SECTION 8 - REINFORCED CONCRETE[1]

Part A
General Requirements and Materials

8.1 APPLICATION

8.1.1 General

The specifications of this section are intended for design of reinforced (non-prestressed) concrete bridge members and structures. Bridge members designed as prestressed concrete shall conform to Section 9.

8.1.2 Notations

a = depth of equivalent rectangular stress block (Article 8.16.2.7)

a_b = depth of equivalent rectangular stress block for balanced strain conditions, inches (Article 8.16.4.2.3)

a_v = shear span, distance between concentrated load and face of support (Articles 8.15.5.8 and 8.16.6.8)

A = effective tension area, in square inches, of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires. When the flexural reinforcement consists of several bar size or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used. For calculation purposes, the thickness of clear concrete cover used to compute A shall not be taken greater than 2 inches. (Article 8.16.8.4)

A_b = area of an individual bar, square inches (Article 8.25.1)

A_c = area of core of spirally reinforced compression member measured to the outside diameter of the spiral, square inches (Article 8.18.2.2.2); area of concrete section resisting shear, square inches (Article 8.16.6.9)

A_{cv} = area of concrete section resisting shear transfer, square inches (Article 8.16.6.4.5)

A_f = area of reinforcement in bracket or corbel resisting moment, square inches (Articles 8.15.5.8 and 8.16.6.8)

A_g = gross area of section, square inches

A_h = area of shear reinforcement parallel to flexural tension reinforcement, square inches (Articles 8.15.5.8 and 8.16.6.8)

A_n = area of reinforcement in bracket or corbel resisting tensile force, N_c (N_{uc}), square inches (Articles 8.15.5.8 and 8.16.6.8)

A_s = area of tension reinforcement, square inches

A'_s = area of compression reinforcement, square inches

A_{sf} = area of reinforcement to develop compressive strength of overhanging flanges of I- and T-sections (Article 8.16.3.3.2)

A_{sh} = total cross sectional area of tie reinforcement including supplementary cross ties within a section having limits of s_t and h_c, square inches (Article 8.18.2.3.1)

A_{st} = total area of longitudinal reinforcement (Articles 8.16.4.1.2 and 8.16.4.2.1)

A_v = area of shear reinforcement within a distance s, square inches (Article 8.15.5.3.2)

A_{vf} = area of shear-friction reinforcement, square inches (Article 8.15.5.4.3)

A_w = area of an individual wire to be developed or spliced, square inches (Articles 8.30.1.2 and 8.30.2)

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[1] The Specifications of Section 8 are patterned after and are in general conformity with the provisions of ACI Standard 318 for reinforced concrete design and its commentary, ACI 318R, published by the American Concrete Institute.

A_1 = loaded area (Articles 8.15.2.1.3 and 8.16.7.2)

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area (Articles 8.15.2.1.3 and 8.16.7.2)

b = width of compression face of member

b_o = perimeter of critical section for slabs and footings (Articles 8.15.5.6.2 and 8.16.6.6.2)

b_v = width of cross section at contact surface being investigated for horizontal shear (Article 8.15.5.5.3)

b_w = web width, or diameter of circular section.
+ For tapered webs, the average width or 1.2
+ times the minimum width, whichever is
+ smaller, inches (Article 8.15.5.1.1)

c = distance from extreme compression fiber to neutral axis (Article 8.16.2.7)

C_m = factor relating the actual moment diagram to an equivalent uniform moment diagram (Article 8.16.5.2.7)

d = distance from extreme compression fiber to centroid of tension reinforcement, inches. For computing shear strength of circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member. For computing horizontal shear strength of composite members, d shall be the distance from extreme compression fiber to centroid of tension reinforcement for entire composite section.

d' = distance from extreme compression fiber to centroid of compression reinforcement, inches

d'' = distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement, inches

d_b = nominal diameter of bar or wire, inches

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto (Article 8.16.8.4)

E_c = modulus of elasticity of concrete, psi (Article 8.7.1)

EI = flexural stiffness of compression member (Article 8.16.5.2.7)

E_s = modulus of elasticity of reinforcement, psi (Article 8.7.2)

f_b = average bearing stress in concrete on loaded area (Articles 8.15.2.1.3 and 8.16.7.1)

f_c = extreme fiber compressive stress in concrete at service loads (Article 8.15.2.1.1)

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_f = fatigue stress range in reinforcement, ksi (Article 8.16.8.3)

f_{min} = algebraic minimum stress level in reinforcement (Article 8.16.8.3)

f_r = modulus of rupture of concrete, psi (Article 8.15.2.1.1)

f_s = tensile stress in reinforcement at service loads, psi (Article 8.15.2.2)

f'_s = stress in compression reinforcement at balanced conditions (Articles 8.16.3.4.3 and 8.16.4.2.3)

f_t = extreme fiber tensile stress in concrete at service loads (Article 8.15.2.1.1)

f_y = specified yield strength of reinforcement, psi

h = overall thickness of member, inches

h_c = core dimension of tied column in the direc- +
tion under consideration (out-to-out of ties) +
(Article 8.18.2.3.1) +

h_f = compression flange thickness of I- and T-sections

I_{cr} = moment of inertia of cracked section transformed to concrete (Article 8.13.3)

I_e = effective moment of inertia for computation of deflection (Article 8.13.3)

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

I_s = moment of inertia of reinforcement about centroidal axis of member cross section

k = effective length factor for compression +
members (Article 8.16.5.2 and Appendix C) +

l_a = additional embedment length at support or at point of inflection, inches (Article .24.2.3)

l_d = development length, inches

l_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook (point of tangency) plus radius of bend and one bar diameter), inches (Article 8.29)

l_{dh} = l_{hb} × applicable modification factor
l_{hb} = basic development length of standard hook in tension, inches
l_u = unsupported length of compression member (Article 8.16.5.2.1)
M = computed moment capacity (Article 8.24.2.3)
M_a = maximum moment in member at stage for which deflection is being computed (Article 8.13.3)
M_b = nominal moment strength of a section at balanced strain conditions (Article 8.16.4.2.3)
M_c = moment to be used for design of compression member (Article 8.16.5.2.7)
M_{cr} = cracking moment (Article 8.13.3)
M_n = nominal moment strength of a section
M_{nx} = nominal moment strength of a section in the direction of the x axis (Article 8.16.4.3)
M_{ny} = nominal moment strength of a section in the direction of the y axis (Article 8.16.4.3)
M_u = factored moment at section
M_{ux} = factored moment component in the direction of the x axis (Article 8.16.4.3)
M_{uy} = factored moment component in the direction of the y axis (Article 8.16.4.3)
M_{1b} = value of smaller end moment on compression member due to gravity loads that result in no appreciable side sway calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature (Article 8.16.5.2.4)
M_{2b} = value of larger end moment on compression member due to gravity loads that result in no appreciable side sway calculated by conventional elastic frame analysis, always positive (Article 8.16.5.2.4)
M_{2s} = value of larger end moment on compression member due to lateral loads or gravity loads that result in appreciable side sway, defined by a deflection Δ, greater than $l_u/1500$, calculated by conventional elastic frame analysis, always positive (Article 8.16.5.2)
n = modular ratio of elasticity
= E_s/E_c (Article 8.15.3.4)
N = design axial load normal to cross section occurring simultaneously with V, to be taken as positive for compression, negative for tension and to include the effects of tension due to shrinkage and creep (Articles 8.15.5.2.2 and 8.15.5.2.3)
N_c = design tensile force applied at top of bracket or corbel acting simultaneously with V, to be taken as positive for tension (Article 8.15.5.8)
N_u = factored axial load normal to the cross section occurring simultaneously with V_u to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep (Article 8.16.6.2.2)
N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension (Article 8.16.6.8)
P_b = nominal axial load strength of a section at balanced strain conditions (Article 8.16.4.2.3)
P_c = critical load (Article 8.16.5.2.7)
P_e = design axial load due to gravity and seismic loading (Articles 8.18.2.2 and 8.18.2.3) +
P_o = nominal axial load strength of a section at zero eccentricity (Article 8.16.4.2.1)
P_n = nominal axial load strength at given eccentricity
P_{nx} = nominal axial load strength corresponding to M_{nx}, with bending considered in the direction of the x axis only (Article 8.16.4.3)
P_{ny} = nominal axial load strength corresponding to M_{ny}, with bending considered in the direction of the y axis only (Article 8.16.4.3)
P_{nxy} = nominal axial load strength with biaxial loading (Article 8.16.4.3)
P_u = factored axial load at given eccentricity
r = radius of gyration of cross section of a compression member (Article 8.16.5.2.2)
s = spacing of shear reinforcement in direction parallel to the longitudinal reinforcement, inches
s_t = vertical spacing of ties, inches (Article 8.18.2.3.1) +
s_w = spacing of wires to be developed or spliced, inches
S = span length, feet
V = design shear force at section (Article 8.15.5.1.1)
v = design shear stress at section (Article 8.15.5.1.1)

V_c = nominal shear strength provided by concrete (Article 8.16.6.1)

v_c = permissible shear stress carried by concrete (Article 8.15.5.2)

v_{dh} = design horizontal shear stress at any cross section (Article 8.15.5.5.3)

v_h = permissible horizontal shear stress (Article 8.15.5.5.3)

V_n = nominal shear strength (Article 8.16.6.1)

V_{nh} = nominal horizontal shear strength (Article 8.16.6.5.3)

V_s = nominal shear strength provided by shear reinforcement (Article 8.16.6.1)

V_u = factored shear force at section (Article 8.16.6.1)

+ v_u = limiting shear stress, psi (Article 8.18.2.1.6)

w_c = weight of concrete, lbs per cubic foot.

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension (Article 8.13.3)

z = quantity limiting distribution of flexural reinforcement (Article 8.16.8.4)

α (alpha) = angle between inclined shear reinforcement and longitudinal axis of member

α_f = angle between shear-friction reinforcement and shear plane (Articles 8.15.5.4 and 8.16.6.4)

β_b (beta) = ratio of area of reinforcement cut off to total area of reinforcement at the section (Article 8.24.1.4.2)

β_c = ratio of long side to short side of concentrated load or reaction area; for a circular concentrated load or reaction area, $\beta_c = 1.0$ (Articles 8.15.5.6.3 and 8.16.6.6.2)

β_d = absolute value of ratio of maximum dead load moment to maximum total load moment, always positive

β_1 = ratio of depth of equivalent compression zone to depth from fiber of maximum compressive strain to the neutral axis (Article 8.16.2.7)

λ (lambda) = correction factor related to unit weight for concrete (Articles 8.15.5.4 and 8.16.6.4)

μ (mu) = coefficient of friction (Article 8.15.5.4.3)

ρ (rho) = tension reinforcement ratio = A_s/bd

ρ' = compression reinforcement ratio = A'_s/bd

ρ_b = reinforcement ratio producing balanced strain conditions (Article 8.16.3.1.1)

ρ_h = the ratio of horizontal shear reinforcement area to gross concrete area of a vertical section in pier walls (Article 8.16.6.9.3) +

ρ_n = the ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section in pier walls (Article 8.18.1.5) +

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member (Article 8.18.2.2.2)

ρ_w = reinforcement ratio used in Equation (8-4) and Equation (8-48) = $A_s/b_w d$

δ_b (delta) = moment magnification factor for members braced against side sway to reflect effects of member curvature between ends of compression member

δ_s = moment magnification factor for members not braced against sidesway to reflect lateral drift resulting from lateral and gravity loads

ϕ (phi) = strength reduction factor (Article 8.16.1.2)

8.1.3 Definitions

The following terms are defined for general use in Section 8. Specialized definitions appear in individual Articles.

Bracket or corbel - Short (haunched) cantilever that projects from the face of a column or wall to support a concentrated load or beam reaction. (Articles 8.15.5.8 and 8.16.6.8)

Compressive strength of concrete (f'_c) - Specified compressive strength of concrete in pounds per square inch (psi).

Concrete, structural lightweight - A concrete containing lightweight aggregate having an air-dry unit weight as determined by "*Method of Test for Unit Weight of Structural Lightweight Concrete*" (ASTM[2] C 567), not exceeding 115 pcf. In this specification, a lightweight concrete without natural sand is termed "all-lightweight concrete" and one in which all fine aggregate consists of normal weight sand is termed "sand-lightweight concrete."

2 American Society for Testing and Materials

Deformed reinforcement - Deformed reinforcing bars, deformed wire, welded smooth wire fabric, and welded deformed wire fabric.

Design load - All applicable loads and forces or their related internal moments and forces used to proportion members. For design by SERVICE LOAD DESIGN, design load refers to loads without load factors. For design by STRENGTH DESIGN METHOD, design load refers to loads multiplied by appropriate load factors.

Design strength - Nominal strength multiplied by a strength reduction factor, ϕ

Development length - Length of embedded reinforcement required to develop the design strength of the reinforcement at a critical section.

Embedment length - Length of embedded reinforcement provided beyond a critical section.

Factored load - Load, multiplied by appropriate load factors, used to proportion members by the STRENGTH DESIGN METHOD.

Nominal strength - Strength of a member or cross section calculated in accordance with provisions and assumptions of the STRENGTH DESIGN METHOD before application of any strength reduction factors.

Plain reinforcement - Reinforcement that does not conform to the definition of deformed reinforcement.

Required strength - Strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in Article 3.22.

Service load - Loads without load factors.

Spiral reinforcement - Continuously wound reinforcement in the form of a cylindrical helix.

Splitting tensile strength (f_{ct}) - Tensile strength of concrete determined in accordance with "*Specifications for Lightweight Aggregates for Structural Concrete*" AASHTO M 195[3] (ASTM C 330).

[3] Standard Specifications for Transportation Materials and Methods of Sampling and Testing

Stirrups or ties - Lateral reinforcement formed of individual units, open or closed, or of continuously wound reinforcement. The term "stirrups" is usually applied to lateral reinforcement in horizontal members and the term "ties" to those in vertical members.

Tension tie member - Member having an axial tensile force sufficient to create tension over the entire cross section and having limited concrete cover on all sides. Examples include: arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.

Yield strength or yield point (f_y) - Specified minimum yield strength or yield point of reinforcement in pounds per square inch.

8.2 CONCRETE

The specified compressive strength, f'_c, of the concrete for each part of the structure shall be shown on the plans. Use f'_c = 3600 psi minimum for reinforced concrete.

8.3 REINFORCEMENT

8.3.1 The yield strength or grade of reinforcement shall be shown on the plans.

8.3.2 *Deleted*

8.3.3 Designs shall, except as shown below, be based on a yield strength, f_y, of 60,000 psi.

8.3.4 Deformed reinforcement shall be used except that plain bars or smooth wire may be used for spirals and ties.

8.3.5 The following structures shall be designed using f_y = 40,000 psi: minor structures, slope and channel paving, sign foundations (pile and footing types), roadside rest facilities, concrete barrier (Type 50 series) and temporary railing.

Part B
Analysis

8.4 GENERAL

All members of continuous and rigid frame structures shall be designed for the maximum effects of the loads specified in Articles 3.2 through 3.22 as determined by the theory of elastic analysis.

8.5 EXPANSION AND CONTRACTION

8.5.1 In general, provision for temperature changes shall be made in simple spans when the span length exceeds 40 feet.

8.5.2 In continuous bridges, the design shall provide for thermal stresses or for the accommodation of thermal movement with rockers, sliding plates, elastomeric pads, or other means.

8.5.3 The coefficient of thermal expansion and contraction for normal weight concrete may be taken as 0.000006 per deg. F.

8.5.4 The coefficient of shrinkage for normal weight concrete may be taken as 0.0002.

8.5.5 Thermal and shrinkage coefficients for lightweight concrete shall be determined for the type of lightweight aggregate used.

8.6 STIFFNESS

8.6.1 Any reasonable assumptions may be adopted for computing the relative flexural and torsional stiffnesses of continuous and rigid frame members. The assumptions made shall be consistent throughout the analysis.

8.6.2 The effect of haunches shall be considered both in determining moments and in design of members.

8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO

8.7.1 The modulus of elasticity, E_c, for concrete may be taken as $w_c^{\,1.5}\, 33\sqrt{f'_c}$ in psi for values of w_c between 90 and 155 pounds per cubic foot. For normal weight concrete ($w_c = 145\ pcf$), E_c may be considered as $57000\sqrt{f'_c}$.

8.7.2 The modulus of elasticity E_s for nonprestressed steel reinforcement may be taken as 29,000,000 psi.

8.7.3 Poisson's ratio may be assumed as 0.2.

8.8 SPAN LENGTH

8.8.1 The span length of members that are not built integrally with their supports shall be considered the clear span plus the depth of the member but need not exceed the distance between centers of supports.

8.8.2 In analysis of continuous and rigid frame members, distances to the geometric centers of members shall be used in the determination of moments. Moments at faces of support may be used for member design. When 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 shall 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 shall be considered as adding to the effective depth.

Column flares which are designed and detailed to be monolithic with a continuous or restrained member shall be considered as fillets. However, no portion of the flares shall be considered as fillets if the flares are designed and detailed as sacrificial flares, or if the flares are separated from the continuous or restrained member by a gap.

8.8.3 The effective span length of slabs shall be as specified in Article 3.24.1.

8.9 CONTROL OF DEFLECTIONS

8.9.1 General

Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect the strength or serviceability of the structure at service load plus impact.

8.9.2 Superstructure Depth Limitations

The minimum depths stipulated in Table 8.9.2 are recommended unless computation of deflection indicates that lesser depths may be used without adverse effects.

TABLE 8.9.2 Recommended Minimum Depths for Constant Depth Members

Superstructure Type	Minimum Depth[a] in Feet	
	Simple Spans	Continuous Spans
Bridge slabs with main reinforcement parallel to traffic	$1.2(S+10)/30$	$(S+10)/30 \geq 0.542$
T-Girders	$0.070\ S$	$0.065\ S$
Box-Girders	$0.060\ S$	$0.055\ S$
Pedestrian Structure Girders	$0.033\ S$	$0.033\ S$

[a] When variable depth members are used, values may be adjusted to account for change in relative stiffness of positive and negative moment sections.

S = span length as defined in Article 8.8, in feet.

8.9.3 Superstructure Deflection Limitations

When making deflection computations, the following criteria are recommended.

