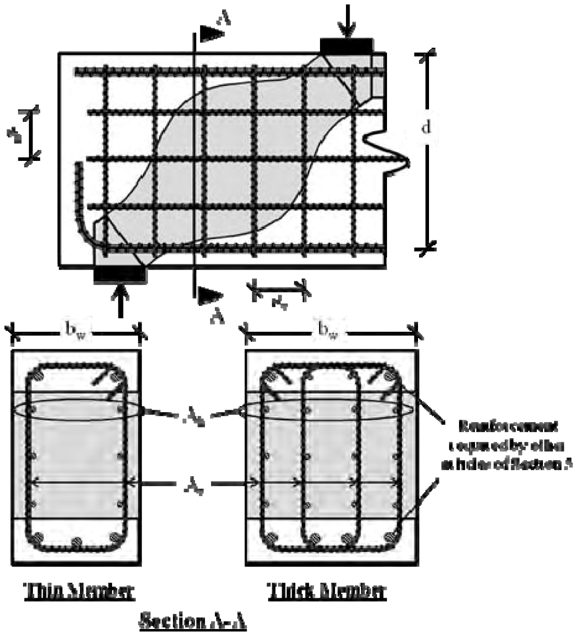


Figure C5.6.3.6-1—Distribution of Crack Control Reinforcement in Compression Strut

5.7—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.1—Assumptions for Service and Fatigue Limit States

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

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

C5.7.1

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

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

5.7.2—Assumptions for Strength and Extreme Event Limit States

5.7.2.1—General

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

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

C5.7.2.1

The first paragraph of C5.7.1 applies.

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

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

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

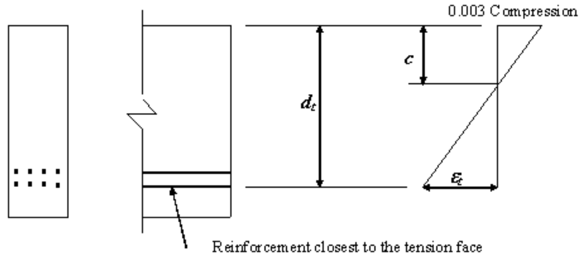


Figure C5.7.2.1-1—Strain Distribution and Net Tensile Strain

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

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

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

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

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

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

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

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

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

5.7.2.2—Rectangular Stress Distribution

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

C5.7.2.2

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

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

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

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

5.7.3—Flexural Members

5.7.3.1—Stress in Prestressing Steel at Nominal Flexural Resistance

5.7.3.1.1—Components with Bonded Tendons

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

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

in which:

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

for T-section behavior:

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

for rectangular section behavior:

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

where:

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

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

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

C5.7.3.1.1

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

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

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

Table C5.7.3.1.1-1—Values of *k*

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

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

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

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

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

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

b_w = width of web (in.)

h_f = depth of compression flange (in.)

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

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

β_1 = stress block factor specified in Article 5.7.2.2

5.7.3.1.2—Components with Unbonded Tendons

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

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

in which:

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

for T-section behavior:

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

for rectangular section behavior:

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

C5.7.3.1.2

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

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

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

where:

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

ℓ_e = effective tendon length (in.)

ℓ_i = length of tendon between anchorages (in.)

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

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

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

5.7.3.1.3—Components with Both Bonded and Unbonded Tendons

5.7.3.1.3a—Detailed Analysis

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

5.7.3.1.3b—Simplified Analysis

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

where:

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

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

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

5.7.3.2—Flexural Resistance

5.7.3.2.1—Factored Flexural Resistance

The factored resistance M_r shall be taken as:

$$M_r = \phi M_n \tag{5.7.3.2.1-1}$$

where:

M_n = nominal resistance (kip-in.)

ϕ = resistance factor as specified in Article 5.5.4.2

C5.7.3.2.1

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

5.7.3.2.2–Flanged Sections

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

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

where:

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

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

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

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

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

C5.7.3.2.2

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

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

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

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

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

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

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

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

β_1 = stress block factor specified in Article 5.7.2.2

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

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

5.7.3.2.3—Rectangular Sections

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

5.7.3.2.4—Other Cross-Sections

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

5.7.3.2.5—Strain Compatibility Approach

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

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

5.7.3.3—Limits for Reinforcement

5.7.3.3.1—Maximum Reinforcement

[PROVISION DELETED IN 2005]

C5.7.3.3.1

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

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

5.7.3.3.2—Minimum Reinforcement

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

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

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

where:

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

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

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

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

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

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

The provisions of Article 5.10.8 shall apply.

5.7.3.4—Control of Cracking by Distribution of Reinforcement

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

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

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

C5.7.3.4

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

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

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

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

in which:

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

where:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

where:

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

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

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

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

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

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

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

5.7.3.5—Moment Redistribution

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

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

C5.7.3.5

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

5.7.3.6—Deformations

5.7.3.6.1—General

The provisions of Article 2.5.2.6 shall be considered.

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

C5.7.3.6.1

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

5.7.3.6.2—Deflection and Camber

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

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

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

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

in which:

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

where:

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

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

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

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

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

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

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

where:

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

C5.7.3.6.2

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

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

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

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

5.7.3.6.3—Axial Deformation

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

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

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

5.7.4—Compression Members

5.7.4.1—General

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

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

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

where:

K = effective length factor specified in Article 4.6.2.5

ℓ_u = unbraced length (in.)

r = radius of gyration (in.)

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

C5.7.4.1

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

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

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

5.7.4.2—Limits for Reinforcement

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

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

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

and:

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

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

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

where:

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

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

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

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

f_y = specified yield strength of reinforcing bars (ksi)

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

f_{pe} = effective prestress (ksi)

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

C5.7.4.2

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

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

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

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

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

5.7.4.3—Approximate Evaluation of Slenderness Effects

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

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

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

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

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

C5.7.4.3

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

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

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

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

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

where:

E_c = modulus of elasticity of concrete (ksi)

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

E_s = modulus of elasticity of longitudinal steel (ksi)

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

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

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

5.7.4.4—Factored Axial Resistance

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

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

in which:

- For members with spiral reinforcement:

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

- For members with tie reinforcement:

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

where:

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

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

C5.7.4.4

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

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

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

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

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

f_y = specified yield strength of reinforcement (ksi)

ϕ = resistance factor specified in Article 5.5.4.2

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

E_p = modulus of elasticity of prestressing tendons (ksi)

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

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

5.7.4.5—Biaxial Flexure

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

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

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

in which:

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

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

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

where:

ϕ = resistance factor for members in axial compression

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

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

C5.7.4.5

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

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

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

P_u = factored applied axial force (kip)

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

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

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

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

P_o = nominal axial resistance of a section at 0.0 eccentricity

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

5.7.4.6—Spirals and Ties

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

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

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

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

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

where:

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

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

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

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

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

5.7.4.7—Hollow Rectangular Compression Members

5.7.4.7.1—Wall Slenderness Ratio

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

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

where:

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

t = thickness of wall (in.)

λ_w = wall slenderness ratio for hollow columns

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

C5.7.4.7.1

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

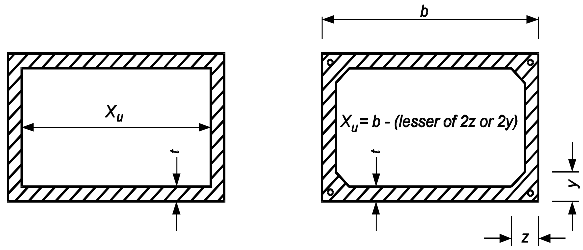


Figure C5.7.4.7.1-1—Illustration of X_u

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

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

5.7.4.7.2a—General

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

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

5.7.4.7.2b—Refined Method for Adjusting Maximum Usable Strain Limit

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

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

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

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

5.7.4.7.2c—Approximate Method for Adjusting Factored Resistance

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

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

- If $\lambda_w \leq 15$, then $\phi_w = 1.0$ (5.7.4.7.2c-1)
- If $15 < \lambda_w \leq 25$, then $\phi_w = 1 - 0.025(\lambda_w - 15)$ (5.7.4.7.2c-2)
- If $25 < \lambda_w \leq 35$, then $\phi_w = 0.75$ (5.7.4.7.2c-3)

5.7.5—Bearing

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

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

in which:

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

where:

P_n = nominal bearing resistance (kip)

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

m = modification factor

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

C5.7.5

The modification factor may be determined as follows:

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

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

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

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

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

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

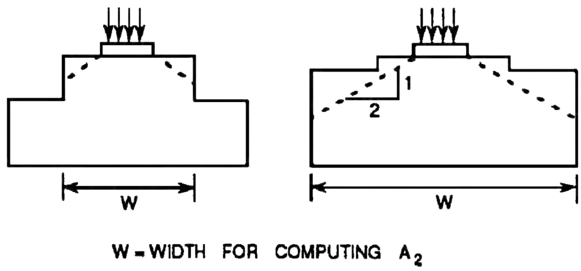


Figure C5.7.5-1—Determination of A_2 for a Stepped Support

5.7.6—Tension Members

5.7.6.1—Factored Tension Resistance

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

The factored resistance to uniform tension shall be taken as:

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

where:

P_n = nominal tension resistance specified in Article 5.6.3.4

ϕ = resistance factor specified in Article 5.5.4.2

5.7.6.2—Resistance to Combinations of Tension and Flexure

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

5.8—SHEAR AND TORSION

5.8.1—Design Procedures

5.8.1.1—Flexural Regions

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

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

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

C5.8.1.1

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

5.8.1.2—Regions Near Discontinuities

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

C5.8.1.2

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

5.8.1.3—Interface Regions

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

5.8.1.4—Slabs and Footings

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

5.8.2—General Requirements

5.8.2.1—General

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

$$T_r = \phi T_n \tag{5.8.2.1-1}$$

C5.8.2.1

where:

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

ϕ = resistance factor specified in Article 5.5.4.2

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

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

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

ϕ = resistance factor as specified in Article 5.5.4.2

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

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

in which:

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

where:

T_u = factored torsional moment (kip-in.)

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

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

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

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

ϕ = resistance factor specified in Article 5.5.4.2

For cellular structures:

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

where:

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

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

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

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

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

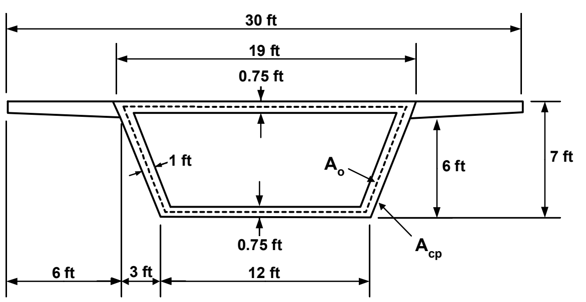


Figure C5.8.2.1-1—Sketch Showing Data Used in Sample Calculation for A_o Shown Below

For solid sections:

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

For box sections:

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

where:

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

T_u = factored torsional moment (kip-in.)

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

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

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

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

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

5.8.2.2—Modifications for Lightweight Concrete

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

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

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

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

C5.8.2.2

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

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

5.8.2.3—Transfer and Development Lengths

The provisions of Article 5.11.4 shall be considered.

C5.8.2.3

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

5.8.2.4—Regions Requiring Transverse Reinforcement

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

- $$V_u > 0.5\phi\left(V_c + V_p\right) \tag{5.8.2.4-1}$$

  or

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

where:

V_u = factored shear force (kip)

V_c = nominal shear resistance of the concrete (kip)

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

ϕ = resistance factor specified in Article 5.5.4.2

C5.8.2.4

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

5.8.2.5—Minimum Transverse Reinforcement

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

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

where:

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

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

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi)

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

C5.8.2.5

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

Additional transverse reinforcement may be required for transverse web bending.

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

where:

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

b_w = width of web (in.)

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi)

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

5.8.2.6—Types of Transverse Reinforcement

Transverse reinforcement to resist shear may consist of:

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

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

C5.8.2.6

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

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

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

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

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

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

5.8.2.7—Maximum Spacing of Transverse Reinforcement

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

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

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

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

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

where:

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

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

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

C5.8.2.7

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

5.8.2.8—Design and Detailing Requirements

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

C5.8.2.8

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

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

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

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

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

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

5.8.2.9—Shear Stress on Concrete

The shear stress on the concrete shall be determined as:

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

where:

ϕ = resistance factor for shear specified in Article 5.5.4.2

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

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

C5.8.2.9

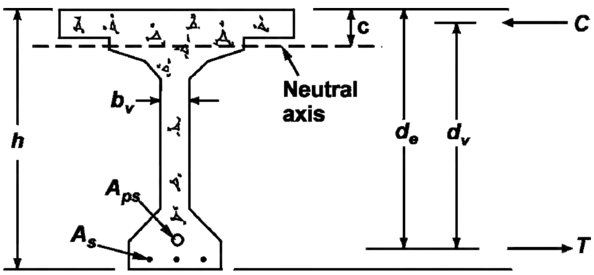


Figure C5.8.2.9-1—Illustration of the Terms b_v and d_v

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

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

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

in which:

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

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

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

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

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

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

where:

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

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

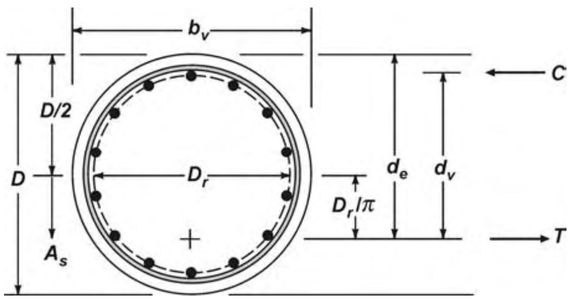


Figure C5.8.2.9-2—Illustration of Terms b_v, d_v, and d_e for Circular Sections

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

5.8.3—Sectional Design Model

5.8.3.1—General

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

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

C5.8.3.1

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

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

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

5.8.3.2—Sections Near Supports

The provisions of Article 5.8.1.2 shall be considered.

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

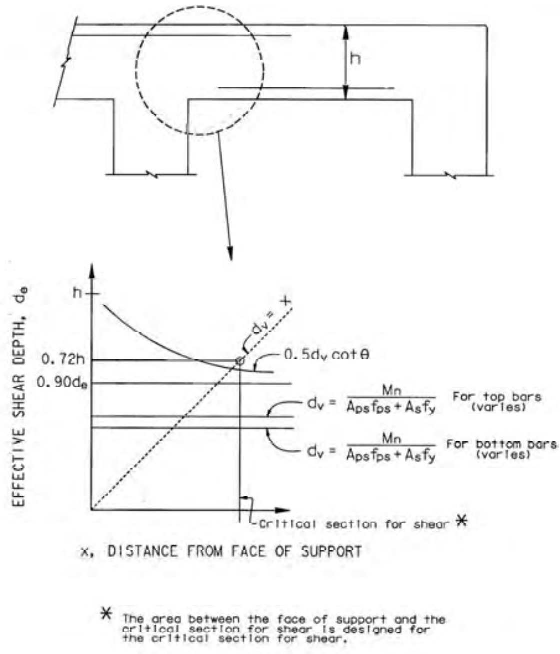


Figure 5.8.3.2-1—Critical Section for Shear

C5.8.3.2

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

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

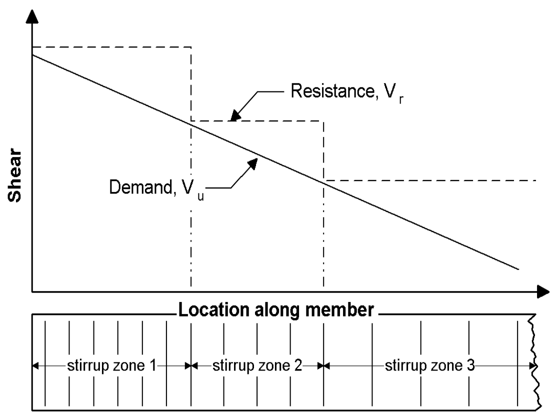


Figure C5.8.3.2-1—Traditional Shear Design

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

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

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

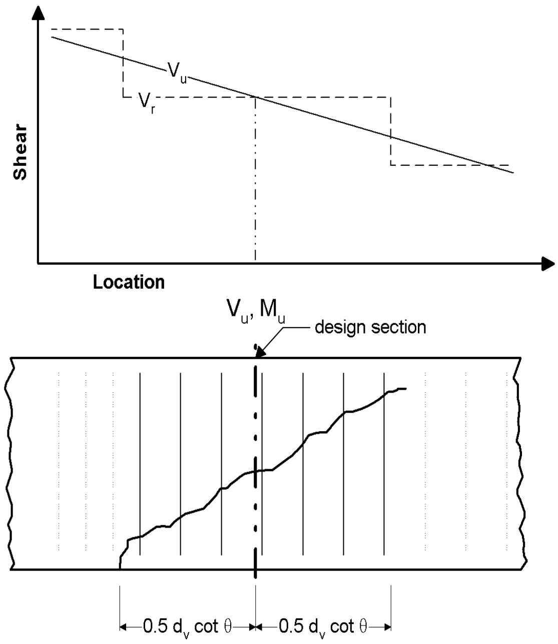


Figure C5.8.3.2-2—Simplified Design Section for Loads Applied at or above the Middepth of the Member

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

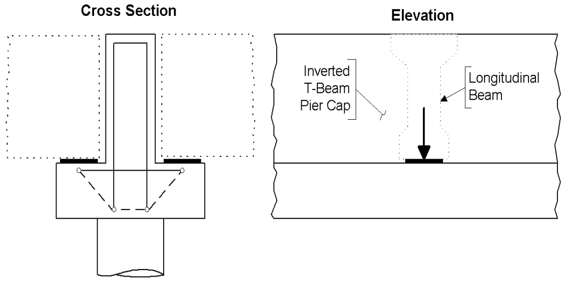


Figure C5.8.3.2-3—Inverted T-Beam Pier Cap

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

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

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

5.8.3.3—Nominal Shear Resistance

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

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

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

in which:

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

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

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

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

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

C5.8.3.3

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

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

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

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

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad (C5.8.3.3\text{-}1)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)

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

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

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

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

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

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

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear; V_p = 0 when Article 5.8.3.4.3 is applied (kip)

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

Where bent longitudinal reinforcement is used, only the center three-fourths of the inclined portion of the bent bar shall be considered effective for transverse reinforcement.

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

Where shear resistance is provided by bent longitudinal reinforcement or a combination of bent longitudinal reinforcement and stirrups, the nominal shear resistance shall be determined using the simplified procedure in accordance with Article 5.8.3.4.1.

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

5.8.3.4—Procedures for Determining Shear Resistance

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

C5.8.3.4

Three complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.8.3.4.1, as described herein, is only applicable for nonprestressed sections. Method 2, as described in Article 5.8.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load. Two approaches are presented in Method 2: a direct calculation, specified in Article 5.8.3.4.2, and an evaluation using tabularized values presented in

Appendix B5. The approaches to Method 2 may be considered statistically equivalent. Method 3, specified in Article 5.8.3.4.3, is applicable for both prestressed and nonprestressed sections in which there is no net axial tensile load and at least minimum shear reinforcement is provided. Axial load effects can otherwise be accounted for through adjustments to the level of effective precompression stress, f_{pc}. In regions of overlapping applicability between the latter two methods, Method 3 will generally lead to somewhat more shear reinforcement being required, particularly in areas of negative moment and near points of contraflexure. If Method 3 leads to an unsatisfactory rating, it is permissible to use Method 2.

5.8.3.4.1—Simplified Procedure for Nonprestressed Sections

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

$\beta = 2.0$
$\theta = 45°$

C5.8.3.4.1

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

5.8.3.4.2—General Procedure

The parameters β and θ may be determined either by the provisions herein, or alternatively by the provisions of Appendix B5.

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

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

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

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

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

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

In Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3, ε_s is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement as shown in Figures 5.8.3.4.2-1 and 5.8.3.4.2-2. In lieu of more involved procedures, ε_s may be determined by Eq. 5.8.3.4.2-4:

C5.8.3.4.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.8.3.1. Such an analysis, see Figure C5.8.3.4.2-1, would show that the shear stresses are not uniform over the depth of the web and that the direction of the principal compressive stresses changes over the depth of the beam. The more direct procedure given herein assumes that the concrete shear stresses are uniformly distributed over an area b_v wide and d_v deep, that the direction of principal compressive stresses (defined by angle θ and shown as D) remains constant over d_v, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. See Figure C5.8.3.4.2-2.

This design procedure (Collins et al, 1994) was derived from the Modified Compression Field Theory (MCFT, Vecchio, and Collins, 1986) which is a comprehensive behavioral model for the response of diagonally cracked concrete subject to in-plane shear and normal stresses. Prior to the 2008 interim revisions, the General Procedure for shear design was iterative and required the use of tables for the evaluation of β and θ. With the 2008 revisions, this design procedure was modified to be non-iterative and algebraic equations were introduced for the evaluation of β and θ. These equations are functionally equivalent to those used in the Canadian design code (A23.2-M04, 2004), were also derived from the MCFT (Bentz et al. 2006), and were evaluated as

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps}} \quad (5.8.3.4.2\text{-}4)$$

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

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

where:

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

where:

A_c = area of concrete on the flexural tension side of the member as shown in Figure 5.8.3.4.2-1 (in.^{2})

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure 5.8.3.4.2-1 (in.^{2})

A_s = area of nonprestressed steel on the flexural tension side of the member at the section under consideration, as shown in Figure 5.8.3.4.2-1 (in.^{2})

a_g = maximum aggregate size (in.)

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value of 0.7 f_{pu} will be appropriate for both pretensioned and post-tensioned members

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

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

s_x = the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003 b_v s_x$, as shown in Figure 5.8.3.4.2-3 (in.)

V_u = factored shear force (kip)

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

The longitudinal strain, ε_s, can be determined by the procedure illustrated in Figure C5.8.3.4.2-3. The actual section is represented by an idealized section consisting of a flexural tension flange, a flexural compression flange, and a web. The area of the compression flange is taken as the area on the flexure compression side of the member, i.e., the total area minus the area of the tension flange as defined by A_c. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u - V_p$, will primarily be carried by diagonal compressive stresses in the web concrete.
These diagonal compressive stresses will result in a longitudinal compressive force in the web concrete of $(V_u - V_p)$ cot θ. Equilibrium requires that this longitudinal compressive force in the web needs to be balanced by tensile forces in the two flanges, with half the force, that is 0.5 $(V_u - V_p)$ cot θ, being taken by each flange. For simplicity, 0.5 cot θ may be taken as = 2.0 and the longitudinal demand due to shear in the longitudinal tension reinforcement becomes $V_u - V_p$ without significant loss of accuracy. After the required axial forces in the two flanges are calculated, the resulting axial strains, ε_t and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure C5.8.3.4.2-4.

For pretensioned members, f_{po} can be taken as the stress in the strands when the concrete is cast around them, i.e., approximately equal to the jacking stress. For post-tensioned members, f_{po} can be conservatively taken as the average stress in the tendons when the post-tensioning is completed.

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

The flexural tension side of the member shall be taken as the half-depth containing the flexural tension zone, as illustrated in Figure 5.8.3.4.2-1.

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

- $|M_u|$ should not be taken less than $|V_u - V_p|d_v$.
- In calculating A_s and A_{ps} the area of bars or tendons terminated less than their development length from the section under consideration should be reduced in proportion to their lack of full development.
- If the value of ε_s calculated from Eq. 5.8.3.4.2-4 is negative, it should be taken as zero or the value should be recalculated with the denominator of Eq. 5.8.3.4.2-4 replaced by ($E_sA_s + E_pA_{ps} + E_cA_{ct}$). However, ε_s should not be taken as less than -0.40×10^{-3}.
- For sections closer than d_v to the face of the support, the value of ε_s calculated at d_v from the face of the support may be used in evaluating β and θ.
- If the axial tension is large enough to crack the flexural compression face of the section, the value calculated from Eq. 5.8.3.4.2-4 should be doubled.
- It is permissible to determine β and θ from Eqs. 5.8.3.4.2-1 through 5.8.3.4.2-3 using a value of ε_s which is greater than that calculated from Eq. 5.8.3.4.2-4 However ε_s should not be taken greater than 6.0×10^{-3}.

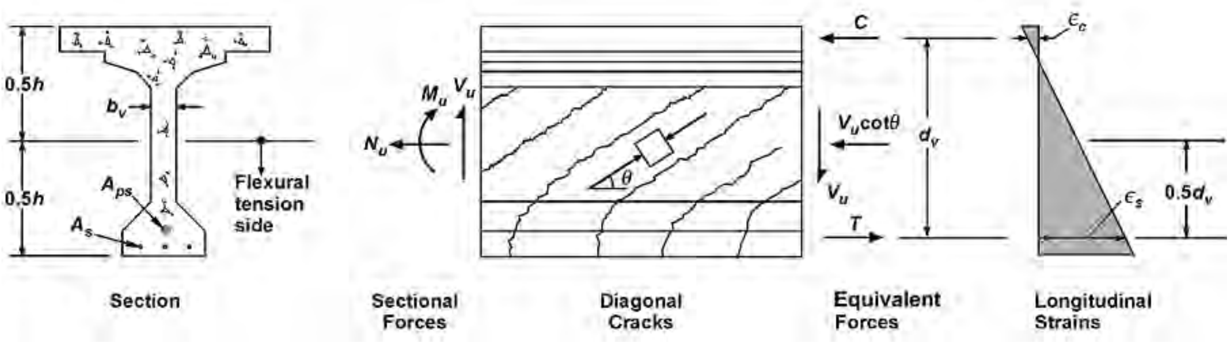


Figure 5.8.3.4.2-1—Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, $V_p = 0$

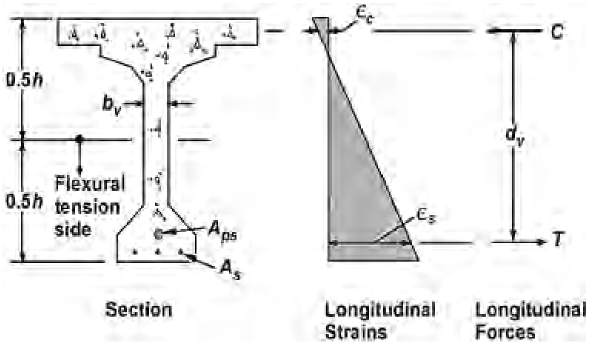


Figure 5.8.3.4.2-2—Longitudinal Strain, ε_s, for Sections Containing Less than the Minimum Amount of Transverse Reinforcement

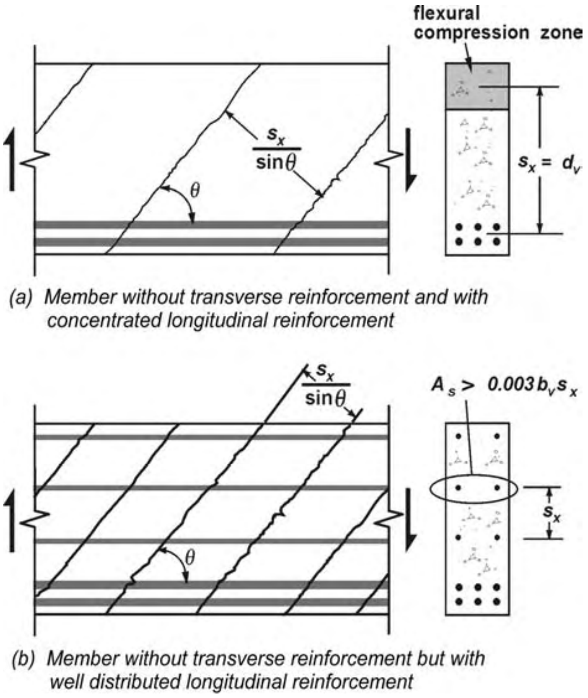


Figure 5.8.3.4.2-3—Definition of Crack Spacing Parameter, s_x

The relationships for evaluating β and θ in Eqs. 5.8.3.4.2-1 and 5.8.3.4.2-2 are based on calculating the stresses that can be transmitted across diagonally cracked concrete. As the cracks become wider, the stress that can be transmitted decreases. For members containing at least the minimum amount of transverse reinforcement, it is assumed that the diagonal cracks will be spaced about 12.0 in. apart. For members without transverse reinforcement, the spacing of diagonal cracks inclined at θ degrees to the longitudinal reinforcement is assumed to be $s_x/\sin\theta$, as shown in Figure 5.8.3.4.2-3. Hence, deeper members having larger values of s_x are calculated to have more widely spaced cracks and hence, cannot transmit such high shear stresses. The ability of the crack surfaces to transmit shear stresses is influenced by the aggregate size of the concrete. Members made from concretes that have a smaller maximum aggregate size will have a larger value of s_{xe} and hence, if there is no transverse reinforcement, will have a smaller shear strength.

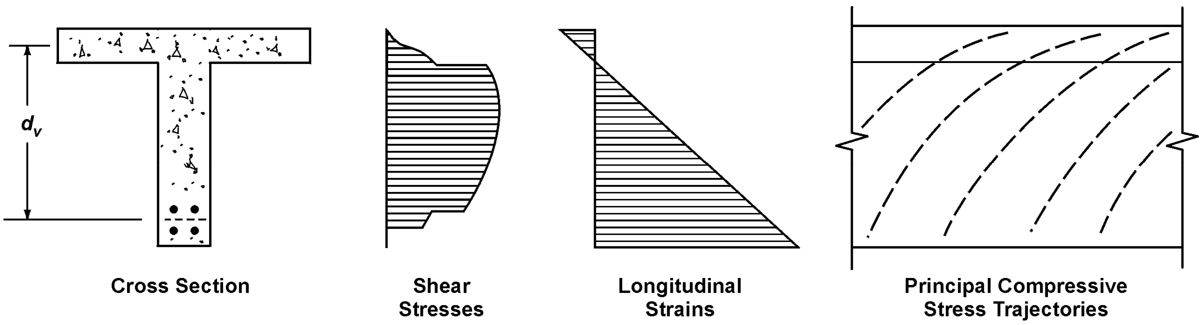


Figure C5.8.3.4.2-1—Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.8.3.1

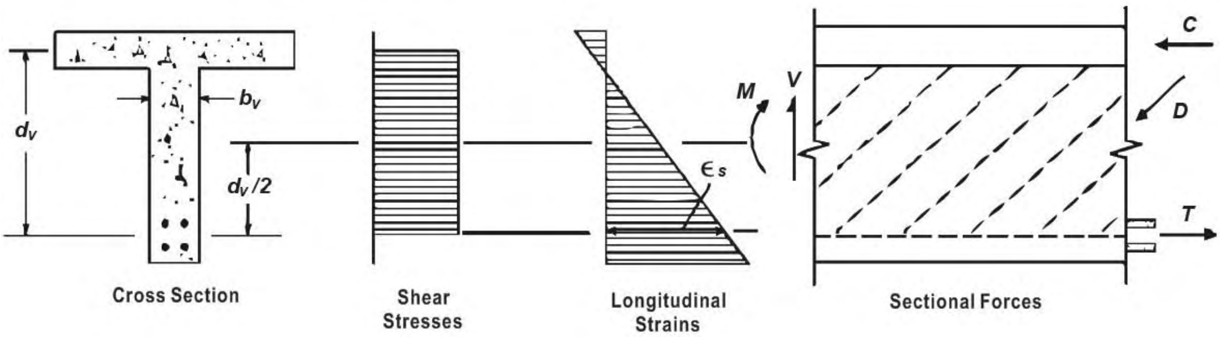


Figure C5.8.3.4.2-2—More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.8.3.4.2

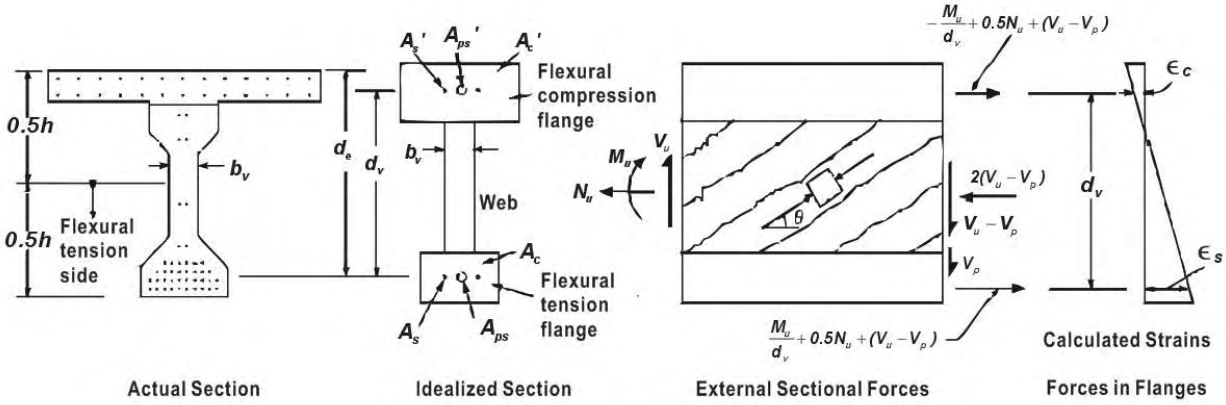


Figure C5.8.3.4.2-3—More Accurate Calculation Procedure for Determining ε_s

5.8.3.4.3—Simplified Procedure for Prestressed and Nonprestressed Sections

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

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

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

V_{ci} shall be determined as:

$$V_{ci} = 0.02\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}b_v d_v \quad (5.8.3.4.3\text{-}1)$$

C5.8.3.4.3

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

The nominal shear resistance V_n is the sum of the shear resistances V_c and V_s provided by the concrete and shear reinforcement, respectively. Both V_c and V_s depend on the type of inclined cracking that occurs at the given section. There are two types of inclined cracking: flexure-shear cracking and web-shear cracking for which the associated resistances are V_{ci} and V_{cw}, respectively. Figure C5.8.3.4.3-1 shows the development of both types of cracking when increasing uniform load was applied to a 63-in. bulb-tee girder. NCHRP Report XX2 (Hawkins et al., 2005).

where:

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

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

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

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

M_{cre} shall be determined as:

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

where:

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

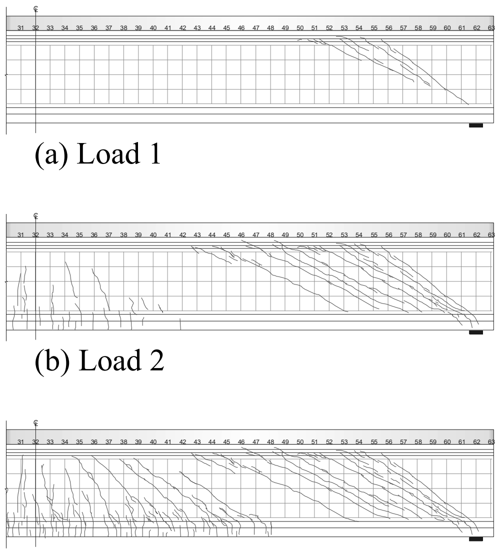
M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

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

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

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

V_{cw} shall be determined as:



(a) Load 1

(b) Load 2

(c) Load 3

Figure C5.8.3.4.3-1—Development of Shear Cracking with Increasing Loads for Uniformly Loaded Bulb Tee Beam; Load 1 < Load 2 < Load 3

Web-shear cracking begins from an interior point in the web of the member before either flange in that region cracks in flexure. In Figure C5.8.3.4.3-1, at load 1, web-shear cracking developed in the web of the member adjacent to the end support. Flexure-shear cracking is initiated by flexural cracking. Flexural cracking increases the shear stresses in the concrete above the flexural crack. In Figure C5.8.3.4.3-1, flexural cracking had developed in the central region of the beam by load 2 and by load 3, the flexural cracks had become inclined cracks as flexural cracking extended towards the end support with increasing load.

For sections with shear reinforcement equal to or greater than that required by Article 5.8.2.5, the shear carried by the concrete may drop below V_c shortly after inclined cracking, and the shear reinforcement may yield locally. However, sections continue to resist increasing shears until resistances provided by the concrete again reach V_c. Thus, V_{ci} and V_{cw} are measures of the resistance that can be provided by the concrete at the nominal shear resistance of the section and are not directly equal to the shears at inclined cracking.

The angle θ of the inclined crack, and therefore of the diagonal compressive stress, is less for a web-shear crack than a flexure-shear crack. Consequently, for a given section the value of V_s associated with web-shear cracking is greater than that associated with flexure-shear cracking.

$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30 f_{pc}\right) b_v d_v + V_p \qquad (5.8.3.4.3\text{-}3)$$

where:

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

V_s shall be determined using Eq. 5.8.3.3-4 with cot θ taken as follows:

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

$\cot\theta = 1.0$

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

$$\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8 \qquad (5.8.3.4.3\text{-}4)$$

V_{ci} is the sum of the shear ($V_i M_{cr} / M_{max}$) required to cause flexural cracking at the given section plus the increment of shear necessary to develop the flexural crack into a shear crack, For a non-composite beam, the total cross section resists all applied shears, dead and live, I_c equals the moment of inertia of the gross section and V_d equals the unfactored dead load shear acting on the section. In this case Eq. 5.8.3.4.3-1 can be used directly.

For a composite beam, part of the dead load is resisted by only part of the final section. Where the final gross concrete section is achieved with only one addition to the initial concrete section (two-stage construction), Eq. 5.8.3.4.3-1 can be used directly. In Eq. 5.8.3.4.3-2 appropriate section properties are used to compute f_d and in Eq. 5.8.3.4.3-1 the shear due to dead load V_d and that due to other loads V_i are separated. V_d is the total shear force due to unfactored dead loads acting on the part of the section carrying the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The term V_i may be taken as $(V_u - V_d)$ and M_{max} as $M_u - M_d$ where V_u and M_u are the factored shear and moment at the given section due to the total factored loads M_d is the moment due to unfactored dead load at the same section.