8.9.3.1 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load plus impact shall not exceed 1/800 of the span, except on bridges in urban areas used in part by pedestrians, whereon the ratio preferably shall not exceed 1/1000.

8.9.3.2 The deflection of cantilever arms due to service live load plus impact preferably should be limited to 1/300 of the cantilever arm except for the case including pedestrian use, where the ratio preferably should be 1/375.

8.10 COMPRESSION FLANGE WIDTH

8.10.1 T-Girder

8.10.1.1 The total width of slab effective as a T-girder flange shall not exceed one-fourth of the span length of the girder. The effective flange width overhanging on each side of the web shall not exceed six times the thickness of the slab or one-half the clear distance to the next web.

8.10.1.2 For girders having a slab on one side only, the effective overhanging flange width shall not exceed 1/12 of the span length of the girder, six times the thickness of the slab, or one-half the clear distance to the next web.

8.10.1.3 Isolated T-girders in which the T-shape is used to provide a flange for additional compression area shall have a flange thickness not less than one-half the width of the girder web and an effective flange width not more than four times the width of the girder web.

8.10.1.4 For integral bent caps, the effective flange width overhanging each side of the bent cap web shall not exceed six times the least slab thickness, or 1/10 the span length of the bent cap. For cantilevered bent caps, the span length shall be taken as two times the length of the cantilever span.

8.10.2 Box Girders

8.10.2.1 The entire slab width shall be assumed effective for compression.

8.10.2.2 For integral bent caps, see Article 8.10.1.4.

8.11 SLABS AND WEB THICKNESS

8.11.1 The thickness of deck slabs shall be designed in accordance with Article 3.24.3 but shall not be less than that specified in Article 8.9.

8.11.2 The thickness of the bottom slab of a box girder shall be not less than 1/16 of the clear span between girder webs or 5 $^1/_2$ inches, except that the thickness need not be greater than the top slab unless required by design.

8.11.3 When required by design, changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

8.12 DIAPHRAGMS

8.12.1 Diaphragms shall be used at the ends of T-girder and box girder spans unless other means are provided to resist lateral forces and to maintain section geometry. Diaphragms may be omitted where tests or structural analysis show adequate strength.

8.12.2 In T-girder construction, one intermediate diaphragm is recommended at the point of maximum positive moment for spans in excess of 40 feet.

8.12.3 Straight box girder bridges and curved box girder bridges with an inside radius of 800 feet or greater do not require intermediate diaphragms. For curved box girder bridges having an inside radius less than 800 feet, intermediate diaphragms are required unless shown otherwise by tests or structural analysis. For such curved box girders, the maximum diaphragm spacing shall be 40 feet for radius 400 feet or less and 80 feet for radius between 400 feet and 800 feet.

8.13 COMPUTATION OF DEFLECTIONS

8.13.1 Computed deflections shall be based on the cross-sectional properties of the entire superstructure section excluding railings, curbs, sidewalks, or any element not placed monolithically with the superstructure section before falsework removal.

8.13.2 Live load deflection may be based on the assumption that the superstructure flexural members act together and have equal deflection. The live loading shall consist of all traffic lanes fully loaded, with reduction in load intensity allowed as specified in Article 3.12. The live loading shall be considered uniformly distributed to all longitudinal flexural members.

8.13.3 Deflections that occur immediately on application of load shall be computed by the usual methods or formulas for elastic deflections. Unless stiffness values are obtained by a more comprehensive analysis, immediate deflections shall be computed taking the modulus of elasticity for concrete as specified in Article 8.7 for normal weight or lightweight concrete and taking the moment of inertia as either I_g or I_e as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (8\text{-}1)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (8\text{-}2)$$

and f_r = modulus of rupture of concrete specified in Article 8.15.2.1.1.

For continuous spans, the effective moments of inertia may be taken as the average of the values obtained from Equation (8-1) for the critical positive and negative moment sections. For prismatic members, effective moment of inertia may be taken as the value obtained from Equation (8-1) at midspan for simple or continuous spans, and at support for cantilever spans.

8.13.4 Unless values are obtained by a more comprehensive analysis, the long-time deflection for both normal weight and lightweight concrete flexural members shall be the immediate deflection caused by the sustained load considered, computed in accordance with Article 8.13.3, multiplied by one of the following factors:

(a) Where the immediate deflection has been based on I_g, the multiplication factor for the long-time deflection shall be taken as 4.

(b) Where the immediate deflection has been based on I_e, the multiplication factor for the long-time deflection shall be taken as 3 - 1.2$(A'_s/A_s) \ge 1.6$.

Part C
Design

8.14 GENERAL

8.14.1 Design Methods

8.14.1.1 The design of reinforced concrete members shall be made either with reference to service loads and allowable stresses as provided in SERVICE LOAD DESIGN or, alternatively, with reference to load factors and strengths as provided in STRENGTH DESIGN.

8.14.1.2 Except as provided herein, all reinforced concrete structures or members shall be designed by STRENGTH DESIGN. Current standard designs by other methods shall be utilized until revised.

8.14.1.3 Structures designed exclusively for carrying railroad traffic and transversely reinforced deck slabs of highway structures shall be designed by SERVICE LOAD DESIGN. AREA Specifications may be required for substructure design of railroad structures.

8.14.1.4 SERVICE LOAD DESIGN may be used at any section where the allowable stress determined by Article 8.16.8.4 is less than 24,000 psi if the amount of reinforcement provided is sufficient to satisfy other requirements for STRENGTH DESIGN.

8.14.1.5 All applicable provisions of this specification shall apply to both methods of design.

8.14.1.6 The strength and serviceability requirements of STRENGTH DESIGN may be assumed to be satisfied for design by SERVICE LOAD DESIGN if the service load stresses are limited to the values given in Article 8.15.2.

8.14.2 Composite Flexural Members

8.14.2.1 Composite flexural members consist of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to superimposed loads as a unit. When considered in design, shoring shall not be removed until the supported elements have developed the design properties required to support all loads and limit deflections and cracking.

8.14.2.2 The entire composite member or portions thereof may be used in resisting the shear and moment. The individual elements shall be investigated for all critical stages of loading and shall be designed to support all loads introduced prior to the full development of the design strength of the composite member. Reinforcement shall be provided as necessary to prevent separation of the individual elements.

8.14.2.3 If the specified strength, unit weight, or other properties of the various elements are different, the properties of the individual elements, or the most critical values, shall be used in design.

8.14.2.4 In calculating the flexural strength of a composite member by strength design, no distinction shall be made between shored and unshored members.

8.14.2.5 When an entire member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Article 8.15.5 or Article 8.16.6 as for a monolithically cast member of the same cross-sectional shape.

8.14.2.6 Shear reinforcement shall be fully anchored into the interconnected elements in accordance with Article 8.27. Extended and anchored shear reinforcement may be included as ties for horizontal shear.

8.14.2.7 The design shall provide for full transfer of horizontal shear forces at contact surfaces of interconnected elements. Design for horizontal shear shall be in accordance with the requirements of Article 8.15.5.5 or Article 8.16.6.5.

8.14.3 Concrete Arches

8.14.3.1 The combined flexure and axial load strength of an arch ring shall be in accordance with the provisions of Articles 8.16.4 and 8.16.5. Slenderness effects in the vertical plane of an arch ring, other than tied arches with suspended roadway, may be evaluated by the approximate procedure of Article 8.16.5.2 with the unsupported length, l_u, taken as one-half the length of the arch ring, and the radius of gyration, r, taken about an axis perpendicular to the plane of the arch at the quarter point of the arch span. Values of the effective length factor, k, given in Table 8.14.3 may be used. In Equation (8-41), C_m shall be taken as 1.0 and ϕ shall be taken as 0.85.

TABLE 8.14.3 Effective Length Factors, *k*

Rise-to-Span Ratio	3-Hinged Arch	2-Hinged Arch	Fixed Arch
0.1 - 0.2	1.16	1.04	0.70
0.2 - 0.3	1.13	1.10	0.70
0.3 - 0.4	1.16	1.16	0.72

8.14.3.2 Slenderness effects between points of lateral support and between suspenders in the vertical plane of a tied arch with suspended roadway, shall be evaluated by a rational analysis taking into account the requirements of Article 8.16.5.1.1.

8.14.3.3 The shape of arch rings shall conform, as nearly as is practicable, to the equilibrium polygon for full dead load.

8.14.3.4 In arch ribs and barrels, the longitudinal reinforcement shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.01, divided equally between the intrados and the extrados. The longitudinal reinforcement shall be enclosed by lateral ties in accordance with Article 8.18.2. In arch barrels, upper and lower levels of transverse reinforcement shall be provided that are designed for transverse bending due to loads from columns and spandrel walls and for shrinkage and temperature stresses.

8.14.3.5 If transverse expansion joints are not provided in the deck slab, the effects of the combined action of the arch rib, columns and deck slab shall be considered. Expansion joints shall be provided in spandrel walls.

8.14.3.6 Walls exceeding 8 feet in height on filled spandrel arches shall be laterally supported by transverse diaphragms or counterforts with a slope greater than 45 degrees with the vertical to reduce transverse stresses in the arch barrel. The top of the arch barrel and interior faces of the spandrel walls shall be waterproofed and a drainage system provided for the fill.

8.15 SERVICE LOAD DESIGN METHOD (ALLOWABLE STRESS DESIGN)

8.15.1 General Requirements

8.15.1.1 Service load stresses shall not exceed the values given in Article 8.15.2.

8.15.1.2 Development and splices of reinforcement shall be as required in Articles 8.24 through 8.32.

8.15.2 Allowable Stresses

8.15.2.1 Concrete

Stresses in concrete shall not exceed the following:

8.15.2.1.1 Flexure

Extreme fiber stress in compression, f_c ..... $0.40\, f_c'$

Extreme fiber stress in compression for transversely reinforced deck slabs, f_c .... 1200 psi +

Extreme fiber stress in tension for plain concrete, f_t .... $0.21 f_r$

Modulus of rupture, f_r, from tests, or, if data are not available:

Normal weight concrete .... $7.5\sqrt{f_c'}$

“Sand-lightweight” concrete .... $6.3\sqrt{f_c'}$

“All-lightweight” concrete .... $5.5\sqrt{f_c'}$

8.15.2.1.2 Shear

For detailed summary of allowable shear stress, v_c, see Article 8.15.5.2.

8.15.2.1.3 Bearing Stress

The bearing stress, f_b, on loaded area shall not exceed $0.30\, f_c'$.

When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be increased by $\sqrt{A_2/A_1}$, but not by more than 2.

When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of the right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.

8.15.2.2 Reinforcement

The tensile stress in the reinforcements, f_s, shall not exceed the following:

Grade 40 reinforcement .... 20,000 psi

Grade 60 reinforcement .... 24,000 psi

\+ Grade 60 reinforcement for transversely
\+ reinforced deck slabs .... 20,000 psi

In straight reinforcement, the range between the maximum tensile stress and the minimum stress caused by live load plus impact shall not exceed the value given in Article 8.16.8.3. Bends in primary reinforcement shall be avoided in regions of high stress range.

8.15.3 Flexure

8.15.3.1 For the investigation of stresses at service loads, the straight-line theory of stress and strain in flexure shall be used with the following assumptions:

8.15.3.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis, except that for deep flexure members with overall depth to span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered.

8.15.3.3 In reinforced concrete members, concrete resists no tension.

8.15.3.4 The modular ratio, $n = E_s/E_c$ may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, the value of n for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength.

8.15.3.5 In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform the compression reinforcement for stress computations. The compressive stress in such reinforcements shall not be greater than the allowable tensile stress.

8.15.4 Compression Members

The combined flexural and axial load capacity of compression members shall be taken as 35 percent of that computed in accordance with the provisions of Article 8.16.4. Slenderness effects shall be included according to the requirements of Article 8.16.5. The term P_u in Equation (8-41) shall be replaced by 2.5 times the design axial load. In using the provisions of Articles 8.16.4 and 8.16.5, ϕ shall be taken as 1.0.

8.15.5 Shear

8.15.5.1 Shear Stress

8.15.5.1.1 Design shear stress, v, shall be computed by:

$$v = \frac{V}{b_w d} \qquad (8\text{-}3)$$

where V is design shear force at section considered, b_w is the width of web, and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable, effects of torsion[4] shall be included.

[4] The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete" - ACI 318 may be used.

8.15.5.1.2 For a circular section, b_w shall be the diameter and *d* need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member.

8.15.5.1.3 For tapered webs, b_w shall be the average width or 1.2 times the minimum width, whichever is smaller.

8.15.5.1.4 When the reaction, in the direction of the applied shear, introduces compression into the end regions of a member, sections located less than a distance *d* from face of support may be designed for the same shear, *V*, as that computed at a distance *d*. An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case, sections closer than *d* to the support shall be designed for *V* at distance *d* plus the major concentrated loads.

8.15.5.2 Shear Stress Carried by Concrete

8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, the allowable shear stress carried by the concrete, v_c, may be taken as $0.95\sqrt{f_c'}$. A more detailed calculation of the allowable shear stress can be made using:

$$v_c = 0.9\sqrt{f_c'} + 1{,}100\,\rho_w\left(\frac{Vd}{M}\right) \le 1.6\sqrt{f_c'} \qquad (8\text{-}4)$$

Note:

(a) *M* is the design moment occurring simultaneously with *V* at the section being considered.

(b) The quantity $\frac{Vd}{M}$ shall not be taken greater than 1.0.

8.15.5.2.2 Shear in Compression Members

For members subject to axial compression, the allowable shear stress carried by the concrete, v_c, may be taken as $0.95\sqrt{f_c'}$. A more detailed calculation can be made using:

$$v_c = 0.9\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f_c'} \qquad (8\text{-}5)$$

The quantity $\frac{N}{A_g}$ shall be expressed in pounds per square inch.

8.15.5.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = 0.9\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f_c'} \qquad (8\text{-}6)$$

Note:

(a) *N* is negative for tension.

(b) The quantity $\frac{N}{A_g}$ shall be expressed in pounds per square inch.

8.15.5.2.4 Shear in Lightweight Concrete

The provisions for shear stress, v_c, carried by the concrete apply to normal weight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified, the shear stress v_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f_c'}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f_c'}$.

(b) When f_{ct} is not specified, the shear stress, v_c, shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

8.15.5.3 Shear Stress Carried by Shear Reinforcement

8.15.5.3.1 Where design shear stress *v* exceeds shear stress carried by concrete v_c, shear reinforcement shall be provided in accordance with this Article. Shear reinforcement shall also conform to the general requirements of Article 8.19.

8.15.5.3.2 When shear reinforcement perpendicular to the axis of the member is used:

$$A_v = \frac{(v - v_c) b_w s}{f_s} \tag{8-7}$$

8.15.5.3.3 When inclined stirrups are used:

$$A_v = \frac{(v - v_c) b_w s}{f_s(\sin\alpha + \cos\alpha)} \tag{8-8}$$

8.15.5.3.4 When shear reinforcement consists of a single bar or single group of parallel bars all bent up at the same distance from the support:

$$A_v = \frac{(v - v_c) b_w s}{f_s \sin\alpha} \tag{8-9}$$

where (v-v_c) shall not exceed $1.5\sqrt{f'_c}$.

8.15.5.3.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area shall be computed by Equation (8-8).

8.15.5.3.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

8.15.5.3.7 Where more than one type of shear reinforcement is used to reinforce the same portion of the member, the required area shall be computed as the sum of the values computed for the various types separately. In such computations, v_c shall be included only once.

8.15.5.3.8 When (v - v_c) exceeds $2\sqrt{f'_c}$, the maximum spacings given in Article 8.19 shall be reduced by one-half.

8.15.5.3.9 The value of (v - v_c) shall not exceed $4\sqrt{f'_c}$.

8.15.5.3.10 When flexural reinforcement located within the width of a member used to compute the shear strength is terminated in a tension zone, shear reinforcement shall be provided in accordance with Article 8.24.1.4.

8.15.5.4 Shear Friction

8.15.5.4.1 Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

8.15.5.4.2 A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vf} across the shear plane may be designed using either Article 8.15.5.4.3 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of paragraph 8.15.5.4.4 through 8.15.5.4.8 shall apply for all calculations of shear transfer strength.

8.15.5.4.3 Shear-friction design method

(a) When shear-friction reinforcement is perpendicular to shear plane, area of shear-friction reinforcement A_{vf} shall be computed by

$$A_{vf} = \frac{V}{f_s \mu} \tag{8-10}$$

where μ is the coefficient of friction in accordance with Article 8.15.5.4.3(c).

(b) When shear-friction reinforcement is inclined to shear plane such that the shear force produces tension in shear-friction reinforcement, area of shear-friction A_{vf} shall be computed by

$$A_{vf} = \frac{V}{f_s(\mu \sin\alpha_f + \cos\alpha_f)} \tag{8-11}$$

where a_f is angle between shear-friction reinforcement and shear plane.

(c) Coefficient of friction μ in Equation (8-10) and Equation (8-11) shall be:

concrete placed monolithically 1.4λ

concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.15.5.4.7 .... 1.0λ

concrete placed against hardened concrete not intentionally roughened 0.6λ

concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.15.5.4.8) 0.7λ

where λ= 1.0 for normal weight concrete; 0.85 for "sand-lightweight" concrete; and 0.75 for "all-lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

8.15.5.4.4 Shear stress v shall not exceed 0.09 f_c' nor 360 psi.

8.15.5.4.5 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_s$, when calculating required A_{vf}.

8.15.5.4.6 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

8.15.5.4.7 For the purpose of Art. 8.15.5.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 1/4 inch.

8.15.5.4.8 When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

8.15.5.5 Horizontal Shear Design for Composite Concrete Flexural Members

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

8.15.5.5.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Paragraph 8.15.5.5.3 or 8.15.5.5.4 or any other shear transfer design method that results in prediction of strength in substantial agreement with result of comprehensive tests.

8.15.5.5.3 Design horizontal shear stress v_{dh} at any cross section may be computed by

$$v_{dh} = \frac{V}{b_v d} \tag{8-11A}$$

where V is design shear force at section considered and d is for entire composite section. Horizontal shear v_{dh} shall not exceed permissible horizontal shear v_h in accordance with the following:

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear stress v_h shall not exceed 36 psi.

(b) When minimum ties are provided in accordance with Paragraph 8.15.5.5.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 36 psi.