Where the final gross section is developed with more than one concrete composite addition to the initial section (multiple-stage construction), it is necessary to trace the build up of the extreme fiber flexural stresses to compute M_{cr}. For each stage in the life history of the member, the increments in the extreme fiber flexural stress at the given section due to the unfactored loads acting on that section are calculated using the section properties existing at that stage. V_d, V_i, and M_{max} are calculated in the same manner as for two-stage construction.

A somewhat lower modulus of rupture is used in evaluating M_{cre} by Eq. 5.8.3.4.3-2 to account for the effects of differential shrinkage between the slab and the girder, and the effects of thermal gradients that can occur over the depth of the girder.

5.8.3.5—Longitudinal Reinforcement

At each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta \qquad (5.8.3.5\text{-}1)$$

where:

C5.8.3.5

Shear causes tension in the longitudinal reinforcement. For a given shear, this tension becomes larger as θ becomes smaller and as V_c becomes larger. The tension in the longitudinal reinforcement caused by the shear force can be visualized from a free-body diagram such as that shown in Figure C5.8.3.5-1.

Taking moments about Point 0 in Figure C5.8.3.5-1, assuming that the aggregate interlock force on the crack, which contributes to V_c, has a negligible moment about Point 0, and neglecting the small difference in location of V_u and V_p leads to the requirement for the tension force in the longitudinal reinforcement caused by shear.

V_s = shear resistance provided by the transverse reinforcement at the section under investigation as given by Eq. 5.8.3.3-4, except V_s shall not be taken as greater than V_u/ϕ (kip)

θ = angle of inclination of diagonal compressive stresses used in determining the nominal shear resistance of the section under investigation as determined by Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

$\phi_f\phi_v\phi_c$ = resistance factors taken from Article 5.5.4.2 as appropriate for moment, shear and axial resistance

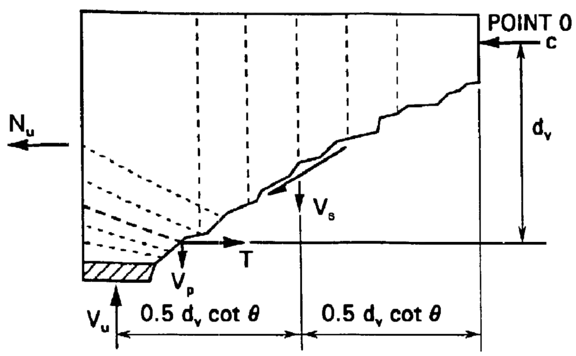


Figure C5.8.3.5-1—Forces Assumed in Resistance Model Caused by Moment and Shear

The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or the load introduces direct compression into the flexural compression face of the member.

Eq. 5.8.3.5-1 shall be evaluated where simply-supported girders are made continuous for live loads. Where longitudinal reinforcement is discontinuous, Eq. 5.8.3.5-1 shall be re-evaluated.

At maximum moment locations, the shear force changes sign, and hence the inclination of the diagonal compressive stresses changes. At direct supports including simply-supported girder ends and bent/pier caps pinned to columns, and at loads applied directly to the top or bottom face of the member, this change of inclination is associated with a fan-shaped pattern of compressive stresses radiating from the point load or the direct support as shown in Figure C5.8.3.5-2. This fanning of the diagonal stresses reduces the tension in the longitudinal reinforcement caused by the shear; i.e., angle θ becomes steeper. The tension in the reinforcement does not exceed that due to the maximum moment alone. Hence, the longitudinal reinforcement requirements can be met by extending the flexural reinforcement for a distance of $d_v$cotθ or as specified in Article 5.11, whichever is greater.

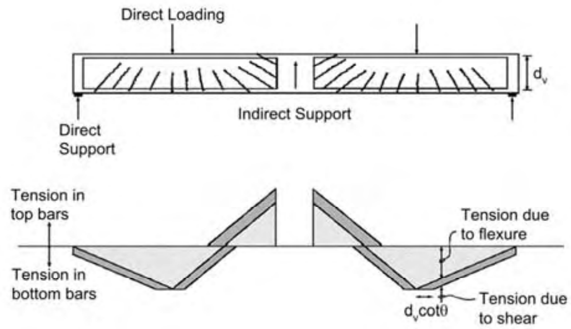


Figure C5.8.3.5-2—Force Variation in Longitudinal Reinforcement Near Maximum Moment Locations

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad (5.8.3.5\text{-}2)$$

Eqs. 5.8.3.5-1 and 5.8.3.5-2 shall be taken to apply to sections not subjected to torsion. Any lack of full development shall be accounted for.

In determining the tensile force that the reinforcement is expected to resist at the inside edge of the bearing area, the values of V_u, V_s, V_p, and θ, calculated for the section d_v from the face of the support may be used. In calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the development length of Article 5.11.2.1.1 or the bi-linear variation of resistance over the transfer and development length of Article 5.11.4.2 may be assumed.

5.8.3.6—Sections Subjected to Combined Shear and Torsion

5.8.3.6.1—Transverse Reinforcement

The transverse reinforcement shall not be less than the sum of that required for shear, as specified in Article 5.8.3.3, and for the concurrent torsion, as specified in Articles 5.8.2.1 and 5.8.3.6.2.

C5.8.3.6.1

The shear stresses due to torsion and shear will add on one side of the section and offset on the other side. The transverse reinforcement is designed for the side where the effects are additive.

Usually the loading that causes the highest torsion differs from the loading that causes the highest shear. Although it is sometimes convenient to design for the highest torsion combined with the highest shear, it is only necessary to design for the highest shear and its concurrent torsion, and the highest torsion and its concurrent shear.

5.8.3.6.2—Torsional Resistance

The nominal torsional resistance shall be taken as:

$$T_n = \frac{2A_o A_t f_y \cot\theta}{s} \qquad (5.8.3.6.2\text{-}1)$$

where:

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

A_t = area of one leg of closed transverse torsion reinforcement in solid members, or total area of transverse torsion reinforcement in the exterior web of cellular members (in.^{2})

θ = angle of crack as determined in accordance with the provisions of Article 5.8.3.4 with the modifications to the expressions for v and V_u herein (degrees)

5.8.3.6.3—Longitudinal Reinforcement

The provisions of Article 5.8.3.5 shall apply as amended, herein, to include torsion.

The longitudinal reinforcement in solid sections shall be proportioned to satisfy Eq. 5.8.3.6.3-1:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5 V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi}\right)^2} \quad (5.8.3.6.3\text{-}1)$$

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

$$A_\ell = \frac{T_n p_h}{2 A_o f_y} \quad (5.8.3.6.3\text{-}2)$$

where:

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

C5.8.3.6.3

To account for the fact that on one side of the section the torsional and shear stresses oppose each other, the equivalent tension used in the design equation is taken as the square root of the sum of the squares of the individually calculated tensions in the web.

5.8.4—Interface Shear Transfer—Shear Friction

5.8.4.1—General

Interface shear transfer shall be considered across a given plane at:

- An existing or potential crack,
- An interface between dissimilar materials,
- An interface between two concretes cast at different times, or
- The interface between different elements of the cross-section.

C5.8.4.1

Shear displacement along an interface plane may be resisted by cohesion, aggregate interlock, and shear-friction developed by the force in the reinforcement crossing the plane of the interface. Roughness of the shear plane causes interface separation in a direction perpendicular to the interface plane. This separation induces tension in the reinforcement balanced by compressive stresses on the interface surfaces.

Adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/flange interfaces in box girders to transfer flange longitudinal forces at the strength limit state. The factored design force for the interface reinforcement is calculated to account for the interface shear force, ΔF, as shown in Figure C5.8.4.1-1, as well as any localized shear effects due to the prestressing force anchorages at the section.

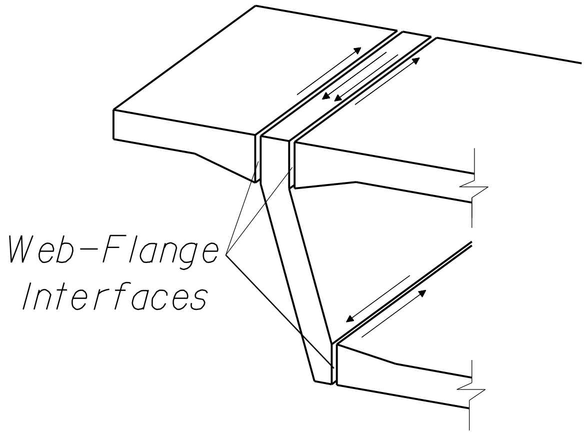


Figure C5.8.4.1-1—Longitudinal Shear Transfer between Flanges and Webs of Box Girder Bridges

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

All reinforcement present where interface shear transfer is to be considered shall be fully developed on both sides of the interface by embedment, hooks, mechanical methods such as headed studs or welding to develop the design yield stress.

Any reinforcement crossing the interface is subject to the same strain as the designed interface reinforcement. Insufficient anchorage of any reinforcement crossing the interface could result in localized fracture of the surrounding concrete.

When the required interface shear reinforcement in girder/slab design exceeds the area required to satisfy vertical (transverse) shear requirements, additional reinforcement must be provided to satisfy the interface shear requirements. The additional interface shear reinforcement need only extend into the girder a sufficient depth to develop the design yield stress of the reinforcement rather than extending the full depth of the girder as is required for vertical shear reinforcement.

The minimum area of interface shear reinforcement specified in Article 5.8.4.4 shall be satisfied.

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

$$V_{ri} = \phi V_{ni} \qquad (5.8.4.1\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \qquad (5.8.4.1\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)

V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip), and

Total load shall include all noncomposite and composite loads.

ϕ = resistance factor for shear specified in Article 5.5.4.2.1. In cases where different weight concretes exist on the two sides of an interface, the lower of the two values of ϕ shall be used.

For the extreme limit state event ϕ may be taken as 1.0.

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

$$V_{ni} = cA_{cv} + \mu\,(A_{vf}f_y + P_c) \qquad (5.8.4.1\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}, \text{ or} \qquad (5.8.4.1\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \qquad (5.8.4.1\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \qquad (5.8.4.1\text{-}6)$$

where:

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

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})

b_{vi} = interface width considered to be engaged in shear transfer (in.)

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in Article 5.8.4.3 (ksi)

μ = friction factor specified in Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

A pure shear friction model assumes interface shear resistance is directly proportional to the net normal clamping force ($A_{vf}f_y + P_c$), through a friction coefficient (μ). Eq. 5.8.4.1-3 is a modified shear-friction model accounting for a contribution, evident in the experimental data, from cohesion and/or aggregate interlock depending on the nature of the interface under consideration given by the first term. For simplicity, the term "cohesion factor" is used throughout the body of this Article to capture the effects of cohesion and/or aggregate interlock such that Eq. 5.8.4.1-3 is analogous to the vertical shear resistance expression of $V_c + V_s$.

Eq. 5.8.4.1-4 limits V_{ni} to prevent crushing or shearing of aggregate along the shear plane.

Eqs. 5.8.4.1-3 and 5.8.4.1-4 are sufficient, with an appropriate value for K_1, to establish a lower bound for the available experimental data; however, Eq. 5.8.4.1-5 is necessitated by the sparseness of available experimental data beyond the limiting K_2 values provided in Article 5.8.4.3.

The interface shear strength Eqs. 5.8.4.1-3, 5.8.4.1-4, and 5.8.4.1-5 are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi to 6.0 ksi; and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

Composite section design utilizing full-depth precast deck panels is not addressed by these provisions. Design specifications for such systems should be established by, or coordinated with, the Owner.

A_{vf} used in Eq. 5.8.4.1-3 is the interface shear reinforcement within the interface area A_{cv}. For a girder/slab interface, the area of the interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.1-3 with $12b_{vi}$ and P_c corresponding to the same one foot of girder length.

In consideration of the use of stay-in-place deck panels, or any other interface details, the Designer shall determine the width of interface, b_{vi}, effectively acting to resist interface shear.

The interface reinforcement is assumed to be stressed to its design yield stress, f_y. However, f_y used in determining the interface shear resistance is limited to 60 ksi because interface shear resistance computed using higher values have overestimated the interface shear resistance experimentally determined in a limited number of tests of pre-cracked specimens.

P_c = permanent net compressive force normal to the shear plane; if force is tensile, $P_c = 0.0$ (kip)

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3.

K_2 = limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

It is conservative to neglect P_c if it is compressive, however, if included, the value of P_c shall be computed as the force acting over the area, A_{cv}. If P_c is tensile, additional reinforcement is required to resist the net tensile force as specified in Article 5.8.4.2.

5.8.4.2—Computation of the Factored Interface Shear Force, V_{ui}, for Girder/Slab Bridges

Based on consideration of a free body diagram and utilizing the conservative envelope value of V_{u1}, the factored interface shear stress for a concrete girder/slab bridge may be determined as:

$$V_{ui} = V_{u1} \div b_{vi} d_v \quad (5.8.4.2\text{-}1)$$

where:

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

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

$$V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_v \quad (5.8.4.2\text{-}2)$$

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

$$A_{vpc} = P_c \div \phi f_y \quad (5.8.4.2\text{-}3)$$

For beams and girders, the longitudinal spacing of the rows of interface shear transfer reinforcing bars shall not exceed 24.0 in.

C5.8.4.2

The following illustrates a free body diagram approach to computation of interface shear in a girder/slab bridge. In reinforced concrete, or prestressed concrete, girder bridges, with a cast-in-place slab, horizontal shear forces develop along the interface between the girders and the slab. The classical strength of materials approach, which is based on elastic behavior of the section, has been used successfully in the past to determine the design interface shear force. As an alternative to the classical elastic strength of materials approach, a reasonable approximation of the factored interface shear force at the strength or extreme event limit state for either elastic or inelastic behavior and cracked or uncracked sections, can be derived with the defined notation and the free body diagram shown in Figure C5.8.4.2-1 as follows:

M_{u2} = maximum factored moment at section 2

V_1 = the factored vertical shear at section 1 concurrent with M_{u2}

M_1 = the factored moment at section 1 concurrent with M_{u2}

Δl = unit length segment of girder

C_1 = compression force above the shear plane associated with M_1

C_{u2} = compression force above the shear plane associated with M_{u2}

$$M_{u2} = M_1 + V_1 \Delta l \quad (C5.8.4.2\text{-}1)$$

$$C_{u2} = M_{u2} \div d_v \quad (C5.8.4.2\text{-}2)$$

$$C_{u2} = M_1 \div d_v + V_1 \Delta l \div d_v \quad (C5.8.4.2\text{-}3)$$

$$C_1 = M_1 \div d_v \qquad \text{(C5.8.4.2-4)}$$

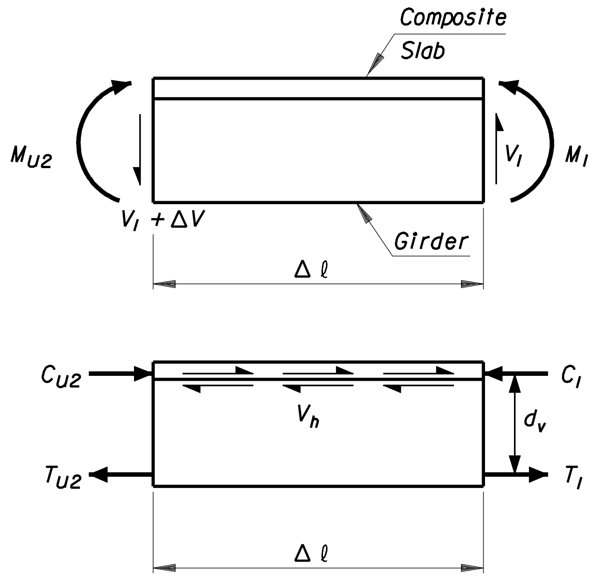


Figure C5.8.4.2-1—Free Body Diagrams

$$V_h = C_{u2} - C_1 \qquad \text{(C5.8.4.2-5)}$$

$$V_h = V_1 \Delta 1 \div d_v \qquad \text{(C5.8.4.2-6)}$$

Such that for a unit length segment:

$$V_{hi} = V_1 \div d_v \qquad \text{(C5.8.4.2-7)}$$

where:

V_{hi} = factored interface shear force per unit length (kips/length)

The variation of V_1 over the length of any girder segment reflects the shear flow embodied in the classical strength of materials approach. For simplicity of design, V_1 can be conservatively taken as V_{u1} (since V_{u1}, the maximum factored vertical shear at section 1, is not likely to act concurrently with the factored moment at section 2); and further, the depth, d_v, can be taken as the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress.

For design purposes, the computed factored interface shear stress of Eq. 5.8.4.2-1 is converted to a resultant interface shear force computed with Eq. 5.8.4.2-1 acting over an area, A_{cv}, within which the computed area of reinforcement, A_{vf}, shall be located. The resulting area of reinforcement, A_{vf}, then defines the area of interface reinforcement required per foot of girder for direct comparison with vertical shear reinforcement requirements.

5.8.4.3—Cohesion and Friction Factors

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

- For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.

c = 0.28 ksi
μ = 1.0
K_1 = 0.3
K_2 = 1.8 ksi for normal-weight concrete
= 1.3 ksi for lightweight concrete

- For normal-weight concrete placed monolithically:

c = 0.40 ksi
μ = 1.4
K_1 = 0.25
K_2 = 1.5 ksi

- For lightweight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.0 ksi

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.5 ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

C5.8.4.3

The values presented provide a lower bound of the substantial body of experimental data available in the literature (Loov and Patnaik, 1994; Patnaik, 1999; Mattock, 2001; Slapkus and Kahn, 2004). Furthermore, the inherent redundancy of girder/slab bridges distinguishes this system from other structural interfaces.

The values presented apply strictly to monolithic concrete. These values are not applicable for situations where a crack may be anticipated to occur at a Service Limit State.

The factors presented provide a lower bound of the experimental data available in the literature (Hofbeck, Ibrahim, and Mattock, 1969; Mattock, Li, and Wang, 1976; Mitchell and Kahn, 2001).

Available experimental data demonstrates that only one modification factor is necessary, when coupled with the resistance factors of Article 5.5.4.2, to accommodate both all-lightweight and sand-lightweight concrete. Note this deviates from earlier specifications that distinguished between all-lightweight and sand-lightweight concrete.

Due to the absence of existing data, the prescribed cohesion and friction factors for nonmonolithic lightweight concrete are accepted as conservative for application to monolithic lightweight concrete.

Tighter constraints have been adopted for roughened interfaces, other than cast-in-place slabs on roughened girders, even though available test data does not indicate more severe restrictions are necessary. This is to account for variability in the geometry, loading and lack of redundancy at other interfaces.

- For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint:

  c = 0.025 ksi
  μ = 0.7
  K_1 = 0.2
  K_2 = 0.8 ksi

For brackets, corbels, and ledges, the cohesion factor, c, shall be taken as 0.0.

Since the effectiveness of cohesion and aggregate interlock along a vertical crack interface is unreliable the cohesion component in Eq. 5.8.4.1-3 is set to 0.0 for brackets, corbels, and ledges.

5.8.4.4—Minimum Area of Interface Shear Reinforcement

Except as provided herein, the cross-sectional area of the interface shear reinforcement, A_{vf}, crossing the interface area, A_{cv}, shall satisfy:

$$A_{vf} \geq \frac{0.05 A_{cv}}{f_y} \qquad (5.8.4.4\text{-}1)$$

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

- The minimum interface shear reinforcement, A_{vf}, need not exceed the lesser of the amount determined using Eq. 5.8.4.4-1 and the amount needed to resist $1.33V_{ui}/\phi$ as determined using Eq. 5.8.4.1-3.

- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, v_{ui} of Eq. 5.8.4.2-1, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored in the slab.

C5.8.4.4

For a girder/slab interface, the minimum area of interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.4-1 with $12b_{vi}$.

Previous editions of these specifications and of the AASHTO Standard Specifications have required a minimum area of reinforcement based on the full interface area; similar to Eq. 5.8.4.4-1, irrespective of the need to mobilize the strength of the full interface area to resist the applied factored interface shear. In 2006, the additional minimum area provisions, applicable only to girder/slab interfaces, were introduced. The intent of these provisions was to eliminate the need for additional interface shear reinforcement due simply to a beam with a wider top flange being utilized in place of a narrower flanged beam.

The additional provision establishes a rational upper bound for the area of interface shear reinforcement required based on the interface shear demand rather than the interface area as stipulated by Eq. 5.8.4.4-1. This treatment is analogous to minimum reinforcement provisions for flexural capacity where a minimum additional overstrength factor of 1.33 is required beyond the factored demand.

With respect to a girder/slab interface, the intent is that the portion of the reinforcement required to resist vertical shear which is extended into the slab also serves as interface shear reinforcement.

5.8.5—Principal Stresses in Webs of Segmental Concrete Bridges

The provisions specified herein shall apply to all types of segmental bridges with internal and/or external tendons.

The principal tensile stress resulting from the long-term residual axial stress and maximum shear and/or maximum shear combined with shear from torsion stress at the neutral axis of the critical web shall not exceed the tensile stress limit of Table 5.9.4.2.2-1 at the Service III limit state of Article 3.4.1 at all stages during the life of the structure, excluding those during construction. When investigating principal stresses during construction, the tensile stress limits of Table 5.14.2.3.3-1 shall apply.

The principal stress shall be determined using classical beam theory and the principles of Mohr's Circle. The width of the web for these calculations shall be measured perpendicular to the plane of the web.

Compressive stress due to vertical tendons provided in the web shall be considered in the calculation of the principal stress. The vertical force component of draped longitudinal tendons shall be considered as a reduction in the shear force due to the applied loads.

Local tensions produced in webs resulting from anchorage of tendons as discussed in Article 5.10.9.2 shall be included in the principal tension check.

Local transverse flexural stress due to out-of-plane flexure of the web itself at the critical section may be neglected in computing the principal tension in webs.

C5.8.5

This principal stress check is introduced to verify the adequacy of webs of segmental concrete bridges for longitudinal shear and torsion.

5.8.6—Shear and Torsion for Segmental Box Girder Bridges

5.8.6.1—General

Where it is reasonable to assume that plane sections remain plane after loading, the provisions presented herein shall be used for the design of segmental post-tensioned concrete box girder bridges for shear and torsion in lieu of the provisions of Article 5.8.3.

The applicable provisions of Articles 5.8.1, 5.8.2, 5.8.4, and 5.8.5 may apply, as modified by the provisions herein.

Discontinuity regions (where the plane sections assumption of flexural theory is not applicable) shall be designed using the provisions of Article 5.8.6.2 and the strut-and-tie model approach of Article 5.6.3. The provisions of Article 5.13.2 shall apply to special discontinuity regions such as deep beams, brackets and corbels, as appropriate.

The effects of any openings or ducts in members shall be considered. In determining the effective web or flange thickness, b_e, the diameters of ungrouted ducts or one-half the diameters of grouted ducts shall be subtracted from the web or flange thickness at the location of these ducts.

The values of $\sqrt{f'_c}$ used in any part of Article 5.8.6 shall not exceed 3.16.

C5.8.6.1

For types of construction other than segmental box girders, the provisions of Article 5.8.3 may be applied in lieu of the provisions of Article 5.8.6.

Discontinuity regions where the plane sections assumption of flexural theory is not applicable include regions adjacent to abrupt changes in cross-sections, openings, dapped ends, regions where large concentrated loads, reactions, or post-tensioning forces are applied or deviated, diaphragms, deep beams, corbels or joints.

The effects of using concrete with $\sqrt{f'_c} > 3.16$ on the allowable stress limits is not well known.

The design yield strength of transverse shear or torsion reinforcement shall be in accordance with Article 5.8.2.8.

5.8.6.2—Loading

Design for shear and torsion shall be performed at the strength limit state load combinations as defined in Article 3.4.1.

The shear component of the primary effective longitudinal prestress force acting in the direction of the applied shear being examined, V_p, shall be added to the load effect, with a load factor of 1.0.

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

The vertical component of inclined tendons shall only be considered to reduce the applied shear on the webs for tendons which are anchored or fully developed by anchorages, deviators, or internal ducts located in the top or bottom 1/3 of the webs.

The effects of factored torsional moments, T_u, shall be considered in the design when their magnitude exceeds the value specified in Article 5.8.6.3.

In a statically indeterminate structure where significant reduction of torsional moment in a member can occur due to redistribution of internal forces upon cracking, the applied factored torsion moment at a section, T_u, may be reduced to ϕT_{cr}, provided that moments and forces in the member and in adjoining members are adjusted to account for the redistribution.

where:

T_u = factored torsional moment (kip-in.)

T_{cr} = torsional cracking moment calculated using Eq. 5.8.6.3-2 (kip-in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

In lieu of a more refined analysis, the torsional loading from a slab may be assumed as linearly distributed along the member.

The effects of axial tension due to creep, shrinkage, and thermal effects in restrained members shall be considered wherever applicable.

C5.8.6.2

Design of prestressed concrete segmental bridges for shear and torsion is based on the strength limit state conditions because little information is available concerning actual shear stress distributions at the service limit state.

This load effect should only be added to the box girder analysis and not transferred into the substructure. Some designers prefer to add this primary prestress force shear component to the resistance side of the equation.

For members subjected to combined shear and torsion, the torsional moments produce shear forces in different elements of the structure that, depending on the direction of torsion, may add to or subtract from the shear force in the element due to vertical shear. Where it is required to consider the effects of torsional moments, the shear forces from torsion need to be added to those from the vertical shear when determining the design shear force acting on a specific element. The possibility of the torsional moment reversing direction should be investigated.

The component of inclined flexural compression or tension, in the direction of the applied shear, in variable depth members shall be considered when determining the design factored shear force.

5.8.6.3—Regions Requiring Consideration of Torsional Effects

For normal weight concrete, torsional effects shall be investigated where:

$$T_u > 1/3\ \phi T_{cr} \tag{5.8.6.3-1}$$

in which:

$$T_{cr} = 0.0632 K \sqrt{f'_c}\, 2 A_o b_e \tag{5.8.6.3-2}$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\sqrt{f'_c}}} \le 2.0 \tag{5.8.6.3-3}$$

where:

T_u = factored torsional moment (kip-in.)

T_{cr} = torsional cracking moment (kip-in.)

K = stress variable K shall not be taken greater than 1.0 for any section where the stress in the extreme tension fiber, calculated on the basis of gross section properties, due to factored load and effective prestress force after losses exceeds $0.19\sqrt{f'_c}$ in tension.

A_o = area enclosed by the shear flow path of a closed box section, including any holes therein (in.^{2})

b_e = effective width of the shear flow path, but not exceeding the minimum thickness of the webs or flanges comprising the closed box section (in.). b_e shall be adjusted to account for the presence of ducts as specified in Article 5.8.6.1.

p_c = the length of the outside perimeter of the concrete section (in.)

f_{pc} = unfactored compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi)

ϕ = resistance factor for shear specified in Article 5.5.4.2

In lieu of a more refined analysis, b_e may be taken as A_{cp}/P_e, where A_{cp} is the area enclosed by the outside perimeter of the concrete cross-section and P_c is the outside perimeter of the concrete cross-section.

When calculating K for a section subject to factored axial force, N_u, f_{pc} shall be replaced with $f_{pc} - N_u/A_g$. N_u shall be taken as a positive value when the axial force is tensile and negative when it is compressive.

5.8.6.4—Torsional Reinforcement

C5.8.6.4

Where consideration of torsional effects is required by Article 5.8.6.3, torsion reinforcement shall be provided, as specified herein. This reinforcement shall be in addition to the reinforcement required to resist the factored shear, as specified in Article 5.8.6.5, flexure and axial forces that may act concurrently with the torsion.

The longitudinal and transverse reinforcement required for torsion shall satisfy:

$$T_u \leq \phi T_n \tag{5.8.6.4-1}$$

The nominal torsional resistance from transverse reinforcement shall be based on a truss model wit 45-degree diagonals and shall be computed as:

$$T_n = \frac{2A_o A_v f_y}{s} \tag{5.8.6.4-2}$$

The minimum additional longitudinal reinforcement for torsion, A_ℓ, shall satisfy:

$$A_\ell \geq \frac{T_u\, p_h}{2\phi\, A_o\, f_y} \tag{5.8.6.4-3}$$

where:

A_v = area of transverse shear reinforcement (in.^{2})

A_ℓ = total area of longitudinal torsion reinforcement in the exterior web of the box girder (in.^{2})

T_u = applied factored torsional moment (kip-in.)

p_h = perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion. p_h may be taken as the perimeter of the centerline of the outermost closed stirrups (in.)

A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2})

f_y = yield strength of additional longitudinal reinforcement (ksi)

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing is taken into account by considering it equivalent to an area of reinforcing steel with a yield force equal to the effective prestressing force.

ϕ = resistance factor for shear specified in Article 5.5.4.2

A_ℓ shall be distributed around the perimeter of the closed stirrups in accordance with Article 5.8.6.6.

Subject to the minimum reinforcement requirements of Article 5.8.6.6, the area of additional longitudinal torsion reinforcement in the flexural compression zone may be reduced by an amount equal to:

$$\frac{M_u}{(0.9 d_e f_y)} \qquad (5.8.6.4\text{-}4)$$

where:

M_u = the factored moment acting at that section concurrent with T_u (kip-in.)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

f_y = specified minimum yield strength of reinforcing bars (ksi)

5.8.6.5—Nominal Shear Resistance

In lieu of the provisions of Article 5.8.3, the provisions herein shall be used to determine the nominal shear resistance of post-tensioned concrete box girders in regions where it is reasonable to assume that plane sections remain plane after loading.

Transverse reinforcement shall be provided when $V_u > 0.5\phi V_c$, where V_c is computed by Eq. 5.8.6.5-4.

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s \qquad (5.8.6.5\text{-}1)$$

$$V_n = 0.379\sqrt{f'_c}\, b_v d_v \qquad (5.8.6.5\text{-}2)$$

and, where the effects of torsion are required to be considered by Article 5.8.6.2, the cross-sectional dimensions shall be such that:

$$V_c = 0.0632\ K\sqrt{f'_c}\ b_v d_v \qquad (5.8.6.5\text{-}3)$$

in which:

$$V_s = \frac{A_v f_y d_v}{s} \qquad (5.8.6.5\text{-}4)$$

$$\left(\frac{V_u}{b_v d_v}\right) + \left(\frac{T_u}{2 A_o b_e}\right) \leq 0.474\sqrt{f'_c} \qquad (5.8.6.5\text{-}5)$$

C5.8.6.5

The expression for V_c has been checked against a wide range of test data and has been found to be a conservative expression.

Eq. 5.8.6.5-4 is based on an assumed 45-degree truss model.

Eq. 5.8.6.4-5 is only used to establish appropriate concrete section dimensions.

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.6.1 (in.)

d_v = 0.8h or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.)

s = spacing of stirrups (in.)

K = stress variable computed in accordance with Article 5.8.6.3.

A_v = area of shear reinforcement within a distance s (in.^{2})

V_u = factored design shear including any normal component from the primary prestressing force (kip)

T_u = applied factored torsional moment (kip-in.)

A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2})

b_e = the effective thickness of the shear flow path of the elements making up the space truss model resisting torsion calculated in accordance with Article 5.8.6.3 (in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

The factored nominal shear resistance, ϕV_n, shall be greater than or equal to V_u.

The applied factored shear, V_u, in regions near supports may be computed at a distance h/2 from the support when the support reaction, in the direction of the applied shear, introduces compression into the support region of the member and no concentrated load occurs within a distance, h, from the face of the support.

5.8.6.6—Reinforcement Details

In addition to the provisions herein, the provisions of Article 5.10 and 5.11 shall also apply to segmental post-tensioned box girders, as applicable.

At any place on the cross-section where the axial tension due to torsion and bending exceeds the axial compression due to prestressing and bending, either supplementary tendons to counter the tension or local longitudinal reinforcement, which is continuous across the joints between segments, shall be required.

Where supplementary tendons are added, they shall be located to provide compression around the perimeter of the closed box section.

Where local longitudinal reinforcement is added, the bars shall be distributed around the perimeter formed by the closed stirrups. Perimeter bar spacing shall not exceed 18.0 in. At least one longitudinal bar shall be placed in each corner of the stirrups. The minimum diameter of the corner bars shall be 1/24 of the stirrup spacing but no less than that of a #5 bar.

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.19\sqrt{f'_c}$, then:

$$s_{max} = 0.8d \leq 36.0 \text{ in.} \quad (5.8.6.6\text{-}1)$$

- If $v_u \geq 0.19\sqrt{f'_c}$, then:

$$s_{max} = 0.4d \leq 18.0 \text{ in.} \quad (5.8.6.6\text{-}2)$$

where:

v_u = the shear stress calculated in accordance with Eq. 5.8.6.5-5 (ksi)

d_v = effective shear depth as defined in Article 5.8.6.5 (in.)

Transverse reinforcement for shear and torsion shall be provided for a distance at least $h/2$ beyond the point they are theoretically required.

Interface shear transfer reinforcement shall be provided as specified in Article 5.8.4.

5.9—PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1—General Design Considerations

5.9.1.1—General

The provisions herein specified shall apply to structural concrete members reinforced with any combination of prestressing tendons and conventional reinforcing bars acting together to resist common force effects. Prestressed and partially prestressed concrete structural components shall be designed for both initial and final prestressing forces. They shall satisfy the requirements at service, fatigue, strength, and extreme event limit states, as specified in Article 5.5, and in accordance with the assumptions provided in Articles 5.6, 5.7, and 5.8.

Unstressed prestressing tendons or reinforcing bars may be used in combination with stressed tendons, provided it is shown that performance of the structure satisfies all limit states and the requirements of Articles 5.4 and 5.6.

Compressive stress limits, specified in Article 5.9.4, shall be used with any applicable service load combination in Table 3.4.1-1, except Service Load Combination III, which shall not apply to the investigation of compression.

C5.9.1.1

The introduction of partial prestressing permits the development of a unified theory of concrete structures in which conventional reinforced and prestressed concrete become boundary cases.

The background material in this Article is based on previous editions of the Standard Specifications and ACI 343, ACI 318, and the *Ontario Highway Bridge Design Code*, the provisions of which are extended herein to accommodate partial prestressing.

Prestressing tendons of high-strength steel bars or strands are generally used, but other materials satisfying desired strength, stiffness, and ductility requirements could also be used, provided that they meet the intent of Article 5.4.1.

Partial prestressing can be considered a design concept that allows one or a combination of the following design solutions:

- A concrete member reinforced with a combination of prestressed and nonprestressed reinforcement designed to simultaneously resist the same force effects, Tensile stress limits, specified in Article 5.9.4, shall be used with any applicable service load combination in Table 3.4.1-1. Service Load Combination III shall apply when investigating tension under live load.

- A prestressed concrete member designed to crack in tension under service load, and
- A prestressed concrete member in which the effective prestress in the prestressed reinforcement is purposely kept lower than its maximum allowable value.

5.9.1.2—Specified Concrete Strengths

The specified strengths, f'_c and f'_{ci}, shall be identified in the contract documents for each component. Stress limits relating to specified strengths shall be as specified in Article 5.9.4.

Concrete strength at transfer shall be adequate for the requirements of the anchorages or for transfer through bond as well as for camber or deflection requirements.

5.9.1.3—Buckling

Buckling of a member between points where concrete and tendons are in contact, buckling during handling and erection, and buckling of thin webs and flanges shall be investigated.

5.9.1.4—Section Properties

For section properties prior to bonding of post-tensioning tendons, effects of loss of area due to open ducts shall be considered.

For both pretensioned or post-tensioned members after bonding of tendons, section properties may be based on either the gross or transformed section.

C5.9.1.4

Bonding means that the grout in the duct has attained its specified strength.

5.9.1.5—Crack Control

Where cracking is permitted under service loads, crack width, fatigue of reinforcement, and corrosion considerations shall be investigated in accordance with the provisions of Articles 5.5, 5.6, and 5.7.

5.9.1.6—Tendons with Angle Points or Curves

The provisions of Article 5.4.6 for the curvature of ducts shall apply.

The provisions of Article 5.10.4 shall apply to the investigation of stress concentrations due to changes in the direction of prestressing tendons.

For tendons in draped ducts that are not nominally straight, consideration shall be given to the difference between the center of gravity of the tendon and the center of gravity of the duct when determining eccentricity.

C5.9.1.6

Vertically draped strand tendons should be assumed to be at the bottom of the duct in negative moment areas and at the top of the duct in positive moment areas. The location of the tendon center of gravity, with respect to the centerline of the duct, is shown for negative moment in Figure C5.9.1.6-1.

Duct Size (IN)	Z (IN)
3 OD and Less	1/2
Over 3 OD to 4	3/4
Over 4 OD	1

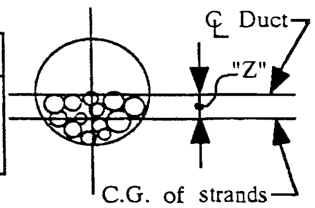


Figure C5.9.1.6-1—Location of Tendon in Duct

5.9.2—Stresses Due to Imposed Deformation

The effects on adjoining elements of the structure of elastic and inelastic deformations due to prestressing shall be investigated. The restraining forces produced in the adjoining structural elements may be reduced due to the effects of creep.

In monolithic frames, force effects in columns and piers resulting from prestressing the superstructure may be based on the initial elastic shortening.

For conventional monolithic frames, any increase in column moments due to long-term creep shortening of the prestressed superstructure is considered to be offset by the concurrent relaxation of deformation moments in the columns due to creep in the column concrete.

The reduction of restraining forces in other members of a structure that are caused by the prestress in a member may be taken as:

- For suddenly imposed deformations

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right), \text{ or} \qquad (5.9.2\text{-}1)$$

- For slowly imposed deformations

$$F' = F\left(1 - e^{-\psi(t,t_i)}\right) / \psi\left(t, t_i\right) \qquad (5.9.2\text{-}2)$$

where:

F = force effect determined using the modulus of elasticity of the concrete at the time loading is applied (kip)

F' = reduced force effect (kip)

$\Psi(t, t_i)$ = creep coefficient at time t for loading applied at time t_i as specified in Article 5.4.2.3.2

e = base of Napierian logarithms

C5.9.2

Additional information is contained in Leonhardt (1964).

5.9.3—Stress Limitations for Prestressing Tendons

The tendon stress due to prestress or at the service limit state shall not exceed the values:

- Specified in Table 5.9.3-1, or
- Recommended by the manufacturer of the tendons or anchorages.

The tendon stress at the strength and extreme event limit states shall not exceed the tensile strength limit specified in Table 5.4.4.1-1.

C5.9.3

For post-tensioning, the short-term allowable of $0.90f_{py}$ may be allowed for short periods of time prior to seating to offset seating and friction losses, provided that the other values in Table 5.9.3-1 are not exceeded.

Table 5.9.3-1—Stress Limits for Prestressing Tendons

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70f_{pu}$	$0.75f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$
At anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.70f_{pu}$	$0.70f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.74f_{pu}$	$0.70f_{pu}$
At service limit state after losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

5.9.4—Stress Limits for Concrete

5.9.4.1—For Temporary Stresses before Losses—Fully Prestressed Components

5.9.4.1.1—Compression Stresses

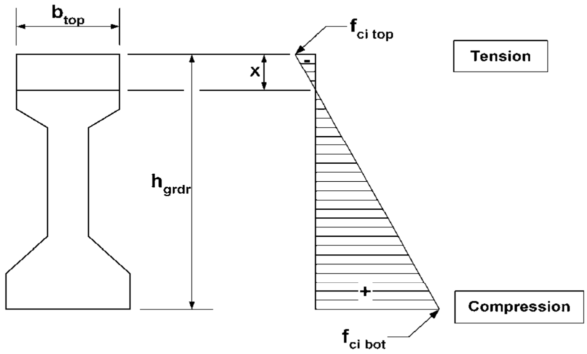
The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0.60 f'_{ci} (ksi).

5.9.4.1.2—Tension Stresses

The limits in Table 5.9.4.1.2-1 shall apply for tensile stresses.

C5.9.4.1.2

Where bonded reinforcement is provided to allow use of the increased tensile limiting stress in areas with bonded reinforcement, the tensile force must be computed. The first step in computing the tensile force, T, is to determine the depth of the tensile zone using the extreme fiber stresses at the location being considered, $f_{ci\ top}$ and $f_{ci\ bot}$. An area is then defined over which the average tensile stress is assumed to act. The tensile force is computed as the product of the average tensile stress and the computed area, as illustrated below. The required area of reinforcement, A_s, is computed by dividing the tensile force by the permitted stress in the reinforcement.



$$T = \frac{f_{ci\ top}}{2} b_{top} x$$

$$A_s = \frac{T}{f_s}$$

where $f_s = 0.5\,f_y \leq 30$ ksi

Figure C5.9.4.1.2-1—Calculation of Tensile Force and Required Area of Reinforcement

Table 5.9.4.1.2-1—Temporary Tensile Stress Limits in Prestressed Concrete before Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	• In precompressed tensile zone without bonded reinforcement	N/A
	• In areas other than the precompressed tensile zone and without bonded reinforcement	$0.0948\sqrt{f'_{ci}} \leq 0.2$ (ksi)
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi.	$0.24\sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	$0.158\sqrt{f'_{ci}}$ (ksi)
Segmentally Constructed Bridges	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5f_y$; with internal tendons or external tendons	$0.0948\sqrt{f'_{ci}}$ maximum tension (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through the joints	No tension
	Transverse Stresses through Joints	
	• For any type of joint	$0.0948\sqrt{f'_{ci}}$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded nonprestressed reinforcement	No tension
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5\ f_y$, not to exceed 30 ksi.	$0.19\sqrt{f'_{ci}}$ (ksi)
	Principal Tensile Stress at Neutral Axis in Web	
	• All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures	$0.110\sqrt{f'_{ci}}$ (ksi)

5.9.4.2—For Stresses at Service Limit State after Losses—Fully Prestressed Components

5.9.4.2.1—Compression Stresses

Compression shall be investigated using the Service Limit State Load Combination I specified in Table 3.4.1-1. The limits in Table 5.9.4.2.1-1 shall apply.

C5.9.4.2.1

The reduction factor, ϕ_w, shall be taken to be equal to 1.0 when the web and flange slenderness ratios, calculated according to Article 5.7.4.7.1, are not greater than 15. When either the web or flange slenderness ratio is greater than 15, the reduction factor, ϕ_w, shall be calculated according to Article 5.7.4.7.2.

Unlike solid rectangular beams that were used in the development of concrete design codes, the unconfined concrete of the compression sides of box girders are expected to creep to failure at a stress far lower than the nominal strength of the concrete. This behavior is similar to the behavior of the concrete in thin-walled columns. The reduction factor, ϕ_w, was originally developed to account for the reduction in the usable strain of concrete in thin-walled columns at the strength limit state. The use of ϕ_w to reduce the stress limit in box girders at the service limit state is not theoretically correct. However, due to the lack of information about the behavior of the concrete at the service limit state, the use of ϕ_w provides a rational approach to account for the behavior of thin components.

The application of Article 5.7.4.7.2 to flanged, strutted, and variable thickness elements requires some judgment. Consideration of appropriate lengths of wall-type element is illustrated in Figure C5.9.4.2.1-1. For constant thickness lengths, the wall thickness associated with that length should be used. For variable thickness lengths, e.g., L_4, an average thickness could be used. For multilength components, such as the top flange, the highest ratio should be used. The beneficial effect of support by struts should be considered. There are no effective length factors shown. The free edge of the cantilever overhang is assumed to be supported by the parapet in Figure C5.9.4.2.1-1.

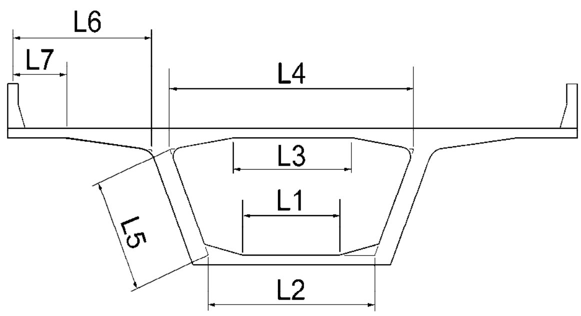


Figure C5.9.4.2.1-1—Suggested Choices for Wall Lengths to be Considered

Table 5.9.4.2.1-1—Compressive Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components

Location	Stress Limit
• In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45f'_c$ (ksi)
• In segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45f'_c$ (ksi)
• Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0.60\,\phi_w f'_c$ (ksi)

5.9.4.2.2—Tension Stresses

For service load combinations that involve traffic loading, tension stresses in members with bonded or unbonded prestressing tendons should be investigated using Load Combination Service III specified in Table 3.4.1-1.

The limits in Table 5.9.4.2.2-1 shall apply.

C5.9.4.2.2

Severe corrosive conditions include exposure to deicing salt, water, or airborne sea salt and airborne chemicals in heavy industrial areas.

See Figure C5.9.4.1.2-1 for calculation of required area of bonded reinforcement.

Table 5.9.4.2.2-1—Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.19\sqrt{f'_c}$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.0948\sqrt{f'_c}$ (ksi)
	• For components with unbonded prestressing tendons	No tension
Segmentally Constructed Bridges	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 fy; internal tendons or external tendons	$0.0948\sqrt{f'_c}$ (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through joints	No tension
	Transverse Stresses through Joints	
	• Tension in the transverse direction in precompressed tensile zone	$0.0948\sqrt{f'_c}$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded reinforcement	No tension
	• In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi	$0.19\sqrt{f'_c}$ (ksi)
	Principal Tensile Stress at Neutral Axis in Web	
	• All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures.	$0.110\sqrt{f'_c}$ (ksi)

5.9.4.3—Partially Prestressed Components

Compression stresses shall be limited as specified in Articles 5.9.4.1.1 and 5.9.4.2.1 for fully prestressed components.

Cracking in the precompressed tensile zone may be permitted. The design of partially prestressed members should be based on a cracked section analysis with various service limit states being satisfied. Tensile stress in reinforcement at the service limit state shall be as specified in Article 5.7.3.4, in which case f_s shall be interpreted as the change in stress after decompression.

5.9.5—Loss of Prestress

5.9.5.1—Total Loss of Prestress

Values of prestress losses specified herein shall be applicable to normal weight concrete only and for specified concrete strengths up to 15.0 ksi, unless stated otherwise.

In lieu of more detailed analysis, prestress losses in members constructed and prestressed in a single stage, relative to the stress immediately before transfer, may be taken as:

- In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \qquad (5.9.5.1\text{-}1)$$

- In post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \qquad (5.9.5.1\text{-}2)$$

where:

Δf_{pT} = total loss (ksi)

Δf_{pF} = loss due to friction (ksi)

Δf_{pA} = loss due to anchorage set (ksi)

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)

C5.9.5.1

For segmental construction, lightweight concrete construction, multi-stage prestressing, and bridges where more exact evaluation of prestress losses is desired, calculations for loss of prestress should be made in accordance with a time-step method supported by proven research data. See references cited in Article C5.4.2.3.2.

Data from control tests on the materials to be used, the methods of curing, ambient service conditions, and pertinent structural details for the construction should be considered.

Accurate estimate of total prestress loss requires recognition that the time-dependent losses resulting from creep, shrinkage, and relaxation are also interdependent. However, undue refinement is seldom warranted or even possible at the design stage because many of the component factors are either unknown or beyond the control of the Designer.

Losses due to anchorage set, friction, and elastic shortening are instantaneous, whereas losses due to creep, shrinkage, and relaxation are time-dependent.

This Article has been revised on the basis of new analytical investigations. The presence of a substantial amount of nonprestressed reinforcement, such as in partially prestressed concrete, influences stress redistribution along the section due to creep of concrete with time, and generally leads to smaller loss of prestressing steel pretension and larger loss of concrete precompression.

The loss across stressing hardware and anchorage devices has been measured from two to six percent (Roberts, 1993) of the force indicated by the ram pressure times the calibrated ram area. The loss varies depending on the ram and the anchor. An initial design value of three percent is recommended.

The extension of the provisions to 15.0 ksi was based on Tadros (2003), which only included normal weight concrete. Consequently, the extension to 15.0 ksi is only valid for members made with normal weight concrete.

5.9.5.2—Instantaneous Losses

5.9.5.2.1—Anchorage Set

The magnitude of the anchorage set shall be the greater of that required to control the stress in the prestressing steel at transfer or that recommended by the manufacturer of the anchorage. The magnitude of the set assumed for the design and used to calculate set loss shall be shown in the contract documents and verified during construction.

C5.9.5.2.1

Anchorage set loss is caused by the movement of the tendon prior to seating of the wedges or the anchorage gripping device. The magnitude of the minimum set depends on the prestressing system used. This loss occurs prior to transfer and causes most of the difference between jacking stress and stress at transfer. A common value for anchor set is 0.375 in., although values as low as 0.0625 in. are more appropriate for some anchorage devices, such as those for bar tendons.

For wedge-type strand anchors, the set may vary between 0.125 in. and 0.375 in., depending on the type of equipment used. For short tendons, a small anchorage seating value is desirable, and equipment with power wedge seating should be used. For long tendons, the effect of anchorage set on tendon forces is insignificant, and power seating is not necessary. The 0.25-in. anchorage set value, often assumed in elongation computations, is adequate but only approximate.

Due to friction, the loss due to anchorage set may affect only part of the prestressed member.

Losses due to elastic shortening may also be calculated in accordance with Article 5.9.5.2.3 or other published guidelines (PCI 1975; Zia et. al. 1979). Losses due to elastic shortening for external tendons may be calculated in the same manner as for internal tendons.

5.9.5.2.2—Friction

5.9.5.2.2a—Pretensioned Construction

For draped prestressing tendons, losses that may occur at the hold-down devices should be considered.

5.9.5.2.2b—Post-Tensioned Construction

Losses due to friction between the internal prestressing tendons and the duct wall may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right) \qquad (5.9.5.2.2b\text{-}1)$$

Losses due to friction between the external tendon across a single deviator pipe may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha + 0.04)}\right) \qquad (5.9.5.2.2b\text{-}2)$$

where:

f_{pj} = stress in the prestressing steel at jacking (ksi)

x = length of a prestressing tendon from the jacking end to any point under consideration (ft)

C5.9.5.2.2b

Where large discrepancies occur between measured and calculated tendon elongations, in-place friction tests are required.

The 0.04 radians in Eq. 5.9.5.2.2b-2 represents an inadvertent angle change. This angle change may vary depending on job-specific tolerances on deviator pipe placement and need not be applied in cases where the deviation angle is strictly controlled or precisely known, as in the case of continuous ducts passing through separate longitudinal bell-shaped holes at deviators. The inadvertent angle change need not be considered for calculation of losses due to wedge seating movement.

K = wobble friction coefficient (per ft of tendon)

μ = coefficient of friction

α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad.)

e = base of Napierian logarithms

Values of K and μ should be based on experimental data for the materials specified and shall be shown in the contract documents. In the absence of such data, a value within the ranges of K and μ as specified in Table 5.9.5.2.2b-1 may be used.

For slender members, the value of x may be taken as the projection of the tendon on the longitudinal axis of the member. A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes. See also Article C5.14.2.3.7 for further discussion of friction and wobble coefficients.

For tendons confined to a vertical plane, α shall be taken as the sum of the absolute values of angular changes over length x.

α_v and α_h may be taken as the sum of absolute values of angular changes over length, x, of the projected tendon profile in the vertical and horizontal planes, respectively.

The scalar sum of α_v and α_h may be used as a first approximation of α.

For tendons curved in three dimensions, the total tridimensional angular change α shall be obtained by vectorially adding the total vertical angular change, α_v, and the total horizontal angular change, α_h.

When the developed elevation and plan of the tendons are parabolic or circular, the α can be computed from:

$$\alpha = \sqrt{\alpha_v^2 + \alpha_h^2} \qquad \text{(C5.9.5.2.2b-1)}$$

When the developed elevation and the plan of the tendon are generalized curves, the tendon may be split into small intervals, and the above formula can be applied to each interval so that:

$$\alpha = \Sigma\Delta\alpha = \Sigma\sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2} \qquad \text{(C5.9.5.2.2b-2)}$$

As an approximation, the tendon may be replaced by a series of chords connecting nodal points. The angular changes, $\Delta\alpha_v$ and $\Delta\alpha_h$, of each chord may be obtained from its slope in the developed elevation and in plan.

Field tests conducted on the external tendons of a segmental viaduct in San Antonio, Texas, indicate that the loss of prestress at deviators is higher than the usual friction coefficient ($\mu = 0.25$) would estimate.

This additional loss appears to be due, in part, to the tolerances allowed in the placement of the deviator pipes. Small misalignments of the pipes can result in significantly increased angle changes of the tendons at the deviation points. The addition of an inadvertent angle change of 0.04 radians to the theoretical angle change accounts for this effect based on typical deviator length of 3.0 ft and placement tolerance of ±3/8 in. The 0.04 value is to be added to the theoretical value at each deviator. The value may vary with tolerances on pipe placement.

The measurements also indicated that the friction across the deviators was higher during the stressing operations than during the seating operations.

See Podolny (1986) for a general development of friction loss theory for bridges with inclined webs and for horizontally curved bridges.

Table 5.9.5.2.2b-1—Friction Coefficients for Post-Tensioning Tendons

Type of Steel	Type of Duct	*K*	μ
Wire or strand	Rigid and semirigid galvanized metal sheathing	0.0002	0.15–0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
High-strength bars	Galvanized metal sheathing	0.0002	0.30

5.9.5.2.3—Elastic Shortening

5.9.5.2.3a—Pretensioned Members

The loss due to elastic shortening in pretensioned members shall be taken as:

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \tag{5.9.5.2.3a-1}$$

where:

f_{cgp} = the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

C5.9.5.2.3a

Changes in prestressing steel stress due to the elastic deformations of the section occur at all stages of loading. Historically, it has been conservative to account for this effect implicitly in the calculation of elastic shortening and creep losses considering only the prestress force present after transfer.

The change in prestressing steel stress due to the elastic deformations of the section may be determined for any load applied. The resulting change may be a loss, at transfer, or a gain, at time of superimposed load application. Where a more detailed analysis is desired, Eq. 5.9.5.2.3a-1 may be used at each section along the beam, for the various loading conditions.

In calculating f_{cgp}, using gross (or net) cross-section properties, it may be necessary to perform a separate calculation for each different elastic deformation to be included. For the combined effects of initial prestress and member weight, an initial estimate of prestress after transfer is used. The prestress may be assumed to be 90 percent of the initial prestress before transfer and the analysis iterated until acceptable accuracy is achieved. To avoid iteration altogether, Eq. C5.9.5.2.3a-1 may be used for the initial section. If the inclusion of an elastic gain due to the application of the deck weight is desired, the change in prestress force can be directly calculated. The same is true for all other elastic gains with appropriate consideration for composite sections.

The total elastic loss or gain may be taken as the sum of the effects of prestress and applied loads.

When calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for and Δf_{pES} should not be included in the prestressing force applied to the transformed section at transfer. Nevertheless, the effective prestress in the strands can be determined by subtracting losses (elastic and time-dependent) from the jacking stress. In other words, when using transformed section properties, the prestressing strand and the concrete are treated together as a composite section in which both the concrete and the prestressing strand are equally strained in compression by a prestressing force conceived as a fictitious external load applied at the level of the strands. To determine the effective stress in the prestressing strands (neglecting time-dependent losses for simplicity) the sum of the Δf_{pES} values considered must be included. In contrast, analysis with gross (or net) section properties involves using the effective stress in the strands at any given stage of loading to determine the prestress force and resulting concrete stresses.

The loss due to elastic shortening in pretensioned members may be determined by the following alternative equation:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}} \tag{C5.9.5.2.3a-1}$$

where:

A_{ps} = area of prestressing steel (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

e_m = average prestressing steel eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

I_g = moment of inertia of the gross concrete section (in.^{4})

M_g = midspan moment due to member self-weight (kip-in.)

5.9.5.2.3b—Post-Tensioned Members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as:

$$\Delta f_{pES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \qquad (5.9.5.2.3b\text{-}1)$$

where:

N = number of identical prestressing tendons

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force after jacking and the self-weight of the member at the sections of maximum moment (ksi)

f_{cgp} values may be calculated using a steel stress reduced below the initial value by a margin dependent on elastic shortening, relaxation, and friction effects.

For post-tensioned structures with bonded tendons, f_{cgp} may be taken at the center section of the span or, for continuous construction, at the section of maximum moment.

For post-tensioned structures with unbonded tendons, the f_{cgp} value may be calculated as the stress at the center of gravity of the prestressing steel averaged along the length of the member.

For slab systems, the value of Δf_{pES} may be taken as 25 percent of that obtained from Eq. 5.9.5.2.3b-1.

C5.9.5.2.3b

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be determined by the following alternative equation:

$$\Delta f_{pES} = \frac{N-1}{2N}\,\frac{A_{ps} f_{pbt}(I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps}(I_g + e_m^2 A_g) + \dfrac{A_g I_g E_{ci}}{E_p}} \qquad (C5.9.5.2.3b\text{-}1)$$

where:

A_{ps} = area of prestressing steel (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

e_m = average eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in Table 5.9.3-1 (ksi)

I_g = moment of inertia of the gross concrete section (in.^{4})

M_g = midspan moment due to member self-weight (kip-in.)

N = number of identical prestressing tendons

f_{pj} = stress in the prestressing steel at jacking (ksi)

For post-tensioned structures with bonded tendons, Δf_{pES} may be calculated at the center section of the span or, for continuous construction, at the section of maximum moment.

For post-tensioned structures with unbonded tendons, Δf_{pES} can be calculated using the eccentricity of the prestressing steel averaged along the length of the member.

For slab systems, the value of Δf_{pES} may be taken as 25 percent of that obtained from Eq. C5.9.5.2.3b-1.

For post-tensioned construction, Δf_{pES} losses can be further reduced below those implied by Eq. 5.9.5.2.3b-1 with proper tensioning procedures such as stage stressing and retensioning.

If tendons with two different numbers of strand per tendon are used, N may be calculated as:

$$N = N_1 + N_2 \frac{A_{sp2}}{A_{sp1}} \quad \text{(C5.9.5.2.3b-2)}$$

where:

N_1 = number of tendons in the larger group

N_2 = number of tendons in the smaller group

A_{sp1} = cross-sectional area of a tendon in the larger group (in.^{2})

A_{sp2} = cross-sectional area of a tendon in the smaller group (in.^{2})

5.9.5.2.3c—Combined Pretensioning and Post-Tensioning

In applying the provisions of Articles 5.9.5.2.3a and 5.9.5.2.3b to components with combined pretensioning and post-tensioning, and where post-tensioning is not applied in identical increments, the effects of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons shall be considered.

C5.9.5.2.3c

See Castrodale and White (2004) for information on computing the effect of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons.

5.9.5.3—Approximate Estimate of Time-Dependent Losses

For standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- members are made from normal-weight concrete,
- the concrete is either steam- or moist-cured,
- prestressing is by bars or strands with normal and low relaxation properties, and
- average exposure conditions and temperatures characterize the site,

the long-term prestress loss, Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using the following formula:

$$\Delta f_{pLT} = 10.0 \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR} \quad \text{(5.9.5.3-1)}$$

in which:

$$\gamma_h = 1.7 - 0.01H \quad \text{(5.9.5.3-2)}$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})} \quad \text{(5.9.5.3-3)}$$

C5.9.5.3

The losses or gains due to elastic deformations at the time of transfer or load application should be added to the time-dependent losses to determine total losses. However, these elastic losses (or gains) must be taken equal to zero if transformed section properties are used in stress analysis.

The approximate estimates of time-dependent prestress losses given in Eq. 5.9.5.3-1 are intended for sections with composite decks only. The losses in Eq. 5.9.5.3-1 were derived as approximations of the terms in the refined method for a wide range of standard precast prestressed concrete I-beams, box beams, inverted tee beams, and voided slabs. The members were assumed to be fully utilized, i.e., level of prestressing is such that concrete tensile stress at full service loads is near the maximum limit. It is further assumed in the development of the approximate method that live load moments produce about one-third of the total load moments, which is reasonable for I-beam and inverted tee composite construction and conservative for noncomposite boxes and voided slabs. They were calibrated with full-scale test results and with the results of the refined method, and found to give conservative results (Al-Omaishi, 2001; Tadros, 2003). The approximate method should not be used for members of uncommon shapes, i.e., having V/S ratios much different from 3.5 in., level of prestressing, or construction staging. The first term in Eq. 5.9.5.3-1 corresponds to creep losses, the second term to shrinkage losses, and the third to relaxation losses.

The commentary to Article 5.9.5.4.2 also gives an alternative relaxation loss prediction method.

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = the average annual ambient relative humidity (%)

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

Δf_{pR} = an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand, 10.0 ksi for stress relieved strand, and in accordance with manufacturers recommendation for other types of strand (ksi)

For girders other than those made with composite slabs, the time-dependent prestress losses resulting from creep and shrinkage of concrete and relaxation of steel shall be determined using the refined method of Article 5.9.5.4.

For segmental concrete bridges, lump sum losses may be used only for preliminary design purposes.

For members of unusual dimensions, level of prestressing, construction staging, or concrete constituent materials, the refined method of Article 5.9.5.4 or computer time-step methods shall be used.

5.9.5.4—Refined Estimates of Time-Dependent Losses

5.9.5.4.1—General

For nonsegmental prestressed members, more accurate values of creep-, shrinkage-, and relaxation-related losses, than those specified in Article 5.9.5.3 may be determined in accordance with the provisions of this Article. For precast pretensioned girders without a composite topping and for precast or cast-in-place nonsegmental post-tensioned girders, the provisions of Articles 5.9.5.4.4 and 5.9.5.4.5, respectively, shall be considered before applying the provisions of this Article.

C5.9.5.4.1

See Castrodale and White (2004) for information on computing the interaction of creep effects for prestressing applied at different times.

Estimates of losses due to each time-dependent source, such as creep, shrinkage, or relaxation, can lead to a better estimate of total losses compared with the values obtained using Article 5.9.5.3. The individual losses are based on research published in Tadros (2003), which aimed at extending applicability of the provisions of these Specifications to high-strength concrete.

For segmental construction and post-tensioned spliced precast girders, other than during preliminary design, prestress losses shall be determined by the time-step method and the provisions of Article 5.9.5, including consideration of the time-dependent construction stages and schedule shown in the contract documents. For components with combined pretensioning and post-tensioning, and where post-tensioning is applied in more than one stage, the effects of subsequent prestressing on the creep loss for previous prestressing shall be considered.

The change in prestressing steel stress due to time-dependent loss, Δf_{pLT}, shall be determined as follows:

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df} \qquad (5.9.5.4.1\text{-}1)$$

where:

Δf_{pSR} = prestress loss due to shrinkage of girder concrete between transfer and deck placement (ksi)

Δf_{pCR} = prestress loss due to creep of girder concrete between transfer and deck placement (ksi)

Δf_{pR1} = prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (ksi)

Δf_{pR2} = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi)

Δf_{pSD} = prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi)

Δf_{pCD} = prestress loss due to creep of girder concrete between time of deck placement and final time (ksi)

Δf_{pSS} = prestress gain due to shrinkage of deck in composite section (ksi)

$(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$
= sum of time-dependent prestress losses between transfer and deck placement (ksi)

$(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$
= sum of time-dependent prestress losses after deck placement (ksi)

The new approach additionally accounts for interaction between the precast and the cast-in-place concrete components of a composite member and for variability of creep and shrinkage properties of concrete by linking the loss formulas to the creep and shrinkage prediction formulae of Article 5.4.2.3.

For concrete containing lightweight aggregates, very hard aggregates, or unusual chemical admixtures, the estimated material properties used in this Article and Article 5.4.2.3 may be inaccurate. Actual test results should be used for their estimation.

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.5, including consideration of the time-dependent construction method and schedule shown in the contract documents.

5.9.5.4.2—Losses: Time of Transfer to Time of Deck Placement

5.9.5.4.2a—Shrinkage of Girder Concrete

The prestress loss due to shrinkage of girder concrete between time of transfer and deck placement, Δf_{pSR}, shall be determined as:

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} \tag{5.9.5.4.2a-1}$$

in which:

$$K_{id} = \frac{1}{1 + \dfrac{E_p}{E_{ci}} \dfrac{A_{ps}}{A_g} \left(1 + \dfrac{A_g e_{pg}^2}{I_g}\right) \left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \tag{5.9.5.4.2a-2}$$

where:

ε_{bid} = concrete shrinkage strain of girder between the time of transfer and deck placement per Eq. 5.4.2.3.3-1

K_{id} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement

e_{pg} = eccentricity of prestressing force with respect to centroid of girder (in.); positive in common construction where it is below girder centroid

$\Psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer per Eq. 5.4.2.3.2-1

t_f = final age (days)

t_i = age at transfer (days)

5.9.5.4.2b—Creep of Girder Concrete

The prestress loss due to creep of girder concrete between time of transfer and deck placement, Δf_{pCR}, shall be determined as:

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b \left(t_d, t_i\right) K_{id} \qquad (5.9.5.4.2b\text{-}1)$$

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 (days)

5.9.5.4.2c—Relaxation of Prestressing Strands

The prestress loss due to relaxation of prestressing strands between time of transfer and deck placement, Δf_{pR1}, shall be determined as:

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right) \qquad (5.9.5.4.2c\text{-}1)$$

where:

f_{pt} = stress in prestressing strands immediately after transfer, taken not less than $0.55f_{py}$ in Eq. 5.9.5.4.2c-1

K_L = 30 for low relaxation strands and 7 for other prestressing steel, unless more accurate manufacturer's data are available

The relaxation loss, Δf_{pR1}, may be assumed equal to 1.2 ksi for low-relaxation strands.

C5.9.5.4.2c

Eqs. 5.9.5.4.2c-1 and 5.9.5.4.3c-1 are given for relaxation losses and are appropriate for normal temperature ranges only. Relaxation losses increase with increasing temperatures.

A more accurate equation for prediction of relaxation loss between transfer and deck placement is given in Tadros et al. (2003):

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K'_L}\frac{\log\ (24t)}{\log(24t_i)}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR}\ )}{f_{pt}}\right] K_{id} \qquad (C5.9.5.4.2c\text{-}1)$$

where the K'_L is a factor accounting for type of steel, equal to 45 for low relaxation steel and 10 for stress relieved steel, t is time in days between strand tensioning and deck placement. The term in the first square brackets is the intrinsic relaxation without accounting for strand shortening due to creep and shrinkage of concrete. The second term in square brackets accounts for relaxation reduction due to creep and shrinkage of concrete. The factor K_{id} accounts for the restraint of the concrete member caused by bonded reinforcement. It is the same factor used for the creep and shrinkage components of the prestress loss. The equation given in Article 5.9.5.4.2c is an approximation of the above formula with the following typical values assumed:

t_i = 0.75 day

t = 120 days

$$\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR}\ )}{f_{pt}}\right] = 0.67$$

K_{id} = 0.8

5.9.5.4.3—Losses: Time of Deck Placement to Final Time

5.9.5.4.3a—Shrinkage of Girder Concrete

The prestress loss due to shrinkage of girder concrete between time of deck placement and final time, Δf_{pSD}, shall be determined as:

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df} \qquad (5.9.5.4.3a\text{-}1)$$

in which:

$$K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_c} \left(1 + \frac{A_c e_{pc}^2}{I_c}\right)\left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]} \qquad (5.9.5.4.3a\text{-}2)$$

where:

ε_{bdf} = shrinkage strain of girder between time of deck placement and final time per Eq. 5.4.2.3.3-1

K_{df} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between deck placement and final time

e_{pc} = eccentricity of prestressing force with respect to centroid of composite section (in.), positive in typical construction where prestressing force is below centroid of section

A_c = area of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio (in.^{2})

I_c = moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio at service (in.^{4})

5.9.5.4.3b—Creep of Girder Concrete

The prestress (loss is positive, gain is negative) due to creep of girder concrete between time of deck placement and final time, Δf_{pCD}, shall be determined as:

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b \left[\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \psi_b \left(t_f, t_d\right) K_{df} \qquad (5.9.5.4.3b\text{-}1)$$

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

5.9.5.4.3c—Relaxation of Prestressing Strands

The prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time, Δf_{pR2}, shall be determined as:

$$\Delta f_{pR2} = \Delta f_{pR1} \tag{5.9.5.4.3c-1}$$

C5.9.5.4.3.c

Research indicates that about one-half of the losses due to relaxation occur before deck placement; therefore, the losses after deck placement are equal to the prior losses.

5.9.5.4.3d—Shrinkage of Deck Concrete

The prestress gain due to shrinkage of deck composite section, Δf_{pSS}, shall be determined as:

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1+0.7\psi_b\left(t_f, t_d\right)\right] \tag{5.9.5.4.3d-1}$$

in which:

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left[1+0.7\psi_d\left(t_f, t_d\right)\right]}\left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right) \tag{5.9.5.4.3d-2}$$

where:

Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi)

ε_{ddf} = shrinkage strain of deck concrete between placement and final time per Eq. 5.4.2.3.3-1

A_d = area of deck concrete (in.^{2})

E_{cd} = modulus of elasticity of deck concrete (ksi)

e_d = eccentricity of deck with respect to the gross composite section, positive in typical construction where deck is above girder (in.)

$\Psi_b(t_f, t_d)$ = creep coefficient of deck concrete at final time due to loading introduced shortly after deck placement (i.e. overlays, barriers, etc.) per Eq. 5.4.2.3.2-1

5.9.5.4.4—Precast Pretensioned Girders without Composite Topping

The equations in Article 5.9.5.4.2 and Article 5.9.5.4.3 are applicable to girders with noncomposite deck or topping, or with no topping. The values for time of "deck placement" in Article 5.9.5.4.2 may be taken as values at time of noncomposite deck placement or values at time of installation of precast members without topping. Time of "deck placement" in Article 5.9.5.4.3 may be taken as time of noncomposite deck placement or values at time of installation of precast members without topping. Area of "deck" for these applications shall be taken as zero.

5.9.5.4.5—Post-Tensioned Nonsegmental Girders

Long-term prestress losses for post-tensioned members after tendons have been grouted may be calculated using the provisions of Articles 5.9.5.4.1 through 5.9.5.4.4. In Eq. 5.9.5.4.1-1, the value of the term $(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$ shall be taken as zero.

5.9.5.5—Losses for Deflection Calculations

For camber and deflection calculations of prestressed nonsegmental members made of normal weight concrete with a strength in excess of 3.5 ksi at the time of prestress, f_{cgp} and Δf_{cdp} may be computed as the stress at the center of gravity of prestressing steel averaged along the length of the member.

5.10—DETAILS OF REINFORCEMENT

5.10.1—Concrete Cover

Minimum concrete cover shall be as specified in Article 5.12.3.

5.10.2—Hooks and Bends

5.10.2.1—Standard Hooks

For the purpose of these Specifications, the term "standard hook" shall mean one of the following:

- For longitudinal reinforcement:

  (a) 180- degree bend, plus a $4.0d_b$ extension, but not less than 2.5 in. at the free end of the bar, or

  (b) 90- degree bend, plus a $12.0d_b$ extension at the free end of the bar.

C5.10.2.1

These requirements are consistent with the requirements of ACI 318 and CRSI's *Manual of Standard Practice.*

- For transverse reinforcement:
  - (a) No. 5 bar and smaller—90-degree bend, plus a $6.0d_b$ extension at the free end of the bar,
  - (b) No. 6, No. 7 and No. 8 bars—90-degree bend, plus a $12.0d_b$ extension at the free end of the bar; and
  - (c) No. 8 bar and smaller—135-degree bend, plus a $6.0\ d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

5.10.2.2—Seismic Hooks

Seismic hooks shall consist of a 135-degree bend, plus an extension of not less than the larger of $6.0d_b$ or 3.0 in. Seismic hooks shall be used for transverse reinforcement in regions of expected plastic hinges. Such hooks and their required locations shall be detailed in the contract documents.

5.10.2.3—Minimum Bend Diameters

The diameter of a bar bend, measured on the inside of the bar, shall not be less than that specified in Table 5.10.2.3-1.

Table 5.10.2.3-1—Minimum Diameters of Bend

Bar Size and Use	Minimum Diameter
No. 3 through No. 5—General	$6.0d_b$
No. 3 through No. 5—Stirrups and Ties	$4.0d_b$
No. 6 through No. 8—General	$6.0d_b$
No. 9, No. 10, and No. 11	$8.0d_b$
No. 14 and No. 18	$10.0d_b$

The inside diameter of bend for stirrups and ties in plain or deformed welded wire fabric shall not be less than $4.0d_b$ for deformed wire larger than D6 and $2.0d_b$ for all other wire sizes. Bends with inside diameters of less than $8.0d_b$ shall not be located less than $4.0d_b$ from the nearest welded intersection.

5.10.3—Spacing of Reinforcement

5.10.3.1 Minimum Spacing of Reinforcing Bars

5.10.3.1.1—Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars,
- 1.5 times the maximum size of the coarse aggregate, or
- 1.5 in.

5.10.3.1.2—Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than:

- The nominal diameter of the bars,
- 1.33 times the maximum size of the coarse aggregate, or
- 1.0 in.

5.10.3.1.3—Multilayers

Except in decks where parallel reinforcing is placed in two or more layers, with clear distance between layers not exceeding 6.0 in., the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than 1.0 in. or the nominal diameter of the bars.

5.10.3.1.4—Splices

The clear distance limitations between bars that are specified in Articles 5.10.3.1.1 and 5.10.3.1.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

5.10.3.1.5—Bundled Bars

The number of parallel reinforcing bars bundled in contact to act as a unit shall not exceed four in any one bundle, except that in flexural members, the number of bars larger than No. 11 shall not exceed two in any one bundle.

Bundled bars shall be enclosed within stirrups or ties.

Individual bars in a bundle, cut off within the span of a member, shall be terminated at different points with at least a 40-bar diameter stagger. Where spacing limitations are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

C5.10.3.1.5

Bundled bars should be tied, wired, or otherwise fastened together to ensure that they remain in their relative position, regardless of their inclination.

5.10.3.2—Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18.0 in. The maximum spacing of spirals, ties, and temperature shrinkage reinforcement shall be as specified in Articles 5.10.6, 5.10.7, and 5.10.8.

5.10.3.3—Minimum Spacing of Prestressing Tendons and Ducts

5.10.3.3.1—Pretensioning Strand

The distance between pretensioning strands, including shielded ones, at each end of a member within the transfer length, as specified in Article 5.11.4.1, shall not be less than a clear distance taken as 1.33 times the maximum size of the aggregate nor less than the center-to-center distances specified in Table 5.10.3.3.1-1.

Table 5.10.3.3.1-1—Center-to-Center Spacings

Strand Size (in.)	Spacing (in.)
0.6 0.5625 Special 0.5625	2.000
0.5000 0.4375 0.50 Special	1.750
0.3750	1.500

If justified by performance tests of full-scale prototypes of the design, the clear distance between strands at the end of a member may be decreased.