(c) When minimum ties are provided in accordance with Paragraph 8.15.5.5.5, and contact surface is clean, free of laitance, and intentionally roughened to a full magnitude of approximately 1/4 inch, shear stress v_h shall not exceed 160 psi.

(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by 8.15.5.5.5, permissible v_h may be increased by $72f_y/40{,}000$ psi.

8.15.5.5.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. Horizontal shear shall not exceed the permissible horizontal shear stress v_h in accordance with Paragraph 8.15.5.5.3.

8.15.5.5.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50\, b_v s / f_y$, and tie spacing s shall not exceed four times the least web width of support element, nor 24 inches.

(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed). All ties shall be adequately anchored into interconnected elements by embedment or hooks.

(c) All beam shear reinforcement shall extend into cast-in-place deck slabs. Extended shear reinforcement may be used in satisfying the minimum tie reinforcement.

8.15.5.6 Special Provisions for Slabs and Footings

8.15.5.6.1 Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions:

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Article 8.15.5.1 through 8.15.5.3, except at footings supported on piles, the shear on the critical section shall be determined in accordance with Article 4.4.7.2.

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but not closer than $d/2$ to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Article 8.15.5.6.2 and 8.15.5.6.3.

8.15.5.6.2 Design shear stress, v shall be computed by:

$$v = \frac{V}{b_o d} \qquad \text{(8-12)}$$

where V and b_o shall be taken at the critical section defined in 8.15.5.6.1(b).

8.15.5.6.3 Design shear stress, v, shall not exceed v_c given by Equation (8-13) unless shear reinforcement is provided in accordance with Article 8.15.5.6.4.

$$v_c = \left(0.8 + \frac{2}{\beta_c}\right)\sqrt{f'_c} \le 1.8\sqrt{f'_c} \qquad \text{(8-13)}$$

β_c is the ratio of long side to short side of concentrated load or reaction area.

8.15.5.6.4 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear stresses computed by Equation (8-12) shall be investigated at the critical section defined in 8.15.5.6.1(b) and at successive sections more distant from the support.

(b) Shear stress v_c at any section shall not exceed $0.9\sqrt{f'_c}$ and v shall not exceed $3\sqrt{f'_c}$.

(c) Where v exceeds $0.9\sqrt{f'_c}$, shear reinforcement shall be provided in accordance with Article 8.15.5.3.

8.15.5.7 *Deleted*

8.15.5.8 Special Provisions for Brackets and Corbels[5]

8.15.5.8.1 Provisions of Paragraph 8.15.5.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_c not larger than V. Distance d shall be measured at face of support.

8.15.5.8.2 Depth at outside edge of bearing area shall not be less than $0.5d$.

8.15.5.8.3 Section at face of support shall be designed to resist simultaneously a shear V, a moment $\left[Va_v + N_c(h-d)\right]$ and a horizontal tensile force N_c. Distance h shall be measured at the face of support.

5 These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-95" contains an example design of beam ledges - Part 17, Example 17-3.

(a) Design of shear-friction reinforcement A_{vf} to resist shear V shall be in accordance with Article 8.15.5.4. For normal weight concrete, shear stress v shall not exceed $0.9\sqrt{f'_c}$ nor 360 psi. For "all-lightweight" or "sand-lightweight" concrete, shear stress v shall not exceed $(0.09 - 0.03 a_v / d) f'_c$ nor $(360 - 126\, a_v / d)$ psi.

(b) Reinforcement A_f to resist moment $[V a_v + N_c (h - d)]$ shall be computed in accordance with Articles 8.15.2 and 8.15.3.

(c) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c / f_s$. Tensile force N_c shall not be taken less than $0.2V$ unless special provisions are made to avoid tensile forces.

(d) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $((2A_{vf} / 3) + A_n)$.

8.15.5.8.4 Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

8.15.5.8.5 Ratio $\rho = A_s / bd$ shall not be taken less than $0.04(f'_c / f_y)$.

8.15.5.8.6 At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;

(b) bending primary tension bars A_s back to form a horizontal loop, or

(c) some other means of positive anchorage.

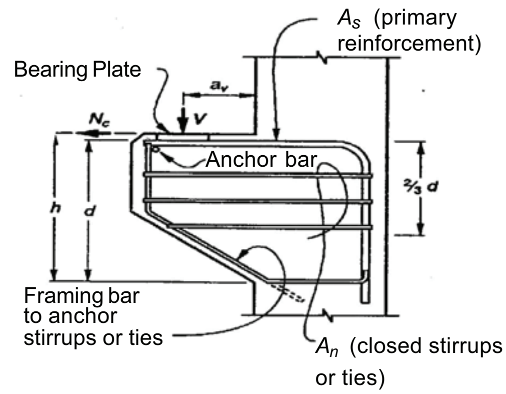


Figure 8.15.5.8

8.15.5.8.7 Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

8.16 STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN)

8.16.1 Strength Requirements

8.16.1.1 Required Strength

The required strength of a section is the strength necessary to resist the factored loads and forces applied to the structure in the combinations stipulated in Article 3.22. All sections of structures and structural members shall have design strengths at least equal to the required strength.

8.16.1.2 Design Strength

8.16.1.2.1 The design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be the nominal strength calculated in accordance with the requirements and assumptions of the strength design method, multiplied by a strength reduction factor ϕ[6].

6 The coefficient ϕ provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in understrength.

8.16.1.2.2 The strength reduction factors, ϕ, shall be as follows:

(a) Flexure .... $\phi = 0.90$
(except Group VII footings .... $\phi = 1.0$
and Group VII columns[7] .... $\phi = 1.2$)

(b) Shear .... $\phi = 0.85$

(c) Axial compression with
spirals .... $\phi = 0.75$
ties .... $\phi = 0.70$
(except Group VII columns[7] .... $\phi = 1.0$)

(d) Bearing on concrete .... $\phi = 0.7$

The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the design axial load strength, ϕP_n, decreases from $0.10 f'_c A_g$ or ϕP_b, whichever is smaller, to zero.

8.16.1.2.3 The development and splice lengths of reinforcement specified in Articles 8.24 through 8.32 do not require a strength reduction factor.

8.16.2 Design Assumptions

8.16.2.1 The strength design of members for flexure and axial loads shall be based on the assumptions given in this article, and on the satisfaction of the applicable conditions of equilibrium of internal stresses and compatibility of strains.

8.16.2.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis.

8.16.2.3 The maximum usable strain at the extreme concrete compression fiber is equal to 0.003.

8.16.2.4 The stress in reinforcement below its specified yield strength, f_y, shall be E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement shall be considered independent of strain and equal to f_y.

[7] For seismic loads (Group VII), the use of increased coefficient ϕ for columns recognizes the overstrength capacity of well confined compression members with axial loads below P_b. For axial loads above P_b, do not use the increased coefficient ϕ without a more detailed analysis to justify the use of higher coefficient ϕ.

8.16.2.5 The tensile strength of the concrete is neglected in flexural calculations.

8.16.2.6 The concrete compressible stress/strain distribution may be assumed to be a rectangle, trapezoid, parabola, or any other shape that results in prediction of strength in substantial agreement with the results of comprehensive tests.

8.16.2.7 A compressive stress/strain distribution, which assumes a concrete stress of $0.85 f'_c$ uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis at a distance $\alpha = \beta_1 c$ from the fiber of maximum compressive strain, may be considered to satisfy the requirements of Article 8.16.2.6. The distance c from the fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis. The factor β_1 shall be taken as 0.85 for concrete strengths, f'_c, up to and including 4,000 psi. For strengths above 4,000 psi, β_1 shall be reduced continuously at a rate of 0.05 for each 1,000 psi of strength in excess of 4,000 psi but β_1 shall not be taken less than 0.65.

8.16.3 Flexure

8.16.3.1 Maximum Reinforcement of Flexural Members

8.16.3.1.1 The ratio of reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section. The portion of ρ_b balanced by compression reinforcement need not be reduced by the 0.75 factor.

8.16.3.1.2 Balanced strain conditions exist at a cross section when the 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.

8.16.3.2 Rectangular Sections with Tension Reinforcement Only

8.16.3.2.1 The design moment strength, ϕM_n, may be computed by:

$$\phi M_n = \phi \left[A_s f_y d \left(1 - 0.6 \frac{\rho f_y}{f'_c} \right) \right] \qquad (8\text{-}15)$$

$$= \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \tag{8-16}$$

where

$$a = \frac{A_s f_y}{0.85 f'_c b} \tag{8-17}$$

8.16.3.2.2 The balanced reinforcement ratio, ρ_b, is given by:

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) \tag{8-18}$$

8.16.3.3 Flanged Sections with Tension Reinforcement Only

8.16.3.3.1 When the compression flange thickness is equal to or greater than the depth of the equivalent rectangular stress block, a, the design moment strength, ϕM_n, may be computed by Equations (8-15) and (8-16).

8.16.3.3.2 When the compression flange thickness is less than a, the design moment strength may be computed by:

$$\phi M_n = \phi\left[\left(A_s - A_{sf}\right)f_y\left(d - \frac{a}{2}\right) + A_{sf} f_y\left(d - 0.5h_f\right)\right] \tag{8-19}$$

where

$$A_{sf} = \frac{0.85 f'_c\left(b - b_w\right)h_f}{f_y} \tag{8-20}$$

$$a = \frac{\left(A_s - A_{sf}\right)f_y}{0.85 f'_c b_w} \tag{8-21}$$

8.16.3.3.3 The balanced reinforcement ratio, ρ_b, is given by:

$$\rho_b = \left(\frac{b_w}{b}\right)\left[\left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87{,}000}{87{,}000 + f_y}\right) + \rho_f\right] \tag{8-22}$$

where

$$\rho_f = \frac{A_{sf}}{b_w d} \tag{8-23}$$

8.16.3.3.4 For T-girder and box-girder construction, the width of the compression face, b, shall be equal to the effective slab width as defined in Article 8.10.

8.16.3.4 Rectangular Sections with Compression Reinforcement

8.16.3.4.1 The design moment, ϕM_n, may be computed as follows:

If

$$\left(\frac{A_s - A'_s}{bd}\right) \geq 0.85\beta_1\left(\frac{f'_c d'}{f_y d}\right)\left(\frac{87{,}000}{87{,}000 - f_y}\right) \tag{8-24}$$

then

$$\phi M_n = \phi\left[\left(A_s - A'_s\right)f_y\left(d - \frac{a}{2}\right) + A'_s f_y\left(d - d'\right)\right] \tag{8-25}$$

where

$$a = \frac{\left(A_s - A'_s\right)f_y}{0.85 f'_c b} \tag{8-26}$$

8.16.3.4.2 When the value of $\left(A_s - A'_s\right)/bd$ is less than the value required by Equation (8-24), so that the stress in the compression reinforcement is less than the yield strength, f_y, or when effects of compression reinforcement are neglected, the design moment strength may be computed by the equations in Article 8.16.3.2. Alternatively, a general analysis may be made based on stress and strain compatibility using the assumptions given in Article 8.16.2.

8.16.3.4.3 The balanced reinforcement ratio ρ_b for rectangular sections with compression reinforcement is given by:

$$\rho_b = \left[\frac{0.85\,\beta_1 f_c'}{f_y} \left(\frac{87{,}000}{87{,}000 + f_y} \right) \right] + \rho' \left(\frac{f_s'}{f_y} \right) \qquad \text{(8-27)}$$

where

$$f_s' = 87{,}000 \left[1 - \left(\frac{d'}{d} \right) \left(\frac{87{,}000 + f_y}{87{,}000} \right) \right] \le f_y \qquad \text{(8-28)}$$

8.16.3.5 Flanged Sections with Compression Reinforcement

8.16.3.5.1 When the compression flange thickness is less than the value of '*a*' determined by Article 8.16.3.4.1, the design moment strength may be computed by:

$$\phi M_n = \phi \left[\begin{array}{l} \left(A_s - A_{sf} - A_s'\right) f_y \left(d - \dfrac{a}{2} \right) + \\ A_{sf} f_y \left(d - 0.5 h_f\right) + A_s' f_y \left(d - d'\right) \end{array} \right] \qquad \text{(8-28A)}$$

where

$$a = \frac{\left(A_s - A_{sf} - A_s'\right) f_y}{0.85\, f_c' b_w} \succ h_f \qquad \text{(8-28B)}$$

and the following condition shall be satisfied:

$$\frac{\left(A_s - A_{sf} - A_s'\right)}{b_w d} \ge \left(\frac{0.85 \beta_1 f_c' d'}{f_y d} \right) \left(\frac{87{,}000}{87{,}000 - f_y} \right) \qquad \text{(8-28C)}$$

8.16.3.5.2 When the value of $\left(A_s - A_{sf} - A_s'\right)/ b_w d$ is less than the limit given by the expression in Article 8.16.3.5.1, the moment strength may be computed by the equations in Article 8.16.3.3.2, or a general analysis based on stress and strain compatibility using the assumptions given in Article 8.16.2 may be performed.

8.16.3.5.3 The balanced reinforcement ratio, ρ_b, for flanged sections with compression reinforcement is given by:

$$\rho_b = \frac{b_w}{b} \left[\left(\frac{0.85\,\beta_1 f_c'}{f_y} \right) \left(\frac{87{,}000}{87{,}000 + f_y} \right) + \rho_f \right] + \rho' \left(\frac{f_s'}{f_y} \right) \qquad \text{(8-28D)}$$

where ρ_f is as defined in Article 8.16.3.3.3 and f_s' is as defined in Article 8.16.3.4.3.

8.16.3.6 Other Cross Sections

For other cross sections the design moment strength, ϕM_n, shall be computed by a general analysis based on stress and strain compatibility using assumptions given in Article 8.16.2. The requirements of Article 8.16.3.1 shall also be satisfied.

8.16.4 Compression Members

8.16.4.1 General Requirements

8.16.4.1.1 The design of members subject to axial load or to combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 8.16.2. Slenderness effects shall be included according to the requirements of Article 8.16.5.

8.16.4.1.2 Members subject to compressive axial load combined with bending shall be designed for the maximum moment that can accompany the axial load. The factored axial load, P_u, at a given eccentricity shall not exceed the design axial strength $\phi P_{n(\max)}$ where

(a) For members with spiral reinforcement conforming to Article 8.18.2.2.

$$P_{n(\max)} = 0.85\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(8-29)}$$

$$\phi = 0.75$$

(b) For members with tie reinforcement conforming to Article 8.18.2.3

$$P_{n(\max)} = 0.80\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(8-30)}$$

$$\phi = 0.70$$

The maximum factored moment, M_u, shall be magnified for slenderness effects in accordance with Article 8.16.5.

8.16.4.2 Compression Member Strengths

The following provisions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

8.16.4.2.1 Pure Compression

The design axial load strength at zero eccentricity, ϕP_o, may be computed by:

$$\phi P_o = \phi\left[0.85 f'_c\left(A_g - A_{st}\right) + A_{st} f_y\right] \quad (8\text{-}31)$$

For design, pure compressive strength is a hypothetical condition since Article 8.16.4.1.2 limits the axial load strength of compression members to 85 and 80 percent of the axial load at zero eccentricity.

8.16.4.2.2 Pure Flexure

The assumptions given in Article 8.16.2 or the applicable equations for flexure given in Article 8.16.3 may be used to compute the design moment strength, ϕM_n, in pure flexure.

8.16.4.2.3 Balanced Strain Conditions

Balanced strain conditions for a cross section are defined in Article 8.16.3.1.2. For a rectangular section with reinforcement in one face, or located in two faces at approximately the same distance from the axis of bending, the balanced load strength, ϕP_b, and balanced moment strength, ϕM_b, may be computed by:

$$\phi P_b = \phi\left[0.85 f'_c b a_b + A'_s f'_s - A_s f_y\right] \quad (8\text{-}32)$$

and

$$\phi M_b = \phi\begin{bmatrix} 0.85 f'_c b a_b\left(d - d'' - \dfrac{a_b}{2}\right) + \\ A'_s f'_s\left(d - d' - d''\right) + A_s f_y d'' \end{bmatrix} \quad (8\text{-}33)$$

where,

$$a_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\beta_1 d \quad (8\text{-}34)$$

and

$$f'_s = 87{,}000\left[1 - \left(\frac{d'}{d}\right)\left(\frac{87{,}000 + f_y}{87{,}000}\right)\right] \le f_y \quad (8\text{-}35)$$

8.16.4.2.4 Combined Flexure and Axial Load

The strength of a cross section is controlled by tension when the nominal axial load strength, P_n, is less than the balanced load strength, P_b, and is controlled by compression when P_n is greater than P_b.

The nominal values of axial load strength, P_n, and moment strength, M_n, must be multiplied by the strength reduction factor, ϕ, for axial compression as given in Article 8.16.1.2.

8.16.4.3 Biaxial Loading

In lieu of a general section analysis based on stress and strain compatibility, the design strength of non-circular members subjected to biaxial bending may be computed by the following approximate expressions:

$$\frac{1}{P_{nxy}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o} \quad (8\text{-}36)$$

when the factored axial load,

$$P_u \ge 0.1 f'_c A_g \quad (8\text{-}37)$$

or

$$\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \le 1 \quad (8\text{-}38)$$

when the factored axial load,

$$P_u \prec 0.1 f'_c A_g \quad (8\text{-}39)$$

8.16.4.4 Hollow Rectangular Compression Members

8.16.4.4.1 The wall slenderness ratio of a hollow rectangular cross section, X_u/t, is defined in Figure 8.16.4.4.1. Wall slenderness ratios greater than 35.0 are not permitted, unless specific analytical and experimental evidence is provided justifying such values.

8.16.4.4.2 The equivalent rectangular stress block method shall not be employed in the design of hollow rectangular compression members with wall thickness ratio of 15 or greater.

8.16.4.4.3 If the wall slenderness ratio is less than 15, then the maximum usable strain at the extreme compression fiber is equal to 0.003. If the wall slenderness is 15 or greater, then the maximum usable strain at the extreme concrete compression fiber is equal to the computed local buckling strain of the widest flange of the cross section, or 0.003, whichever is less.

8.16.4.4.4 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.