The minimum clear distance between groups of bundled strands shall not be less than 1.33 times the maximum size of the aggregate or 1.0 in.

Pretensioning strands in a member may be bundled to touch one another in an essentially vertical plane at and between hold-down locations. Strands bundled in any manner, other than a vertical plane, shall be limited to four strands per bundle.

C5.10.3.3.1

The requirement to maintain the clear spacing within the transfer zone is to ensure the strands are separated sufficiently to properly transfer their prestressing force to the surrounding concrete and to reduce the stress concentration around the strands at the ends of pretensioned components at release.

Some jurisdictions limit the clear distance between pretensioning strands to not less than twice the nominal size of aggregate to facilitate placing and compaction of concrete.

5.10.3.3.2—Post-Tensioning Ducts—Girders Straight in Plan

Unless otherwise specified herein, the clear distance between straight post-tensioning ducts shall not be less than 1.5 in. or 1.33 times the maximum size of the coarse aggregate. For precast segmental construction when post-tensioning tendons extend through an epoxy joint between components, the clear spacing between post-tensioning ducts shall not be less than the greater of the duct internal diameter or 4.0 in.

Ducts may be bundled together in groups not exceeding three, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 3.0 ft of anchorages.

For groups of bundled ducts in construction other than segmental, the minimum clear horizontal distance between adjacent bundles shall not be less than 4.0 in. When groups of ducts are located in two or more horizontal planes, a bundle shall contain no more than two ducts in the same horizontal plan.

C5.10.3.3.2

The minimum vertical clear distance between bundles shall not be less than 1.5 in. or 1.33 times the maximum size of coarse aggregate.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 3.0 in.

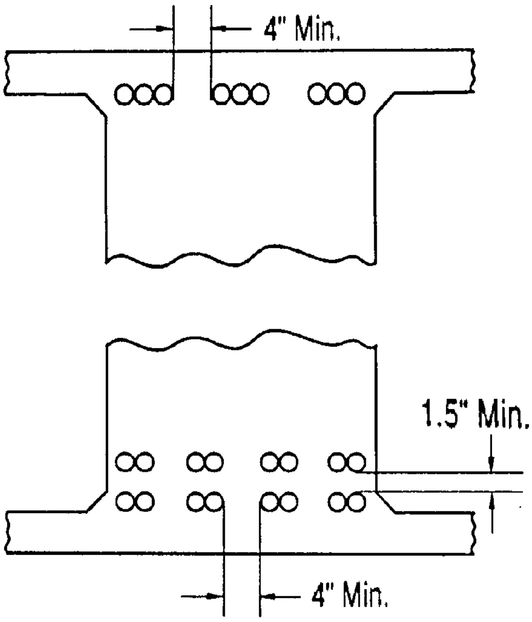


Figure C5.10.3.3.2-1—Examples of Acceptable Arrangements for Ducts Not Curved in the Horizontal Plan

5.10.3.3.3—Post-Tensioning Ducts—Girders Curved in Plan

The minimum clear distance between curved ducts shall be as required for tendon confinement as specified in Article 5.10.4.3. The spacing for curved ducts shall not be less than that required for straight ducts.

5.10.3.4—Maximum Spacing of Prestressing Tendons and Ducts in Slabs

Pretensioning strands for precast slabs shall be spaced symmetrically and uniformly and shall not be farther apart than 1.5 times the total composite slab thickness or 18.0 in.

Post-tensioning tendons for slabs shall not be farther apart, center-to-center, than 4.0 times the total composite minimum thickness of the slab.

C5.10.3.4

The 4.0 times depth of slab requirement for the maximum spacing of transverse post-tensioning ducts in deck slabs is new and reflects common practice. The composite thickness refers to slabs with bonded overlays.

5.10.3.5—Couplers in Post-Tensioning Tendons

The contract documents shall specify that not more than 50 percent of the longitudinal post-tensioning tendons be coupled at one section and that the spacing between adjacent coupler locations be not closer than the segment length or twice the segment depth. The void areas around couplers shall be deducted from the gross section area and moment of inertia when computing stresses at the time post-tensioning force is applied.

C5.10.3.5

European experience indicates that the prestressing force decreases locally in the region of a coupler. This is believed to result, in part, from increased creep caused by high compressive stresses in the reduced concrete section due to coupling of tendons. Cracking has not been observed in bridges where the number of tendons coupled at a section has been limited to 50 percent of the total number of tendons.

5.10.4—Tendon Confinement

5.10.4.1—General

Tendons shall be located within the reinforcing steel stirrups in webs, and, where applicable, between layers of transverse reinforcing steel in flanges and slabs. For ducts in the bottom flanges of variable depth segments, nominal confinement reinforcing shall be provided around the duct at each segment face. The reinforcement shall not be less than two rows of No. 4 hairpin bars at both sides of each duct 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.

C5.10.4.1

This Article is based primarily on the recommendation from Breen and Kashima (1991).

5.10.4.2—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 18.0 in. or 1.5 times the slab thickness in each direction.

C5.10.4.2

The hairpin bars are provided to prevent slab delamination along the plane of the post-tensioning ducts.

5.10.4.3—Effects of Curved Tendons

Reinforcement shall be used to confine curved tendons. 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. Spacing of the confinement reinforcement shall not exceed either 3.0 times the outside diameter of the duct or 24.0 in.

Where tendons are located in curved webs or flanges or are curved around and close to re-entrant corners or internal voids, additional concrete cover and/or confinement reinforcement shall be provided. The distance between a re-entrant corner or void and the near edge of the duct shall not be less than 1.5 duct diameters.

When a tendon curves in two planes, the in-plane and out-of-plane forces shall be added together vectorially.

C5.10.4.3

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.

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.

It is not the purpose of this Article to encourage the use of curved tendons around re-entrant corners or voids. Where possible, this type of detail should be avoided.

5.10.4.3.1—In-Plane Force Effects

In-plane deviation force effects due to the change in direction of tendons shall be taken as:

$$F_{u\text{-}in} = \frac{P_u}{R} \qquad (5.10.4.3.1\text{-}1)$$

where:

C5.10.4.3.1

In-plane forces occur, for example, in anchorage blisters or curved webs, as shown in Figures C5.10.4.3.1-1 and C5.10.4.3.1-2. 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.

$F_{u\text{-}in}$ = the in-plane deviation force effect per unit length of tendon (kips/ft)

P_u = the tendon force factored as specified in Article 3.4.3 (kip)

R = the 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 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.

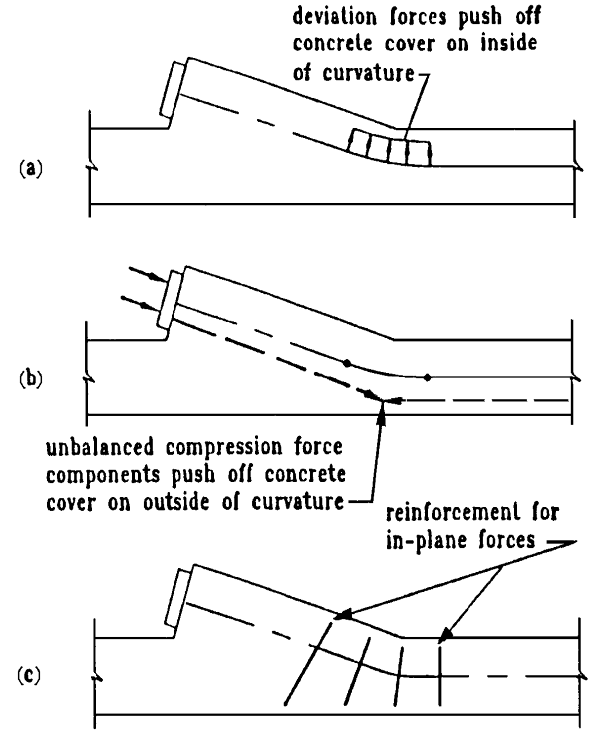


Figure C5.10.4.3.1-1—In-Plane Forces in a Blister

The shear resistance of the concrete cover against pull-out by deviation forces, V_r, shall be taken as:

$$V_r = \phi V_n \tag{5.10.4.3.1-2}$$

in which:

$$V_n = 0.125 d_c \sqrt{f'_{ci}} \tag{5.10.4.3.1-3}$$

where:

V_n = nominal shear resistance of two shear planes per unit length (kips/in.)

ϕ = resistance factor for shear specified in Article 5.5.4.2

d_c = minimum concrete cover over the tendon duct, plus one-half of the duct diameter (in.)

The two shear planes for which Eq. 5.10.4.3.1-3 gives V_n are as indicated in Figure C5.10.4.3.1-2 for single and multiple tendons.

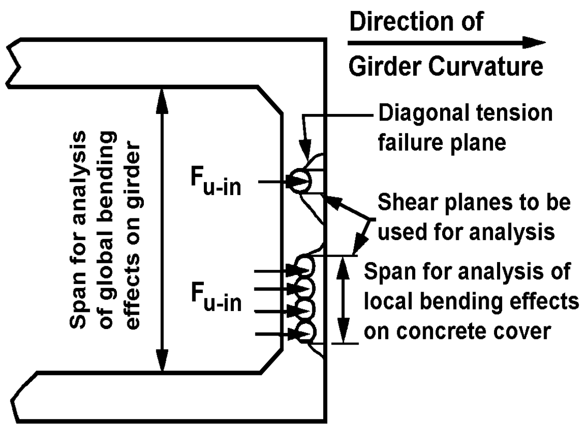


Figure C5.10.4.3.1-2—In-Plane Force Effects in Curved Girders Due to Horizontally Curved Tendons

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)

If the factored in-plane deviation force exceeds the factored shear resistance of the concrete cover, as specified in Eq. 5.10.4.3.1-2, fully anchored tie-backs to resist the in-plane deviation forces shall be provided in the form of either nonprestressed or prestressed reinforcement.

Where stacked ducts are used in curved girders, the moment resistance of the concrete cover, acting in flexure, shall be investigated.

For curved girders, the global flexural effects of out-of-plane forces shall be investigated.

Where 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. 5.10.4.3.1-2;
- Providing confinement reinforcement to resist the radial force; or
- Specifying that each inner duct be grouted before the adjacent outer duct is stressed.

5.10.4.3.2—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:

$$F_{u\text{-}out} = \frac{P_u}{\pi R} \qquad (5.10.4.3.2\text{-}1)$$

where:

$F_{u\text{-}out}$ = out-of-plane force effect per unit length of tendon (kip/ft)

P_u = tendon force, factored as specified in Article 3.4.3 (kip)

R = radius of curvature of the tendon in a vertical plane at the considered location (ft)

C5.10.4.3.2

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 C5.10.4.3.2-1. Small out-of-plane forces may be resisted by concrete in shear; otherwise, spiral reinforcement is most effective to resist out-of-plane forces.

If the factored shear resistance given by Eq. 5.10.4.3.1-2 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.

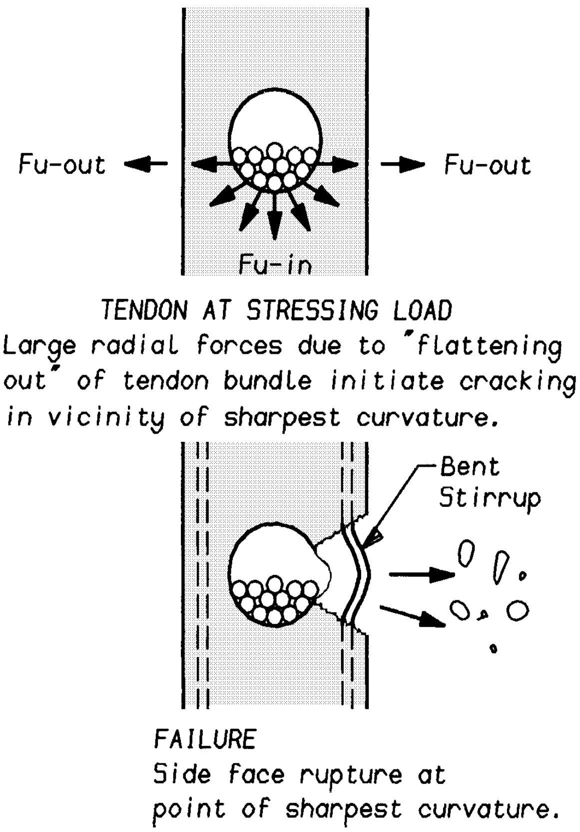


Figure C5.10.4.3.2-1—Effects of Out-of-Plane Forces

5.10.5—External Tendon Supports

Unless a vibration analysis indicates otherwise, the unsupported length of external tendons shall not exceed 25.0 ft.

5.10.6—Transverse Reinforcement for Compression Members

5.10.6.1—General

The provisions of Article 5.10.11 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.

C5.10.6.1

Article 5.10.11.2 applies to Seismic Zone 1 but has no additional requirements for transverse reinforcement for compression members.

5.10.6.2—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 either 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 5.10.11.3 and 5.10.11.4.1 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 5.10.11.4.3.

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.

5.10.6.3—Ties

In tied compression members, all longitudinal bars or bundles shall be enclosed by lateral ties that shall be equivalent to:

- 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 shall not exceed the least dimension of the compression member or 12.0 in. Where two or more bars larger than No. 10 are bundled together, the spacing shall not exceed half the least dimension of the member or 6.0 in.

Deformed wire or welded wire fabric of equivalent area may be used instead of bars.

C5.10.6.3

Figure C5.10.6.3-1 illustrates the placement of restraining ties in compression members which are not designed for plastic hinging.

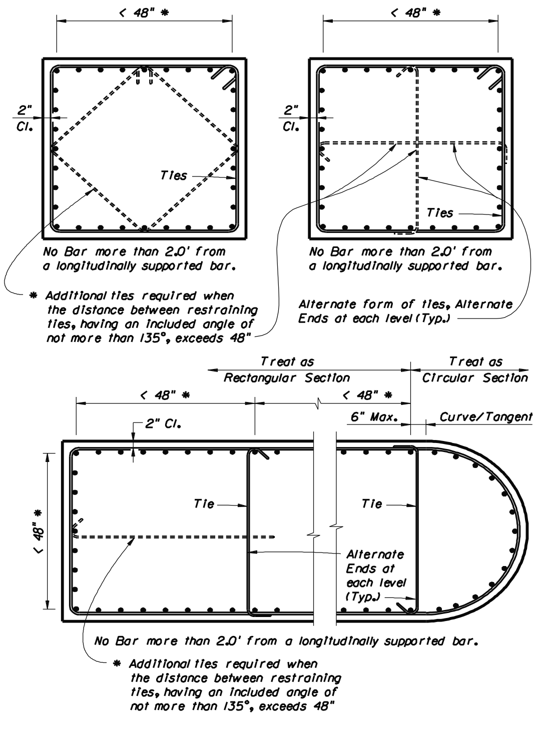


Figure C5.10.6.3-1—Acceptable Tie Arrangements

No longitudinal bar or bundle shall be more than 24.0 in., measured along the tie, from a restrained bar or bundle. A restrained bar or bundle is one which has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. 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 tie from such a laterally supported bar or bundle and the tie reinforcement shall meet the requirements of Articles 5.10.11.4.1d through 5.10.11.4.1f. Where the bars or bundles are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

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.

Columns in Seismic Zones 2, 3, and 4 are designed for plastic hinging. The plastic hinge zone is defined in Article 5.10.11.4.1c. Additional requirements for transverse reinforcement for bridges in Seismic Zones 2, 3, and 4 are specified in Articles 5.10.11.3 and 5.10.11.4.1. Plastic hinging may be used as a design strategy for other extreme events, such as ship collision.

5.10.7—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 5.10.6 or by welded wire fabric of equivalent area.

5.10.8—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 fabric, or prestressing tendons.

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.8\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \qquad (5.10.8\text{-}2)$$

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 yield strength of reinforcing bars ≤75 ksi

C5.10.8

The comparable equation in ACI 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 and 207.2R. The coefficient in Eq. 5.10.8-1 is the product of 0.0018, 60 ksi, and 12.0 in./ft and, therefore, has the units kips/in.-ft.

Eq. 5.10.8-1 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:

$$A_s \geq \frac{1.3A_g}{Perimeter(fy)} \qquad (C5.10.8\text{-}1)$$

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:

- 3.0 times the component thickness, or 18.0 in.
- 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 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 in. or less in thickness
- Side faces of buried footings 36 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 5.10.3.4. Where the spacing is greater than 54.0 in., bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

Permanent prestress of 0.11 ksi is equivalent to the resistance of the steel specified in Eq. 5.10.8-1 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.

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.

5.10.9—Post-Tensioned Anchorage Zones

5.10.9.1—General

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.

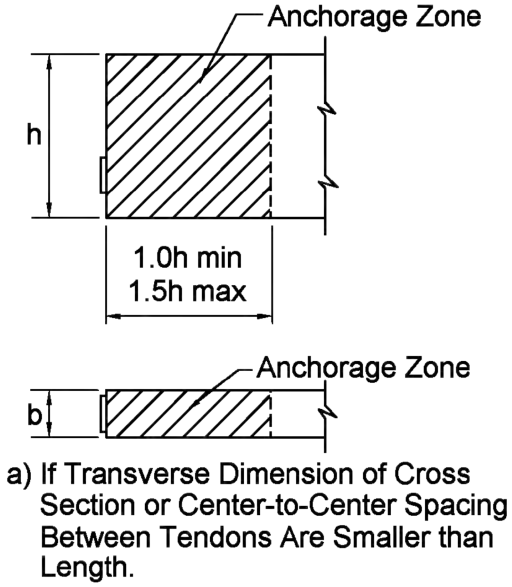
C5.10.9.1

With slight modifications, the provisions of Article 5.10.9 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 C5.10.9.1-1.



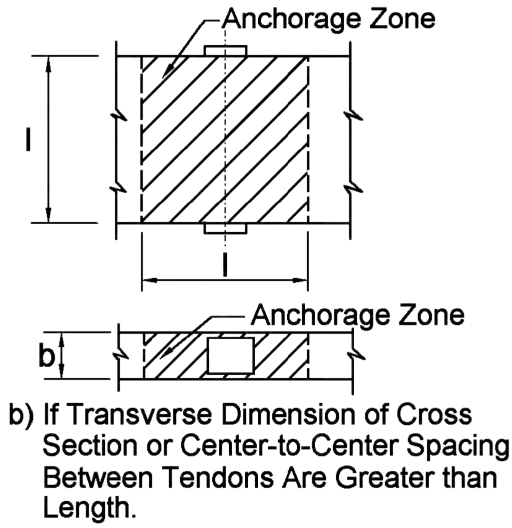


Figure C5.10.9.1-1—Geometry of the Anchorage Zones

5.10.9.2—General Zone and Local Zone

5.10.9.2.1—General

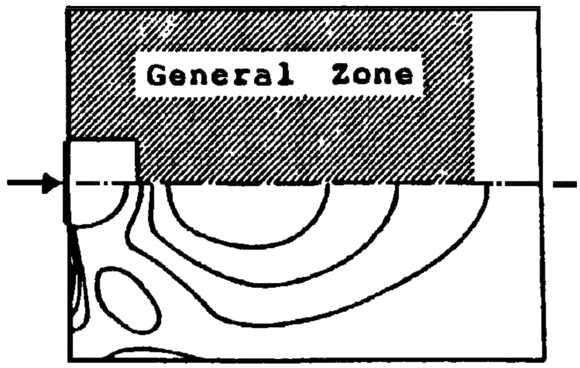
For design purposes, the anchorage zone shall be considered as comprised of two regions:

- The general zone, for which the provisions of Article 5.10.9.2.2 apply, and
- The local zone, for which the provisions of Article 5.10.9.2.3 apply.

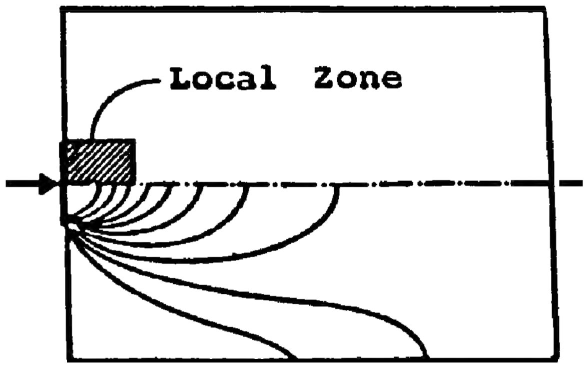
C5.10.9.2.1

For intermediate anchorages, large tensile stresses may exist behind the anchor. These tensile stresses result from the compatibility of deformations ahead of and behind the anchorage.

Figure C5.10.9.1-1 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 C5.10.9.1-1a). The region of high compressive stresses immediately ahead of the anchorage device is the local zone (Figure C5.10.9.1-1b).



a) Principal Tensile Stresses and the General Zone



b) Principal Compressive Stresses and the Local Zone

Figure C5.10.9.2.1-1—General Zone and Local Zone

5.10.9.2.2—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 5.10.9.1.

Design of general zones shall comply with the requirements of Article 5.10.9.3.

C5.10.9.2.2

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, such as high bearing and confining stresses, and general zone effects, such as 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.

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5.10.9.2.3—Local Zone

Design of local zones shall either comply with the requirements of Article 5.10.9.7 or be based on the results of acceptance tests as specified in Article 5.10.9.7.3 and 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.

C5.10.9.2.3

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 immediately ahead of the anchorage device and any integral confining reinforcement. The dimensions of the local zone are defined in Article 5.10.9.7.1.

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.

5.10.9.2.4—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 5.10.9.7. 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 5.10.9.7.3 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.

C5.10.9.2.4

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.

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.

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5.10.9.3—Design of the General Zone

5.10.9.3.1—Design Methods

For the design of general zones, the following design methods, conforming to the requirements of Article 5.10.9.3.2, may be used:

- Equilibrium-based inelastic models, generally termed as "strut-and-tie models;"
- Refined elastic stress analyses as specified in Section 4; or
- Other approximate methods, 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 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.

C5.10.9.3.1

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 5.10.9.6 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 5.10.9.4 and 5.10.9.5 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.

5.10.9.3.2—Design Principles

Compressive stresses in the concrete ahead of basic anchorage devices shall satisfy the requirements of Article 5.10.9.7.2.

The compressive stresses in the concrete ahead of the anchorage device shall be investigated at a distance, measured from the concrete bearing surface, not less than:

- The depth to the end of the local confinement reinforcement, or
- The smaller lateral dimension of the anchorage device.

C5.10.9.3.2

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.

These compressive stresses may be determined using the strut-and-tie model procedures of Article 5.10.9.4, an elastic stress analysis according to Article 5.10.9.5, or the approximate method outlined in Article 5.10.9.6.2.

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 model procedures of Article 5.10.9.4, an elastic stress analysis according to Article 5.10.9.5, or the approximate method outlined in Article 5.10.9.6.3. Three-dimensional effects shall be considered for the determination of the bursting reinforcement requirements.

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. 5.10.9.6.3-2;
- 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.

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 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 C5.10.9.3.2-1, may be considered acceptable.

The edge tension forces may be determined using the strut-and-tie models, procedures of Article 5.10.9.4, elastic analysis according to Article 5.10.9.5, or approximate methods of Article 5.10.9.6.4.

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.

Strut-and-tie models may be used for larger anchor spacings.

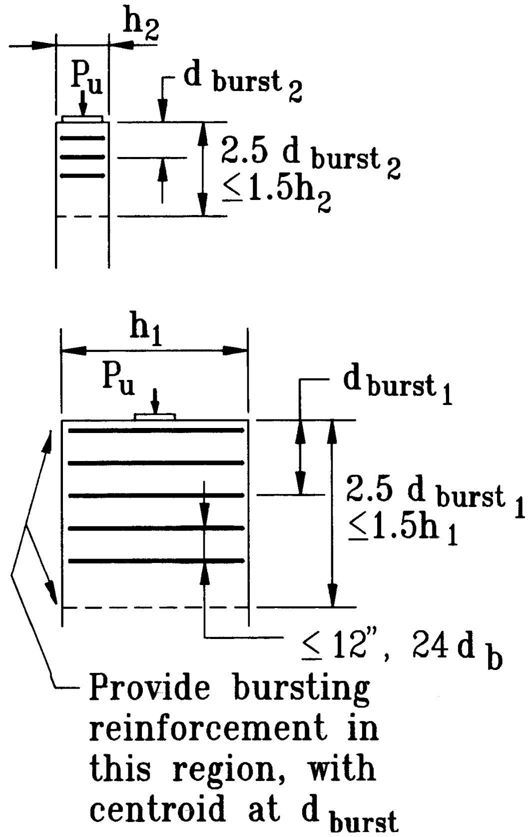


Figure C5.10.9.3.2-1—Arrangement for Bursting Reinforcement

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.

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 C5.10.9.3.2-1. 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.

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.

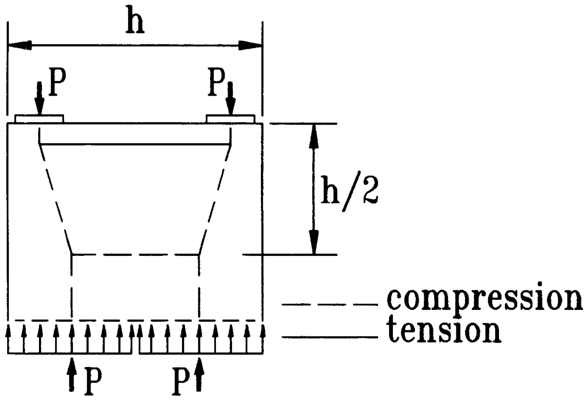


Figure C5.10.9.3.2-2—Path of Forces for Multiple Anchorages

Figure C5.10.9.3.2-3 illustrates the location of the edge tension forces.

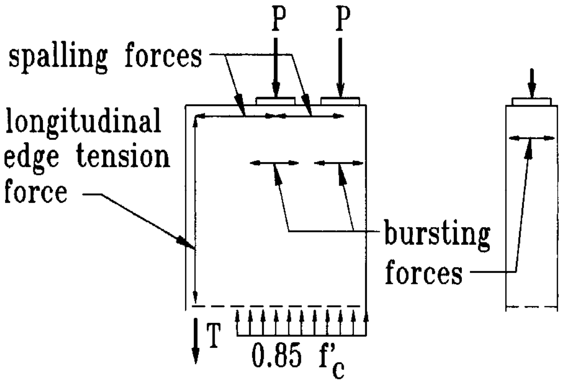


Figure C5.10.9.3.2-3—Edge Tension Forces

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 C5.10.9.3.2-4 illustrates the reinforcement requirements for anchorage zones.

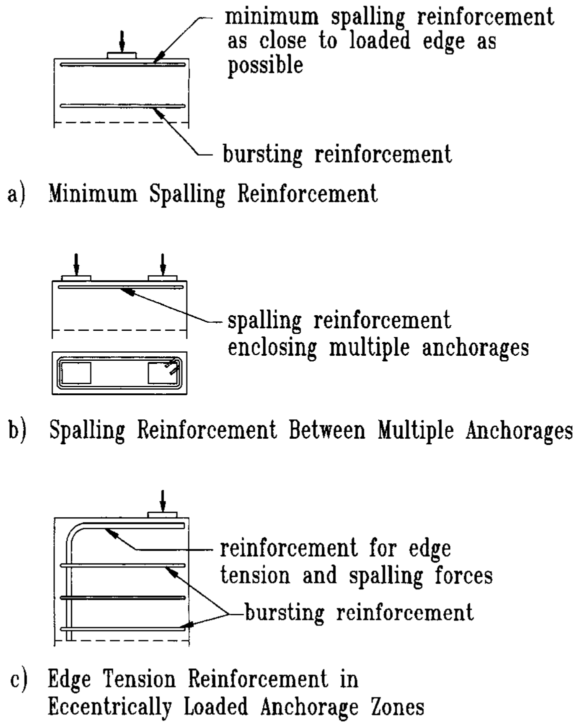


Figure C5.10.9.3.2-4—Arrangement of Anchorage Zone Reinforcement

5.10.9.3.3—Special Anchorage Devices

Where special anchorage devices that do not satisfy the requirements of Article 5.10.9.7.2 are to be used, reinforcement similar in configuration and at least equivalent in volumetric ratio to the supplementary skin reinforcement permitted under the provisions of the *AASHTO LRFD Bridge Construction Specifications*, Article 10.3.2.3.4, shall be furnished in the corresponding regions of the anchorage zone.

5.10.9.3.4—Intermediate Anchorages

5.10.9.3.4a—General

Intermediate anchorages shall not be used in regions where significant tension is generated behind the anchor from other loads. Whenever 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.

C5.10.9.3.4a

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.

5.10.9.3.4b—Tie-Backs

Unless otherwise specified herein, bonded reinforcement shall be provided to tie-back at least 25 percent of the intermediate anchorage unfactored stressing force into the concrete section behind the anchor. Stresses in this bonded reinforcement shall not exceed a maximum of $0.6f_y$ or 36 ksi. If permanent compressive stresses are generated behind the anchor from other loads, the amount of tie-back reinforcement may be reduced using Eq. 5.10.9.3.4b-1.

$$T_{ia} = 0.25\,P_s - f_{cb}\,A_{cb} \qquad (5.10.9.3.4b\text{-}1)$$

where:

T_{ia} = the tie-back tension force at the intermediate anchorage (kip)

P_s = the maximum unfactored anchorage stressing force (kip)

f_{cb} = the unfactored dead load compressive stress in the region behind the anchor (ksi)

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})

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 a distance of one plate width or half the length of the blister or rib ahead of the anchor as well as at the same distance behind the anchor. The centroid of this reinforcement shall coincide with the tendon axis, where possible. For blisters and ribs, the reinforcement shall be placed in the continuing section near that face of the flange or web from which the blister or rib is projecting.

5.10.9.3.4c—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.

C5.10.9.3.4c

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.

Reinforcement, as specified in Article 5.10.9.3.2, shall be provided to resist tensile forces due to transfer of the anchorage force from the blister or rib into the overall structure.

5.10.9.3.5—Diaphragms

For tendons anchored in diaphragms, concrete compressive stresses shall be limited within the diaphragm as specified in Article 5.10.9.3.2 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 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.

C5.10.9.3.5

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 strut-and-tie models or elastic analysis. Approximate methods, such as the symmetric prism, suggested by Guyon (1953), do not apply.

The more general methods of Article 5.10.9.4 or Article 5.10.9.5 are used to determine this reinforcement.

5.10.9.3.6—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 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.

$$T_1 = 0.10\,P_u\left(1-\frac{a}{s}\right) \quad (5.10.9.3.6\text{-}1)$$

$$T_2 = 0.20\,P_u\left(1-\frac{a}{s}\right) \quad (5.10.9.3.6\text{-}2)$$

where:

T_1 = the edge tension force (kip)

T_2 = the bursting force (kip)

C5.10.9.3.6

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 5.10.9.3.2.

Reinforcement to resist edge tension force is placed in the plane of the slab and normal to the tendon axis.

P_u = the factored tendon load on an individual anchor (kip)

a = the anchor plate width (in.)

s = the anchorage spacing (in.)

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.

5.10.9.3.7—Deviation Saddles

Deviation saddles shall be designed using the strut-and-tie model or using methods based on test results.

C5.10.9.3.7

Deviation saddles are disturbed regions of the structure and can be designed using the strut-and-tie model. Tests of scale-model deviation saddles have provided important information on the behavior of deviation saddles regions. Design and detailing guidelines presented in Beaupre et al. (1988) should result in safe and serviceable designs.

5.10.9.4—Application of the Strut-and-Tie Model to the Design of General Zone

5.10.9.4.1—General

The flow of forces in the anchorage zone may be approximated by a strut-and-tie model as specified in Article 5.6.3.

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.

C5.10.9.4.1

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 C5.10.9.4.1-1 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 C5.10.9.4.1-2.

Figure C5.10.9.4.1-3 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.

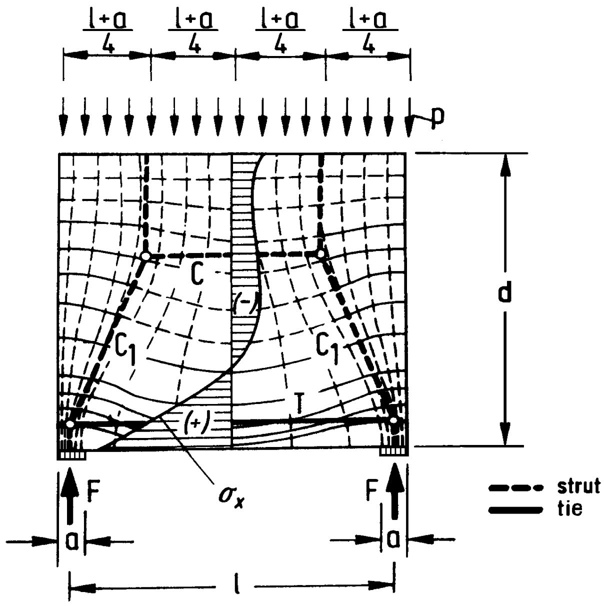


Figure C5.10.9.4.1-1—Principal Stress Field and Superimposed Strut-and-Tie Model

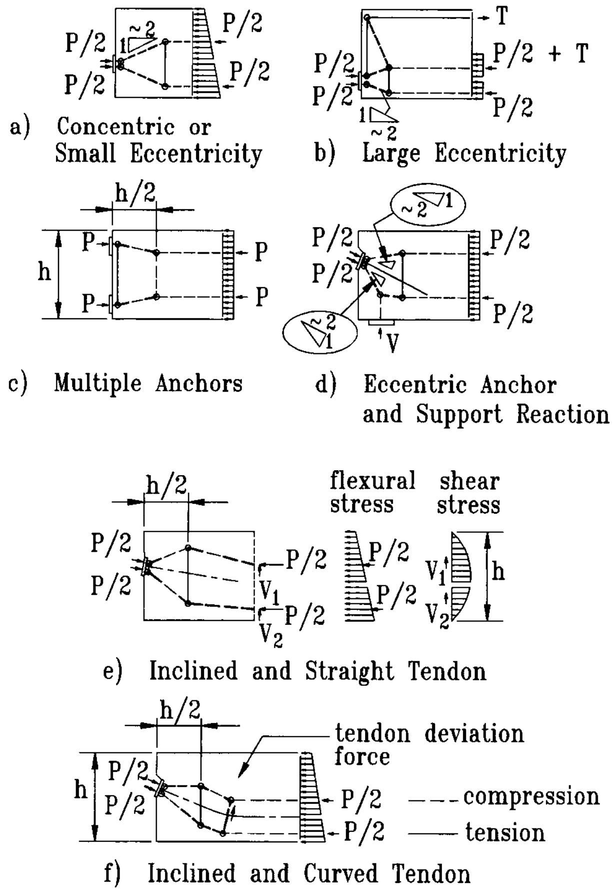


Figure C5.10.9.4.1-2—Strut-and-Tie Models for Selected Anchorage Zones

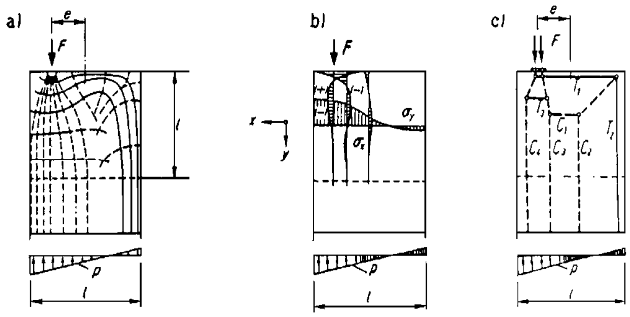


Figure C5.10.9.4.1-3—Strut-and-Tie Model for the Outer Regions of the General Zone

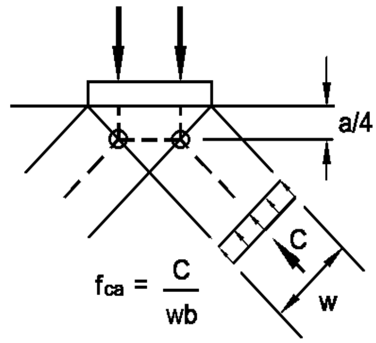
5.10.9.4.2—Nodes

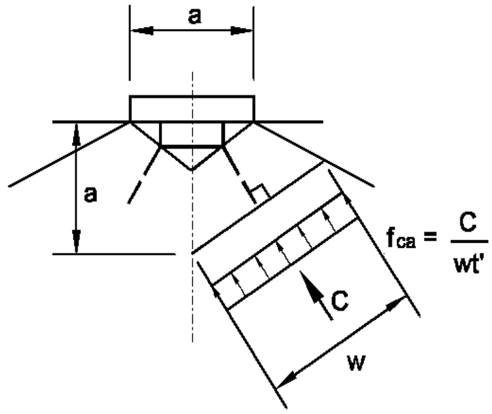
Local zones that satisfy the requirements of Article 5.10.9.7 or Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications* may be considered as properly detailed and are adequate nodes. The other nodes in the anchorage zone may be considered adequate if the effective concrete stresses in the struts satisfy the requirements of Article 5.10.9.4.3, and the tension ties are detailed to develop the full yield strength of the reinforcement.

C5.10.9.4.2

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 5.10.9.7, 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*.

The local zone nodes for the development of a strut-and-tie model may be selected at a depth of $a/4$ ahead of the anchorage plate, as shown in Figure C5.10.9.4.2-1.





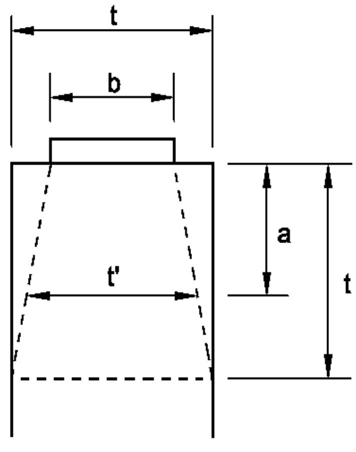


Figure C5.10.9.4.2-1—Critical Sections for Nodes and Compressive Struts

5.10.9.4.3—Struts

The factored compressive stress shall not exceed the limits specified in Article 5.10.9.3.1.

In anchorage zones, the critical section for compression 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.

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 compression strut varies linearly from the transverse lateral dimension of the anchor at the surface of the concrete to the total thickness of the section at a depth equal to the thickness of the section.

The compression stresses should be assumed to act parallel to the axis of the strut and to be uniformly distributed over its cross-section.

C5.10.9.4.3

For strut-and-tie models oriented on the elastic stress distribution, the nominal concrete strength specified in Article 5.10.9.3.1 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.

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 C5.10.9.4.2-1(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, as shown in Figure C5.10.9.4.2-1(b) (Burdet, 1990).

The determination of the dimension of the strut in the direction of the thickness of the member is illustrated in Figure C5.10.9.4.2-1(c).

5.10.9.4.4—Ties

Ties consisting of nonprestressed or prestressed reinforcement shall resist the total tensile force.

Ties shall extend beyond the nodes to develop the full-tension 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.

C5.10.9.4.4

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.

5.10.9.5—Elastic Stress Analysis

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.

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.

C5.10.9.5

Elastic analysis of anchorage zone problems has been found acceptable and useful, even though the development of cracks in the anchorage zone may cause stress redistributions (Burdet, 1990).

Results of a linear elastic analysis can be adjusted by smoothing out local stress maxima to reflect the nonlinear behavior of concrete at higher stresses.

The location and magnitude of the bursting force should be obtained by integration of the tensile bursting stresses along the tendon path. This procedure gives a conservative estimate of the reinforcement required in the anchorage zone. A reinforcement arrangement deviating from the elastic stress distribution, i.e., a uniform distribution of bursting reinforcement, is acceptable as long as the centroid of the bursting reinforcement coincides with the location of the bursting force.

5.10.9.6—Approximate Stress Analyses and Design

5.10.9.6.1—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. 5.10.9.6.2-1 through 5.10.9.6.3-2, 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. 5.10.9.6.3-1 and 5.10.9.6.3-2, is between −5.0 degrees and +20.0 degrees.

C5.10.9.6.1

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 C5.10.9.6.1-1 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.

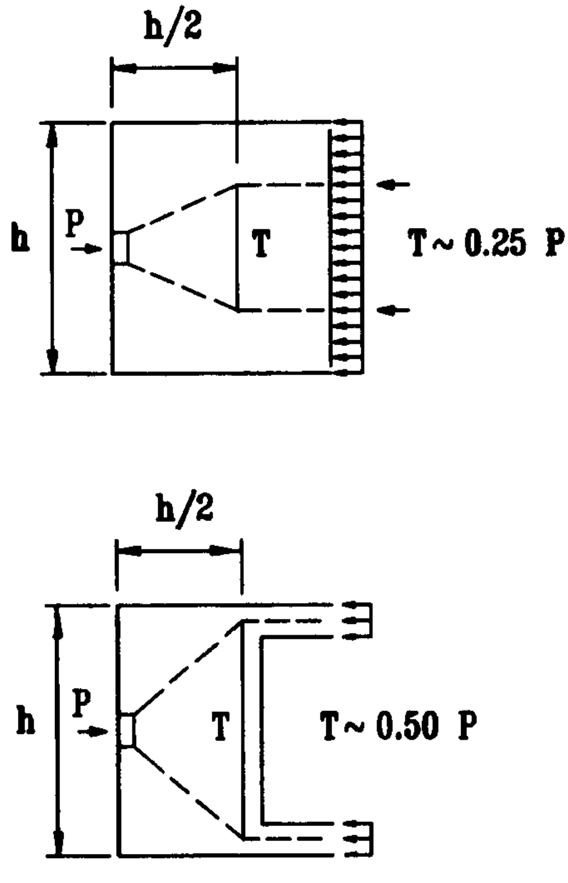


Figure C5.10.9.6.1-1—Effect of Discontinuity in Anchorage Zone

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 C5.10.9.6.1-2. The approximate equations for bursting forces are based on finite element analyses for a single anchor acting on a rectangular cross-section. Eq. 5.10.9.6.3-1 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. 5.10.9.6.3-2.

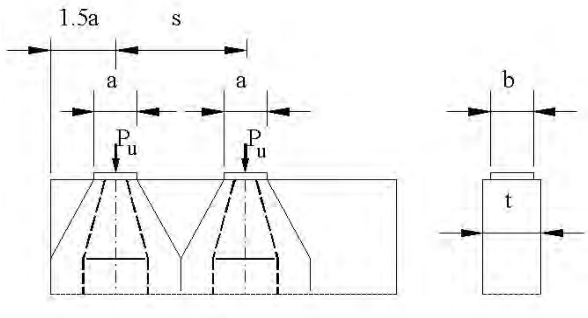


Figure C5.10.9.6.1-2—Edge Distances and Notation

5.10.9.6.2—Compressive Stresses

The concrete compressive stress ahead of the anchorage devices, f_{ca}, calculated using Eq. 5.10.9.6.2-1, shall not exceed the limit specified in Article 5.10.9.3.1:

$$f_{ca} = \frac{0.6 P_u \kappa}{A_b \left(1 + \ell_c \left(\frac{1}{b_{eff}} - \frac{1}{t}\right)\right)} \tag{5.10.9.6.2-1}$$

C5.10.9.6.2

This check of concrete compressive stresses is not required for basic anchorage devices satisfying Article 5.10.9.7.2.

Eqs. 5.10.9.6.2-1 and 5.10.9.6.2-2 are based on a strut-and-tie model for a single anchor with the concrete stresses determined as indicated in Figure C5.10.9.6.2-1 (Burdet, 1990), with the anchor plate width, *b*, and member thickness, *t*, being equal. Eq. 5.10.9.6.2-1 was modified to include cases with values of $b < t$.

in which:

if $a \le s < 2a_{eff}$, then :

$$\kappa = 1 + \left(2 - \frac{s}{a_{eff}}\right)\left(0.3 + \frac{n}{15}\right) \quad (5.10.9.6.2\text{-}2)$$

if $s \ge 2a_{eff}$, then :

$$\kappa = 1 \quad (5.10.9.6.2\text{-}3)$$

where:

κ = correction factor for closely spaced anchorages

a_{eff} = lateral dimension of the effective bearing area measured parallel to the larger dimension of the cross-section (in.)

b_{eff} = lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross-section (in.)

P_u = factored tendon force (kip)

t = member thickness (in.)

s = center-to-center spacing of anchorages (in.)

n = number of anchorages in a row

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.)

A_b = effective bearing area (in.^{2})

The effective bearing area, A_b, in Eq. 5.10.9.6.2-1 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:

- If A_{plate} controls, A_{plate} shall not be taken larger than $4/\pi A_{conf}$.
- If A_{conf} controls, the maximum dimension of A_{conf} shall not be more than twice the maximum dimension of A_{plate} or three times the minimum dimension of A_{plate}. If any of these limits is violated, the effective bearing area, A_b, shall be based on A_{plate}.
- 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. 5.10.9.6.2-1.

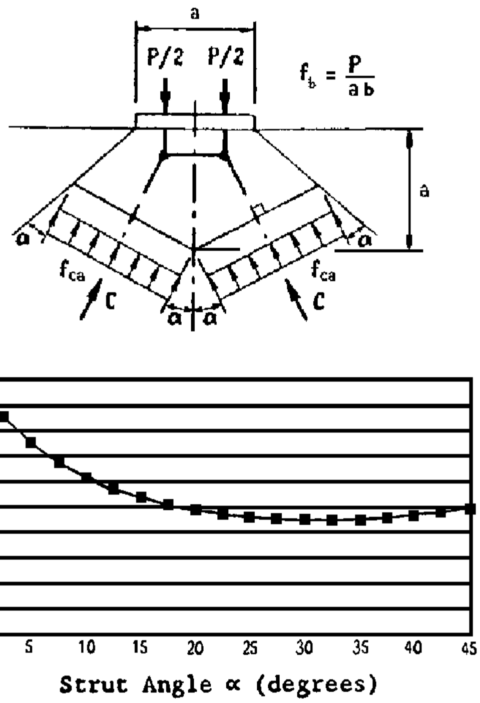


Figure C5.10.9.6.2-1—Local Zone and Strut Interface

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 C5.10.9.6.2-2.

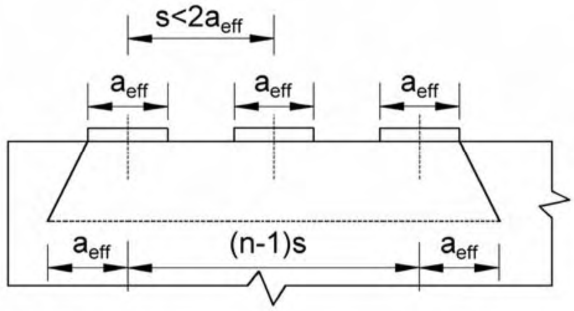


Figure C5.10.9.6.2-2—Closely Spaced Multiple Anchorages

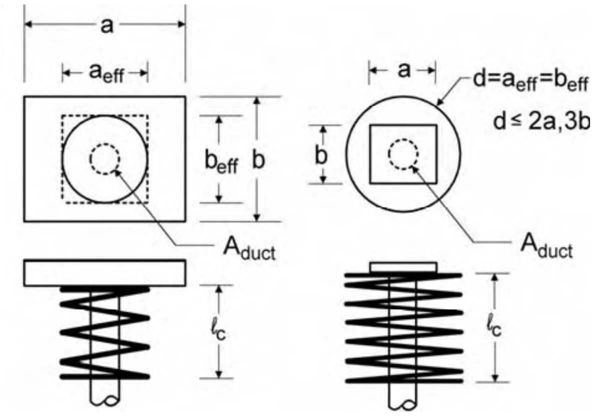


Figure C5.10.9.6.2-3—Effective Bearing Area

5.10.9.6.3—Bursting Forces

The bursting forces in anchorage zones, T_{burst}, may be taken as:

$$T_{burst} = 0.25 \sum P_u \left(1 - \frac{a}{h}\right) + 0.5 \left|\Sigma \left(P_u \sin \alpha\right)\right| \tag{5.10.9.6.3-1}$$

The location of the bursting force, d_{burst}, may be taken as:

$$d_{burst} = 0.5(h - 2e) + 5e \sin \alpha \tag{5.10.9.6.3-2}$$

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)

C5.10.9.6.3

Eqs. 5.10.9.6.3-1 and 5.10.9.6.3-2 are based on the results of linear elastic stress analyses (Burdet, 1990). Figure C5.10.9.6.3-1 illustrates the terms used in the equations.

d_{burst} = distance from anchorage device to the centroid of the bursting force, T_{burst} (in.)

a = lateral dimension of the anchorage device or group of devices in the direction considered (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.)

h = lateral dimension of the cross-section in the direction considered (in.)

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.

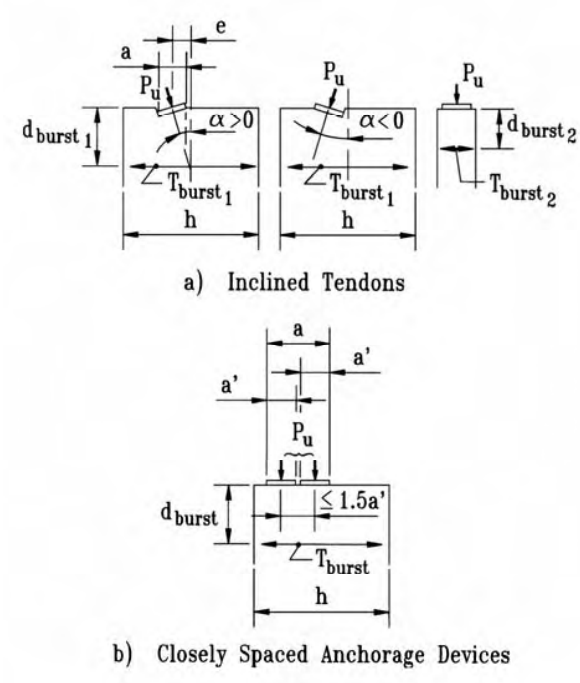


Figure C5.10.9.6.3-1—Notation for Eqs. 5.10.9.6.3-1 and 5.10.9.6.3-2

5.10.9.6.4—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 5.10.9.3.2.

C5.10.9.6.4

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 C5.10.9.6.4-1. 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.

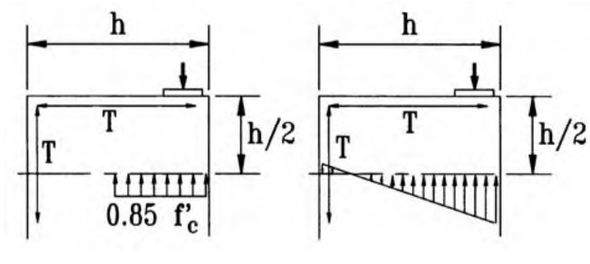


Figure C5.10.9.6.4-1—Determination of Edge Tension Forces for Eccentric Anchorages

5.10.9.7—Design of Local Zones

5.10.9.7.1—Dimensions of Local Zone

Where either:

- The manufacturer has not provided edge distance recommendations, or
- Edge distance have been recommended by the manufacturer, but they have not been independently verified.

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 reinforcing steel for the particular application and environment.

The cover required for corrosion protection shall be as specified in Article 5.12.3.

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.

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 to be less than:

- The maximum width of the local zone;
- The length of the anchorage device confining reinforcement; or
- 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.

C5.10.9.7.1

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 C5.10.9.7.1-1 illustrates the local zone.

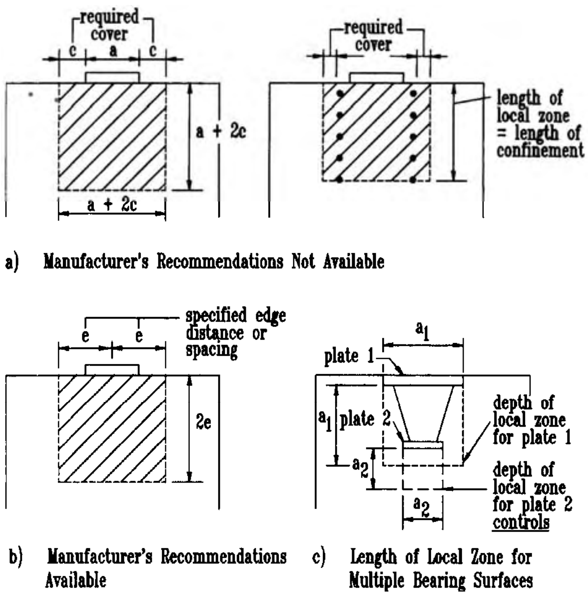


Figure C5.10.9.7.1-1—Geometry of the Local Zone

For closely spaced anchorages, an enlarged local zone enclosing all individual anchorages should also be considered.

5.10.9.7.2—Bearing Resistance

Normal anchorage devices shall comply with the requirements specified herein. Special anchorage devices shall comply with the requirements specified in Article 5.10.9.7.3.

When general zone reinforcement satisfying Article 5.10.9.3.2 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 5.10.9.7.1, the factored bearing resistance of anchorages shall be taken as:

$$P_r = \phi f_n A_b \tag{5.10.9.7.2-1}$$

for which f_n is the lesser of:

$$f_n = 0.7\, f'_{ci} \sqrt{\frac{A}{A_g}}\text{, and} \tag{5.10.9.7.2-2}$$

$$f_n = 2.25\, f'_{ci} \tag{5.10.9.7.2-3}$$

where:

ϕ = resistance factor specified in Article 5.5.4.2

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})

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} = nominal concrete strength at time of application of tendon force (ksi)

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:

C5.10.9.7.2

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 Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications*. Alternatively, these limits may be exceeded if an anchorage system passes the acceptance test. Figures C5.10.9.7.2-1, C5.10.9.7.2-2, and C5.10.9.7.2-3 illustrate the specifications of Article 5.10.9.7.2 (Roberts, 1990).

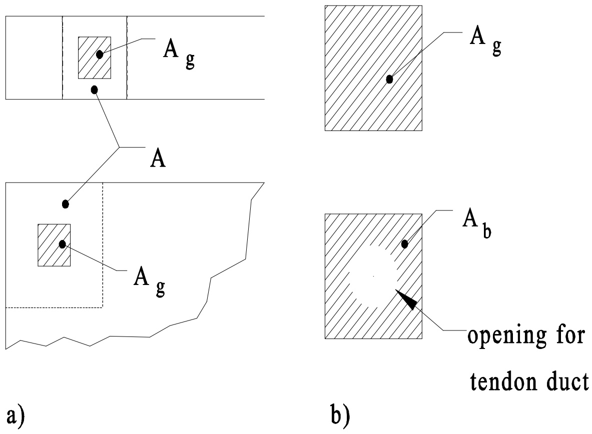


Figure C5.10.9.7.2-1—Area of Supporting Concrete Surface in Eq. 5.10.9.7.2-2

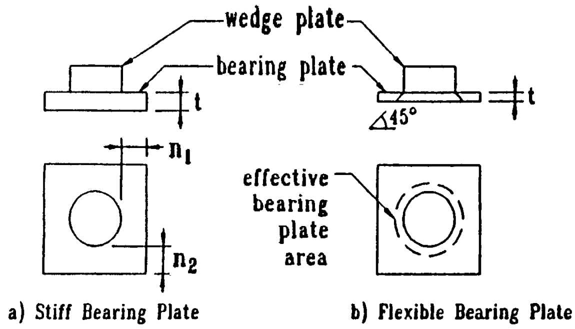


Figure C5.10.9.7.2-2—Effective Bearing Plate Area for Anchorage Devices with Separate Wedge Plate

$$n/t \leq 0.08\left(\frac{E_b}{f_b}\right)^{0.33} \quad (5.10.9.7.2\text{-}4)$$

where:

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)

n = projection of base plate beyond the wedge hole or wedge plate, as appropriate (in.)

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.

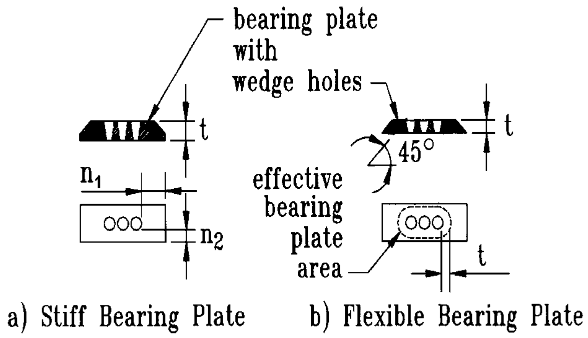


Figure C5.10.9.7.2-3—Effective Bearing Plate Area for Anchorage Device without Separate Wedge Plate

A larger effective bearing area may be calculated by assuming an effective area and checking the new f_b and n/t values.

5.10.9.7.3—Special Anchorage Devices

Special anchorage devices that do not satisfy the requirements specified in Article 5.10.9.7.2 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*.

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.

C5.10.9.7.3

Most anchorage devices fall in this category and still have to pass the acceptance test of Article 10.3.2.3 of *AASHTO LRFD Bridge 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 Article 10.3.2.3 of *AASHTO LRFD Bridge Construction Specifications*.

In addition to any required confining reinforcement, the acceptance test of special anchorage devices, supplementary skin reinforcement is permitted by Article 10.3.2.3.4 of *AASHTO LRFD Bridge Construction Specifications*. Equivalent reinforcement should also be placed in the actual structure. Other general zone reinforcement in the corresponding portion

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.

5.10.10—Pretensioned Anchorage Zones

5.10.10.1—Splitting Resistance

The splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

$$P_r = f_s A_s \tag{5.10.10.1-1}$$

where:

f_s = stress in steel not to exceed 20 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 $h/4$ 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 four 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.

of the anchorage zone may be counted toward this reinforcement requirement.

C5.10.10.1

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 C5.10.10.1-1 and C5.10.10.1-2 show examples of splitting reinforcement for tub girders and voided slabs.

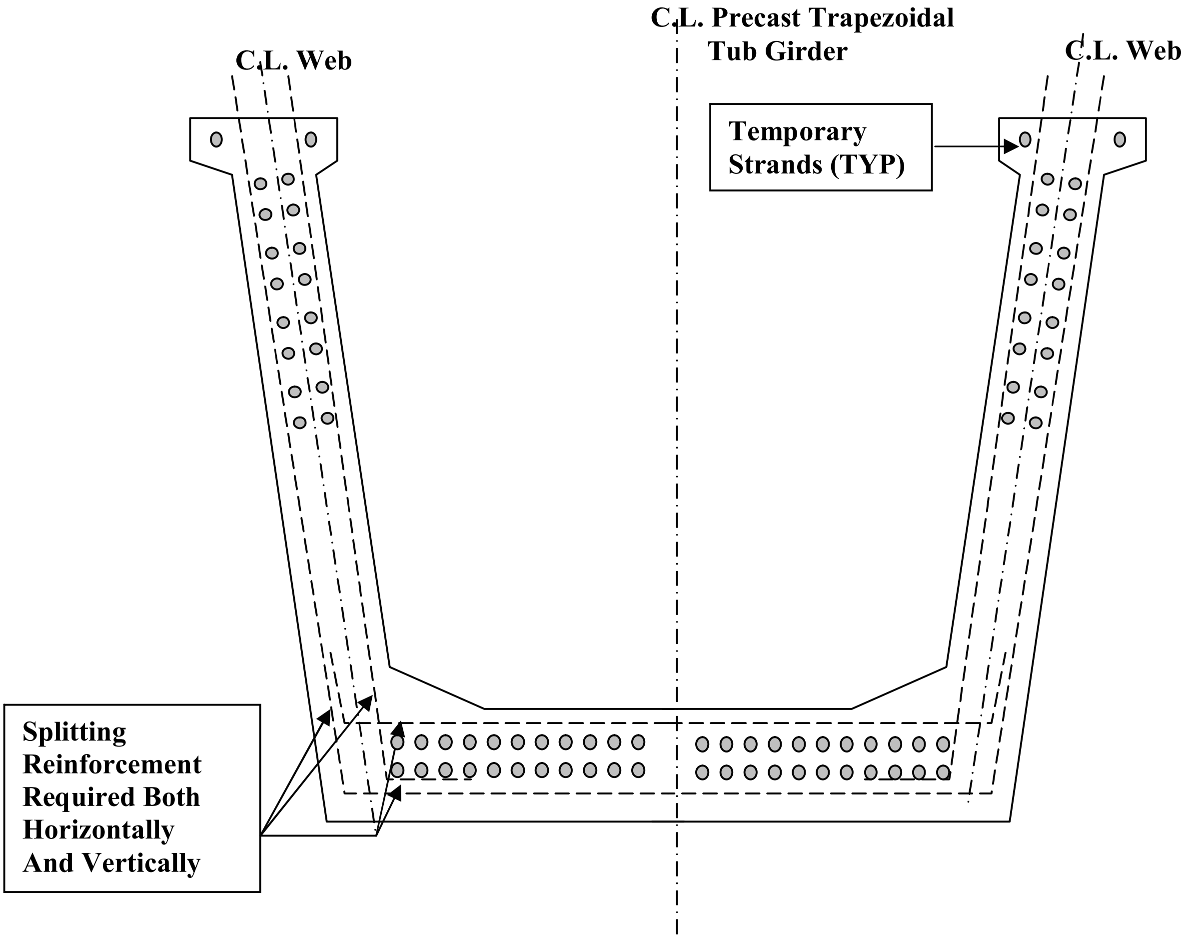


Figure C5.10.10.1-1—Precast Trapezoidal Tub Girder

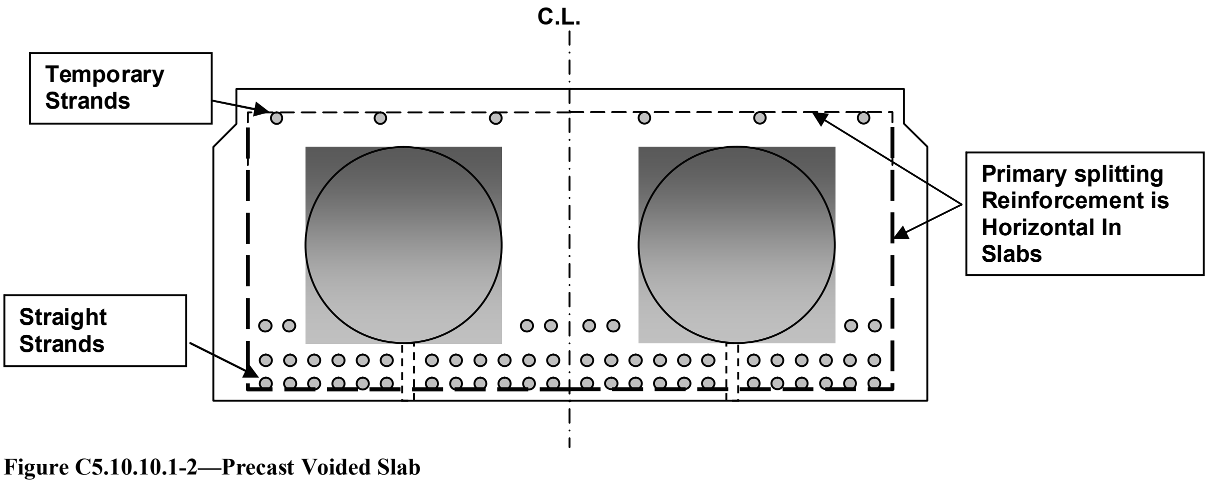


Figure C5.10.10.1-2—Precast Voided Slab

5.10.10.2—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.

5.10.11—Provisions for Seismic Design

5.10.11.1—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, reinforcing steel 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.

Bridges located in Seismic Zone 2 shall satisfy the requirements in Article 5.10.11.3. Bridges located in Seismic Zones 3 and 4 shall satisfy the requirements specified in Article 5.10.11.4.

C5.10.11.1

These Specifications are based on the work by the Applied Technology Council in 1979–1980. The Loma Prieta earthquake of 1989 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), and FHWA (2006).

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.

The Loma Prieta earthquake confirmed the vulnerability of columns with inadequate core confinement and inadequate anchorage of longitudinal reinforcement. New areas of concern that emerged include:

- Lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;

- Lack of adequate strength in joints between columns and bent caps under transverse dynamic loads; and

- Inadequate reinforcement for torsion, particularly in outrigger-type bent caps.

The purpose of the additional design requirements of this Article is to increase the probability that the design of the components of a bridge are consistent with the overall design philosophy of ATC 6, especially for bridges located in Seismic Zones 2, 3, and 4, and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this Article for bridges located in Seismic Zones 2, 3, and 4 are to ensure that a column is provided with reasonable ductility and is forced to yield in flexure and that the potential for a shear, compression, or loss of anchorage mode of failure is minimized. The additional design requirements for piers provide for some inelastic resistance; however, the R-factor specified for piers in Section 4 is to ensure that the anticipated inelastic resistance is significantly less than that of columns.

The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics,
- Design force level,
- Periods of vibration of the bridge,
- Shape of the inelastic hysteresis loop of the columns,
- Elastic damping coefficient,
- Contributions of foundation and soil conditions to structural flexibility, and
- Plastic hinge length of the column.

The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of vibration of the bridge. This ratio will be an indicator of the number of yield excursions and hence of the cumulative ductility demand.

5.10.11.2—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.

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, except that:

C5.10.11.2

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 end of the previous Zone 2. 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.

- 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 5.10.11.4.1d and 5.10.11.4.1e.

5.10.11.3—Seismic Zone 2

The requirements of Article 5.10.11.4 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.

C5.10.11.3

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 reinforcing steel. 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 1000 yr.

5.10.11.4—Seismic Zones 3 and 4

5.10.11.4.1—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 5.10.11.4.2 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

C5.10.11.4.1

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 5.10.11.4.1 or 5.10.11.4.2 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 5.10.11.4.2 would not be applicable.

5.10.11.4.1a—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.

C5.10.11.4.1a

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 one percent steel have also not exhibited good ductility (Halvorsen, 1987). The four 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 six 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 1000 years. The four percent figure is consistent with that recommended in recent publications by NCHRP (2002, 2006) and MCEER/ATC (2003).

5.10.11.4.1b—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 extreme load cases, as 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.

C5.10.11.4.1b

Columns are required to be designed biaxially and to be investigated for both the minimum and maximum axial forces. The previous edition of these Specifications reduced the flexural resistance factor from 0.9 to 0.5 as the axial load increased from 0 to $0.20 f'_c A_g$, because of the trend toward a reduction in ductility capacity as the axial load increases. This requirement is relaxed in this edition but a P-Δ requirement has been added (Article 4.7.4.5) to limit the demand on ductility capacity due to excessive deflection. Also, the R-factors have been maintained at their previous levels (Article 3.10.7) even though the return period of the design earthquake has been increased from 500 to 1,000 years. In both the NCHRP 12-49 and 20-7(193) provisions, the recommended flexural resistance factor is 1.0. However, since the current Specifications are force-based and do not explicitly calculate the ductility demand as in both 12-49 and 20-7(193) provisions, limiting the factor to 0.9 is considered justified in lieu of a more rigorous analysis.

5.10.11.4.1c—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 5.8.3.

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 5.8.3, 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 5.8.3 to zero at zero compression force.

C5.10.11.4.1c

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 end region shall be assumed to extend from the soffit 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:
    - 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 but shall not extend less than 18.0 in. above the mud line.

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 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.

5.10.11.4.1d—Transverse Reinforcement for Confinement at Plastic Hinges

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 5.10.11.4.1e.

For a circular column, the volumetric ratio of spiral or seismic hoop reinforcement, ρ_s, shall satisfy either that required in Article 5.7.4.6 or:

$$\rho_s \geq 0.12 \frac{f'_c}{f_y} \qquad (5.10.11.4.1d\text{-}1)$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

f_y = yield strength of reinforcing bars (ksi)

Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

C5.10.11.4.1d

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 5.10.11.4.1c should govern; these requirements are not in addition to those of Article 5.10.11.4.1c.

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.

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.

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall satisfy either:

$$A_{sh} \geq 0.30\, s h_c \frac{f'_c}{f_y}\left[\frac{A_g}{A_c} - 1\right] \qquad (5.10.11.4.1d\text{-}2)$$

or

$$A_{sh} \geq 0.12\, s h_c \frac{f'_c}{f_y} \qquad (5.10.11.4.1d\text{-}3)$$

where:

s = vertical spacing of hoops, not exceeding 4.0 in. (in.)

A_c = area of column core (in.^{2})

A_g = gross area of column (in.^{2})

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})

f_y = yield strength of tie or spiral reinforcement (ksi)

h_c = core dimension of tied column in the direction under consideration (in.)

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. All cross-ties shall have seismic hooks as specified in Article 5.10.2.2.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135 degrees, with an extension of not less than six diameters but not less than 3.0 in at one end and a hook of not less than 90 degrees with an extension of not less than six diameters at the other end.

Figures C5.10.11.4.1d-2 and C5.10.11.4.1d-4 illustrate the use of Eqs. 5.10.11.4.1d-2 and 5.10.11.4.1d-3. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column. Figure C5.10.11.4.1d-4 shows the distance to be utilized for h_c and the direction of the corresponding reinforcement for both principal directions of a rectangular column.

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 C5.10.11.4.1d-3. Spacing of longitudinal bars of a maximum of 8.0 in. center-to-center is also recommended to help confine the column core.

Examples of transverse column reinforcement are shown herein.

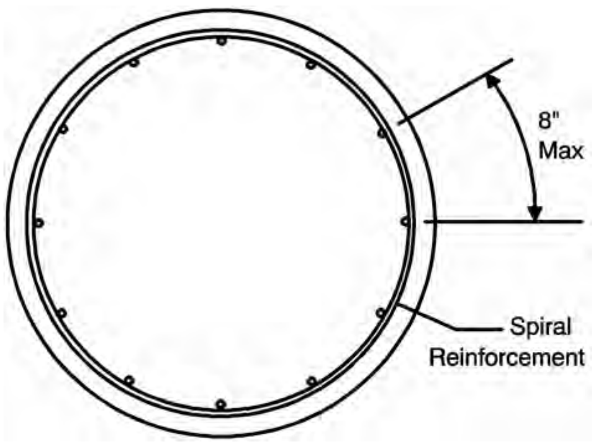


Figure C5.10.11.4.1d-1—Single Spiral

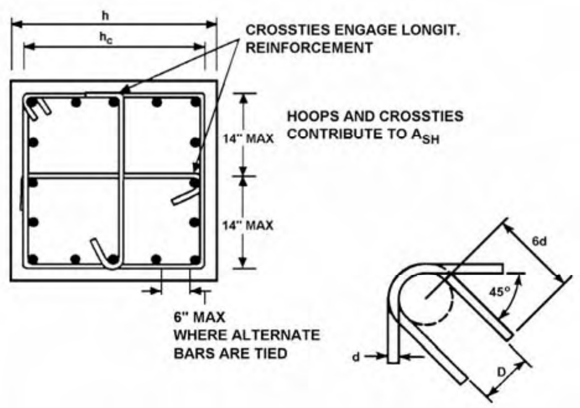


Figure C5.10.11.4.1d-2—Column Tie Details

- The hooks shall engage peripheral longitudinal bars.
- The 90-degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcing elements with 135- degree hooks having a six diameter but not less than a 3.0 in. extension at each end.
- A continuously wound tie shall have at each end a 135-degree hook with a six diameter but not less than a 3.0 in. extension that engages the longitudinal reinforcement.

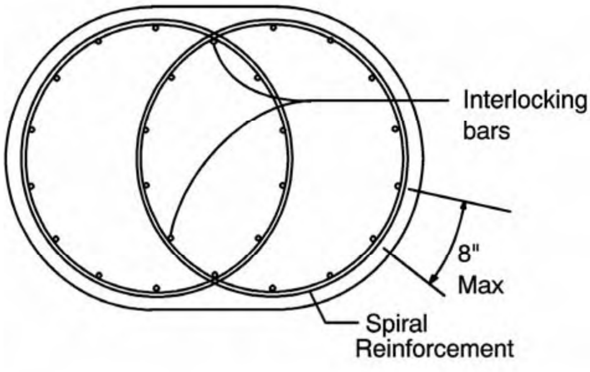


Figure C5.10.11.4.1d-3—Column Interlocking Spiral Details

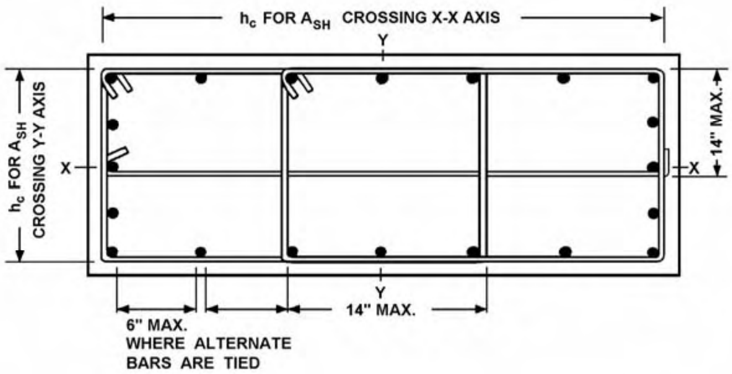


Figure C5.10.11.4.1d-4—Column Tie Details

5.10.11.4.1e—Spacing of Transverse Reinforcement for Confinement

Transverse reinforcement for confinement shall be:

- Provided at the top and bottom of the column 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 5.10.11.4.3;
- 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 not less than the maximum cross-sectional dimension or 18.0 in. above the mud line; and

- Spaced not to exceed one-quarter of the minimum member dimension or 4.0 in. center-to-center.

5.10.11.4.1f—Splices

The provisions of Article 5.11.5 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 4.0 in. or one-quarter of the minimum member dimension.

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 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.

C5.10.11.4.1f

It is 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.

Splices in seismic-critical elements should be designed for ultimate behavior under seismic deformation demands. Recommendations for acceptable strains are provided in Table C5.10.11.4.1f-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 C5.10.11.4.1f-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 latter of which is described in ACI (2001). 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 C5.10.11.4.1f-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 A615/A615M steel is generally not permitted by Caltrans because of weldability and ductility concerns, and was not investigated.

5.10.11.4.2—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 5.10.11.4.1, 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 take as the lesser of:

$$V_r = 0.253\sqrt{f_c'}bd\text{, and} \qquad (5.10.11.4.2\text{-}1)$$

$$V_r = \phi V_n \qquad (5.10.11.4.2\text{-}2)$$

in which:

$$V_n = \left[0.063\sqrt{f_c'} + \rho_h f_y\right]bd \qquad (5.10.11.4.2\text{-}3)$$

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.

C5.10.11.4.2

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.

5.10.11.4.3—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. The development length for all longitudinal steel shall be 1.25 times that required for the full yield strength of reinforcing as specified in Article 5.11.

Column transverse reinforcement, as specified in Article 5.10.11.4.1d, 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:

C5.10.11.4.3

A column connection, as referred to in this Article, is the vertical extension of the column area into the adjoining member.

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

- For normal weight aggregate concrete:

$V_n \leq 0.380\, bd\sqrt{f'_c}$, and (5.10.11.4.3-1)

- For lightweight aggregate concrete:

$V_n \leq 0.285\ bd\sqrt{f'_c}$ (5.10.11.4.3-2)

specified. The factored shear resistance for joints made with lightweight aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75 percent of that in normal weight aggregate concrete.