8.16.4.4.5 In lieu of the provisions of Articles 8.16.4.4.2, 8.16.4.4.3 and 8.16.4.4.4, the following approximate method may be used to account for the strength reduction due to wall slenderness. The maximum usable strain at the extreme concrete compression fiber shall be taken as 0.003 for all wall slenderness ratios up to and including 35.0. A strength reduction factor ϕ_w shall be applied in addition to the usual strength reduction factor, ϕ, in Article 8.16.1.2. The strength reduction factor ϕ_w shall be taken as 1.0 for all wall slenderness ratios up to and including 15.0. For wall slenderness ratios greater than 15.0 and less than or equal to 25.0, the strength reduction factor ϕ_w shall be reduced continuously at a rate of 0.025 for every unit increase in wall slenderness ratio above 15.0. For wall slenderness ratios greater than 25.0 and less than or equal to 35.0, the strength reduction factor ϕ_w shall be taken as 0.75.

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

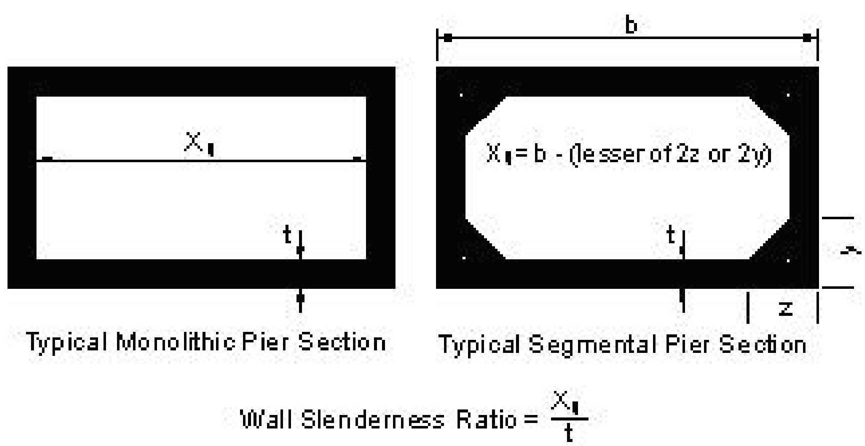


FIGURE 8.16.4.4.1 Definition of Wall Slenderness Ratio

8.16.4.5 Probable Plastic Moment

8.16.4.5.1 The probable plastic moment is defined as the maximum moment which can be expected to actually develop in a well confined column section at yield.

8.16.4.5.2 For well-confined sections with axial loads below P_b (Article 8.1.2) the probable plastic moment may be assumed to be 1.3 times the nominal moment. For loads above P_b, a more detailed analysis shall be performed.

8.16.4.6 Special Provisions for Column and Pier Wall Hinges

8.16.4.6.1 The design shear force, V_u, and the associated axial force, P_u, shall be adequately transferred from superstructure to support, from support to foundation, or at intermediate locations in the support considered hinged.

8.16.4.6.2 The design compressive axial load strength shall be computed in accordance with Article 8.16.4.2.1 for all Group loads except Group VII. For Group VII loads, the design compressive axial load strength shall be computed by:

$$\phi P_o = \phi 0.85 f_c'\left(A_g - A_{st}\right) + A_{st} f_y \qquad \text{(8-39A)}$$

where,

$\phi = 0.90$

P_u shall not exceed ϕP_o.

8.16.4.6.3 The design tensile axial load strength may be computed by:

$$\phi P_n = \phi A_{st} f_y \qquad \text{(8-39B)}$$

where $\phi = 0.90$ for all loads except Group VII, and $\phi = 1.0$ for Group VII loads.

8.16.4.6.4 The design shear strength shall be in accordance with Article 8.16.6.4. The area of longitudinal hinge reinforcement, A_{st}, in excess of, A_s, may be used for the required area, A_{vf}.

8.16.4.6.5 In hinges, the longitudinal reinforcement shall be placed close to the center of the core to minimize moment strength. The longitudinal hinge reinforcement shall be developed on both sides of the hinge interface.

8.16.5 Slenderness Effects in Compression Members

8.16.5.1 General Requirements

8.16.5.1.1 The design of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall include the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflections on the moments and forces, and the effect of the duration of the loads.

8.16.5.1.2 In lieu of the procedure described in Article 8.16.5.1.1, slenderness effects of compression members may be evaluated in accordance with the approximate procedure in Article 8.16.5.2.

8.16.5.1.3 In lieu of the procedure described in Article 8.16.5.1.1, slenderness effects in compression members shall be neglected when proportioning them for the Group VII load combination.

8.16.5.2 Approximate Evaluation of Slenderness Effects

8.16.5.2.1 The unsupported length, l_u, of a compression member shall be the clear distance between slabs, girders, or other members capable of providing lateral support for the compression member. Where haunches are present, the unsupported length shall be measured to the lower extremity of the haunch in the plane considered.

8.16.5.2.2 The radius of gyration, r, may be assumed equal to 0.30 times the overall dimension in the direction in which stability is being considered for rectangular compression members, and 0.25 times the diameter for circular compression members. For other shapes, r may be computed for the gross concrete section.

8.16.5.2.3 For compression members braced against sidesway, the effective length factor, k, shall be taken as 1.0, unless an analysis shows that a lower value may be used. For compression members not braced against sidesway, k shall be determined with due consideration of cracking and reinforcement on relative stiffness and shall be greater than 1.0.

8.16.5.2.4 For compression members braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than $34-(12M_{1b}/M_{2b})$.

8.16.5.2.5 For compression members not braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than 22.

8.16.5.2.6 For all compression members where kl_u/r is greater than 100, an analysis as defined in Article 8.16.5.1 shall be made.

8.16.5.2.7 Compression members shall be designed using the factored axial load, P_u, derived from a conventional elastic analysis and a magnified factored moment, M_c. P_u shall not exceed ϕP_c.

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \quad (8\text{-}40)$$

where

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \geq 1.0 \quad (8\text{-}41)$$

$$\delta_s = \frac{C_m}{1 - \frac{\sum P_u}{\phi \sum P_c}} \geq 1.0 \quad (8\text{-}41A)$$

and

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} \quad (8\text{-}42)$$

For members braced against sidesway, δ_s shall be taken as 1.0. For members not braced against sidesway, δ_b shall be evaluated as for a braced member and δ_s for an unbraced member.

In lieu of a more precise calculation, EI may be taken either as:

$$EI = Q\left[\frac{\frac{E_c I_g}{5} + E_s I_s}{(1 + \beta_d)}\right] \quad (8\text{-}43)$$

where

$$Q = 2.014 - 11.42(\rho_t) \quad (8\text{-}43A)$$

and

$$\rho_t = \frac{A_{st}}{A_g} \quad (8\text{-}43B)$$

Equation (8-43A) applies for values of P/P_o less than 0.6 and for values of ρ_t between 0.01 and 0.06. For values of P/P_o greater than 0.6, the value of Q shall vary linearly from the values in Equation (8-43A) to 1.0 at P/P_o of 0.9 (See Figure 8.16.5), or the value of EI may be taken conservatively as:

$$EI = \frac{\frac{E_c I_g}{2.5}}{(1 + \beta_d)} \quad (8\text{-}44)$$

where β_d is the ratio of maximum dead load moment to maximum total load moment and is always positive. For members braced against sidesway and without transverse loads between supports, C_m may be taken as:

$$C_m = 0.6 + 0.4\left(\frac{M_{1b}}{M_{2b}}\right) \quad (8\text{-}45)$$

but not less than 0.4.

For all other cases C_m shall be taken as 1.0.

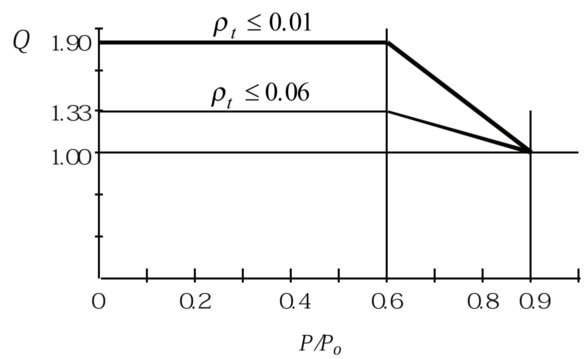


FIGURE 8.16.5 EI Correction Factor Q (Equation 8-43A)

8.16.5.2.8 If computations show that there is no moment at either end of a compression member braced or unbraced against sidesway or that computed end eccentricities are less than $(0.6 + 0.03h)$ inches, M_{2b} and M_{2s} in Equation (8-40) shall be based on a minimum eccentricity of $(0.6 + 0.03h)$ inches about each principal axis separately. The ratio M_{1b}/M_{2b} in Equation (8-45) shall be determined by either of the following:

(a) When the computed end eccentricities are less than $(0.6 + 0.03h)$ inches, the computed end moments may be used to evaluate M_{1b}/M_{2b} in Equation (8-45).

(b) If computations show that there is essentially no moment at either end of the member, the ratio M_{1b}/M_{2b} shall be equal to one.

8.16.5.2.9 In structures that are not braced against sidesway, the flexural members framing into the compression member shall be designed for the total magnified end moments of the compression member at the joint.

8.16.5.2.10 When compression members are subject to bending about both principal axes, the moment about each axis shall be magnified by d, computed from the corresponding conditions of restraint about that axis.

8.16.5.2.11 When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of δ_s shall be computed for the member group with ΣP_u and ΣP_c equal to the summations for all columns in the group.

8.16.6 Shear

8.16.6.1 Shear Strength

8.16.6.1.1 Design of cross sections subject to shear shall be based on:

$$V_u \leq \phi V_n \tag{8-46}$$

where V_u is factored shear force at the section considered and V_n is the nominal shear strength computed by:

$$V_n = V_c + V_s \tag{8-47}$$

where V_c is the nominal shear strength provided by the concrete in accordance with Article 8.16.6.2, and V_s is the nominal shear strength provided by the shear reinforcement in accordance with Article 8.16.6.3. Whenever applicable, effects of torsion[8] shall be included.

8.16.6.1.2 When the reaction, in the direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from the face of support may be designed for the same shear V_u as that computed at a distance d. An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case sections closer than d to the support shall be designed for V at a distance d plus the major concentrated loads.

8 The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete" ACI318 may be used.

8.16.6.2 Shear Strength Provided by Concrete

8.16.6.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, V_c, shall be computed by:

$$V_c = \left(1.9\sqrt{f'_c} + 2{,}500\,\rho_w \frac{V_u d}{M_u}\right) b_w d \tag{8-48}$$

or

$$V_c = 2\sqrt{f'_c}\, b_w d \tag{8-49}$$

where b_w is the width of web and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable, effects of torsion shall be included. For a circular section, b_w shall be the diameter and d need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member. For tapered webs, b_w shall be the average width or 1.2 times the minimum width, whichever is smaller.

Note:

(a) V_c shall not exceed $3.5\sqrt{f'_c}\, b_w d$ when using a more detailed calculation.

(b) The quantity $V_u d/M_u$ shall not be greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section being considered.

8.16.6.2.2 Shear in Compression Members

For members subject to axial compression, V_c may be computed by:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000\, A_g}\right)\sqrt{f'_c}\,(b_w d) \tag{8-50}$$

or

$$V_c = 2\sqrt{f'_c}\, b_w d \tag{8-51}$$

Note:

The quantity N_u/A_g shall be expressed in pounds per square inch.

8.16.6.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$V_c = 2\left(1 + \frac{N_u}{500\,A_g}\right)\sqrt{f'_c}\,(b_w d) \tag{8-52}$$

Note:

(a) N_u is negative for tension.

(b) The quantity N_u/A_g shall be expressed in pounds per square inch.

(c) V_c shall not be taken less than zero.

8.16.6.2.4 Shear in Lightweight Concrete

The provisions for shear stress, v_c, carried by the concrete apply to normal weight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply:

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

(b) When f_{ct} is not specified, V_c shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

8.16.6.3 Shear Strength Provided by Shear Reinforcement

8.16.6.3.1 Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Equations (8-46) and (8-47), but not less than that required by Article 8.19. Shear strength V_s shall be computed in accordance with Articles 8.16.6.3.2 through 8.16.6.3.10.

8.16.6.3.2 When shear reinforcement perpendicular to the axis of the member is used:

$$V_s = \frac{A_v f_y d}{s} \tag{8-53}$$

where A_v is the area of shear reinforcement within a distance s.

8.16.6.3.3 When inclined stirrups are used:

$$V_s = \frac{A_v f_y (\sin\alpha + \cos\alpha) d}{s} \tag{8-54}$$

8.16.6.3.4 When a single bar or a single group of parallel bars all bent up at the same distance from the support is used:

$$V_s = A_v f_y \sin\alpha \le 3\sqrt{f'_c}\,(b_w d) \tag{8-55}$$

8.16.6.3.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, shear strength V_s shall be computed by Equation (8-54).

8.16.6.3.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

8.16.6.3.7 Where more than one type of shear reinforcement is used to reinforce the same portion of the member, shear strength V_s shall be computed as the sum of the V_s values computed for the various types.

8.16.6.3.8 When shear strength V_s exceeds $4\sqrt{f'_c}\,b_w d$, spacing of shear reinforcement shall not exceed one-half the maximum spacing given in Article 8.19.3.

8.16.6.3.9 Shear strength V_s shall not be taken greater than $8\sqrt{f'_c}\,b_w d$.

8.16.6.3.10 When flexural reinforcement, located within the width of a member used to compute the shear strength, is terminated in a tension zone, shear reinforcement shall be provided in accordance with Article 8.24.1.4.

8.16.6.4 Shear Friction

8.16.6.4.1 Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

8.16.6.4.2 Design of cross sections subject to shear transfer as described in paragraph 8.16.6.4.1 shall be based on Equation (8-46), where shear strength V_n is calculated in accordance with provisions of paragraph 8.16.6.4.3 or 8.16.6.4.4.

8.16.6.4.3 A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vf} across the shear plane may be designed using either paragraph 8.16.6.4.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of paragraph 8.16.6.4.5 through 8.16.6.4.9 shall apply for all calculations of shear transfer strength.

8.16.6.4.4 Shear-friction Design Method

(a) When shear-friction reinforcement is perpendicular to shear plane, shear strength V_n shall be computed by:

$$V_n = A_{vf} f_y \mu \qquad (8\text{-}56)$$

where μ is coefficient of friction in accordance with paragraph (c).

(b) When shear-friction reinforcement is inclined to shear plane, such that shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by:

$$V_n = A_{vf} f_y \left(\mu \sin \alpha_f + \cos \alpha_f\right) \qquad (8\text{-}56A)$$

where α_f is angle between shear-friction reinforcement and shear plane.

(c) Coefficient of friction μ in Equation (8-56) and Equation (8-56A) shall be:

Concrete placed monolithically.....................1.4λ

Concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.16.6.4.8....................1.0λ

Concrete placed against hardened concrete not intentionally roughened........................0.6λ

Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.16.6.4.9)............................0.7λ

where λ = 1.0 for normal weight concrete; 0.85 for "sand-lightweight" concrete; and 0.75 for "all-lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

8.16.6.4.5 Shear strength V_n shall not be taken greater than $0.2 f'_c A_{cv}$ nor $800 A_{cv}$ in pounds, where A_{cv} is area of concrete section resisting shear transfer.

8.16.6.4.6 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf} f_y$, when calculating required A_{vf}.

8.16.6.4.7 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

8.16.6.4.8 For the purpose of Article 8.16.6.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately $^1/_4$ inch.

8.16.6.4.9 When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

8.16.6.5 Horizontal Shear Strength for Composite Concrete Flexural Members

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

8.16.6.5.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of paragraph 8.16.6.5.3 or 8.16.6.5.4, or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

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

$$V_u \leq \phi V_{nh} \qquad (8\text{-}57)$$

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

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

(b) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_v d$, in pounds.

(c) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately ¼ inch, shear strength V_{nh} shall not be taken greater than $350b_v d$, in pounds.

(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by paragraph 8.16.6.5.5, shear strength V_{nh} may be increased by $(160f_y/40{,}000)\,b_v d$, in pounds.

8.16.6.5.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} in accordance with paragraph 8.16.6.5.3, except that length of segment considered shall be substituted for d.

8.16.6.5.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing s shall not exceed four times the least web width of support element, nor 24 inches.

(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks.

(c) All beam shear reinforcement shall extend into +
cast-in-place deck slabs. Extended shear rein- +
forcement may be used in satisfying the minimum +
tie reinforcement. +

8.16.6.6 Special Provisions for Slabs and Footings

8.16.6.6.1 Shear strength of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions:

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.1 through 8.16.6.3 except at footings supported on piles the shear on the critical section shall be determined in accordance with Article 4.4.11.3.2. •

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but need not approach closer than $d/2$ to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.6.2 and 8.16.6.6.3.

8.16.6.6.2 Design of slab or footing for two-way action shall be based on Equation (8-46), where shear strength V_n shall not be taken greater than shear strength V_c given by Equation (8-58), unless shear reinforcement is provided in accordance with Article 8.16.6.6.3.

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\,b_o d \leq 4\sqrt{f'_c}\,b_o d \qquad (8\text{-}58)$$

β_c is the ratio of long side to short side of concentrated load or reaction area, and b_o is the perimeter of the critical section defined in Article 8.16.6.6.1(b).

8.16.6.6.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear strength V_n shall be computed by Equation (8-47), where shear strength V_c shall be in accordance with paragraph (d) and shear strength V_s shall be in accordance with paragraph (e).

(b) Shear strength shall be investigated at the critical section defined in 8.16.6.6.1(b), and at successive sections more distant from the support.

(c) Shear strength V_n shall not be taken greater than $6\sqrt{f'_c}\,b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).

(d) Shear strength V_c at any section shall not be taken greater than $2\sqrt{f'_c}\,b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).

(e) Where the factored shear force V_u exceeds the shear strength ϕV_c as given in paragraph (d), the required area A_v and shear strength V_s of shear reinforcement shall be calculated in accordance with Article 8.16.6.3.

8.16.6.7 Special Provisions for Box Culverts

8.16.6.7.1 For top slabs of box culverts under 2 feet or more fill, and for sidewalls and invert slab regardless of fill height, shear strength V_c may be computed by:

$$V_c = 3.0\sqrt{f'_c}\,bd \qquad (8\text{-}59)$$

For top slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.5 should be used.