5.10.11.4.4—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:

$V_n = \left(A_{vf} f_y + 0.75P_u\right)$ (5.10.11.4.4-1)

where:

A_{vf} = the total area of reinforcement, including flexural reinforcement (in.^{2})

P_u = the minimum factored axial load as specified in Article 3.10.9.4 for columns and piers (kip)

C5.10.11.4.4

Eq. 5.10.11.4.4-1 is based on Eq. 11-26 of ACI 318-89 but is restated to reflect dowel action and frictional resistance.

5.10.12—Reinforcement for Hollow Rectangular Compression Members

5.10.12.1—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.

5.10.12.2—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.

5.10.12.3—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 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.

5.10.12.4—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.

5.10.12.5—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.

5.11—DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.1—General

5.11.1.1—Basic Requirements

The calculated force effects in the reinforcement at each section shall be developed on each side of that section by embedment length, hook, mechanical device, or a combination thereof. Hooks and mechanical anchorages may be used in developing bars in tension only.

C5.11.1.1

Most of the provisions in this Article are based on ACI 318-89 and its attendant commentary.

5.11.1.2—Flexural Reinforcement

5.11.1.2.1—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 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 5.11.2, 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:

C5.11.1.2.1

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 cut off 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 5.8, 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.

5.11.1.2.2—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.

C5.11.1.2.2

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 5.11.2.1, satisfied Eq. C5.11.1.2.2-1:

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \qquad \text{(C5.11.1.2.2-1)}$$

where:

M_n = nominal flexural strength, 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 = the 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. C5.11.1.2.2-1 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. C5.11.1.2.2-1 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 C5.11.1.2.2-1.

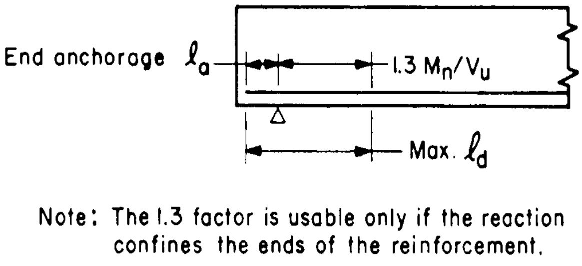


Figure C5.11.1.2.2-1—End Confinement

These provisions are now supplemented by the provisions of Article 5.8, which account for the need to provide longitudinal reinforcement to react the horizontal component of inclined compression diagonals that contribute to shear resistance.

Reinforcement with specified yield strengths in excess of 75.0 ksi may require longer extensions than required by this Article.

5.11.1.2.3—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 effective depth of the member,
- 12.0 times the nominal diameter of bar, and
- 0.0625 times the clear span.

5.11.1.2.4—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.

C5.11.1.2.4

Reinforcing details for developing continuity through joints are suggested in the ACI Detailing Manual.

As of this writing (*Fall 1997*), much research on moment resisting joints and especially on the seismic response thereof is in progress. The reports on this work should be consulted as they become available.

5.11.2—Development of Reinforcement

For reinforcement conforming to the requirements of ASTM A1035/A1035M, the value of f_y used in this Article shall be taken as 100 ksi.

C5.11.2

Although the specified yield strength of reinforcing bars used in design shall not exceed 75.0 ksi, tests have shown that a longer development length is needed with reinforcement conforming to ASTM A1035/A1035M to achieve a ductility comparable to that achieved with reinforcement conforming to AASHTO M 31. Limited tests have shown a lack of ductility in tension splices of reinforcement conforming to ASTM A1035 when compared to the behavior of splices with reinforcement conforming to AASHTO M 31, when the splice length is calculated using the maximum design yield strength of 75.0 ksi. However, when the splice length of the ASTM A1035/A1035M reinforcement is determined using its specified minimum yield strength of 100 ksi, more ductility is achieved. Consequently, it is proposed to use 100 ksi until additional research indicates an alternative value.

5.11.2.1—Deformed Bars and Deformed Wire in Tension

5.11.2.1.1—Tension Development Length

The tension development length, ℓ_d, shall not be less than the product of the basic tension development length, ℓ_{db}, specified herein and the modification factor or factors specified in Articles 5.11.2.1.2 and 5.11.2.1.3. The tension development length shall not be less than 12.0 in., except for lap splices specified in Article 5.11.5.3.1 and development of shear reinforcement specified in Article 5.11.2.6.

The basic tension development length, ℓ_{db}, in in. shall be taken as:

- For No. 11 bar and smaller .... $\dfrac{1.25\, A_b f_y}{\sqrt{f'_c}}$

  but not less than .... $0.4\, d_b f_y$

- For No. 14 bars .... $\dfrac{2.70\, f_y}{\sqrt{f'_c}}$

- For No. 18 bars .... $\dfrac{3.5\, f_y}{\sqrt{f'_c}}$

- For deformed wire .... $\dfrac{0.95\, d_b f_y}{\sqrt{f'_c}}$

where:

A_b = area of bar or wire (in.^{2})

f_y = specified yield strength of reinforcing bars (ksi)

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

d_b = diameter of bar or wire (in.)

5.11.2.1.2—Modification Factors which Increase ℓ_d

The basic development length, ℓ_{db}, shall be multiplied by the following factor or factors, as applicable:

- For top horizontal or nearly horizontal reinforcement, so placed that more than 12.0 in. of fresh concrete is cast below the reinforcement... 1.4
- For lightweight aggregate concrete where f_{ct} (ksi) is specified... $\dfrac{0.22\sqrt{f'_c}}{f_{ct}} \geq 1.0$
- For all-lightweight concrete where f_{ct} is not specified.. 1.3
- For sand-lightweight concrete where f_{ct} is not specified.. 1.2

Linear interpolation may be used between all-lightweight and sand-lightweight provisions when partial sand replacement is used.

- For epoxy-coated bars with cover less than $3d_b$ or with clear spacing between bars less than $6d_b$ 1.5
- For epoxy-coated bars not covered above............ 1.2

The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken to be greater than 1.7.

5.11.2.1.3—Modification Factors which Decrease ℓ_d

The basic development length, ℓ_{db}, modified by the factors as specified in Article 5.11.2.1.2, may be multiplied by the following factors, where:

- Reinforcement being developed in the length under consideration is spaced laterally not less than 6.0 in. center-to-center, with not less than 3.0 in. clear cover measured in the direction of the spacing 0.8
- 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 .... $\dfrac{(A_s \text{ required})}{(A_s \text{ provided})}$
- Reinforcement is enclosed within a spiral composed of bars of not less than 0.25 in. in diameter and spaced at not more than a 4.0 in. pitch 0.75

5.11.2.2—Deformed Bars in Compression

5.11.2.2.1—Compressive Development Length

The development length, ℓ_d, for deformed bars in compression shall not be less than either the product of the basic development length specified herein and the applicable modification factors specified in Article 5.11.2.2.2 or 8.0 in.

The basic development length, ℓ_{db}, for deformed bars in compression shall satisfy:

$$\ell_{db} \geq \frac{0.63\, d_b f_y}{\sqrt{f'_c}} \qquad (5.11.2.2.1\text{-}1)$$

or:

$$\ell_{db} \geq 0.3\, d_b f_y \qquad (5.11.2.2.1\text{-}2)$$

where:

f_y = specified yield strength of reinforcing bars (ksi)

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

d_b = diameter of bar (in.)

5.11.2.2.2—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…….. $\dfrac{(A_s \text{ required})}{(A_s \text{ provided})}$

- 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……………………………………….. 0.75

5.11.2.3—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 5.11.2.1.2 and 5.11.2.1.3, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

5.11.2.4—Standard Hooks in Tension

5.11.2.4.1—Basic Hook Development Length

The development length, ℓ_{dh}, in in., for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall not be less than:

- The product of the basic development length ℓ_{hb}, as specified in Eq. 5.11.2.4.1-1, and the applicable modification factor or factors, as specified in Article 5.11.2.4.2;
- 8.0 bar diameters; or
- 6.0 in.

Basic development length, ℓ_{hb}, for a hooked-bar with yield strength, f_y, not exceeding 60.0 ksi shall be taken as:

$$\ell_{hb} = \frac{38.0\,d_b}{\sqrt{f'_c}} \qquad (5.11.2.4.1\text{-}1)$$

C5.11.2.4.1

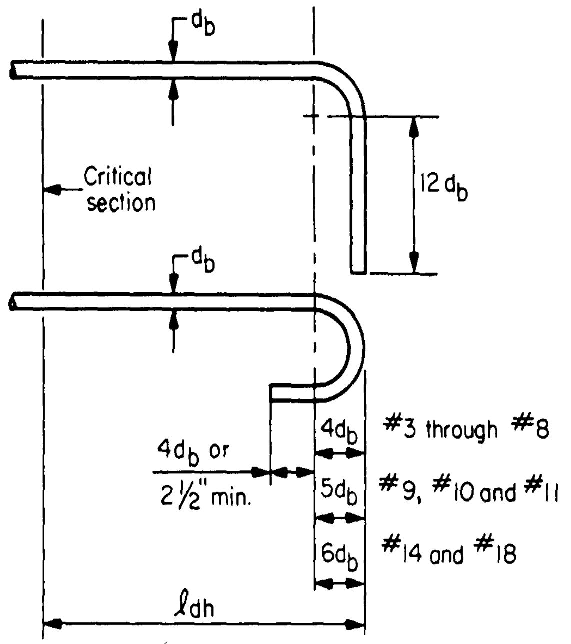


Figure C5.11.2.4-1—Hooked-Bar Details for Development of Standard Hooks (ACI)

where:

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

5.11.2.4.2—Modification Factors

Basic hook development length, ℓ_{hb}, shall be multiplied by the following factor or factors, as applicable, where:

- Reminforcement has a yield strength exceeding 60.0 ksi…………………………………… $\dfrac{f_y}{60.0}$
- Side cover for No. 11 bar and smaller, normal to plane of hook, is not less than 2.5 in., and 90° hook, cover on bar extension beyond hook not less than 2.0 in. ………………………… 0.7
- Hooks for No. 11 bar and smaller enclosed vertically or horizontally within ties or stirrup ties which are spaced along the full development length, ℓ_{dh}, at a spacing not exceeding $3d_b$…………………………….. 0.8
- Anchorage or development of full yield strength is not required, or where reinforcement is provided in excess of that required by analysis………………………… $\dfrac{(A_s \text{ required})}{(A_s \text{ provided})}$
- Lightweight aggregate concrete is used……. 1.3
- Epoxy-coated reinforcement is used……….. 1.2

C5.11.2.4.2

Recent tests indicate that the development length for hooked-bars should be increased by 20 percent to account for reduced bond when reinforcement is epoxy-coated. The proposed change was adopted by ACI Committee 318 in the 1992 edition of the *Building Code Requirements for Reinforced Concrete* (Hamad et al., 1990).

5.11.2.4.3—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 5.11.2.4.3-1. The factor for transverse reinforcement, as specified in Article 5.11.2.4.2, shall not apply.

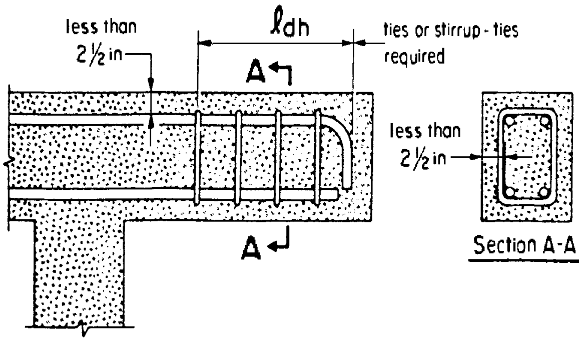


Figure 5.11.2.4.3-1—Hooked-Bar Tie Requirements

5.11.2.5—Welded Wire Fabric

5.11.2.5.1—Deformed Wire Fabric

For applications other than shear reinforcement, the development length, ℓ_{hd}, in in., of welded deformed wire fabric, measured from the point of critical section to the end of wire, shall not be less than either:

- The product of the basic development length and the applicable modification factor or factors, as specified in Article 5.11.2.2.2, or
- 8.0 in., except for lap splices, as specified in Article 5.11.6.1.

The development of shear reinforcement shall be taken as specified in Article 5.11.2.6.

The basic development length, ℓ_{hd}, for welded deformed wire fabric, with not less than one cross wire within the development length at least 2.0 in. from the point of critical section, shall satisfy:

$$\ell_{hd} \leq 0.95\, d_b \frac{f_y - 20.0}{\sqrt{f_c'}}\text{, or} \qquad (5.11.2.5.1\text{-}1)$$

$$\ell_{hd} \leq 6.30 \frac{A_w f_y}{s_w \sqrt{f_c'}} \qquad (5.11.2.5.1\text{-}2)$$

where:

A_w = area of an 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 welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 5.11.2.1.1.

5.11.2.5.2—Plain Wire Fabric

The yield strength of welded plain wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 2.0 in. from the point of critical section. Otherwise, the development length, ℓ_d, measured from the point of critical section to outermost cross wire shall be taken as:

$$\ell_d = 8.50 \frac{A_w f_y}{s_w \sqrt{f'_c}} \tag{5.11.2.5.2-1}$$

The development length shall be modified for reinforcement in excess of that required by analysis as specified in Article 5.11.2.4.2, and by the factor for lightweight concrete specified in Article 5.11.2.1.2, where applicable. However, ℓ_d shall not be taken to be less than 6.0 in., except for lap splices as specified in Article 5.11.6.2.

5.11.2.6—Shear Reinforcement

5.11.2.6.1—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.

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 middepth, $h/2$, as specified for development length for that part of the stress in the reinforcement required to satisfy Eq. 5.8.3.3-4.

5.11.2.6.2—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, and for No. 6, No. 7 and No. 8 bars with f_y of 40.0 ksi or less:

  A standard hook around longitudinal reinforcement, and

- For No. 6, No. 7 and No. 8 stirrups with f_y greater than 40.0 ksi:

  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:

$$\ell_e \geq \frac{0.44\, d_b\, f_y}{\sqrt{f'_c}} \qquad (5.11.2.6.2\text{-}1)$$

5.11.2.6.3—Anchorage of Wire Fabric Reinforcement

Each leg of welded plain wire fabric 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 fabric, 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 middepth 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.

C5.11.2.6.3

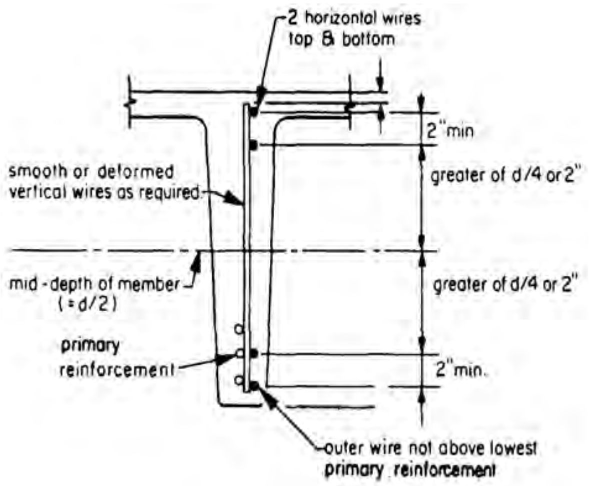


Figure C5.11.2.6.3-1—Anchorage of Single-Leg Welded Wire Fabric Shear Reinforcement, ACI (1989)

5.11.2.6.4—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 1.7 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 with the tension force resulting from factored loads, A_bf_y, not exceeding 9.0 kips per leg, may be considered adequate if the stirrup legs extend the full available depth of the member.

Transverse torsion reinforcement shall be made fully continuous and shall be anchored by 135-degree standard hooks around longitudinal reinforcement.

5.11.3—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.

C5.11.3

Standard details for such devices have not been developed.

5.11.4—Development of Prestressing Strand

5.11.4.1—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 5.11.4.2.

The effects of debonding shall be considered as specified in Article 5.11.4.3.

C5.11.4.1

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.

5.11.4.2—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:

$$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \qquad (5.11.4.2\text{-}1)$$

where:

d_b = nominal strand diameter (in.)

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (ksi)

C5.11.4.2

An October, 1988 FHWA memorandum mandated a 1.6 multiplier on Eq. 5.11.4.2-1 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. 5.11.4.2-1 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 C5.11.4.2-1. This idealized variation of strand stress may be used for analyzing sections within the transfer and development length at the end of pretensioned members.

f_{pe} = effective stress in the prestressing steel after losses (ksi)

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.

The variation of design stress in the pretensioned strand from the free end of the strand may be calculated as follows:

- From the point where bonding commences to the end of transfer length:

$$f_{px} = \frac{f_{pe}\ell_{px}}{60d_b} \qquad (5.11.4.2\text{-}2)$$

- From the end of the transfer length and to the end of the development of the strand:

$$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{(\ell_d - 60d_b)}\left(f_{ps} - f_{pe}\right) \qquad (5.11.4.2\text{-}3)$$

where:

ℓ_{px} = distance from free end of pretensioned strand to section of member under consideration (in.)

f_{px} = design stress in pretensioned strand at nominal flexural strength at section of member under consideration (ksi)

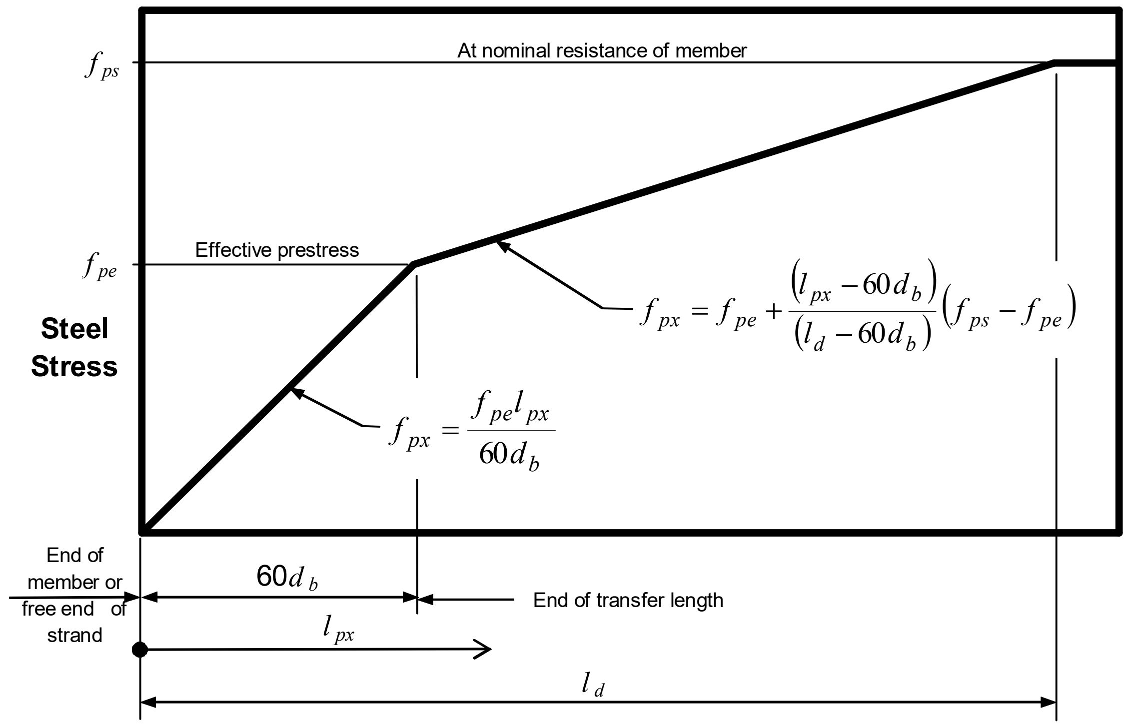


Figure C5.11.4.2-1—Idealized Relationship between Steel Stress and Distance from the Free End of Strand

5.11.4.3—Partially Debonded Strands

Where a portion or portions of a pretensioning strand are not bonded and where tension exists in the precompressed tensile zone, the development length, measured from the end of the debonded zone, shall be determined using Eq. 5.11.4.2-1 with a value of $\kappa = 2.0$.

The number of partially debonded strands should not exceed 25 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed 40 percent of the strands in that row.

The length of debonding of any strand shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated. Not more than 40 percent of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section.

Debonded strands shall be symmetrically distributed about the centerline of the member. Debonded lengths of pairs of strands that are symmetrically positioned about the centerline of the member shall be equal.

Exterior strands in each horizontal row shall be fully bonded.

C5.11.4.3

Tests completed by the Florida Department of Transportation (Shahawy, Robinson, and Batchelor, 1993; Shahawy and Batchelor, 1991) indicate that the anchored strength of the strands is one of the primary contributors to the shear resistance of prestressed concrete beams in their end zones. The recommended limit of 25 percent of debonded strands is derived from those tests. Shear capacity was found to be inadequate with full-scale girders where 40 percent of strands were debonded.

Some states have had success with greater percentages of partially debonded strands. Successful past practice should always be considered, but the shear resistance in the region should be thoroughly investigated with due regard to the reduction in horizontal force available when considering the free body diagram in Figure C5.8.3.5-1 and to all other determinations of shear capacity by any of the provisions of this section.

Research at various institutions was conducted validating that pretensioned strands that are partially debonded have a longer development length.

5.11.5—Splices of Bar Reinforcement

For reinforcement conforming to the requirements of ASTM A1035/A1035M, the value of f_y used in this Article shall be taken as 100 ksi.

5.11.5.1—Detailing

Permissible locations, types, and dimensions of splices, including staggers, for reinforcing bars shall be shown in the contract documents.

5.11.5.2—General Requirements

5.11.5.2.1—Lap Splices

The lengths of lap for lap splices of individual bars shall be as specified in Articles 5.11.5.3.1 and 5.11.5.5.1.

Lap splices within bundles shall be as specified in Article 5.11.2.3. 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 one-fifth the required lap splice length or 6.0 in.

5.11.5.2.2—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 gage points clear of the splice sleeve:

- For bar sizes up to No. 14…………….. 0.01 in.
- For No. 18 bars……………………….. 0.03 in.

C5.11.5.2.2

The stress versus slip criteria has been developed by the California Department of Transportation.

Types of mechanical connectors in use include the sleeve-threaded type, the sleeve-filler metal type and the sleeve-swaged type, of which many are proprietary, commercially available devices. The contract documents should include a testing and approval procedure wherever a proprietary type of connector is used.

Basic information about the various types of proprietary mechanical connection devices is given in ACI 439.3R (1991).

5.11.5.2.3—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.

C5.11.5.2.3

The limitation of a full-welded splice to only butt-welded bars that was included in previous editions of the Standard Specifications was deleted. The purpose of this requirement is unknown, but it may have been an indirect consequence of concern about fatigue of other types of welded splices. It should be noted that this Article requires all welding of reinforcing bar splices to conform to the latest edition of the AWS Code, and that this Code limits lap welded splices to bar size No. 6 and smaller.

5.11.5.3—Splices of Reinforcement in Tension

C5.11.5.3

The tension development length, ℓ_d, used as a basis for calculating splice lengths should include all of the modification factors specified in Article 5.11.2.

5.11.5.3.1—Lap Splices in Tension

The length of lap for tension lap splices shall not be less than either 12.0 in. or the following for Class A, B or C splices:

Class A splice……………………………….. 1.0 ℓ_d

Class B splice…………………………………1.3 ℓ_d

Class C splice…………………………………1.7 ℓ_d

The tension development length, ℓ_d, for the specified yield strength shall be taken in accordance with Article 5.11.2.

The class of lap splice required for deformed bars and deformed wire in tension shall be as specified in Table 5.11.5.3.1-1.

Table 5.11.5.3.1-1—Classes of Tension Lap Splices

Ratio of $\frac{(A_s \text{ as provided})}{(A_s \text{ as required})}$	Percent of A_s Spliced with Required Lap Length		
	50	75	100
≥2	A	A	B
<2	B	C	C

5.11.5.3.2—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.

C5.11.5.3.2

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.

5.11.5.4—Splices in Tension Tie Members

Splices of reinforcement in tension 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.

C5.11.5.4

A tension 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.

5.11.5.5—Splices of Bars in Compression

5.11.5.5.1—Lap Splices in Compression

The length of lap, ℓ_c, for compression lap splices shall not be less than 12.0 in. or as follows:

- If $f_y \leq 60.0$ ksi then:

$$\ell_c = 0.5mf_yd_b$$

or: (5.11.5.5.1-1)

- If $f_y > 60.0$ ksi then:

$$\ell_c = m\left(0.9f_y - 24.0\right)d_b \qquad (5.11.5.5.1\text{-}2)$$

in which:

- Where the specified concrete strength, 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 = diameter of 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.

C5.11.5.5.1

The effective area of the ties is the area of the legs perpendicular to the thickness of the component, as seen in cross-section.

5.11.5.5.2—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 5.11.5.2.2 and 5.11.5.2.3, respectively.

5.11.5.5.3—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.

5.11.6—Splices of Welded Wire Fabric

5.11.6.1—Splices of Welded Deformed Wire Fabric in Tension

When measured between the ends of each fabric sheet, the length of lap for lap splices of welded deformed wire fabric with cross wires within the lap length shall not be less than $1.3\ell_{hd}$ or 8.0 in. The overlap measured between the outermost cross wires of each fabric sheet shall not be less than 2.0 in.

Lap splices of welded deformed wire fabric with no cross wires within the lap splice length shall be determined as for deformed wire in accordance with the provisions of Article 5.11.5.3.1.

5.11.6.2—Splices of Welded Smooth Wire Fabric 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 fabric sheet shall not be less than:

- The sum of one spacing of cross wires plus 2.0 in., or
- $1.5\ell_d$, or
- 6.0 in.

where:

ℓ_d = development length specified in Article 5.11.2 (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 fabric sheet shall not be less than $1.5\ell_d$ or 2.0 in.

5.12—DURABILITY

5.12.1—General

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:

C5.12.1

Design considerations for durability include concrete quality, protective coatings, minimum cover, distribution and size of reinforcement, details, and crack widths. Further guidance can be found in ACI Committee Report 222 (ACI, 1987) and Posten et al. (1987).

- Air-entrainment of the concrete is required,
- Epoxy-coated or galvanized reinforcement is required,
- Special concrete additives are required,
- The concrete is expected to be exposed to salt water or to sulfate soils or water, and
- Special curing procedures are required.

Protective measures for durability shall satisfy the requirements specified in Article 2.5.2.1.

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 cement content,
- Low W/C ratio, and
- Thorough curing, preferably with water.

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.

Sulfate soils or water, sometimes called alkali, contain high levels of sulfates of sodium, potassium, calcium, or magnesia. 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 or the *Concrete Manual* (1981).

5.12.2—Alkali-Silica Reactive Aggregates

The contract documents shall prohibit the use of aggregates from sources that are known to be excessively alkali-silica reactive.

If aggregate of limited reactivity is used, the contract documents shall require the use of either low-alkali-type cements or a blend of regular cement and pozzolanic materials, provided that their use has been proven to produce concrete of satisfactory durability with the proposed aggregate.

C5.12.2

Alkali-silica reactive aggregates occur throughout the world. In the United States, most are found in the West and Midwest. In most states, public agencies have identified locations where reactive aggregates occur. When in doubt, the Designer should investigate this possibility.

Excessive reactivity is generally determined by tests (ASTM C227) made on aggregates prior to their use. Although the line of demarcation between nonreactive and reactive combinations is not clearly defined, expansion when tested per ASTM C227 is generally considered to be excessive if it is greater than 0.05 percent at three months or 0.10 percent at six months. Expansions greater than 0.05 percent at three months should not be considered excessive where the six-month expansion remains below 0.10 percent. Data for the three-month test should be considered only when six-month results are not available.

Reference to AASHTO M 80 will not specifically prohibit use of reactive aggregates as AASHTO M 80. It only requires the use of low-alkali cements or additives.

More guidance on this is contained in ACI 201.2R.

5.12.3—Concrete Cover

Cover for unprotected prestressing and reinforcing steel shall not be less than that specified in Table 5.12.3-1 and modified for *W*/*C* ratio, unless otherwise specified either herein or in Article 5.12.4.

Concrete cover and placing tolerances shall be shown in the contract documents.

Cover for pretensioned prestressing strand, anchorage hardware, and mechanical connections for reinforcing bars or post-tensioned prestressing strands shall be the same as for reinforcing steel.

Cover for metal ducts for post-tensioned tendons shall not be less than:

- That specified for main reinforcing steel,
- One-half the diameter of the duct, or
- That specified in Table 5.12.3-1.

For decks exposed to tire studs or chain wear, additional cover shall be used to compensate for the expected loss in depth due to abrasion, as specified in Article 2.5.2.4.

Modification factors for *W*/*C* ratio shall be the following:

- For $W/C \leq 0.40$ 0.8
- For $W/C \geq 0.50$ 1.2

Minimum cover to main bars, including bars protected by epoxy coating, shall be 1.0 in.

Cover to ties and stirrups may be 0.5 in. less than the values specified in Table 5.12.3-1 for main bars but shall not be less than 1.0 in.

C5.12.3

The concrete cover modification factor used in conjunction with Table 5.12.3-1 recognizes the decreased permeability resulting from a lower *W*/*C* ratio.

Minimum cover is necessary for durability and prevention of splitting due to bond stresses and to provide for placing tolerance.

Table 5.12.3-1—Cover for Unprotected Main Reinforcing Steel (in.)

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments - General - Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0

5.12.4—Protective Coatings

Protection 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. Cover to epoxy-coated steel may be as shown for interior exposure in Table 5.12.3-1.

C5.12.4

Specifications for acceptable epoxy coatings are included in the materials section of *AASHTO LRFD Bridge Construction Specifications*.

5.12.5—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.

C5.12.5

In certain cases, such as the tieing together of longitudinal precast elements by transverse post-tensioning, 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 or metal tubing. 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.

5.13—SPECIFIC MEMBERS

5.13.1—Deck Slabs

Requirements for deck slabs in addition to those specified in Section 5 shall be as specified in Section 9.

5.13.2—Diaphragms, Deep Beams, Brackets, Corbels, and Beam Ledges

5.13.2.1—General

Diaphragms, brackets, corbels, beam ledges, and other deep members subjected primarily to shear and torsion and whose depth is large relative to their span shall be designed as specified herein.

Deep beams shall be analyzed and designed by either the strut-and-tie model, specified in Article 5.6.3, or another recognized theory.

C5.13.2.1

For a structural depth that is large relative to span length, the definition of a deep component, given in Article 5.2, may be used.

As noted in the Commentary for Article 5.6.3, the sectional design model method is not valid for some deep members; they should be designed by a strut-and-tie model.

Another recognized theory for design of these components can be found in Article 11.8 of ACI 318.

5.13.2.2—Diaphragms

Unless otherwise specified, diaphragms shall be provided at abutments, piers, and hinge joints to resist lateral forces and transmit loads 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 angle points in girders.

For spread box beams and for curved box girders having an inside radius less than 800 ft, intermediate diaphragms shall be used.

Diaphragms may be omitted where tests or structural analysis show them to be unnecessary.

Diaphragms should be designed by the strut-and-tie method, where applicable.

In bridges with post-tensioned diaphragms, the diaphragm tendons must be effectively tied into the diaphragms with bonded nonprestressed reinforcement to resist tendon forces at the corners of openings in the diaphragms.

C5.13.2.2

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.

Figure C5.13.2.2-1 illustrates the application of the strut-and-tie model to analysis of forces in a prestressed interior diaphragm of a box girder bridge.

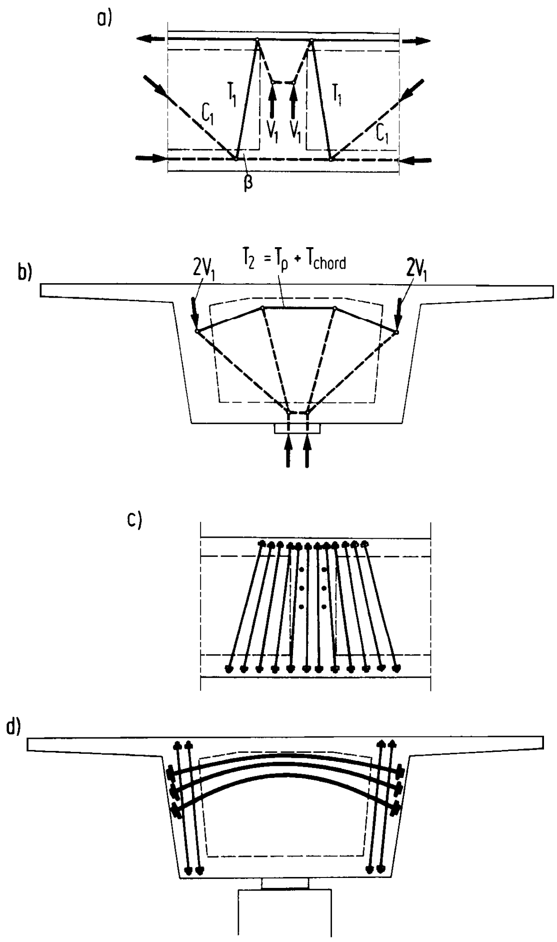


Figure C5.13.2.2-1—Diaphragm of a Box Girder Bridge: (a) Disturbed Regions and Model of the Web near the Diaphragm; (b) Diaphragm and Model; (c) and (d) Prestressing of the Web and the Diaphragm (Schlaich et al., 1987)

5.13.2.3—Detailing Requirements for Deep Beams

The factored tensile resistance, N_R in kips, of transverse pair of reinforcing bars shall satisfy:

$$N_R = \phi f_y A_s \geq 0.12 b_v s \quad (5.13.2.3\text{-}1)$$

where:

b_v = width of web (in.)

f_y = yield strength of reinforcing steel (ksi)

A_s = area of steel in distance s (in.^{2})

ϕ = resistance factor specified in Article 5.5.4.2

s = spacing of reinforcement (in.)

C5.13.2.3

Figure C5.13.2.3-1 shows an application of the strut-and-tie model to analysis of deep beams.

The spacing of transverse reinforcement, s, shall not exceed d/4 or 12.0 in.

Bonded longitudinal bars shall be well distributed over each face of the vertical elements in pairs. The tensile resistance of a bonded reinforcement pair shall not be less than that specified in Eq. 5.13.2.3-1. The vertical spacing between each pair of reinforcement, s, shall not exceed either d/3 or 12.0 in. For components whose width is less than 10.0 in., a single bar of the required tensile resistance may be used in lieu of a pair of longitudinal bars.

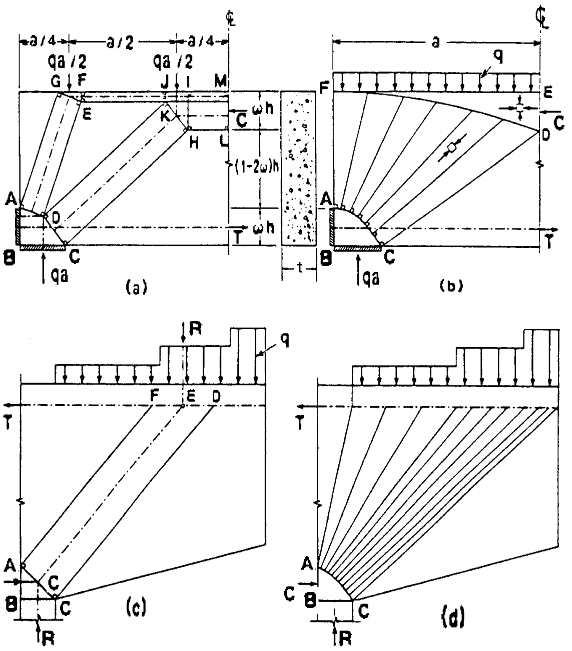
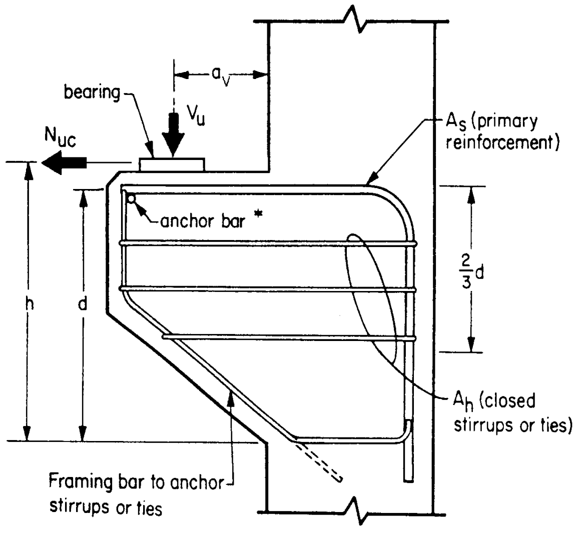


Figure C5.13.2.3-1—Fan Action: (a) Strut-and-Tie Model of Uniformly-Loaded Deep Beam; (b) Fan-Shaped Stress Field; (c) Strut-and-Tie System for Equivalent Single-Load *R* Replacing Distributed-Load *q*; (d) Continuous Fan Developed from Discrete Strut

5.13.2.4—Brackets and Corbels

5.13.2.4.1—General

Components in which a_v, as shown in Figure 5.13.2.4.1-1, 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.



*Welded to primary reinforcement

Figure 5.13.2.4.1-1—Notation

C5.13.2.4.1

Figure C5.13.2.4.1-1 illustrates the application of strut-and-tie models to analysis of brackets and corbels.

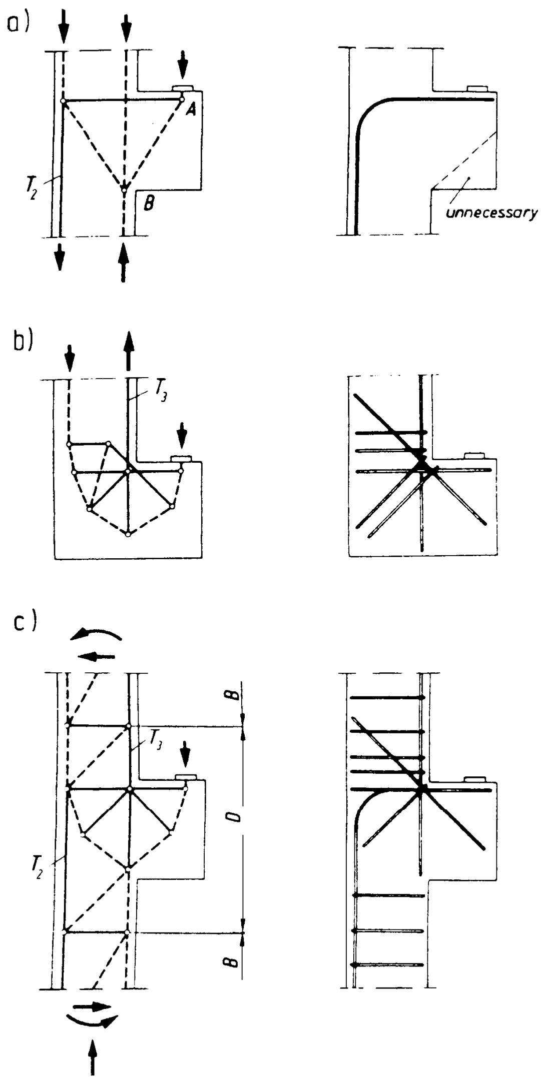
The section at the face of support shall be designed to resist simultaneously a factored shear force V_u, a factored moment

$$M_u = V_u a_v + N_{uc}(h-d) \qquad (5.13.2.4.1\text{-}1)$$

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.

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.



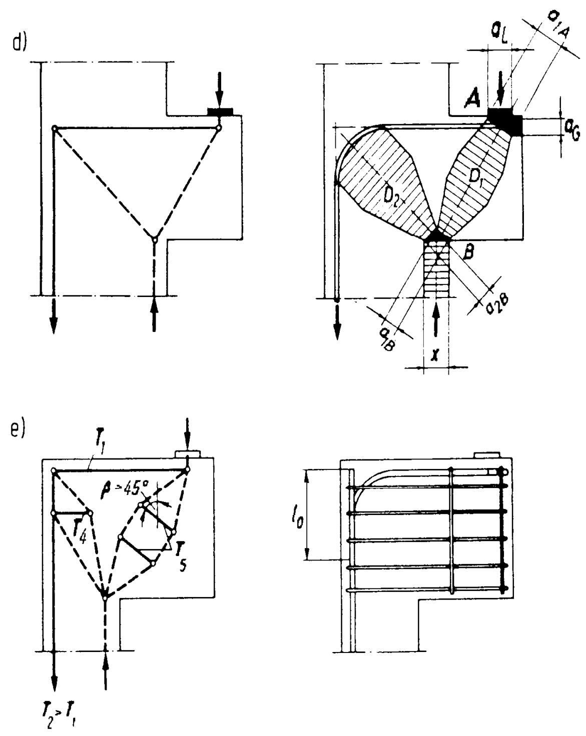


Figure C5.13.2.4.1-1—Different Support Conditions Leading to Different Strut-and-Tie Models and Different Reinforcement Arrangements of Corbels and Beam Ledges (Schlaich et al., 1987)

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.

5.13.2.4.2—Alternative to Strut-and-Tie Model

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 5.6.3 or the provisions of Article 5.13.2.4.1, with the following exceptions:

- Design of shear-friction reinforcement, A_{vf}, to resist the factored shear force, V_u, shall be as specified in Article 5.8.4, except that:

  For normal weight concrete, nominal shear resistance, V_n, shall satisfy:

  $$V_n = 0.2 f'_c b_w d_e \text{ and} \quad (5.13.2.4.2\text{-}1)$$

  $$V_n = 0.8\, b_w\, d_e \quad (5.13.2.4.2\text{-}2)$$

  For all lightweight or sand-lightweight concretes, nominal shear resistance, V_n, shall satisfy:

  $$V_n = \left(0.2 - 0.07 a_v / d\right) f'_c b_w d_e \text{ (kips) and} \quad (5.13.2.4.2\text{-}3)$$

  $$V_n = \left(0.8 - 0.28\, a_v / d_e\right) b_w d \text{ (kips)} \quad (5.13.2.4.2\text{-}4)$$

- Reinforcement, A_s, to resist the factored force effects shall be determined as for ordinary members subjected to flexure and axial load.
- Area of primary tension reinforcement, A_s, shall satisfy:

  $$A_s \geq \frac{2A_{vf}}{3} + A_n \text{, and} \quad (5.13.2.4.2\text{-}5)$$

- The area of closed stirrups or ties placed within a distance equal to $2d_e/3$ from the primary reinforcement shall satisfy:

  $$A_h \geq 0.5\left(A_s - A_n\right) \quad (5.13.2.4.2\text{-}6)$$

in which:

$$A_n \geq N_{uc} / \phi f_y \quad (5.13.2.4.2\text{-}7)$$

where:

b_w = web width (in.)

d_e = depth of center of gravity of steel (in.)

A_{vf} = area of shear friction steel (in.^{2})

5.13.2.5—Beam Ledges

5.13.2.5.1—General

As illustrated in Figure 5.13.2.5.1-1, 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.

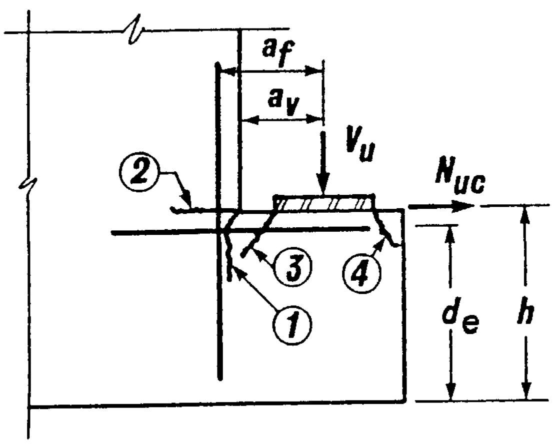


Figure 5.13.2.5.1-1—Notation and Potential Crack Locations for Ledge Beams

Beam ledges may be designed in accordance with either the strut-and-tie model or the provisions of Articles 5.13.2.5.2 through 5.13.2.5.5. Bars shown in Figures 5.13.2.5.2-1 through 5.13.2.5.5-2 shall be properly developed in accordance with Article 5.11.1.1.

C5.13.2.5.1

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 5.13.2.5.3-1. In addition, beam ledges are supported primarily by tension ties to the supporting member, whereas corbels utilize a compression 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 C5.13.2.5.1-1.

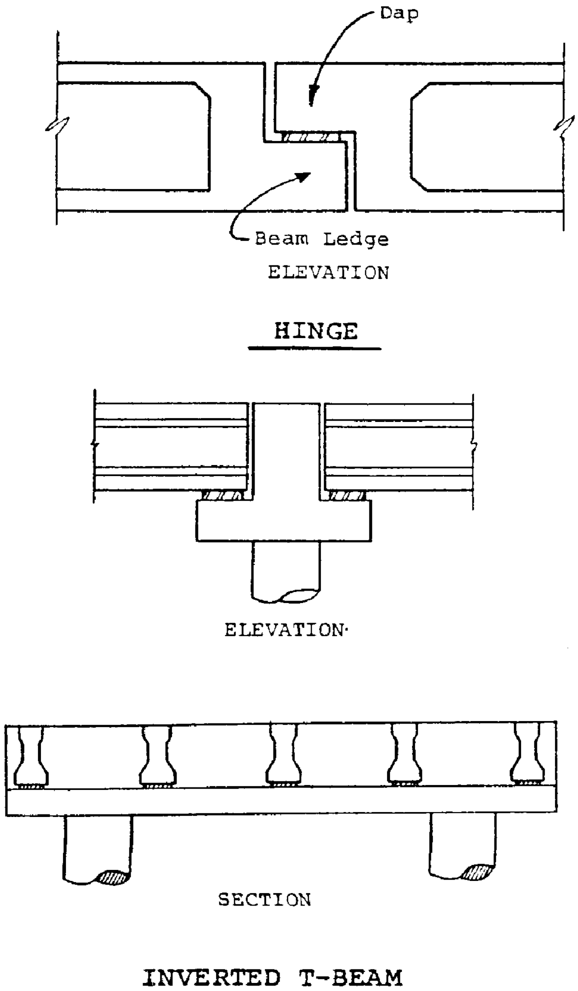


Figure C5.13.2.5.1-1—Examples of Beam Ledges

5.13.2.5.2—Design for Shear

Design of beam ledges for shear shall be in accordance with the requirements for shear friction specified in Article 5.8.4. Nominal interface shear resistance shall satisfy Eqs. 5.13.2.4.2-1 through 5.13.2.4.2-4 wherein the width of the concrete face, b_w, assumed to participate in resistance to shear shall not exceed S, (W + 4a_v), or 2c, as illustrated in Figure 5.13.2.5.2-1.

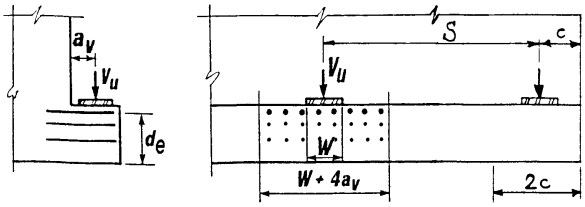


Figure 5.13.2.5.2-1—Design of Beam Ledges for Shear

5.13.2.5.3—Design for Flexure and Horizontal Force

The area of total primary tension reinforcement, A_s, shall satisfy the requirements of Article 5.13.2.4.2.

The primary tension reinforcement shall be spaced uniformly within the region (W + 5a_f) or 2c, as illustrated in Figure 5.13.2.5.3-1, except that the widths of these regions shall not overlap.

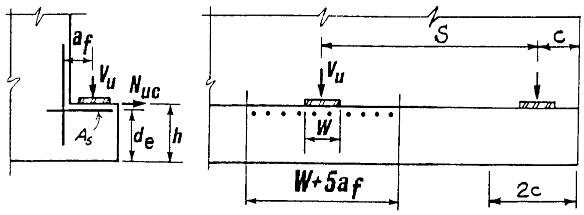


Figure 5.13.2.5.3-1—Design of Beam Ledges for Flexure and Horizontal Force

5.13.2.5.4—Design for Punching Shear

The truncated pyramids assumed as failure surfaces for punching shear, as illustrated in Figure 5.13.2.5.4-1, shall not overlap.

Nominal punching shear resistance, V_n, in kips, shall be taken as:

- At interior pads, or exterior pads where the end distance c is greater than $S/2$:

$$V_n = 0.125 \sqrt{f_c'} (W + 2L + 2d_e)\, d_e \tag{5.13.2.5.4-1}$$

C5.13.2.5.4

The area of concrete resisting the punching shear for each concentrated load is shown in Figure 5.13.2.5.4-1. 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.

- At exterior pads where the end distance c is less than $S/2$ and $c - 0.5W$ is less than d_e:

$$V_n = 0.125\sqrt{f_c'}\,(W + L + d_e)\,d_e \quad (5.13.2.5.4\text{-}2)$$

- At exterior pads where the end distance c is less than $S/2$, but $c - 0.5W$ is greater than d_e:

$$V_n = 0.125\sqrt{f_c'}\left(0.5W + L + d_e + c\right)d_e \quad (5.13.2.5.4\text{-}3)$$

where:

f'_c = specified strength of concrete at 28 days (ksi)

W = width of bearing plate or pad as shown in Figure 5.13.2.5.4-1 (in.)

L = length of bearing pad as shown in Figure 5.13.2.5.4-1 (in.)

d_e = effective depth from extreme compression fiber to centroid of tensile force (in.)

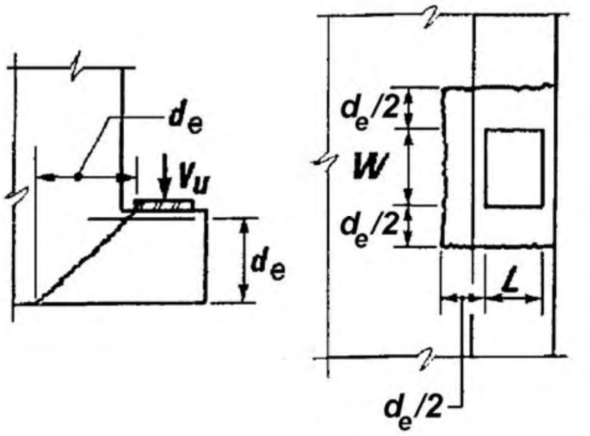


Figure 5.13.2.5.4-1—Design of Beam Ledges for Punching Shear

5.13.2.5.5—Design of Hanger Reinforcement

The hanger reinforcement specified herein shall be provided in addition to the lesser 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 5.13.2.5.5-1.

Using the notation in Figure 5.13.2.5.5-1, the nominal shear resistance, V_n, in kips, for single-beam ledges shall be taken as:

- For the service limit state:

$$V_n = \frac{A_{hr}\left(0.5 f_y\right)}{s}\left(W + 3a_v\right) \qquad (5.13.2.5.5\text{-}1)$$

- For the strength limit state:

$$V_n = \frac{A_{hr} f_y}{s} S \qquad (5.13.2.5.5\text{-}2)$$

where:

A_{hr} = area of one leg of hanger reinforcement as illustrated in Figure 5.13.2.5.5-1 (in.^{2})

S = spacing of bearing places (in.)

s = spacing of hangers (in.)

f_y = yield strength of reinforcing steel (ksi)

a_v = distance from face of wall to the load as illustrated in Figure 5.13.2.5.5-1 (in.)

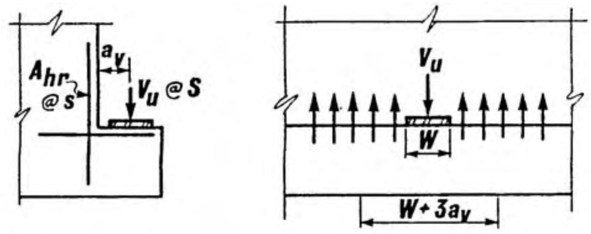


Figure 5.13.2.5.5-1—Single-Ledge Hanger Reinforcement

Using the notation in Figure 5.13.2.5.5-2, the nominal shear resistance of the ledges of inverted T-beams shall be the lesser of that specified by Eq. 5.13.2.5.5-2 and Eq. 5.13.2.5.5-3.

$$V_n = \left(0.063\sqrt{f_c'} b_f d_f\right) + \frac{A_{hr} f_y}{s}\left(W + 2d_f\right) \qquad (5.13.2.5.5\text{-}3)$$

where:

d_f = distance from top of ledge to compression reinforcement as illustrated in Figure 5.13.2.5.5-2 (in.)

The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than d_f.

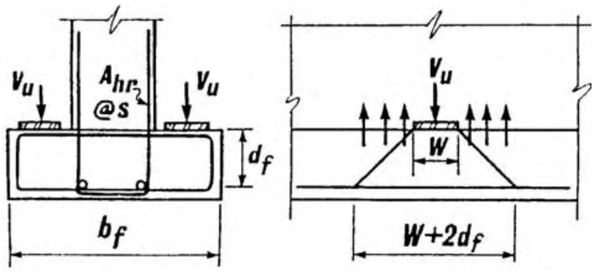


Figure 5.13.2.5.5-2—Inverted T-Beam Hanger Reinforcement

Inverted T-beams shall satisfy the torsional moment provisions as specified in Articles 5.8.3.6 and 5.8.2.1.

5.13.2.5.6—Design for Bearing

For the design for bearings supported by beam ledges, the provisions of Article 5.7.5 shall apply.

5.13.3—Footings

5.13.3.1—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.

C5.13.3.1

Although the provisions of Article 5.13.3 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.

5.13.3.2—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.

C5.13.3.2

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."

5.13.3.3—Resistance Factors

For determination of 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.

5.13.3.4—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.

C5.13.3.4

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.

5.13.3.5—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. 5.13.3.5-1, 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. 5.13.3.5-1.

$$A_{s\text{-}BW} = A_{s\text{-}SD}\left(\frac{2}{\beta + 1}\right) \qquad (5.13.3.5\text{-}1)$$

where:

β = ratio of the long side to the short side of footing

$A_{s\text{-}BW}$ = area of steel in the band width (in.^{2})

$A_{s\text{-}SD}$ = total area of steel in short direction (in.^{2})

5.13.3.6—Shear in Slabs and Footings

5.13.3.6.1—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 5.8.3.2.
- 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

C5.13.3.6.1

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 C5.13.3.6.1-1, in which d_v is the effective depth for shear.

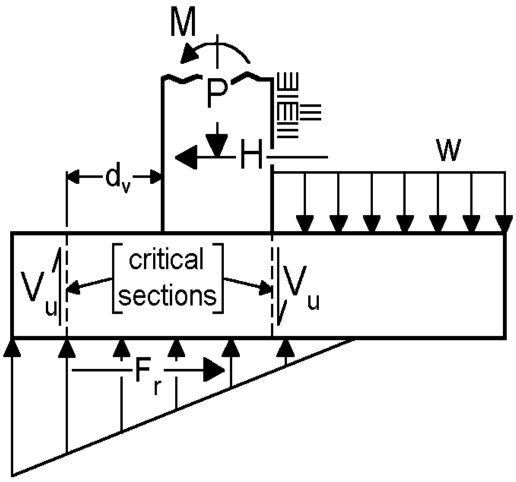


Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

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.

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.

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.

5.13.3.6.2—One-Way Action

For one-way action, the shear resistance of the footing or slab shall satisfy the requirements specified in Article 5.8.3, except for culverts with over 2.0 ft or more of fill, for which the provisions of Article 5.14.5.3 shall apply.

5.13.3.6.3—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:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f'_c}\, b_o d_v \leq 0.126\sqrt{f'_c}\, b_o d_v \quad (5.13.3.6.3\text{-}1)$$

where:

β_c = ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted

b_o = perimeter of the critical section (in.)

d_v = effective shear depth (in.)

Where $V_u > \phi V_n$, shear reinforcement shall be added in compliance with Article 5.8.3.3, 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:

$$V_n = V_c + V_s \leq 0.192\sqrt{f'_c}\, b_o d_v \quad (5.13.3.6.3\text{-}2)$$

in which:

$$V_c = 0.0632\sqrt{f'_c}\, b_o d_v \text{, and} \quad (5.13.3.6.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v}{s} \quad (5.13.3.6.3\text{-}4)$$

5.13.3.7—Development of Reinforcement

For the development of reinforcement in slabs and footings, the provisions of Article 5.11 shall apply.

Critical sections for development of reinforcement shall be assumed to be at the locations specified in Article 5.13.3.4 and at all other vertical planes where changes of section or reinforcement occur.

5.13.3.8—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 5.7.5, for either surface.

C5.13.3.6.3

The traditional expression for punching shear resistance has been retained.

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 actually resist the critical shear for the group under consideration. One such situation is illustrated in Figure C5.13.3.6.3-1.

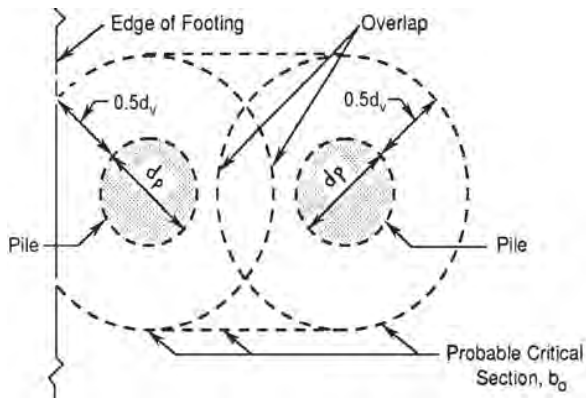


Figure C5.13.3.6.3-1—Modified Critical Section for Shear with Overlapping Critical Perimeters

Lateral forces shall be transferred from the pier to the footing in accordance with shear-transfer provisions specified in Article 5.8.4 on the basis of the appropriate bulleted item in Article 5.8.4.3.

Reinforcement shall be provided across the interface between supporting and supported member, either by extending the main longitudinal column or 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.

5.13.4—Concrete Piles

5.13.4.1—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 $1.25 f_y A_s$.

C5.13.4.1

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.

In addition to the requirements specified in Articles 5.13.4.1 through 5.13.4.5, piles used in the seismic zones shall conform to the requirements specified in Article 5.13.4.6.

5.13.4.2—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.

C5.13.4.2

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.

5.13.4.3—Precast Reinforced Piles

5.13.4.3.1—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.

C5.13.4.3.1

A 1.0-in. connection chamfer is desirable, but smaller chamfers have been used successfully. Local experience should be considered.

5.13.4.3.2—Reinforcing Steel

Longitudinal reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile. The area of reinforcing steel 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 5.13.4.4.3.

5.13.4.4—Precast Prestressed Piles

5.13.4.4.1—Pile Dimensions

Prestressed concrete piles may be octagonal, square, or circular and shall conform to the minimum dimensions specified in Article 5.13.4.3.1.

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.

5.13.4.4.2—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.

5.13.4.4.3—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.

C5.13.4.4.3

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."

5.13.4.5—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 need be approved by the Engineer before any driving is done.

C5.13.4.5

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.

5.13.4.5.1—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 5.13.4.3.

5.13.4.5.2—Reinforcing Steel

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 reinforcing steel 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.

For cast-in-place concrete piling, clear distance between parallel longitudinal, and parallel transverse reinforcing bars shall not be less than five times the maximum aggregate size or 5.0 in., except as noted in Article 5.13.4.6 for seismic requirements.

5.13.4.6—Seismic Requirements

5.13.4.6.1—Zone 1

No additional design provisions need be considered for Zone 1.

5.13.4.6.2—Zone 2

5.13.4.6.2a—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 5.11.2.

Concrete-filled pipe piles shall be anchored with steel dowels as specified in Article 5.13.4.1, 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 ten percent of the factored axial compressive resistance of the pile.

5.13.4.6.2b—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 either 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 2.0 ft or 1.5 pile diameters, whichever is greater. See Articles 5.10.11.3 and 5.10.11.4.

C5.13.4.6.2b

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 reinforcing steel.

5.13.4.6.2c—Precast Reinforced Piles

For precast reinforced piles, the longitudinal steel shall not be less than 1.0 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.

5.13.4.6.2d—Precast Prestressed Piles

For precast prestressed piles, the ties shall conform to the requirements of precast piles, as specified in Article 5.13.4.6.2c.

5.13.4.6.3—Zones 3 and 4

5.13.4.6.3a—General

In addition to the requirements specified for Zone 2, piles in Zones 3 and 4 shall conform to the provisions specified herein.

5.13.4.6.3b—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 2.0 pile diameters or 24.0 in. If an analysis of the bridge and pile system 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 5.13.4.6.2, shall extend thereto.

C5.13.4.6.3b

Note the special requirements for pile bents given in Article 5.10.11.4.1.

5.13.4.6.3c—Volumetric Ratio for Confinement

The volumetric ratio of transverse reinforcement within the confinement length shall be that for columns, as specified in Article 5.10.11.4.1d.

5.13.4.6.3d—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 5.10.11.4.1d, 5.10.11.4.1e, and 5.10.11.4.1f.

C5.13.4.6.3d

See Article C5.13.4.6.2b.

5.13.4.6.3e—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 5.10.11.4.1d.

5.14—PROVISIONS FOR STRUCTURE TYPES

5.14.1—Beams and Girders

5.14.1.1—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 5.8.4.

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.

C5.14.1.1

These provisions supplement the appropriate provisions of other Articles of these Specifications.

This Article applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.14.2. 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 5.14.4.3.

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.

5.14.1.2—Precast Beams

5.14.1.2.1—Preservice Conditions

The preservice conditions of prestressed girders for shipping and erection shall be the responsibility of the contractor.

C5.14.1.2.1

AASHTO LRFD Bridge Construction Specifications places the responsibility on the Contractor to provide adequate devices and methods for the safe storage, handling, erection, and temporary bracing of precast members.

5.14.1.2.2—Extreme Dimensions

The thickness of any part of precast concrete beams shall not be less than:

Top flange .... 2.0 in.
Web, non post-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.

C5.14.1.2.2

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 5.14.1.3.2 may be used.

5.14.1.2.3—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 reinforcing steel.

C5.14.1.2.3

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 mild steel bars, with their tails embedded in the concrete or threaded anchorage devices that are cast into the concrete.

5.14.1.2.4—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.

C5.14.1.2.4

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.

5.14.1.2.5—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 concrete strength specified in the contract documents.

C5.14.1.2.5

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.

5.14.1.3—Spliced Precast Girders

5.14.1.3.1—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. Therefore, 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 Article 5.14.2 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.

C5.14.1.3.1

Bridges consisting of spliced precast girder segments have been constructed in a variety of locations for many different reasons. An extensive database of spliced girder bridge projects has been compiled and is present in the appendix to Castrodale and White (2004).

Splicing of girder segments is generally performed in place, but may be performed prior to erection. The final structure may be a simple span or a continuous span unit.

In previous editions of these Specifications, spliced precast girder bridges were considered as a special case of both conventional precast girders and segmental construction. However, it is more appropriate to classify this type of structure as a conventional bridge with additional requirements at the splice locations that are based on provisions developed for segmental construction. The cross-section for bridges utilizing segmented precast girders is typically comprised of several girders with a composite deck.

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 5.14.2.5.

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 5.9.5. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered.

When 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 5.14.4.3.

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 5.14.1.4.

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 5.14.2.3.8a) 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.

Deck girder bridges are often spliced because the significant weight of the cross-section, which is comprised of both a girder and deck, may exceed usual limits for handling and transportation.

5.14.1.3.2—Joints between Segments

5.14.1.3.2a—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 5.14.2.4.2.

The sequence of placing concrete for the closure joints and deck shall be specified in the contract documents.

C5.14.1.3.2a

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.

5.14.1.3.2b—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 5.14.2.4.2.

C5.14.1.3.2b

When 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.

5.14.1.3.2c—Details of Match-Cast Joints

Match-cast joints for spliced precast girder bridges shall be detailed in accordance with Article 5.14.2.4.2.

C5.14.1.3.2c

One or more large shear keys may be used with spliced girders rather than the multiple small amplitude shear keys indicated in Article 5.14.2.4.2. The shear key proportions specified in Article 5.14.2.4.2 should be used.

5.14.1.3.2d—Joint Design

Stress limits for temporary concrete stresses in joints before losses specified in Article 5.9.4.1 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 5.9.4.2 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.

Resistance factors for joints specified in Article 5.5.4.2.2 for segmental construction shall apply.

The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations.

5.14.1.3.3—Girder Segment Design

Stress limits for temporary concrete stresses in girder segments before losses specified in Article 5.9.4.1 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 5.9.4.2 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 5.7.3.4 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.

C5.14.1.3.3

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.*

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.

5.14.1.3.4—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.

C5.14.1.3.4

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).

Post-tensioning shall be shown on the contract documents according to the requirements of Article 5.14.2.3.9.

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 specified 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.

See Article 5.10.3.5 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 5.14.2.3.10e for deck overlay provisions.

5.14.1.4—Bridges Composed of Simple Span Precast Girders Made Continuous

5.14.1.4.1—General

The provisions of this Article shall apply at the service and strength limit states as applicable.

When the requirements of Article 5.14.1.4 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 in Article 5.14.1.4.

The requirements specified in Article 5.14.1.4 supplement the requirements of other sections of these Specifications for fully 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 the requirements of Article 5.14.1.4.

C5.14.1.4.1

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 that positive moment connections be provided in all bridges detailed as continuous for live load.

5.14.1.4.2—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 5.14.1.4.4.

Restraint moments shall not be included in any combination when the effect of the restraint moment is to reduce the total moment.

C5.14.1.4.2

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 not desirable, 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.

5.14.1.4.3—Material Properties

Creep and shrinkage properties of the girder concrete and the shrinkage properties of the deck slab concrete shall be determined from either:

- 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.

C5.14.1.4.3

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, 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:

$$\varepsilon_{effective} = \varepsilon_{sh}\left(\frac{A_c}{A_{tr}}\right) \quad \text{(C5.14.1.4.3-1)}$$

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. C5.14.1.4.3-1 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.

5.14.1.4.4—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 deck slab shrinkage may be taken to be zero.
- Computation of restraint moments shall not be required.
- A positive moment connection shall be provided with a factored resistance, φMn, not less than $1.2M_{cr}$, as specified in Article 5.14.1.4.9.

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.

C5.14.1.4.4

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 7 days was reported to be a realistic minimum. However, the use of 7 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 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 C5.14.1.4.2) 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.

5.14.1.4.5—Degree of Continuity at Various Limit States

Both a positive and negative moment connection, as specified in Articles 5.14.1.4.8 and 5.14.1.4.9, 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 5.14.1.4.4 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.

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.

C5.14.1.4.5

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 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 5.14.1.4.4 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.

5.14.1.4.6—Service Limit State

Simple-span precast girders made continuous shall be designed to satisfy service limit state stress limits given in Article 5.9.4. 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 5.9.4.1.2-1 for other than segmentally constructed bridges shall apply. The specified 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 5.9.4.2.2-1.

C5.14.1.4.6

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 5.9.4.2.2-1. 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 specified concrete 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.

5.14.1.4.7—Strength Limit State

The connections between precast girders and a continuity diaphragm shall be designed for the strength limit state.

The reinforcement in the deck slab shall be proportioned to resist negative design moments at the strength limit state.

C5.14.1.4.7

The continuity diaphragm is not prestressed concrete so the stress limits for the service limit state do not apply. Connections to it are therefore designed using provisions for reinforced concrete elements.

5.14.1.4.8—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 5.11.1.2.3. 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 5.11.5. 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 5.7.3 shall apply to the reinforcement in the deck slab and at negative moment connections at continuity diaphragms.

C5.14.1.4.8

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, 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.

5.14.1.4.9—Positive Moment Connections

5.14.1.4.9a—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:

- Mild 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.

C5.14.1.4.9a

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 0.6 M_{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

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 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 5.7.3, except Article 5.7.3.3.2, 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 5.14.1.4.4:

- Factored positive restraint moment, or
- $0.6M_{cr}$

The cracking moment M_{cr} shall be computed using Eq. 5.7.3.6.2-2 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.

$1.2M_{cr}$ provides only minor improvement in continuity behavior over a connection with a capacity of $1.2M_{cr}$. Therefore, it is recommended that the 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 5.7.3.3.2 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 5.7.3.3.2 does not apply. Allowing positive moment connections with lower quantities of reinforcement will relieve congestion in continuity diaphragms.

5.14.1.4.9b—Positive Moment Connection Using Mild Reinforcement

The anchorage of mild reinforcement used for positive moment connections shall satisfy the requirements of Article 5.11 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.

C5.14.1.4.9b

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.

5.14.1.4.9c—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 providing a development length as specified in Article 5.11.4.

The stress in the strands used for design, as a function of the total length of the strand, shall not exceed:

$$f_{psl} = \frac{(\ell_{dsh} - 8)}{0.228} \quad (5.14.1.4.9c\text{-}1)$$

$$f_{pul} = \frac{(\ell_{dsh} - 8)}{0.163} \quad (5.14.1.4.9c\text{-}2)$$

where:

ℓ_{dsh} = total length of extended strand (in.)

f_{psl} = stress in the strand at the service limit state Cracked section shall be assumed (ksi)

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.

Strands that are debonded or shielded 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.

Eqs. 5.14.1.4.9c-1 and 5.14.1.4.9c-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).

5.14.1.4.9d—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.

C5.14.1.4.9d

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.

5.14.1.4.10—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.

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.

C5.14.1.4.10

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 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 5.6.3 may be used to design a continuity diaphragm for these conditions.

5.14.1.5—Cast-in-Place Girders and Box and T-Beams

5.14.1.5.1—Flange and Web Thickness

5.14.1.5.1a—Top Flange

The thickness of top flanges serving as deck slabs shall be:

- As determined in Section 9;
- As required for anchorage and cover for transverse prestressing, if used; and
- Not less than 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.

5.14.1.5.1b—Bottom Flange

The bottom flange thickness shall be not less than:

- 5.5 in.;
- the distance between fillets or webs of nonprestressed girders and beams divided by 16; or
- 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.

5.14.1.5.1c—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.

C5.14.1.5.1c

For adequate field placement and consolidation of concrete, a minimum web thickness of 8.0 in. is needed for webs without prestressing ducts; 12.0 in. is needed for webs with only longitudinal or vertical ducts; and 15.0 in. is needed for webs with both longitudinal and vertical ducts. For girders over about 8.0 ft in depth, these dimensions should be increased to compensate for the increased difficulty of concrete placement.

5.14.1.5.2—Reinforcement

5.14.1.5.2a—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.

5.14.1.5.2b—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.

C5.14.1.5.2b

This provision is intended to apply to both reinforced and prestressed boxes.

5.14.2—Segmental Construction

5.14.2.1—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.

The provisions herein shall apply only to segmental construction using normal-weight concrete.

C5.14.2.1

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 joined 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 5.14.1.3.

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, although many of the specification provisions are also applicable to them.

Lightweight concrete has been infrequently used for segmental bridge construction. Provision for the use of lightweight aggregates represents a significant complication of both design and construction specifications. Given this complication and questions concerning economic benefit, use of lightweight aggregates for segmental bridges is not explicitly covered.

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 5.14.2.5.

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 be required to prepare working drawings and calculations based on his choice of methods for review and approval by the Engineer before work begins.

5.14.2.2—Analysis of Segmental Bridges

5.14.2.2.1—General

The analysis of segmentally constructed bridges shall conform to the requirements of Section 4 and those specified herein.

5.14.2.2.2—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 5.14.2.3.

5.14.2.2.3—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.

C5.14.2.2.3

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 creep models, have been published (AASHTO, 1989). 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.

Joints in segmental girders made continuous by unbonded post-tensioning steel 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.

5.14.2.3—Design

5.14.2.3.1—Loads

In addition to the loads specified in Section 3, the construction loads specified in Articles 5.14.2.3.2 through 5.14.2.3.4 shall be considered.

5.14.2.3.2—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 maxima 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 (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)

C5.14.2.3.2

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 show 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 5.14.2.3.3-1.

The load *DIFF* allows for possible variations in cross-section weight due to construction irregularities.

CEQ = specialized construction equipment: the load from segment delivery trucks and any special equipment, including a formtraveler launching gantry, beam and winch, truss, or similar major auxiliary structure and the maximum loads 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 (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 5.14.2.3.6

SH = shrinkage in accordance with Article 5.14.2.3.6

T = thermal: the sum of the effects due to uniform temperature variation (*TU*) and temperature gradients (*TG*) (°F)

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. 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.

5.14.2.3.3—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 5.14.2.3.3-1, for which the following notes apply:

- Note 1: equipment not working,
- Note 2: normal erection, and
- Note 3: moving equipment.

Stress limits shall conform to Article 5.9.4.

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 compressive strength at the time of load application.

Tensile stresses in concrete due to construction loads shall not exceed the values specified in Table 5.14.2.3.3-1, except for structures with less than 60 percent of their tendon capacity provided by internal tendons, the tensile stresses shall not exceed $0.095\sqrt{f'_c}$. The requirements of Table 5.14.2.3.3-1 shall apply to vertically post-tensioned substructures. The requirements of Table 5.14.2.3.3-1 shall not be applied to construction of cast-in-place substructures supporting segmental superstructures.

C5.14.2.3.3

The stresses in Table 5.14.2.3.3-1 limit construction load stresses to less than the modulus of rupture of the concrete for structures with internal tendons and Type A joints. The construction load stresses should not, therefore, generate any cracking.

Table 5.14.2.3.3-1—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	*DIFF*	*U*	*CE CLL*	*IE*	*CLE*	*WS*	*WUP*	*WE*	*CR*	*SH*	*TU*	*TG*	*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	0.7	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	0.7	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	0.3	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	0.3	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

5.14.2.3.4—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 5.14.2.3.4a and 5.14.2.3.4b.

5.14.2.3.4a—Superstructures

- For maimum force effects:

$$\Sigma\gamma Q = 1.1(DC + DIFF) + 1.3CEQ + A + AI \quad (5.14.2.3.4a\text{-}1)$$

- For minimum force effects:

$$\Sigma\gamma Q = DC + CEQ + A + AI \quad (5.14.2.3.4a\text{-}2)$$

C5.14.2.3.4a

Eqs. 5.14.2.3.4a-1 and 5.14.2.3.4a-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 5.14.2.3.3-1.

5.14.2.3.4b—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}.

C5.14.2.3.4b

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. The loads *CLL* and *WE* may be included and used in place of *LL* and *WL*, respectively, but a reduced load factor may be appropriate if the construction equipment is well-defined during design.

5.14.2.3.5—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.

C5.14.2.3.5

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.

5.14.2.3.6—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.

C5.14.2.3.6

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 calculated as:

$$E_{eff} = \frac{E_c}{\psi(t,t_i)+1} \quad \text{(C5.14.2.3.6-1)}$$

where:

$\Psi(t, t_i)$ = the creep coefficient

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, 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, 1989; 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.

5.14.2.3.7—Prestress Losses

The applicable provisions of Article 5.9.5 shall apply.

C5.14.2.3.7

The friction and wobble coefficients in Article 5.9.5.2.2 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 5.14.2.3.8.

5.14.2.3.8—Provisional Post-Tensioning Ducts and Anchorages

5.14.2.3.8a—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.