8.16.6.8 Special Provisions for Brackets and Corbels[9]

8.16.6.8.1 Provisions of Article 8.16.6.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance d shall be measured at face of support.

8.16.6.8.2 Depth at outside edge of bearing area shall not be less than $0.5d$.

8.16.6.8.3 Section at face of support shall be designed to resist simultaneously a shear V_u, a moment $[V_u a_v + N_{uc}(h-d)]$, and a horizontal tensile force N_{uc}. Distance h shall be measured at the face of support.

(a) In all design calculations in accordance with paragraph 8.16.6.8, strength reduction factor ϕ shall be taken equal to 0.85.

(b) Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with Article 8.16.6.4. For normal weight concrete, shear strength V_n shall not be taken greater than $0.2 f'_c b_w d$ nor $800\, b_w d$ in pounds. For "all-lightweight" or "sand-lightweight" concrete, shear strength V_n shall not be taken greater than $\left(0.2 - 0.07\, a_v/d\right) f'_c b_w d$ nor $(800\text{-}280 a_v/d) b_w d$ in pounds.

(c) Reinforcement A_f to resist moment $[V_u a_v + N_{uc}(h-d)]$ shall be computed in accordance with Articles 8.16.2 and 8.16.3.

(d) Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \le \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2 V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.

(e) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$, or $(2A_{vf}/3 + A_n)$.

8.16.6.8.4 Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

8.16.6.8.5 Ratio $\rho = A_s/b_d$ shall not be less than $0.04\left(f'_c/f_y\right)$.

8.16.6.8.6 At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

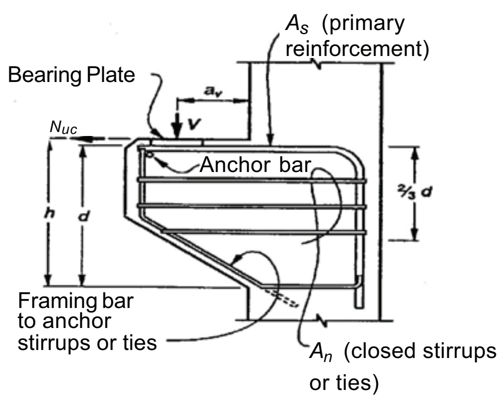
[9] These provisions do not apply to beam ledges. The PCA publication, *Notes on ACI 318-95*, contains an example design of beam ledges - Part 17, Example 17.3

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars,

(b) bending primary tension bars A_s back to form a horizontal loop, or

(c) some other means of positive anchorage.

8.16.6.8.7 Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).



8.16.6.9 Special Provision for Pier Walls

8.16.6.9.1 The shear strength of a pier wall in its weak direction shall be determined from Articles 8.16.6.2 and 8.16.6.3.

8.16.6.9.2 In regions outside the plastic hinge zone, the shear strength V_n in the strong direction of the pier wall shall be:

$$V_n = v_u A_c \tag{8-59A}$$

Where, v_u is the limiting shear stress; A_c is the area of a horizontal section of the pier wall and is given by h times d. d may be taken as 0.8 times the wall length.

8.16.6.9.3 The limiting shear stress shall be determined in accordance with the following formula:

$$v_u = 2\sqrt{f_c'} + \rho_h f_y \tag{8-59B}$$

but shall not be taken greater than $8\sqrt{f_c'}$.

8.16.6.9.4 For lightweight aggregate concrete, V_n shall be multiplied by 0.75. The reinforcement required for shear shall be continuous and distributed uniformly.

8.16.6.10 Compression Member Connection to Caps

8.16.6.10.1 Design the connection between the compression member and the bent or pier cap as specified in Article 3.22. The development length for all longitudinal steel shall be in accordance with Articles 8.24 through 8.30.

8.16.6.10.2 The shear strength in the joint of a pier or bent reinforced for enclosure, in the direction under consideration, shall not exceed $12\sqrt{f_c'}b_w d$ for normal-weight aggregate concrete, $9\sqrt{f_c'}b_w d$ nor for lightweight aggregate concrete.

8.16.6.11 Special Seismic Provisions for Columns, Pier Walls and Piles

8.16.6.11.1 The design shear force V_u on each principal axis of each column, pier wall or pile shall be the value determined from the loading combinations in Article 3.22 except for Group VII. V_u for Group VII shall be the lesser of the shear forces resulting from plastic hinging or unreduced elastic ARS seismic forces in columns or pier walls.

8.16.6.11.2 The amount of transverse reinforcement provided shall not be less than that required by Article 8.18.2 for confinement or by Article 8.19.1 for minimum shear reinforcement. For calculating the area or spacing of Grade 60 transverse reinforcement, f_y shall be taken as 60 ksi.

8.16.6.11.3 The member shear resistance in regions away from plastic hinges shall conform to Articles 8.16.6.2, 8.16.6.3 and 8.16.6.9.

8.16.6.11.4 The following provisions will determine the member shear resistance in regions of plastic hinges as described in Article 8.16.6.11.1.

(a) Concrete
The shear strength of the concrete, V_c, shall be in accordance with Article 8.16.6.2 when the axial load associated with the shear produces an average compression stress in excess of $0.1f'_c$ over the core concrete area of the support member. As P/A_c increases from 0.0 to $0.1f'_c$, strength V_c increases linearly from 0 to a maximum value of $2\sqrt{f'_c}bd$.

(b) Reinforcement
The shear strength of the transverse reinforcement V_s shall be computed by Article 8.16.6.3. Only the core section of the member shall be considered.

A_v shall be the area of the least number of bars or wires in a single layer of transverse reinforcement that can be intersected by a plane normal to the direction of the applied shear.

In walls, A_v shall be taken as the area of horizontal shear reinforcement within a distance s.

8.16.6.11.5 When calculating the shear strength of the section, d may be taken as 0.8 times the member length measured in the direction of the shear force under consideration or as 0.8 times the section diameter.

8.16.7 Bearing Strength

8.16.7.1 The bearing stress, f_b, on concrete shall not exceed $\phi 0.85 f'_c$ except as provided in Articles 8.16.7.2, 8.16.7.3 and 8.16.7.4.

8.16.7.2 When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not by more than 2.

8.16.7.3 When 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 or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

8.16.7.4 When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.

8.16.8 Serviceability Requirements

8.16.8.1 Application

For flexural members designed with reference to load factors and strengths by Strength Design Method, stresses at service load shall be limited to satisfy the requirements for fatigue in Article 8.16.8.3, and for distribution of reinforcement in Article 8.16.8.4. The requirements for control of deflections in Article 8.9 shall also be satisfied.

8.16.8.2 Service Load Stresses

For investigation of stresses at service loads to satisfy the requirements of Articles 8.16.8.3 and 8.16.8.4, the straight-line theory of stress and strain in flexure shall be used and the assumptions given in Article 8.15.3 shall apply.

8.16.8.3 Fatigue Stress Limits

The range between a maximum tensile stress and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed:

$$f_f = 21 - 0.33 f_{min} + 8\left(\frac{r}{h}\right) \qquad (8\text{-}60)$$

where:

f_f = stress range in kips per square inch;
f_{min} = algebraic minimum stress level, (tension positive, compression negative) in kips per square inch;
$\frac{r}{h}$ = ratio of base radius to height of rolled-on transverse deformations; when the actual value is not known, use 0.3.

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

Fatigue stress limits need not be considered for concrete deck slabs with primary reinforcement perpendicular to traffic and designed in accordance with the approximate methods given under Article 3.24.3 Case A.

8.16.8.4 Distribution of Flexural Reinforcement

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones. When the design yield strength, f_y, for tension reinforcement exceeds 40,000 psi, the bar sizes and spacing at maximum positive and negative moment sections shall be chosen so that the calculated stress in the reinforcement at service load, f_s, in ksi does not exceed the value computed by:

$$f_s = \frac{z}{(d_c A)^{1/3}} \leq 0.6 f_y \qquad (8\text{-}61)$$

• but shall not be less than $0.4 f_y$

In Equation (8-61),

A = effective tension area, in square inches, of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires. When the flexural reinforcement consists of several bar or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used.

d_c = distance measured from extreme tension fiber to the center of the closest bar or wire in inches. • For calculation purposes, the thickness of • clear concrete cover used to compute d_c shall • not be taken greater than 2 inches.

z = a factor related to the exposure conditions of the structure and based on the maximum crack width permitted. Values for various exposure conditions are:

(a) 100 Kips/inch for structures in direct contact with sea water or subjected to sea water spray.

(b) 130 Kips/inch for structures not in direct contact with sea water or subjected to sea water spray but located within 1,000 feet of ocean or tidal water and for bridge decks located in environmental Area III (severe climate).

(c) 170 Kips/inch for all cases other than as listed above.

Where members are exposed to very aggressive exposure or corrosive environments as specified in Article 8.22, protection should be provided as discussed in Article 8.22 and Table 8.22.1, or by furnishing other methods of protection such as a waterproofing system, in addition to • satisfying Equation (8-61). •

Part D
Reinforcement

8.17 REINFORCEMENT OF FLEXURAL MEMBERS

8.17.1 Minimum Reinforcement

8.17.1.1 At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete specified in Article 8.15.2.1.1.

8.17.1.2 The requirements of Article 8.17.1.1 may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22.

8.17.2 Distribution of Reinforcement

8.17.2.1 Flexural Tension Reinforcement in Zones of Maximum Tension

8.17.2.1.1 Where flanges of T-girders and box-girders are in tension, tension reinforcement shall be distributed over an effective tension flange width equal to $^{1}/_{10}$ the girder span length or a width as defined in Article 8.10.1, whichever is smaller. If the actual slab width, center-to-center of girder webs, exceeds the effective tension flange width, and for excess portions of the deck slab overhang, additional longitudinal reinforcement with area not less than 0.4 percent of the excess slab area shall be provided in the excess portions of the slab.

8.17.2.1.2 For integral bent caps of T-girder and box-girder construction, tension reinforcement shall be placed within a width not to exceed the web width plus an overhanging slab width on each side of the bent cap web equal to one-fourth the average spacing of the intersecting girder webs or a width as defined in Article 8.10.1.4 for integral bent caps, whichever is smaller.

8.17.2.1.3 For the distribution of negative moment tensile reinforcement continuous over a support, the effective tension flange width shall be computed separately on each side of the support in accordance with paragraphs 8.17.2.1.1 and 8.17.2.1.2. The larger of the two effective flange widths shall be used for the uniform distribution of the reinforcement into both spans.

8.17.2.1.4 If the depth of the side face of a member exceeds 2 feet, longitudinal reinforcement having a total area at least equal to 10 percent of the area of the flexural tension reinforcement shall be placed near the side faces of the member and distributed in the zone of flexural tension with a spacing not more than the web width or 12 inches.

For continuous structures, the area of flexural tension reinforcement shall be taken as the maximum at any single section, either positive or negative. Minimum size of the side face reinforcement shall be No. 4.

Such reinforcement may be included in computing the flexural capacity only if a stress and strain compatibility analysis is made to determine stresses in the individual bars or wires.

8.17.2.1.5 For girders, the top side face bar on each face of the girder web shall be a No. 8 bar.

8.17.2.1.6 In bent caps, reinforcement shall be placed approximately three inches below the construction joint between the deck and cap, or lower if necessary to clear prestressing ducts. This reinforcement shall be designed by Load Factor methods taking M_u as 1.3 times the dead load negative moment of that portion of the cap and superstructure located beneath the construction joint and within 10 feet of each side face of the cap. Service load checks and shear design are not required for this condition. This reinforcement may be included in computing the flexural capacity of the cap only if a stress and strain compatibility analysis is made to determine the stress in the bars.

8.17.2.2 Transverse Deck Slab Reinforcement in T-Girders and Box-Girders

At least one-third of the bottom layer of the transverse reinforcement in the deck slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook. If the slab extends beyond the last girder web, such reinforcement shall extend into the slab overhang and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook.

8.17.2.3 Bottom Slab Reinforcement for Box-Girders

8.17.2.3.1 Minimum distributed reinforcement of 0.4 percent of the flange area shall be placed in the bottom slab parallel to the girder span. A single layer of reinforcement may be provided. The spacing of such reinforcement shall not exceed 18 inches.

8.17.2.3.2 Minimum 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 inches. All transverse reinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook or equal.

8.17.3 Lateral Reinforcement of Flexural Members

8.17.3.1 Compression reinforcement used to increase the strength of flexural members shall be enclosed by ties or stirrups which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Welded wire fabric of equivalent area may be used instead of bars. The spacing of ties shall not exceed 16 longitudinal bar diameters. Such stirrups or ties shall be provided throughout the distance where the compression reinforcement is required. This paragraph does not apply to reinforcement located in a compression zone which has not been considered as compression reinforcement in the design of the member.

8.17.3.2 Torsion reinforcement, where required, shall consist of closed stirrups, closed ties, or spirals, combined with longitudinal bars. See Article 8.15.5.1.1 or 8.16.6.1.1.

8.17.3.3 Closed stirrups or ties may be formed in one piece by overlapping the standard end hooks of ties or stirrups around a longitudinal bar, or may be formed in one or two pieces by splicing with Class C splices (lap of $1.7l_d$).

8.17.3.4 In seismic areas, where an earthquake that could cause major damage to construction has a high probability of occurrence, lateral reinforcement shall be designed and detailed to provide adequate strength and ductility to resist expected seismic movements.

8.17.4 Reinforcement for Hollow Rectangular Compression Members

8.17.4.1 The area of longitudinal reinforcement in the cross section shall not be less than 0.01 times the gross area of concrete in the cross section.

8.17.4.2 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.

8.17.4.3 The center-to-center lateral spacing of longitudinal reinforcing bars shall be no greater than 1.5 times the wall thickness, or 18 inches, whichever is less.

8.17.4.4 The center-to-center longitudinal spacing of lateral reinforcing bars shall be no greater than 1.25 times the wall thickness, or 12 inches, whichever is less.

8.17.4.5 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 not to exceed 24 inches.

8.17.4.6 For segmentally constructed members, additional cross ties shall be provided along the top and bottom edges of each segment. The cross ties 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.

8.17.4.7 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 are not 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.

8.17.4.8 When 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, then 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 substituted.

8.17.4.9 Post-tensioning ducts located in the corners of the cross section shall be anchored into the corner regions with closed hoops, or by stirrups having a 90-degree bend at each end which encloses at least one longitudinal bar near the outer face of the cross section.

8.18 REINFORCEMENT OF COMPRESSION MEMBERS

8.18.1 Maximum and Minimum Longitudinal Reinforcement

8.18.1.1 The area of longitudinal reinforcement for compression members shall not exceed 0.08 times the gross area, A_g, of the section.

8.18.1.2 The minimum area of longitudinal reinforcement shall not be less than 0.01 times the gross area, A_g, of the section. When the cross section is larger than that required by consideration of loading, a reduced effective area may be used. The reduced effective area shall not be less than that which would require one percent of longitudinal reinforcement to carry the loading. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bars shall be No. 5.

8.18.1.4 The center-to-center spacing of interlocking spirals or hoop cages in oblong columns shall not be greater than 0.75 times the diameter of the cage. The overlaps shall be interlocked by a minimum of four bars.

8.18.1.5 The minimum vertical shear reinforcement ratio ρ_n, in a pier wall shall not be less than 0.0025. The reinforcement determined by ρ_n shall be spaced uniformly along both faces at a spacing not exceeding 12 inches. ρ_n shall not be less than ρ_h (Article 8.18.2.1.6).

8.18.2 Lateral Reinforcement

8.18.2.1 General

8.18.2.1.1 Lateral reinforcement for compression members shall consist of either spiral reinforcement, hoops, or of a combination of lateral ties and cross ties. Ties shall only be used when it is not practical to provide spiral or hoop reinforcement. Where longitudinal bars are required outside the spiral or hoop reinforcement, they shall have lateral support provided by bars spaced and hooked as required for cross ties. The hooked bars shall extend into the core of the spiral or hoop by a full development length.

8.18.2.1.2 Reinforcement required for Article 8.18.2.1.1 may be used to satisfy shear requirements of Article 8.16.6.9.

8.18.2.1.3 Lateral reinforcement shall extend at the same spacing into the footing to the point of tangency of the column bar hooks but may be discontinuous at the top footing reinforcement.

8.18.2.1.4 Lateral reinforcement for compression members shall be continued into the cap a distance equal to the lesser of:

(a) one-half the maximum dimension of the confined core section of the compression member at the cap soffit;

(b) the development length of straight main reinforcement from compression members;

(c) the straight portion of hooked main reinforcement from compression members.

This lateral reinforcement may be discontinuous at the bottom flexural reinforcement of the cap.

8.18.2.1.5 In a compression member that has a larger cross section than that required by conditions of loading, the lateral reinforcement requirements may be waived where structural analysis or tests show adequate strength and feasibility of construction.

8.18.2.1.6 In pier walls, the minimum horizontal shear reinforcement ratio ρ_h shall not be less than 0.0025. For pier walls designed as columns, provisions in Article 8.18.2.3.1 shall apply.

8.18.2.1.7 The vertical spacing of horizontal shear reinforcement (ties) in pier walls shall not exceed the least dimension of the wall or 12 inches, whichever is smaller.

When vertical reinforcement is comprised of bars larger than No. 10 bundled together with more than two bars in any one bundle, the maximum spacing of horizontal reinforcement shall be one-half of that specified above.

8.18.2.1.8 In plastic hinge zones, the maximum spacing of horizontal shear reinforcement shall be one-half of that specified in Article 8.18.2.1.7.

8.18.2.1.9 Cross-ties and horizontal shear reinforcement in pier walls shall conform to Articles 8.18.2.3.3 and 8.18.2.3.4.

8.18.2.2 Spiral or Hoops

Spiral or hoop reinforcement for compression members shall conform to the following:

8.18.2.2.1 Spirals or hoops shall consist of evenly spaced continuous bar or wire, with a minimum diameter of 0.20 inch for members having a minimum dimension of 20 inches or less, and 0.348 inch for members having a minimum dimension greater than 20 inches.

8.18.2.2.2 Ratio of spiral or hoop reinforcement ρ_s shall not be less than the value given by:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (8\text{-}62)$$

where f_y is the specified yield strength of spiral or hoop reinforcement but not more than 60,000 psi.

In potential plastic hinge zone, as defined in Article

3.21.8, ρ_s shall not be less than:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}\left(0.5 + \frac{1.25P_e}{f'_c A_g}\right) \quad \text{(8-62A)}$$

for columns less than or equal to 3 feet in diameter or least dimension,

or

$$\rho_s = 0.12\frac{f'_c}{f_y}\left(0.5 + \frac{1.25P_e}{f'_c A_g}\right) \quad \text{(8-62B)}$$

for columns larger than 3 feet in diameter or least dimension.

However, ρ_s shall not be not less than that required by Equation (8-62).

If the cover over the core in a plastic hinge zone exceeds two inches at any point, the value of A_g used to determine ρ_s for the plastic hinge zone shall be limited to the area of a reduced section having not more than two inches of cover. The reduced section used to calculate A_g shall be adequate for all applied loads associated with the plastic hinge.

8.18.2.2.3 *Deleted*

8.18.2.2.4 *Deleted*

8.18.2.2.5 *Deleted*

8.18.2.2.6 Splices in spiral or hoop reinforcement shall be accomplished by welding or mechanical couplers.

8.18.2.2.7 Spirals or hoops shall be of such size and so assembled as to permit handling and placing without distortion from designed dimensions.

8.18.2.2.8 Spirals or hoops shall be held firmly in place by attachment to the longitudinal reinforcement and true to line by vertical spacers.

8.18.2.3 Ties

Tie reinforcement for compression members shall conform to the following:

8.18.2.3.1 All bars shall be enclosed by lateral ties which are at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18 and bundled longitudinal bars.

The total cross sectional area (A_{sh}) of tie reinforcement for a rectangular column shall not be less than:

$$A_{sh} = 0.30 s_t h_c \frac{f'_c}{f_y}\left(\frac{A_g}{A_c} - 1\right) \quad \text{(8-62C)}$$

In plastic hinge zones, A_{sh} shall not be less than either

$$A_{sh} = 0.30 s_t h_c \frac{f'_c}{f_y}\left(\frac{A_g}{A_c} - 1\right)\left(0.5 + \frac{1.25P_e}{f'_c A_g}\right) \quad \text{(8-62D)}$$

or

$$A_{sh} = 0.12 s_t h_c \frac{f'_c}{f_y}\left(0.5 + \frac{1.25P_e}{f'_c A_g}\right) \quad \text{(8-62E)}$$

whichever is greater, but not less than that required by Equation (8-62C). s_t shall not be less than 2 inches. Use area A_g for plastic hinge zones as defined in Article 8.18.2.2.2.

8.18.2.3.2 *Deleted*

8.18.2.3.3 Ties shall be located vertically not more than half a tie spacing above the footing or other support and shall be spaced as provided herein to not more than half a tie spacing below the lowest horizontal reinforcement in members supported above.

8.18.2.3.4 Lateral tie reinforcement, shall be provided by single or overlapping closed ties, or a single closed tie combined with cross ties.

Ties shall be so arranged that every corner and alternate longitudinal bar or bundle of bars shall have lateral support, but no intermediate bar or bundle shall be farther than 6 inches clear on either side from such a laterally supported bar or bundle. Corner bars shall be considered laterally supported if the included angle of the tie does not exceed 135 degrees.

Closed ties shall be terminated with 135 degree hooks. The hook extensions shall be the larger of 10 tie diameters or 6 inches.

Cross ties shall be hooked at both ends and placed normal across core section h_c. Each hook will engage the perimeter tie at a longitudinal bar on opposite face of the column. Hook extensions shall be the same as for closed ties. Hook details shall be in accordance with either of the following:

(a) Continuous ties with 135degree hook on one end and 90degree hook on the other. Cross ties shall be alternated so that hooks of the same degree are not adjacent to each other both vertically and horizontally.

(b) Lap spliced tie with 180degree hook at each end.

8.18.2.4 *Deleted*

8.19 LIMITS FOR SHEAR REINFORCEMENT

8.19.1 Minimum Shear Reinforcement

8.19.1.1 A minimum area of shear reinforcement shall be provided in all flexural members, except slabs and footings, where:

(a) For design by Strength Design, factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c.

(b) For design by Service Load Design, design shear stress v exceeds one-half the permissible shear stress carried by concrete v_c.

8.19.1.2 Where shear reinforcement is required by Article 8.19.1.1, or by analysis, the area provided shall not be less than:

$$A_v = \frac{50 b_w s}{f_y} \qquad (8\text{-}63)$$

where b_w and s are in inches.

8.19.1.3 Minimum shear reinforcement requirements may be waived if it is shown by test that the required ultimate flexural and shear capacity can be developed when shear reinforcement is omitted.

8.19.2 Types of Shear Reinforcement

8.19.2.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to the axis of the member or making an angle of 45degrees or more with the longitudinal tension reinforcement.

(b) Welded wire fabric with wires located perpendicular to the axis of the member.

(c) Longitudinal reinforcement with a bent portion making an angle of 30degrees or more with the longitudinal tension reinforcement.

(d) Combinations of stirrups and bent longitudinal reinforcement.

(e) Spirals.

8.19.2.2 Shear reinforcement shall be developed at both ends in accordance with the requirements of Article 8.27.

8.19.3 Spacing of Shear Reinforcement

Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed $d/2$ or 24 inches. Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree line extending toward the reaction from the mid-depth of the member, $d/2$, to the longitudinal tension reinforcement shall be crossed by at least one line of shear reinforcement.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1 Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least $^1/_8$ square inch per foot in each direction.

8.20.2 The spacing of shrinkage and temperature reinforcement shall not exceed three times the wall or slab thickness, or 18 inches.

8.21 SPACING LIMITS FOR REINFORCEMENT

8.21.1 For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than $1^1/_2$ bar diameters, $1^1/_2$ times the maximum size of the coarse aggregate, or $1^1/_2$ inches.

8.21.1.1 The maximum spacing of lateral reinforcement in compression members shall not exceed the smaller of one-fifth of the least dimension of the cross-section , 6 times the nominal diameter of the longitudinal reinforcement, or 8 inches.

8.21.1.2 The maximum spacing of longitudinal reinforcement in compression members shall be 8 inches. The maximum spacing of the inner circle or row of concentric longitudinal reinforcement enclosed in lateral reinforcement may be increased to 16 inches if confinement does not control the spacing of the lateral reinforcement.

8.21.2 For precast concrete (manufactured under plant control conditions) the clear distance between parallel bars in a layer shall be not less than 1 bar diameter, or $1^1/_3$ times the maximum size of the coarse aggregate, or 1 inch.

8.21.3 Where positive or negative reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers not less than 1 inch.

8.21.4 The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.21.5 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle. Bars larger than No. 11 shall be limited to two in any one bundle in beams. Bundled bars shall be located within stirrups or ties. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40 bar diameters apart. Where spacing limitations are based on bar diameter, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8.21.6 In walls and slabs, the primary flexural reinforcement shall be spaced not farther apart than $1^1/_2$ times the wall or slab thickness, or 18 inches.

8.21.7 For cast-in-place concrete piling, the entire length of piles 24 inches and greater in diameter and the portion below 15 feet from the top of piles less than 24 inches in diameter, the clear distance between parallel longitudinal and tranverse reinforcing bars shall not be less than 5 times the maximum aggregate size or 5 inches.

8.22 PROTECTION AGAINST CORROSION

8.22.1 The minimum concrete cover for protection of reinforcement against corrosion due to chlorides shall be as provided in Table 8.22.1. "Corrosive" water or soil contains more than 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than 2,000 ppm and/or a pH of less than 5.5 shall be considered non-corrosive in determining minimum cover from Table 8.22.1, but shall conform to the requirements of Article 8.22.6.

Marine atmosphere includes both the atmosphere over land within 1,000 feet of ocean or tidal water, and the atmosphere above the splash zone. Tidal water, from corrosion considerations, is any body of water having a chloride content greater than or equal to 500 ppm. The splash zone is defined as the region from the Mean Lower Low Water (MLLW) elevation to 20 feet above the Mean Higher High Water (MHHW) elevation and/or a horizontal distance of 20 feet from the edge of water. The concrete cover in structural elements that are in direct contact with ocean spray shall be based on the requirements for a chloride concentration greater than 10,000 ppm in the corrosive splash zone.

8.22.2 For bundled bars, the minimum concrete cover in non-corrosive atmosphere shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches. For concrete in contact with non-corrosive soil or water, the minimum cover shall be 3 inches. In corrosive environments, the cover shall be the same as that specified in Table 8.22.1, except that it shall not be less than the cover specified for bundled bars in non-corrosive environments.

8.22.3 The minimum concrete cover for protection of ducts in corrosive environments shall be the same as that specified for reinforcement in Table 8.22.1, except that:

(a) the concrete cover over the duct shall not be less than one-half the diameter of the duct; and,

(b) when epoxy-coated reinforcement is required, the minimum concrete cover over the duct shall be increased by $^1/_2$ inch beyond that specified for reinforcement in Table 8.22.1, but shall not be less than that specified in (a).

TABLE 8.22.1 Minimum Concrete Cover (inches) for 75-year Design Life

	Exposure condition										
	Non-corrosive Atmosphere/ soil/water	Marine Atmosphere	Corrosive soil above MLLW level			Corrosive soil below MLLW level	Corrosive water permanently below MLLW level	Corrosive splash zone			Deicing salt, snow run-off, or snow blower spray
			Chloride Concentration (ppm)					Chloride concentration (ppm)			
			500 – 5,000	5,001 – 10,000	Greater than 10,000			500 – 5,000	5,001 – 10,000	Greater than 10,000	
			(a)	**(a)**	**(a)**	**(a)**	**(a),(b)**	**(a),(b)**	**(a),(b)**	**(a),(b)**	**(a),(c),(e)**
Footings & pile caps	3	3	3	4	5	3	2	2	3	3.5	2.5
Walls, columns & cast-in-place piles	2	3	3	4	5	3	2	2	3	3.5	2.5
Precast piles and pile extensions	2	2[d]	2[d]	2[b], [d]	3[b], [d]	2[d]	2	2	2[d]	2.5[d]	2[d]
Top surface of deck slabs	2	2.5						2.5	2.5	2.5[d]	2.5
Bottom surface of deck slabs	1.5	1.5						2	2.5	2.5[d]	2.5
Bottom slab of box girders	1.5	1.5						2	2.5	2.5[d]	1.5
Cast-in-place "I" and "T" girders; cast exposed faces of box-girder webs, bent caps, diaphragms, and hinged joints **(f)**	1.5	3						2	2.5	2.5[d]	3
Curbs & railings	1	1[b]						1	1	1[d]	1
Concrete surface not exposed to weather, soil or water	Principal reinforcement: 1.5 inches Stirrups, ties and spirals: 1.0 inch										

General Notes:

1. Mineral admixtures conforming to ASTM Designation C618 Type F or N, are required for all exposure conditions, except for 'non-corrosive' exposure conditions.
2. For protection of bundled bars, ducts and /or prestressing steel, see Articles 8.22.2, 8.22.3 and 8.22.4.
3. The minimum cover at the corners, beveled edges, and curved surfaces shall be the same as that in the corresponding structural elements.

Footnotes:

(a) The maximum water to cementitious material ratio shall not exceed 0.40.
(b) Use pre-fabricated epoxy coated reinforcing bars (ECR).
(c) Use post-fabricated ECR.
(d) Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N, may be required.
(e) The minimum concrete cover and other requirements in structural elements exposed to de-icing salt, snow run-off, or snow blower spray shall be adopted only where the structural elements are directly exposed to these corrosive conditions, otherwise the requirements specified for non-corrosive conditions shall be adopted.
(f) For precast "I" and "T" girders, the minimum cover may be reduced (depending on site conditions).

8.22.4 In corrosive environments, the minimum concrete cover to prestressing steel not placed within ducts, shall be the same as that specified for reinforcement (Table 8.22.1), except that when epoxy-coated reinforcement is required per Table 8.22.1, the prestressing steel shall either be epoxy-coated or the minimum concrete cover to the prestressing steel shall be increased by 1 inch beyond that specified in Table 8.22.1.

8.22.5 Exposed reinforcement, inserts, and plates intended for bonding with future extensions, as well as other types shall be protected from corrosion. All other ferrous hardware, attachments, installations etc. shall conform to the requirements of Table 8.22.1, or shall be protected by hot-dip galvanizing or an equivalent protective method. Appropriate reductions in requirements are permitted depending on the interim conditions and/or exposure duration.

8.22.6 The durability of concrete may be adversely affected by contact with acids and sulfates in soil or water. The minimum requirements for protection of concrete against acid and sulfate exposure shall conform to the requirements in Table 8.22.2.

TABLE 8.22.2 Minimum Requirements for Protection of Reinforced and Unreinforced Concrete against Acid and Sulfate Exposure Conditions

Soil or Water pH	Sulfate concentration in soil or water (ppm)	Cement type required
7.1 to 14	0 to 1499	Type I-P (MS) modified or Type II modified
5.6 to 7	1500 to 1999	Type I-P (MS) modified or Type II modified [a]
3 to 5.5 [c]	2000 to 15000 [c]	Type II modified or Type V [b]

General Notes:

1. Recommendations for cement type shall apply when the pH shown in Column 1 and/or the sulfate concentration shown in Column 2 exist.
2. The table lists soil/water pH and Sulfate concentration in increasing levels of severity. If the soil/water pH and the sulfate concentration are at different levels of severity, then the recommendation for the more severe level shall apply.

Footnotes:

(a) Maximum water to cementitious material ratio shall not exceed 0.45

(b) The minimum cementitious material content shall be 658 pounds per cubic yard with a 25% mineral admixture replacement by weight. Maximum water to cementitious material ratio shall not exceed 0.40.

(c) Additional mitigation measures will be needed for conditions where pH is less than 3 and/or the sulfate concentration exceeds 15,000 ppm. Mitigation measures may include additional concrete cover and/or protective coatings.

8.23 HOOKS AND BENDS

8.23.1 Standard Hooks

The term "standard hook" as used herein, shall mean one of the following:

(a) 180-deg bend plus $4d_b$ extension, but not less than 2.5 inches at free end of bar.
(b) 90-deg bend plus $12d_b$ extension at free end of bar.
(c) For stirrup and tie hooks:
   (1) No. 5 bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar, or
   (2) No. 6, No. 7 and No. 8 bar, 90-deg bend plus $12d_b$ extension at free end of bar, or
   (3) No. 8 bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

8.23.2 Minimum Bend Diameters

8.23.2.1 For reinforcing bars, the diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values given in Table 8.23.2.1.

TABLE 8.23.2.1 Minimum Diameters of Bend

Bar Size	Minimum Diameter
Nos. 3 through 8	6 bar diameters
Nos. 9,10 and 11	8 bar diameters
Nos. 14 and 18	10 bar diameters

8.23.2.2 For Grade 40 bars of size No. 3 to No. 11 inclusive, with bends not exceeding 180 degrees, the minimum diameter of bend shall not be less than 5 bar diameters.

8.23.2.3 The inside diameter of bend for stirrups and ties shall not be less than 4 bar diameters for sizes No. 5 and smaller. For bars larger than size No. 5, the diameter of bend shall be in accordance with Table 8.23.2.1.

8.23.2.4 The inside diameter of bend in smooth or deformed welded wire fabric for stirrups and ties shall not be less than 4-wire diameters for deformed wire larger than D6 and 2-wire diameters for all other wires. Bends with inside diameters of less than 8-wire diameters shall not be less than 4-wire diameters from the nearest welded intersection.

8.24 DEVELOPMENT OF FLEXURAL REINFORCEMENT

8.24.1 General

8.24.1.1 The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

8.24.1.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent. The provisions of Article 8.24.2.3 must also be satisfied.

8.24.1.2.1 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member, 15 bar diameters, or $^1/_{20}$ of the clear span, whichever is greater, except at supports of simple spans and at the free ends of cantilevers.

8.24.1.2.2 Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

8.24.1.3 Tension reinforcement may be developed by bending across the web in which it lies or by making it continuous with the reinforcement on the opposite face of the member.

8.24.1.4 Flexure reinforcement within the portion of the member used to calculate the shear strength shall not be terminated in a tension zone unless one of the following conditions is satisfied:

8.24.1.4.1 The shear at the cutoff point does not exceed two-thirds of that permitted, including the shear strength of shear reinforcement provided.

8.24.1.4.2 Stirrup area in excess of that required for shear is provided along each terminated bar over a distance from the termination point equal to three-fourths the effective depth of the member. The excess stirrup area, A_v, shall not be less than $60b_ws/f_y$. Spacing, s, shall not exceed $d/(8\beta_b)$ where β_b is the ratio of the area of reinforcement cutoff to the total area of tension reinforcement at the section.

8.24.1.4.3 For No. 11 bars and smaller, the continuing bars provided double the area required for flexure at the cutoff point and the shear does not exceed three-fourths of that permitted.

8.24.1.5 Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which the tension reinforcement is not parallel to the compression face.

8.24.2 Positive Moment Reinforcement

8.24.2.1 At least one-third the positive moment reinforcement in simple members or at simple supports of continuous members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. In beams, such reinforcement shall extend into the support at least 6 inches.

8.24.2.2 When a flexural member is part of the lateral load resisting system, the positive moment reinforcement required to be extended into the support by Article 8.24.2.1 shall be anchored to develop the specified yield strength, f_y, in tension at the face of the support.

8.24.2.3 At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that l_d computed for f_y by Article 8.25 satisfies Equation (8-64); except Equation (8-64) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \le \frac{M}{V} + l_a \qquad \text{(8-64)}$$

where M is the computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed. V is the maximum shear force at the section. l_a at a support shall be the embedment length beyond center of support. At a point of inflection, l_a shall be limited to the effective depth of the member or $12d_b$, whichever is greater. The value M/V in the development length limitation may be increased by 30 percent when the ends of the reinforcement are confined by a compressive reaction.

8.24.3 Negative Moment Reinforcement

8.24.3.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

8.24.3.2 Negative moment reinforcement shall have an embedment length into the span as required by Article 8.24.1.

8.24.3.3 At least one-third of the total tension reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, 12 bar diameters or $^1/_{16}$ of the clear span, whichever is greater.

8.25 DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

The development length, l_d, in inches shall be computed as the product of the basic development length defined in Article 8.25.1 and the applicable modification factor or factors defined in Articles 8.25.2 and 8.25.3, but l_d shall be not less than that specified in Article 8.25.4.