5.14.2.3.8b—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.

C5.14.2.3.8b

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.

5.14.2.3.8c—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 positive moment and negative moment post-tensioning force.

C5.14.2.3.8c

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 positive moment and negative moment post-tensioning force 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.

5.14.2.3.9—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."

C5.14.2.3.9

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.

The concrete cross-section shall be proportioned to accommodate an assumed post-tensioning system, reinforcing steel, 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.

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, 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.

5.14.2.3.10—Box Girder Cross-Section Dimensions and Details

5.14.2.3.10a—Minimum Flange Thickness

Top and bottom flange thickness shall not be less than any of the following:

- 1/30 the clear span between webs or haunches. A lesser dimension will require transverse ribs at a spacing equal to the clear span between webs or haunches.
- 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.

Transverse post-tensioning or pretensioning shall be used where the clear span between webs or haunches is 15.0 ft or larger. Strands used for transverse pre-tensioning shall be 0.5 in. diameter or less.

C5.14.2.3.10a

A top flange thickness of 9.0 in. is preferable in the area of anchorages for transverse post-tensioning tendons. A minimum flange thickness of 8.0 in. is recommended.

5.14.2.3.10b—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.

5.14.2.3.10c—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.

5.14.2.3.10d—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 1/1000 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.

C5.14.2.3.10d

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 $L/3200$ in the main span. The deflection limit of $L/1000$ 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.

5.14.2.3.10e—Overlays

Overlays shall be considered for all bridge decks exposed to freeze thaw cycles and application of deicing chemicals. The governing authority should consider providing additional protection against penetration of chlorides. For all types of segmental bridges (precast and cast-in-place), it is recommended that this additional protection be provided by the addition of a minimum of 1.5 in. of concrete cover, added as an overlay or alternatively a waterproof membrane with bituminous overlay. The governing authority may require specific materials and placement techniques stipulated by local practices.

C5.14.2.3.10e

Overlays are encouraged instead of the inclusion of additional monolithic concrete because an overlay will add protection at the critical segment joint. Delamination of overlays is generally due to poor installation practices or material selection and can be resolved. It is not recommended that the additional cover be obtained by merely increasing concrete covers. The added cover will not add protection across the segment joint which is the area of most concern due to the ability of the water to migrate to the tendon and reinforcement.

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.

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.

5.14.2.3.11—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.

C5.14.2.3.11

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.

Superstructure prestressing steel shall be designed to remain below yield for the combined dead load plus seismic demands. The stress in the prestressing steel may be computed by detailed moment curvature analysis, with the stress in bonded prestressing steel computed by strain compatibility with the section and the stress in unbonded prestressing steel computed using global displacement compatibility between bonded sections of tendons located within the span.

The California Department of Transportation evaluates capacity of concrete substructures 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.

5.14.2.4—Types of Segmental Bridges

5.14.2.4.1—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.

C5.14.2.4.1

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. 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.

5.14.2.4.2—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 5.14.2.4.2-1. Shear keys shall also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single-element keys.

C5.14.2.4.2

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.

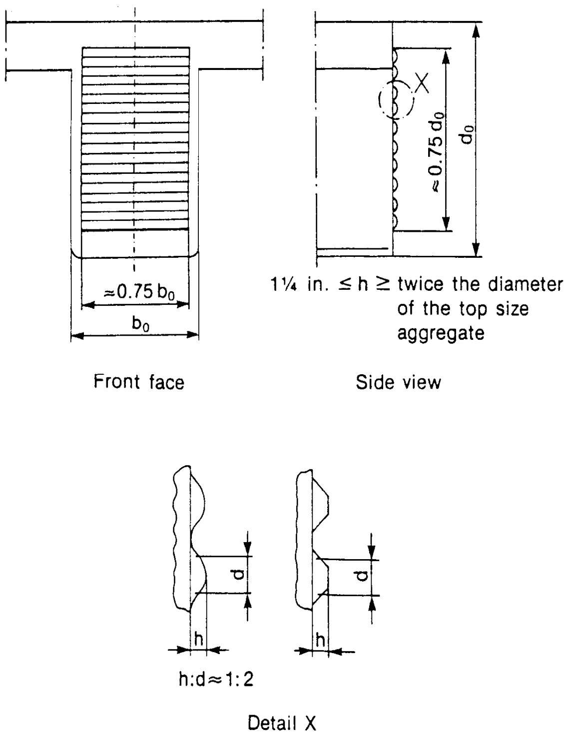


Figure 5.14.2.4.2-1—Example of Fine Indentation Shear Keys

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 reinforcing and the need for a multiple layer protection system. Failures of some post-tensioning reinforcing 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.

AASHTO LRFD Bridge Construction Specifications requires 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.

5.14.2.4.3—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.

C5.14.2.4.3

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.

5.14.2.4.4—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 5.14.2.3.3. 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.

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.

C5.14.2.4.4

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 55 mph corresponds to the load factor 0.30 in Table 3.4.1-1.

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.

5.14.2.4.5—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.

C5.14.2.4.5

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.

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.

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.

5.14.2.4.6—Incrementally Launched Construction

5.14.2.4.6a—General

Stresses under all stages of launching shall not exceed the limits specified in Article 5.9.4 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 zero and four 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.

C5.14.2.4.6a

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.

5.14.2.4.6b—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 5.14.2.3 shall be superimposed upon those from gravity loads. Concrete tensile stresses due to the combined moments shall not exceed $0.221\sqrt{f'_c}$.

5.14.2.4.6c—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.

C5.14.2.4.6c

The dimensional restrictions on placement of launching bearings are shown in Figure C5.14.2.4.6c-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 C5.14.2.4.6c-2.

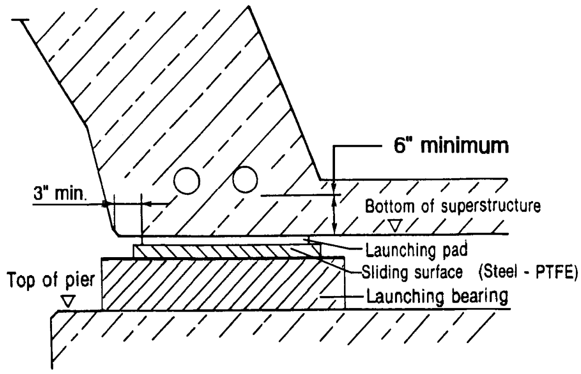


Figure C5.14.2.4.6c-1—Location of Launching Pads

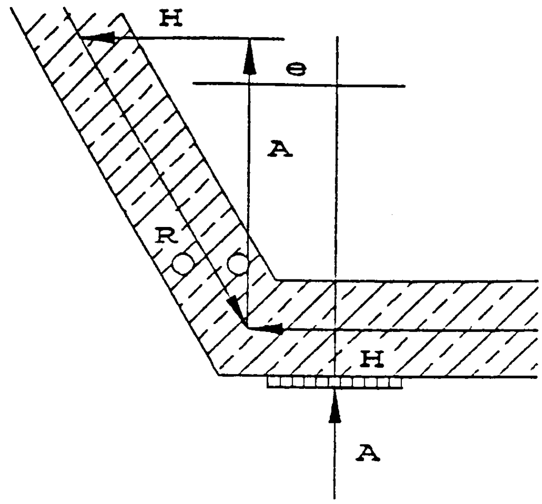


Figure C5.14.2.4.6c-2—Eccentric Reaction at Launching Pads

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 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.

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 reinforcing shall be equivalent to No. 4 bars spaced at 5.0 in.

5.14.2.4.6d—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 5.14.2.4.6b.
- 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 Section 5.14.2.4.6a 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.

5.14.2.5—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

C5.14.2.5

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.

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 Professional Engineer responsible for the redesign shall become the Engineer of Record.

"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, 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."

5.14.2.6—Segmentally Constructed Bridge Substructures

5.14.2.6.1—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 5.7.4.7. The area of discontinuous longitudinal nonprestressed reinforcement may be as specified in Article 5.14.2.6.3.

C5.14.2.6.1

Nonsegmentally constructed substructures are addressed in Sections 10 and 11 and in Article 5.14.2.3.4b.

5.14.2.6.2—Construction Load Combinations

Tensile stresses in vertically prestressed substructures during construction shall be computed for applicable load combinations of Table 5.14.2.3.3-1.

5.14.2.6.3—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 5.10.8.

C5.14.2.6.3

Minimum longitudinal reinforcement of hollow, rectangular precast segmental piers is based on Article 5.10.8 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.

5.14.3—Arches

5.14.3.1—General

The shape of an arch shall be selected with the objective of minimizing flexure under the effect of combined permanent and transient loads.

5.14.3.2—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 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 reinforcing 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.

Spandrel walls shall be provided with expansion joints. Temperature reinforcing shall be provided corresponding to the joint spacing.

The spandrel wall shall be jointed at the springline.

C5.14.3.2

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.

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.

The ACI 207.2R73 *Manual of Concrete Practice* contains a discussion of joint spacing and temperature reinforcement of restrained walls.

The spandrel fill shall be provided with effective drainage. Filters shall be provided to prevent clogging of drains with fine material.

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.

5.14.4—Slab Superstructures

5.14.4.1—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.

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:

$$\frac{100}{\sqrt{L}} \leq 50\% \quad (5.14.4.1\text{-}1)$$

- For longitudinal prestressed construction:

$$\frac{100}{\sqrt{L}} \frac{f_{pe}}{60} \leq 50\% \quad (5.14.4.1\text{-}2)$$

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 5.10.8.

C5.14.4.1

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.

5.14.4.2—Cast-in-Place Voided Slab Superstructures

5.14.4.2.1—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 5.14.1.5.1b.

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.

C5.14.4.2.1

Cross-sections of alternative typical round-voided concrete deck system, taken between piers, are shown in Figure C5.14.4.2.1-1, in which *PT* denotes post-tensioning.

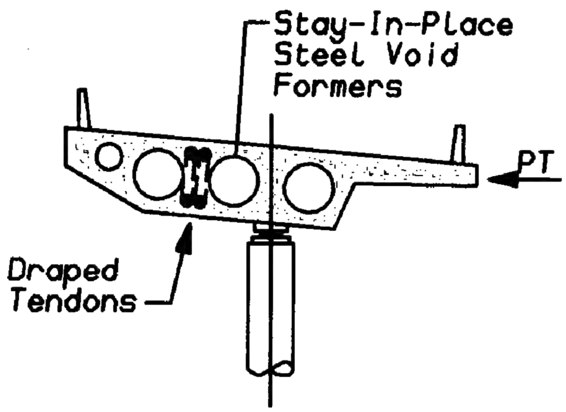


Figure C5.14.4.2.1-1—Cross-Section of Typical Voided Concrete Deck System

The dimensions provided for spacing and size of voids in this Article are based on past experience and are expected to provide safe results. They may be taken as preliminary design values.

5.14.4.2.2—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.

C5.14.4.2.2

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.

5.14.4.2.3—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 5.10.9. 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.

C5.14.4.2.3

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 a strut-and-tie model is advisable.

5.14.4.2.4—General Design Requirements

For voided slabs conforming to the provisions of Article 5.14.4.2.1, 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 5.9.5, shall not be less than 0.5 ksi. When transversely post-tensioned, no additional reinforcing steel need be applied to the concrete above the circular voids.

Transverse shrinkage and temperature steel at the bottom of the voided slab shall satisfy the requirements specified in Article 5.10.8.

C5.14.4.2.4

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.

5.14.4.2.5—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.

C5.14.4.2.5

Recent 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 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 5.10.6.

5.14.4.2.6—Drainage of Voids

Adequate drainage of the voids shall be provided in accordance with the provisions of Article 2.6.6.5.

C5.14.4.2.6

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.

5.14.4.3—Precast Deck Bridges

5.14.4.3.1—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 5.14.1.3.2.

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.

C5.14.4.3.1

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.

5.14.4.3.2—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.

C5.14.4.3.2

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.

5.14.4.3.3—Shear-Flexure Transfer Joints

5.14.4.3.3a—General

Precast longitudinal components may be joined together by transverse post-tensioning, cast-in-place closure joints, a structural overlay, or a combination thereof.

C5.14.4.3.3a

These joints are intended to provide full continuity and monolithic behavior of the deck.

5.14.4.3.3b—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 5.14.4.3.3d.

C5.14.4.3.3b

From the modeling point of view, these precast concrete deck systems are not different from cast-in-place ones of the same geometry.

5.14.4.3.3c—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.

C5.14.4.3.3c

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 cable 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.

5.14.4.3.3d—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.

C5.14.4.3.3d

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.

5.14.4.3.3e—Cast-in-Place Closure Joint

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.

5.14.4.3.3f—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 5.10.8. The top surface of the precast components shall be roughened.

C5.14.4.3.3f

The composite overlay should be regarded as a structural component and should be designed and detailed accordingly.

5.14.5—Additional Provisions for Culverts

5.14.5.1—General

The soil structure aspects of culvert design are specified in Section 12.

5.14.5.2—Design for Flexure

The provisions of Article 5.7 shall apply.

5.14.5.3—Design for Shear in Slabs of Box Culverts

The provisions of Article 5.8 apply unless modified herein. For slabs of box culverts under 2.0 ft or more fill, shear strength V_c may be computed by:

$$V_c = \left(0.0676\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u} \right) bd_e \qquad (5.14.5.3\text{-}1)$$

but V_c shall not exceed $0.126\sqrt{f'_c}\, bd_e$

where:

A_s = area of reinforcing steel in the design width (in.^{2})

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

V_u = shear from factored loads (kip)

M_u = moment from factored loads (kip-in.)

b = design width (in.)

For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than $0.0948\sqrt{f'_c}bd_e$, and V_c for slabs simply supported need not be taken to be less than $0.0791\sqrt{f'_c}bd$. The quantity $V_u d_e/M_u$ 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 5.8 and 5.13.3.6 shall apply to slabs of box culverts under less than 2.0 ft of fill and to sidewalls.

C5.14.5.3

Eq. 5.14.5.3-1, as originally proposed, included an additional multiplier to account for axial compression. Because the effect was considered relatively small, it was deleted from Eq. 5.14.5.3-1. However, if the Designer wishes, effect of axial compression may be included by multiplying the results of Eq. 5.14.5.3-1 by the quantity $(1+0.04\ N_u/V_u)$.

The lower limits of $0.0948\sqrt{f'_c}$ and $0.0791\sqrt{f'_c}$ are compared with test results in Figure C5.14.5.3-1.

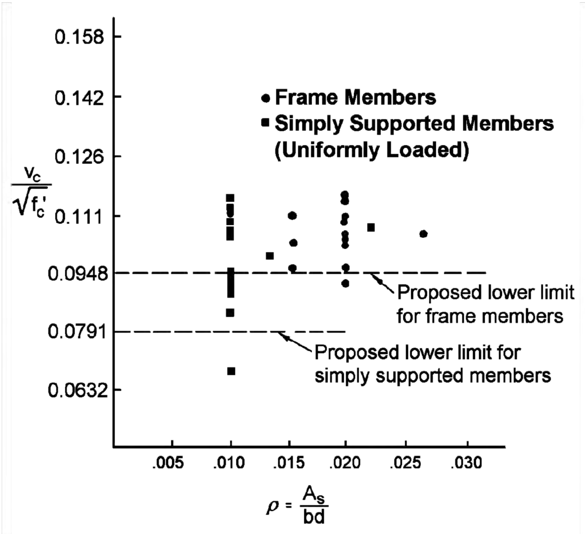


Figure C5.14.5.3-1—Culvert Test Results

5.15—REFERENCES

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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 (5.14.1.2.2)
    b. Bottom Flange (5.14.1.2.2)
    c. Webs (5.14.1.2.2)
    d. Structure Depth (2.5.2.6.3)
    e. Minimum Reinforcement (5.7.3.3.2) (5.7.3.4)
    f. Lifting Devices (5.14.1.2.3)
    g. Joints (5.14.1.3.2)
  2. CIP T-Beams and Multiweb Box Girders (5.14.1.5)
    a. Top Flange (5.14.1.5.1a)
    b. Bottom Flange (5.14.1.5.1b)
    c. Webs (5.14.1.5.1c)
    d. Structure Depth (2.5.2.6.3)
    e. Reinforcement (5.14.1.5.2)
      (1) Minimum Reinforcement (5.7.3.3.2) (5.7.3.4)
      (2) Temperature and Shrinkage Reinforcement (5.10.8)
    f. Effective Flange Widths (4.6.2.6)
    g. Strut-and-Tie Areas, if Any (5.6.3)
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) (5.5.4.2.1)
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)

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 (5.9.5)
2. Stress Limitations for P/S Tendons (5.9.3)
3. Stress Limitations for P/S Concrete (5.9.4)
a. Before Losses (5.9.4.1)
b. After Losses (5.9.4.2)
4. Durability (5.12)
5. Crack Control (5.7.3.4)
6. Fatigue, if Applicable (5.5.3)
7. Deflection and Camber (2.5.2.6.2) (3.6.1.3.2) (5.7.3.6.2)

J. Investigate Strength Limit State
1. Flexure
a. Stress in P/S Steel—Bonded Tendons (5.7.3.1.1)
b. Stress in P/S Steel—Unbonded Tendons (5.7.3.1.2)
c. Flexural Resistance (5.7.3.2)
d. Limits for Reinforcement (5.7.3.3)
2. Shear (Assuming No Torsional Moment)
a. General Requirements (5.8.2)
b. Sectional Design Model (5.8.3)
(1) Nominal Shear Resistance (5.8.3.3)
(2) Determination of β and θ (5.8.3.4)
(3) Longitudinal Reinforcement (5.8.3.5)
(4) Transverse Reinforcement (5.8.2.4) (5.8.2.5) (5.8.2.6) (5.8.2.7)
(5) Horizontal Shear (5.8.4)

K. Check Details
1. Cover Requirements (5.12.3)
2. Development Length—Reinforcing Steel (5.11.1) (5.11.2)
3. Development Length—Prestressing Steel (5.11.4)
4. Splices (5.11.5) (5.11.6)
5. Anchorage Zones
a. Post-Tensioned (5.10.9)
b. Pretensioned (5.10.10)
6. Ducts (5.4.6)
7. Tendon Profile Limitation
a. Tendon Confinement (5.10.4)
b. Curved Tendons (5.10.4)
c. Spacing Limits (5.10.3.3)
8. Reinforcement Spacing Limits (5.10.3)
9. Transverse Reinforcement (5.8.2.6) (5.8.2.7) (5.8.2.8)
10. Beam Ledges (5.13.2.5)

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 (5.14.4.1)
F. Investigate Distribution Reinforcement (5.14.4.1)

G. If Not Solid
   1. Check if Voided Slab or Cellular Construction (5.14.4.2.1)
   2. Check Minimum and Maximum Dimensions (5.14.4.2.1)
   3. Design Diaphragms (5.14.4.2.3)
   4. Check Design Requirements (5.14.4.2.4)

A5.5—SUBSTRUCTURE DESIGN

A. Establish Minimum Seat Width
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 (5.7.4)
   1. Factored Axial Resistance (5.7.4.4)
   2. Biaxial Flexure (5.7.4.5)
   3. Slenderness Effects (4.5.3.2.2) (5.7.4.3)
   4. Transverse Reinforcement (5.7.4.6)
   5. Shear (Usually EQ and Ship Collision Induced) (3.10.9.4.3)
   6. Reinforcement Limits (5.7.4.2)
   7. Bearing (5.7.5)
   8. Durability (5.12)
   9. Detailing (As in Step A5.3K) and Seismic (5.10.11)
E. Design Foundations (Structural Considerations)
   1. Scour
   2. Footings (5.13.3)
   3. Abutments (Section 11)
   4. Pile Detailing (5.13.4)

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 5.8.3.4.2. The procedure in this Appendix utilizes tabularized values of β and θ instead of Eqs. 5.8.3.4.2-1, 5.8.3.4.2-2, and 5.8.3.4.2-3. Appendix B5 is a complete presentation of the general procedures in LRFD Design (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 5.8.2.5, the values of β and θ shall be as specified in Table B5.2-1. In using this table, εx shall be taken as the calculated longitudinal strain at the middepth 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 5.8.2.5, the values of β and θ shall be as specified in Table B5.2-2. In using this table, εx 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.

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 5.8.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})} \quad \text{(B5.2-1)}$$

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 5.8.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \quad \text{(B5.2-2)}$$

CB5.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.8.3.1. Such an analysis (see Figure 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.

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 of this capacity to redistribute, it is appropriate to use the middepth 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, 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_c. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u - V_p$, will primarily be carried by diagonal compressive stresses in the web concrete. These

The initial value of ε_x should not be taken greater than 0.002.

- If the value of ε_x from Eqs. B5.2-1 or B5.2-2 is negative, the strain shall be taken as:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_c A_c + E_s A_s + E_p A_{ps}\right)} \tag{B5.2-3}$$

where:

A_c = 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_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})

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. For the usual levels of prestressing, a value of $0.7f_{pu}$ will be appropriate for both pretensioned and post-tensioned members (ksi)

M_u = factored moment, not to be taken less than V_ud_v (kip-in.)

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

V_u = factored shear force (kip)

Within the transfer length, f_{po} shall be increased linearly from zero at the location where the bond between the strands and 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 s_{xe}, used in Table B5.2-2, shall be determined as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ in.} \tag{B5.2-4}$$

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. B5.2-1, B5.2-2, and B5.2-3. 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:

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2} \tag{CB5.2-1}$$

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. B5.2-1. For members containing less than the minimum amount of transverse reinforcement, Eq. B5.2-2 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. B5.2-1 through B5.2-3. 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 Eqs. B5.2-1 through B5.2-3 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. B5.2-1 through B5.2-3.

Absolute value signs were added to Eqs. B5.2-1 through B5.2-3 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. B5.2-1 through B5.2-3 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.

where:

a_g = maximum aggregate size (in.)

s_x = the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003b_vs_x$, as shown in Figure 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.
- The value of ε_x calculated from Eqs. B5.2-2 and B5.2-3 should not be taken as less than -0.20×10^{-3}.
- 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. B5.2-2 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 that that calculated from Eqs. B5.2-2 and B5.2-3; however, ε_x shall not be taken greater than 3.0×10^{-3}.

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. B5.2-1, B5.2-2, or B5.2-3, 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.

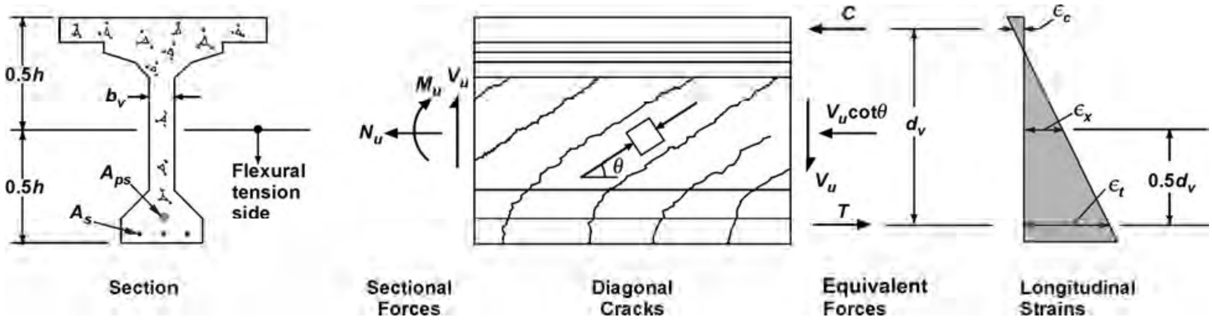


Figure B5.2-1—Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, $V_p = 0$

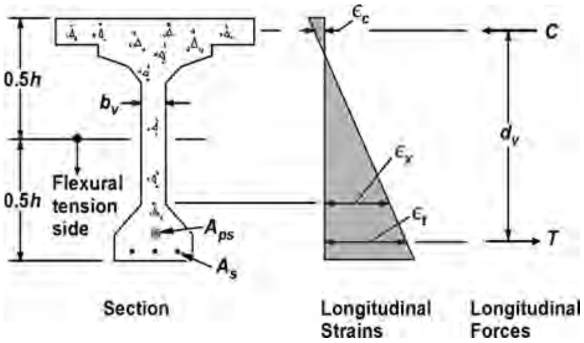
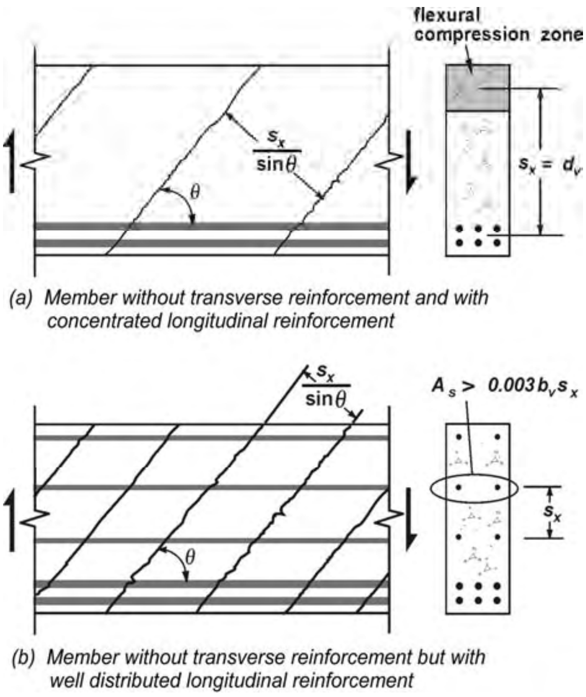


Figure B5.2-2—Longitudinal Strain, ε_x, for Sections Containing Less than the Minimum Amount of Transverse Reinforcement



(a) Member without transverse reinforcement and with concentrated longitudinal reinforcement

(b) Member without transverse reinforcement but with well distributed longitudinal 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 εx, 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 moment capacity M_n can be found by calculating, from Eqs. B5.2-1 through B5.2-3, the moment required to cause this chosen value of ε_x, and calculating, from Eq. 5.8.3.5-1, the moment required to yield the reinforcement. The predicted moment capacity will be the lower of these two values. In using Eqs. 5.8.2.9-1, 5.8.3.5-1, and Eqs. B5.2-1 through B5.2-3 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.

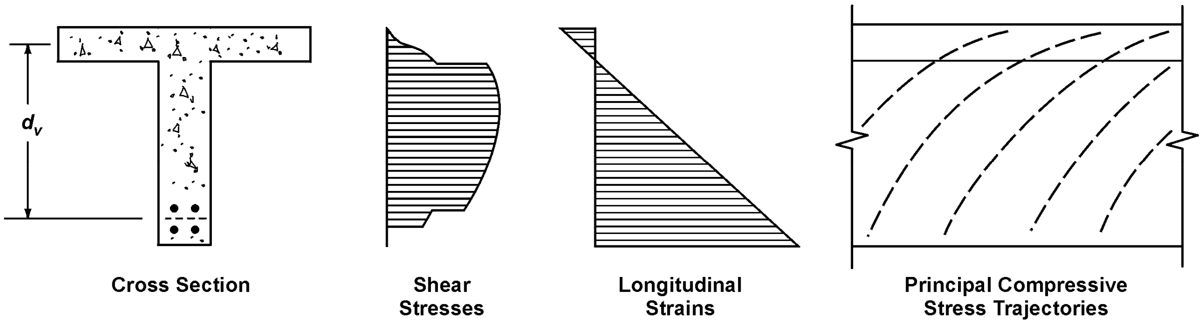


Figure CB5.2-1—Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.8.3.1

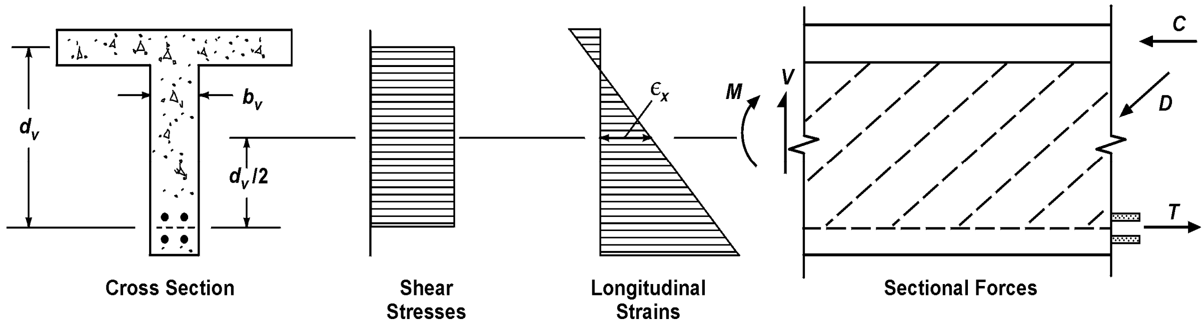


Figure CB5.2-2—More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.8.3.4.2

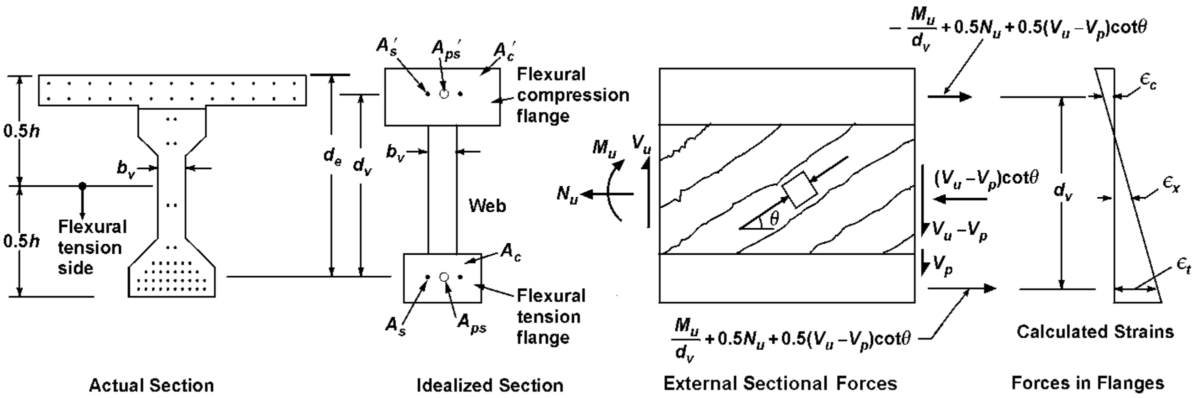


Figure CB5.2-3—More Accurate Calculation Procedure for Determining ε_x

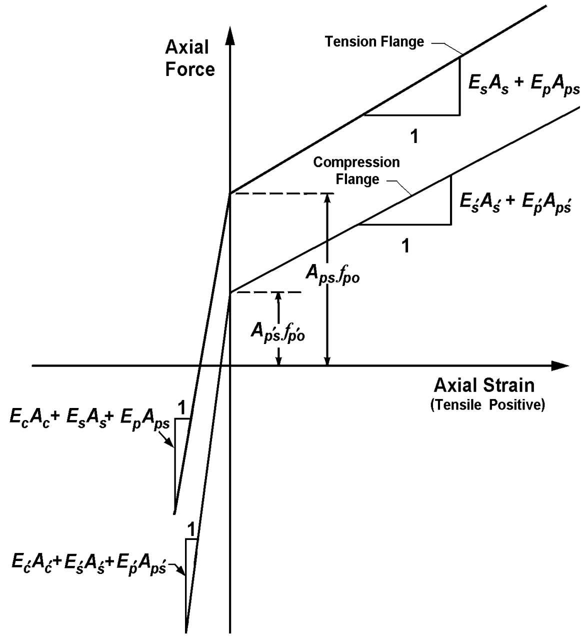


Figure CB5.2-4—Assumed Relations between Axial Force in Flange and Axial Strain of Flange

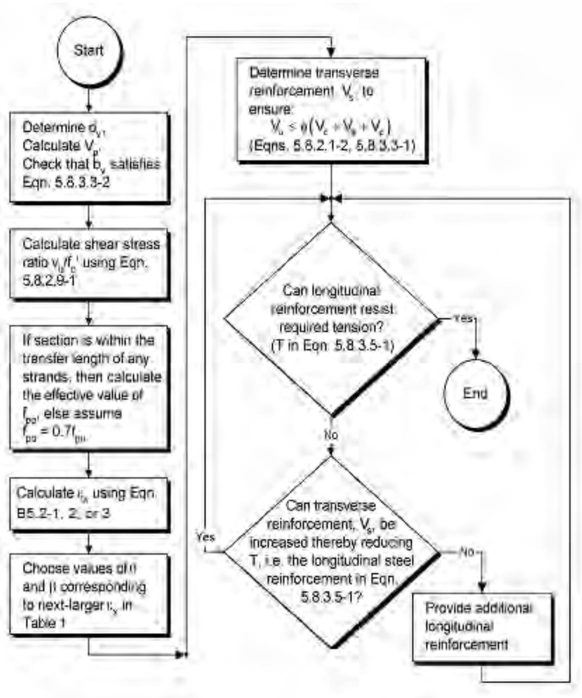


Figure CB5.2-5—Flow Chart for Shear Design of Section Containing at Least Minimum Transverse Reinforcement

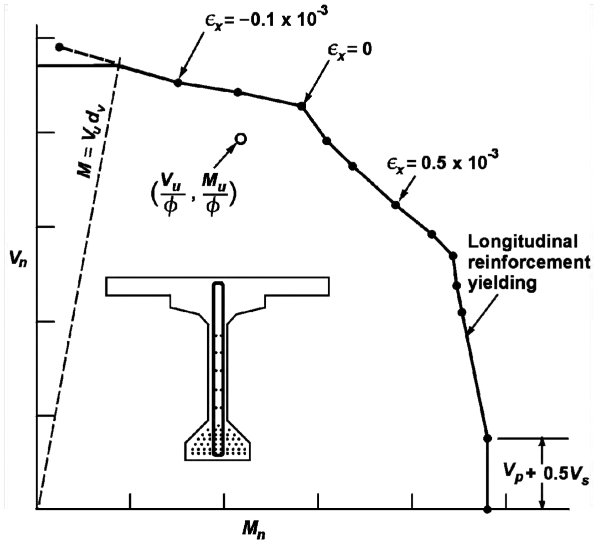


Figure CB5.2-6—Typical Shear-Moment Interaction Diagram

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

Table B5.2-2—Values of θ and β for Sections with Less than Minimum Transverse Reinforcement

s_{xe}, in.	$\varepsilon_x \times 1000$										
	≤–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

APPENDIX C5—UPPER LIMITS FOR ARTICLES AFFECTED BY CONCRETE COMPRESSIVE STRENGTH

Article*	Upper Limit, ksi	
	10.0	15.0**
5.1—Scope		By exception
5.4.2.1—Compressive Strength		By exception
5.4.2.3—Shrinkage and Creep		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	
5.6.3.3.3—Limiting Compressive Stress in Strut	X	
5.6.3.5—Proportioning of Node Regions	X	
5.6.3.6—Crack Control Reinforcement	X	
5.7.2—Assumptions for Strength and Extreme Event Limit States	X	
5.7.3.1—Stress in Prestressing Steel at Nominal Flexural Resistance	X	
5.7.3.2—Flexural Resistance	X	
5.7.3.3—Limits for Reinforcement	X	
5.7.3.4—Control of Cracking by Distribution of Reinforcement	X	
5.7.3.5—Moment Redistribution	X	
5.7.3.6—Deformations	X	
5.7.4.2—Limits for Reinforcement	X	
5.7.4.3—Approximate Evaluation of Slenderness Effects	X	
5.7.4.4—Factored Axial Resistance	X	
5.7.4.5—Biaxial Flexure	X	
5.7.4.6—Spirals and Ties	X	
5.7.4.7—Hollow Rectangular Compression Members	X	
5.7.5—Bearing	X	
5.8.2.1—General	X	
5.8.2.3—Transfer and Development Lengths	X	
5.8.2.7—Maximum Spacing of Transverse Reinforcement	X	
5.8.3—Sectional Design Model	X	
5.8.4—Interface Shear Transfer—Shear Friction	X	
5.8.6—Shear and Torsion for Segmental Box Girder Bridges	X	
5.9.1—General Design Considerations	X	
5.9.4—Stress Limits for Concrete	X	
5.9.5—Loss of Prestress		X
5.10.4.3—Effects of Curved Tendons	X	
5.10.6.2—Spirals	X	
5.10.6.3—Ties	X	
5.10.8—Shrinkage and Temperature Reinforcement	X	
5.10.9.3.1—Design Methods	X	
5.10.9.4—Application of the Strut-and-Tie Model to the Design of General Zone	X	
5.10.9.7.2—Bearing Resistance	X	
5.10.11.4—Seismic Zones 3 and 4	X	
5.11.2.1—Deformed Bars and Deformed Wire in Tension	X	
5.11.2.2—Deformed Bars in Compression	X	
5.11.2.3—Bundled Bars	X	
5.11.2.4—Standard Hooks in Tension	X	

Article*	Upper Limit, ksi	
	10.0	15.0**
5.11.2.5—Welded Wire Fabric	X	
5.11.2.6—Shear Reinforcement	X	
5.11.4.1—General	X	
5.11.4.2—Bonded Strand	X	
5.11.4.3—Partially Debonded Strands	X	
5.11.5.3—Splices of Reinforcement in Tension	X	
5.11.5.5—Splices of Bars in Compression	X	
5.13.2.4—Brackets and Corbels	X	
5.13.2.5—Beam Ledges	X	
5.13.3.6—Shear in Slabs and Footings	X	
5.13.4.6—Seismic Requirements	X	
5.14.1—Beams and Girders	X	
5.14.2.3—Design	X	
5.14.5—Additional Provisions for Culverts	X	

* Applies to all subarticles of the listed Article
** Normal weight concrete only

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