8.25.1 The basic development length shall be:

No. 11 bar[10] and smaller............................ $\dfrac{0.04A_bf_y}{\sqrt{f'_c}}$

but not less than[11]...................................... $0.0004d_bf_y$

10 The constant has the unit of 1/in.
11 The constant has the unit of in.^{2}/lb.

No. 14 bars[12] .... $\dfrac{0.085 f_y}{\sqrt{f'_c}}$

No. 18 bars[12] .... $\dfrac{0.11 f_y}{\sqrt{f'_c}}$

deformed wire .... $\dfrac{0.03 d_b f_y}{\sqrt{f'_c}}$

8.25.2 The basic development length shall be multiplied by the following applicable factor or factors:

8.25.2.1 Top reinforcement shall be considered as only those horizontal bars or hooks which have more than 12 inches of concrete cast in the member below the bar or hook as a part of the same pour that encases the bar 1.4

8.25.2.2 Lightweight aggregate concrete

when f_{ct} is specified .... $\dfrac{6.7\sqrt{f'_c}}{f_{ct}}$

but not less than 1.0

When f_{ct} is not specified "all lightweight" concrete 1.33

"sand lightweight" concrete 1.18

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

8.25.2.3 Bars coated with epoxy with cover less than $3d_b$ or clear spacing between bars less than $6d_b$ 1.5

All other cases 1.15

The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy coated reinforcement need not be taken greater than 1.7

12 The constant has the unit of in. ^{2}/in.

8.25.3 The basic development length, modified by the appropriate factors of Article 8.25.2, may be multiplied by the following factors when:

8.25.3.1 Reinforcement being developed in the length under consideration is spaced laterally at least 6 inches on center with at least 3 inches clear cover measured in the direction of the spacing 0.8

8.25.3.2 Anchorage or development for reinforcement strength is not specifically required or reinforcement in flexural members is in excess of that required by analysis

(A_s required)/(A_s provided)

8.25.3.3 Reinforcement is enclosed within a spiral of not less than 1/4 inch in diameter and not more than 4-inch pitch 0.75

8.25.4 The development length, l_d, shall not be less than 12 inches except in the computation of lap splices by Article 8.32.3 and development of shear reinforcement by Article 8.27.

8.26 DEVELOPMENT OF DEFORMED BARS IN COMPRESSION

The development length, l_d, in inches, for deformed bars in compression shall be computed as the product of the basic development length of Article 8.26.1 and applicable modification factors of Article 8.26.2, but l_d shall not be less than 8 inches.

8.26.1 The basic development length

shall be .... $\dfrac{0.02 d_b f_y}{\sqrt{f'_c}}$

but not less than[13] .... $0.0003 d_b f_y$

13 The constant has a unit of square inches per pound.

8.26.2 The basic development length may be multiplied by applicable factors when:

8.26.2.1 Anchorage or development for reinforcement strength is not specifically required, or reinforcement is in excess of that required by analysis

(A_s required)/(A_s provided)

8.26.2.2 Reinforcement is enclosed in a spiral of not less than $^1/_4$ inch in diameter and not more than 4-inch pitch..0.75

8.27 DEVELOPMENT OF SHEAR REINFORCEMENT

8.27.1 Shear reinforcement shall extend at least to the centroid of the tension reinforcement, and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its design yield strength.

8.27.2 The ends of single leg, single U, or multiple U-stirrups shall be anchored by one of the following means:

8.27.2.1 A standard hook plus an embedment of the stirrup leg length of at least 0.5 l_d between the mid-depth of the member d/2 and the point of tangency of the hook.

8.27.2.2 An embedment length of l_d above or below the mid-depth of the member on the compression side but not less than 24 bar or wire diameters or, for deformed bars or deformed wire, 12 inches.

8.27.2.3 Bending around the longitudinal reinforcement through at least 180 degrees. Hooking or bending stirrups around the longitudinal reinforcement shall be considered effective anchorage only when the stirrups make an angle of at least 45 degrees with the longitudinal reinforcement.

8.27.2.4 For each leg of welded smooth wire fabric forming single U-stirrups, either:

8.27.2.4.1 Two longitudinal wires at 2-inch spacing along the member at the top of the U.

8.27.2.4.2 One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced at least 2 inches 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 of not less than 8-wire diameters.

8.27.2.5 For each end of a single leg stirrup of welded smooth or welded deformed wire fabric, there shall be two longitudinal wires at a minimum spacing of 2 inches and with the inner wire at least the greater of d/4 or 2 inches from mid-depth of member d/2. Outer longitudinal wire at the tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

8.27.3 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when the laps are $1.7 l_d$.

8.27.4 Between the anchored ends, each bend in the continuous portion of a single U- or multiple U-stirrup shall enclose a longitudinal bar.

8.27.5 Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with the longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond the mid-depth, d/2, as specified for development length in Article 8.25 for that part of the stress in the reinforcement required to satisfy Equation (8-8) or Equation (8-54).

8.28 DEVELOPMENT OF 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 33 percent for a four-bar bundle.

8.29 DEVELOPMENT OF STANDARD HOOKS IN TENSION

8.29.1 Development length l_{dh} in inches, for deformed bars in tension terminating in a standard hook (Article

8.23.1) shall be computed as the product of the basic development length l_{hb} of Paragraph 8.29.2 and the applicable modification factor or factors of Paragraph 8.29.3, but l_{dh} shall not be less than $8d_b$ or 6 inches, whichever is greater.

8.29.2 Basic development length l_{hb} for a hooked bar with f_y equal to 60,000 psi shall be.................................... $\dfrac{1{,}200d_b}{\sqrt{f'_c}}$

8.29.3 Basic development length l_{hb} shall be multiplied by applicable modification factor or factors for:

8.29.3.1 Bar Yield Strength

Bars with f_y other than 60,000 psi...................................... $\dfrac{f_y}{60{,}000}$

8.29.3.2 Concrete Cover

For No. 11 bar and smaller, side cover (normal to plane of hook) not less than $2^1/_2$ inches, and for 90 deg hook, cover on bar extension beyond hook not less than 2 inches..0.7

8.29.3.3 Ties or Stirrups

For No. 11 bar and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development length l_{dh} not greater than $3d_b$, where d_b diameter of hooked bar............0.8

8.29.3.4 Excess Reinforcement

Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis

(A_s required)/(A_s provided)

8.29.3.5 Lightweight aggregate concrete..........1.3

8.29.3.6 Epoxy-coated reinforcement hooked bars with epoxy coating 1.2

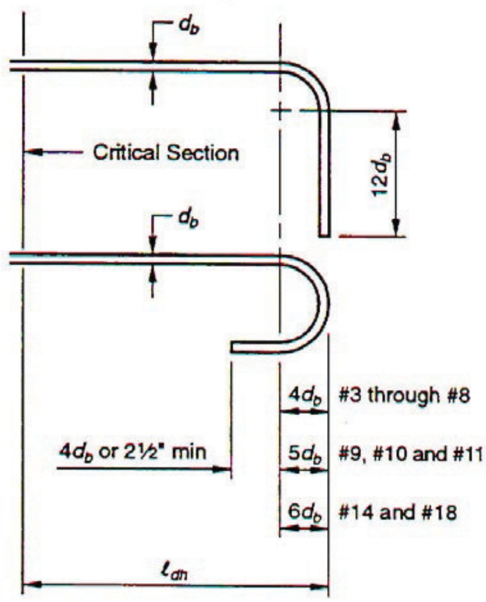


FIGURE 8.29.1 Hooked Bar Details for Development of Standard Hooks

8.29.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than $2^1/_2$ inches, hooked bar shall be enclosed within ties or stirrups spaced along the full development length l_{dh} not greater than $3\,d_b$, where d_b is diameter of hooked bar. For this case, factor of Article 8.29.3.3 shall not apply.

8.29.5 Hooks shall not be considered effective in developing bars in compression.

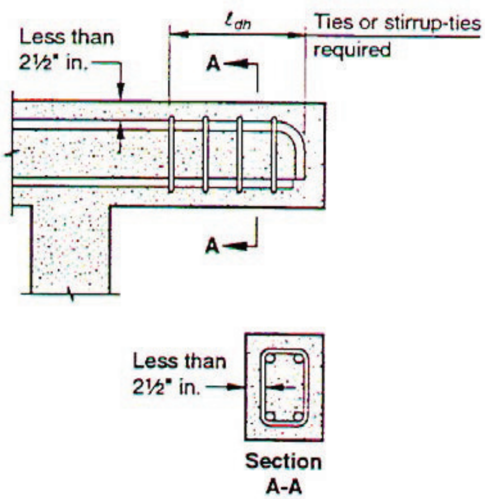


FIGURE 8.29.4 Hooked Bar Tie Requirements

8.30 DEVELOPMENT OF WELDED WIRE FABRIC IN TENSION

8.30.1 Deformed Wire Fabric

8.30.1.1 The development length, l_d, in inches of welded deformed wire fabric measured from the point of critical section to the end of wire shall be computed as the product of the basic development length of Article 8.30.1.2 or 8.30.1.3 and the applicable modification factor or factors of Articles 8.25.2 and 8.25.3, but l_d shall not be less than 8 inches except in computation of lap splices by Article 8.32.5 and development of shear reinforcement by Article 8.27.

8.30.1.2 The basic development length of welded deformed wire fabric, with at least one cross wire within the development length not less than 2 inches from the point of critical section, shall be:

$$\frac{0.03 d_b \left(f_y - 20{,}000\right)^{14}}{\sqrt{f'_c}} \qquad (8\text{-}65)$$

but not less than

$$0.20 \frac{A_w}{s_w} \frac{f_y}{\left(\sqrt{f'_c}\right)} \qquad (8\text{-}66)$$

8.30.1.3 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 8.25.

8.30.2 Smooth Wire Fabric

The yield strength of welded smooth wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 2 inches from the point of critical section. However, development length l_d measured from the point of critical section to outermost cross wire shall not be less than:

$$0.27 \frac{A_w}{s_w} \frac{f_y}{\left(\sqrt{f'_c}\right)} \qquad (8\text{-}67)$$

modified by (A_s required)/(A_s provided) for reinforcement in excess of that required by analysis and by factor of Article 8.25.2 for lightweight aggregate concrete, but l_d shall not be less than 6 inches except in computation of lap splices by Article 8.32.6.

8.31 MECHANICAL ANCHORAGE

8.31.1 Any mechanical device shown by tests to be capable of developing the strength of reinforcement without damage to concrete may be used as anchorage.

8.31.2 Development of reinforcement may consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between point of maximum bar stress and the mechanical anchorage.

[14] 20,000 has units of psi.

8.32 SPLICES OF REINFORCEMENT

Splices of reinforcement shall be made only as shown on the design drawings or as specified, or as authorized by the Engineer.

8.32.1 Lap Splices

8.32.1.1 Lap splices shall not be used for bars larger than No. 11, except as provided in Articles 8.32.4.1 and 4.4.9.7.

8.32.1.2 Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle. The length of lap, as prescribed in Articles 8.32.3 or 8.32.4, shall be increased by 20 percent for a three-bar bundle and 33 percent for a four-bar bundle. Individual bar splices within the bundle shall not overlap.

8.32.1.3 Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than $^1/_5$ the required length of lap or 6 inches.

8.32.1.4 The length, l_d, shall be the development length for the specified yield strength, f_y, as given in Article 8.25.

8.32.1.5 Lap splices shall not be used in longitudinal reinforcing bars within zones of possible plastic hinging of the member.

8.32.2 Welded Splices and Mechanical Connections

8.32.2.1 Welded splices or other mechanical connections may be used.

8.32.2.2 *Deleted*

8.32.2.3 *Deleted*

8.32.2.4 *Deleted*

8.32.3 Splices of Deformed Bars and Deformed Wire in Tension

8.32.3.1 The minimum length of lap for tension lap splices shall be as required for Class A, B, or C splice, but not less than 12 inches.

Class A splice................................ 1.0 l_d
Class B splice................................ 1.3 l_d
Class C splice................................ 1.7 l_d

8.32.3.2 Lap splices of deformed bars and deformed wire in tension shall conform to Table 8.32.3.2

TABLE 8.32.3.2 Tension Lap Splices.

(A_s provided)/(A_s required)[a]	Maximum percent of A_s spliced within required lap length		
	50	75	100
Equal to or Greater than 2	Class A	Class A	Class B
Less than 2	Class B	Class C	Class C

[a] Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

8.32.3.3 *Deleted*

8.32.3.4 *Deleted*

8.32.3.4.1 *Deleted*

8.32.3.4.2 *Deleted*

8.32.3.5 Splices in tension tie members shall be made with a full welded splice or a full mechanical connection.

8.32.4 Splices of Bars in Compression

8.32.4.1 Lap Splices in Compression

The minimum length of lap for compression lap splices shall be $0.0005 f_y d_b$ in inches, but not less than 12 inches. When the specified concrete strength, f'_c, is less than 3,000 psi, the length of lap shall be increased by one-third.

When bars of different size are lap spliced in compression, splice length shall be the larger of: development length of the larger bar, or splice length of smaller bar. Bar sizes No. 14 and No. 18 may be lap spliced to No. 11and smaller bars.

In compression members where ties along the splice have an effective area not less than $0.0015hs$, the lap splice length may be multiplied by 0.83, but the lap length shall not be less than 12 inches. The effective area of the ties shall be the area of the legs perpendicular to dimension h.

In compression members when spirals are used for lateral restraint along the splice, the lap splice length may be multiplied by 0.75, but the lap length shall not be less than 12 inches.

\+ **8.32.4.2** *Deleted*

\+ **8.32.4.3** *Deleted*

8.32.5 Splices of Welded Deformed Wire Fabric in Tension

8.32.5.1 The minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall not be less than $1.7l_d$ or 8 inches, and the overlap measured between the outermost cross wires of each fabric sheet shall not be less than 2 inches.

8.32.5.2 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 Article 8.32.3.1.

8.32.6 Splices of Welded Smooth Wire Fabric in Tension

\+ The minimum lap for lap splices of welded smooth wire
\+ fabric shall be such that the overlap between the outer-
\+ most cross wires of each fabric sheet is not less than the
\+ larger of 1.5 l_d, 6 inches, or the spacing of the cross wires
\+ plus 2 inches.

\+ **8.32.6.1** *Deleted*

\+ **8.32.6.2** *Deleted*

Section 8:
Reinforced Concrete Commentary

8.8.2 Research on rectangular columns with one-way flares (UCSD report # SSRP – 97/06: "Seismic performance of flared columns) has demonstrated that flared columns which have a separation gap between the bridge soffit and the top of the flares have better ductility than columns which have flares monolithic with the superstructure. In the UCSD tests, the monolithic flares were not fully confined as is typically required of columns.

In flared columns with gaps, only the column reinforcement was continued into the superstructure. These flares were tested with various degrees of confinement to minimize cracking and spalling under moderate seismic events as well as to prevent flare separation under stronger earthquakes.

On the basis of UCSD test results, gapped flares are the recommended choice.

8.12.3 The only requirement in AASHTO is for intermediate diaphragms to be spaced at a maximum of 40 feet for curved box-girder bridges having an inside radius of less than 800 feet. Caltrans requires a less stringent spacing for bridges on curves of moderate radii from construction considerations.

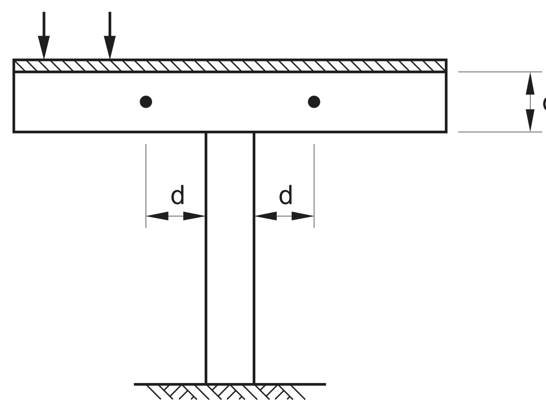
8.15.2.2 The stress limitation on Grade 60 rebars has been imposed in order to maintain the same stress levels in deck rebars that would be obtained using WSD.

8.15.5.5.5 Ties for Horizontal Shear

(c) AASHTO adopted to eliminate this article in 1992 to make deck replacement easier. Since such reinforcement improves the composite action between the deck and the girder, Caltrans has retained this article.

8.15.5.6 Special Provisions for Slabs and Footings

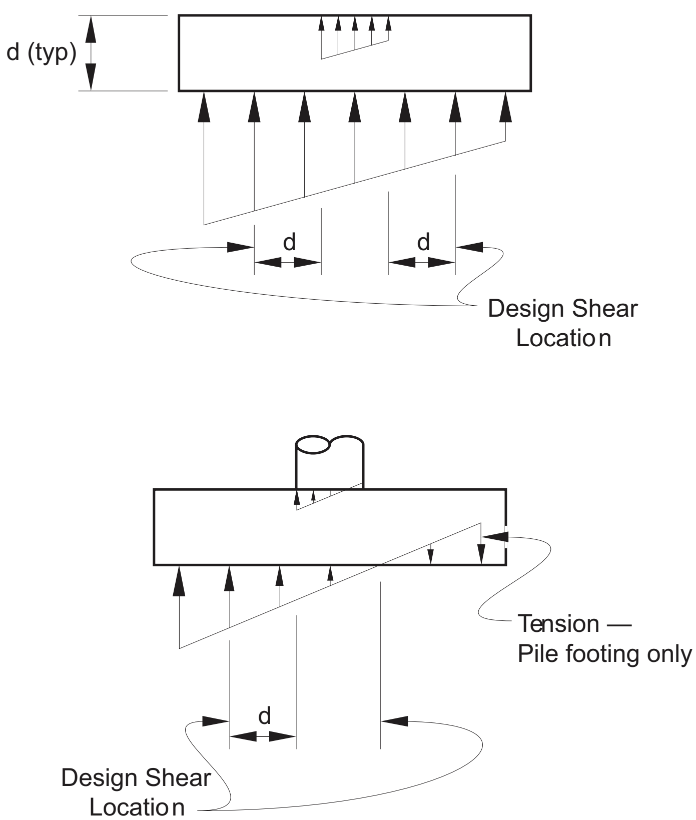
These provisions require the shear section for beam action to be at the column face when there is no compressive force. Articles 8.15.5.1.4 and 8.16.6.1.2 define the "Design Shear Section" as a point '*d*' from the face of support if compression is introduced into the "end region" of the member. For a single column bent, we design for shear at a point '*d*' from the column face (Figure C.8.15.5.6A).



Typical Single Column Bent

FIGURE C.8.15.5.6A Typical Single Column Bent

A pile footing is to be treated like an inverted 'T'. The footing then, is the shear member and the "end region" is the area where the column joins the footing. If the combined axial and moment loads produce compression on the joint area, the design shear section may be taken at a distance '*d*' from the column face. Otherwise, the section is designed for shear (Figure C.8.15.5.6B).



Typical Footings

FIGURE C.8.15.5.6B Typical Footings

8.16.1.2 Design Strength

Since ultimate strength is used for flexural design of footings for seismic forces, ϕ was increased to 1.00 for flexure in Group VII. The value of ϕ for Group VII forces in columns was increased by about 1.3 to take advantage of the overstrength capacity of well confined column members.

8.16.3.5 Flanged Sections with Compression Reinforcement

These equations are based on compatibility and equilibrium of cross sections (*Ref: Reinforced Concrete Structures: Park and Paulay*).

8.16.4.5 Probable Plastic Moment

In general, seismic analysis and design including calculation of probable plastic moment in a column, is discussed in the Caltrans Seismic Design Criteria. Earthquake forces could take a column to its yield capacity (probable plastic moment). The design details in a column must ensure that plastic hinging can occur. Forces in the columns and adjoining elements (example: superstructure and footing) are based on the column plastic moment, which in turn is based on potential overstrength capacity of the column materials (expected strength).

Generally, the probable plastic moment depends on the following four factors:

1) The actual size of the column and the actual amount of reinforcing.
2) The effect of increased f_y for both over-specification and for strain hardening effects.
3) The effect of increased f'_c for both over-specification and confinement provided by the transverse reinforcement. Also, the concrete will gradually increase in strength with time.
4) The effect of an actual ultimate compressive strain above 0.003.

Actual Size and Reinforcement Configuration

The Design Engineer should select the minimum column section and reinforcing steel structurally possible. As these parameters increase, the probable moment increases. That will lead to an increase in the foundation size and cost. The Engineer must also consider that column size will influence whether the column is functioning above or below balanced axial load P_b. For columns designed above P_b (for compression), the probable yield moment will usually be greater than that for the same section designed below P_b (for tension). A size and reinforcement selection which forces the design below P_b is preferable, especially in high seismic areas. However, the selection of size and reinforcement must meet the aesthetic requirements which may be the controlling factor. The designer should be actively involved in the aesthetic selection process to encourage the use of economical members.

Increase in Yield Strength of Reinforcement (f_y)

TransLab test data shows that the average yield strength of Grade 60 reinforcement is about 67,000 psi or about 12% over the minimum specified value. Combining this increase with an estimate of the effect of strain hardening beyond yield, it is realistic to assume f_y at 75,000 psi or 25% over minimum yield strength.

Increase in f'_c

Ref: Priestly, Park and Potanangaroa; ASCE STRUCTURAL JOURNAL, Jan. 1981.

f'_{cc} = enhanced f'_c due to confinement

$$f'_{cc} = f'_c \left(1 + 2.05\rho_s \frac{f_y}{f'_c}\right)$$

f'_c = unconfined strength

ρ_s = ratio of volume of spiral reinforcement to the volume of the core concrete

f_y = spiral steel yield strength

Assuming $\rho_s = 0.013$ (ACI 318/SEAOC/ATC Spec)

f'_{cc} = 3.25 (1 + 2.05 * 0.013 * 60/3.25)
= 4.85 ksi which is 1.49 f'_c.

or

f'_{cc} = 1.5 f'_c is a realistic estimate.

Ultimate Compressive Strain (ε)

Although tests on unconfined concrete show 0.003 as a reasonable strain at first crushing, tests on confined column sections show a marked increase in this value. Priestly (1981) found 0.0074 as a minimum average with 0.01 as an average. Blume, Newmark and Corning as well as Penzien [(Berkeley) EERC 75-19] also support a 0.01 value. Assume 0.01 as a realistic value.

As a rule of thumb, it is generally satisfactory to assume the probable plastic moment to be 1.3 times the yield moment for axial loads below P_b.

As shown on the "Probable Moment Capacity" plot (Figure C.8.16.4.5), a factor of 1.3 may be in considerable error for axial loads above P_b.

For computer generated "probable" plastic moments under high axial load conditions, the Engineer must compute the probable capacity using the increased realistic values (i.e., f'_c = 4,870 psi, f_y = 75,000 psi and ε = 0.01 for corresponding concrete design strength of 3,250 psi and steel yield strength of 60,000 psi).

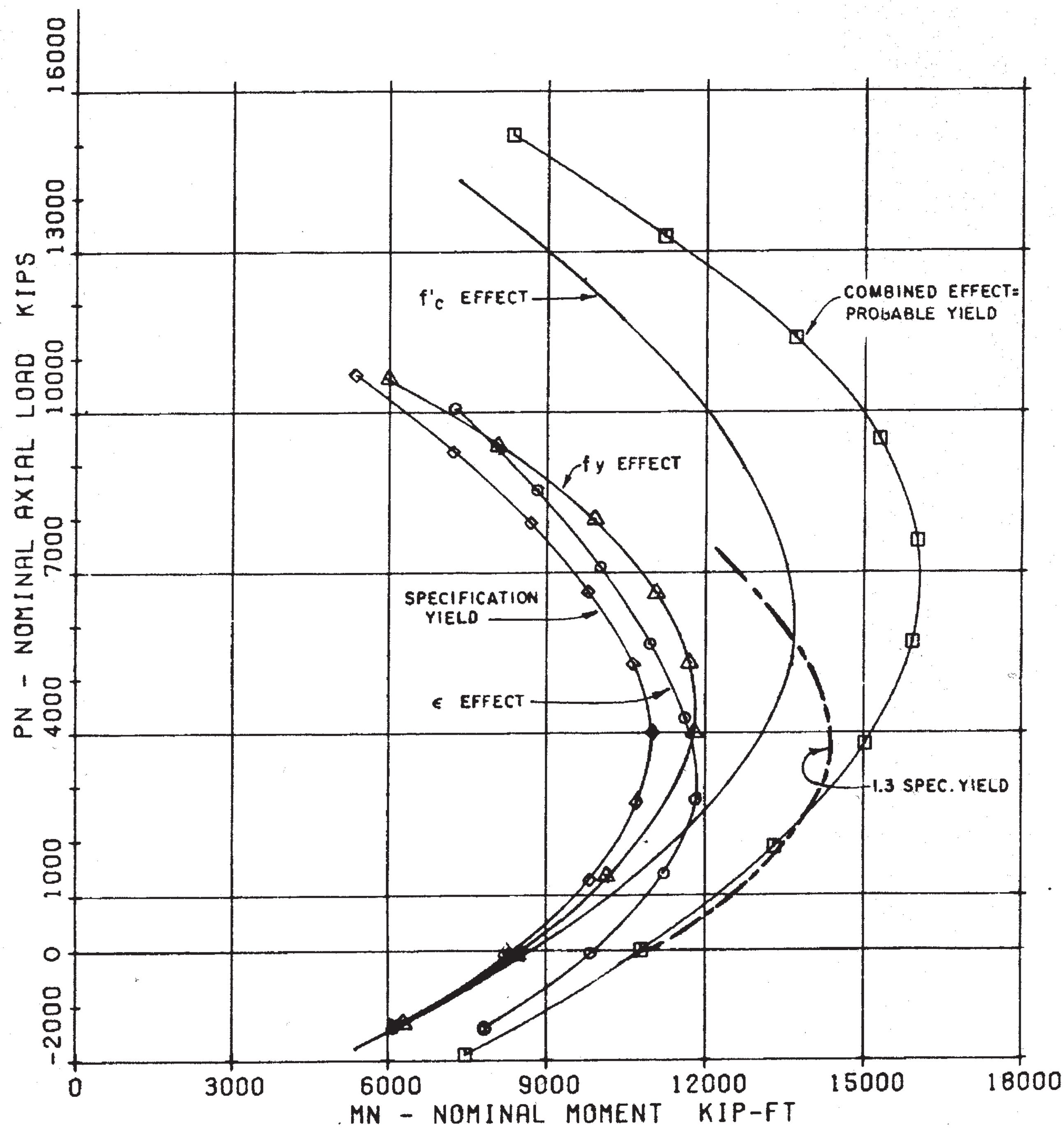


FIGURE C.8.16.4.5 Plot Showing Development of Probable Moment Capacity

5 feet – 6 inch diameter round column	f'_c = 3,250 psi to 4,870 psi
A_s = 44- No. 11 Bars (2%)	f_y = 60 ksi to 75 ksi
	ε = 0.003 to 0.01

8.16.4.6 Special Provisions for Column and Pier Wall Hinges

The traditional method of designing and detailing a pin connection at the base of a column is to group several large diameter rebars (reinforcing bars) at the center of the column. Caltrans' current design criteria of using elastic seismic forces or plastic hinging forces results in large shear forces. Typical rebar (reinforcing bars) clusters may have to be replaced by such devices as a cluster of rebars in a spiral cage, H-beam, cylindrical steel shell, etc., to develop the required shear capacity.

Oblong columns may perform as pinned columns about their weak axis, but may perform as fixed columns about their strong axis if not detailed properly. The Engineer should ensure that the designed connection matches the dynamic model. The connection must be fully developed on both sides of the interface between supported and supporting member. Concrete stress levels must be checked if the designed pin connection performs as a fixed connection about the strong axis.

When calculating the design strength for keys, assume the following conditions:

1) f'_c shall be the concrete strength of the supporting or supported member, which ever is less.

2) A_g shall be the contact area at the interface.

3) A_{st} shall be the area, or the vertical component of the area of the longitudinal rebars, crossing the interface and connecting the supporting and supported members.

8.16.6.5.5 Ties for Horizontal Shear

(c) See commentary for Article 8.15.5.5.5.

8.16.6.6 Special Provisions for Slabs and Footings

See commentary for Article 8.15.5.6.

8.16.6.7 Special Provisions for Box Culverts

Caltrans has provided a simplified version of the corresponding AASHTO equation. In addition to a more detailed equation, AASHTO provides a lower bound value of $3\sqrt{(f'_c)}$ for shear strength of slabs which are monolithic with the walls. However, Caltrans adopts this lower bound value as a default for slabs and walls. Caltrans also uses horizontal pressure distribution whose value exceeds that recommended by AASHTO, and a vertical pressure distribution whose value corresponds to the upper bound AASHTO value. Hence, Caltrans' equation provides a simple and conservative approach to the shear strength equation.

8.16.6.9 Special Provision for Pier Walls

The equations for determining shear strength are based on the corresponding equations in ACI-318 (1995). However, equations for shear strength based on more detailed calculations have been omitted. The upper bound on the shear strength has been added similar to the guidelines in AASHTO.

This specification also assures that pier walls are reinforced sufficiently to resist lateral forces in a direction parallel to its long dimension. Note that the foundation in this case may be subjected to very large forces. The foundation for piers in the transverse direction must be evaluated to assure that its capacity is less than or equal to the ultimate shear capacity of the pier wall. This approach will cause the foundation to act as a fuse and prevent a catastrophic failure of the pier wall. This is an exception to the requirements of 4.5.6.5.1. The foundation should be stable for large lateral loads.

In special circumstances, where the "fuse action" of the footing is limited (Example: Footing anchored in rock), alternate strategies such as isolation should be considered.

8.16.6.10 Compression Member Connection to Caps

The connection of columns to bent caps in a direction parallel to the bent, will generally be satisfied by other design criteria. However, in "I" or "T" girder bridges, or in dropped caps, the shear force in a direction normal to the bent must be developed. For "T" girders or "I" girders with integral bent caps, a partial slab or diaphragm may be necessary to satisfy plastic hinging requirements (Figure C.8.16.6.10). In many instances, additional shear reinforcement and/or an increased cap width will be satisfactory.

8.16.6.11 Special Seismic Provision for Columns, Pier Walls and Piles

The shear and confinement specifications were added to define the shear and confinement reinforcement in vertical support members, primarily at the location of plastic hinges. The design shear force in pile shafts shall consider the reduction in shear along the length of the shaft as the load is transferred to the surrounding soil.

8.16.8.4 Distribution of Flexural Reinforcement

Typically, the concrete cover to a reinforcing bar is increased to improve corrosion resistance of the structure. However, increasing the cover leads to an increase in the calculated crack width in the extreme tension fiber. Therefore, to comply with the serviceability requirements of Eq. (8-61), the amount of reinforcement has to be increased. In order to eliminate this penalty for providing increased

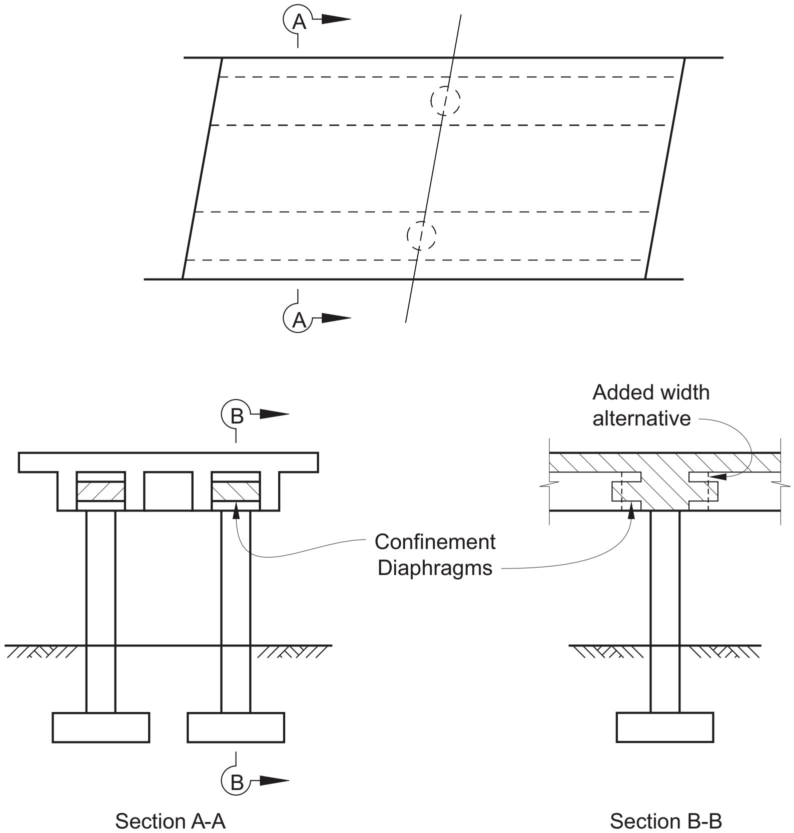


FIGURE C.8.16.6.10 Compression Member/Cap Connection

concrete cover based on other controlling specifications or practice, ACI committee 224 recommended that the value of concrete cover used to determine d_c in Eq.(8-61) be limited to 2 inches. Additional information is available in the following reference: "DEBATE: Crack width, cover and corrosion", Concrete International, May 1985.

The controlling value of steel stress f_s given by Eq. (8-61) need not be less than $0.4*f_y$ to be consistent with Service Load Design (SLD) practice. Under SLD, serviceability requirements of Load Factor Design are satisfied by default, and hence serviceability investigations are not required.

8.17.2.1.5 The additional reinforcement is provided to account for any unexpected settlement of falsework during construction.

8.18.1.4 Interlocking Spirals

In rectangular or oblong columns, confinement is provided through interlocking hoop/spirals. The maximum limitation for center-to-center spacing of the spirals was established by a geometrical relationship for stability normal to the bent. A minimum spacing of 0.50 times the spiral diameter is recommended to avoid overlaps of more than two spirals. Revise the column shape, size, number of columns, etc., to avoid a closer spacing (Figure C.8.18.1.4).

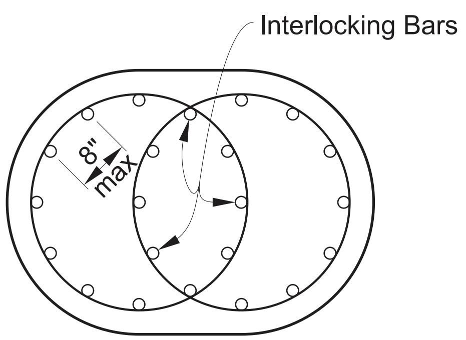


FIGURE C.8.18.1.4 Interlocking Spirals

Note: If "interlocking bars" are also used to provide column load capacity, then they must be fully developed into the cap and footing as required. If these bars are used solely for interlocking, then they shall extend into the cap and footing the same distance as that of the spirals.

8.18.2.2 Spiral Reinforcement

These changes have been incorporated (from the SEAOC recommendations and the New Zealand code on Design of Concrete Structures) to account for the axial load effects on the column. The equations ensure that axial load strength is preserved after cover concrete spalls, as well as ensure that adequate moment capacity is maintained with further plastic rotation.

Testing of reinforced columns at the University of Canterbury, New Zealand has shown that the required confinement of column reinforcement is directly proportional to the axial load applied. Test results in New Zealand have shown that satisfactory results are obtained by multiplying the generally accepted expressions for volumetric ratio ρ_s given by:

$$0.45\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_y} \qquad \text{[AASHTO]} \quad \text{(C-1)}$$

or

$$0.12\frac{f_c'}{f_y} \qquad \text{[SEAOC]} \quad \text{(C-2)}$$

by the expression

$$\left(0.5+1.25\frac{P_e}{f_c'A_g}\right) \qquad \text{(C-3)}$$

The revised specification provides that the volumetric ratio, ρ_s, shall not be less than:

$$0.45\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_y}\left(0.5+1.25\frac{P_e}{f_c'A_g}\right) \qquad \text{(C-4)}$$

for columns less than 3 feet in dimension

or

$$0.12\frac{f_c'}{f_y}\left(0.5+1.25\frac{P_e}{f_c'A_g}\right) \qquad \text{(C-5)}$$

for columns larger than 3 feet in dimension;

but, not less than

$$0.45\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_y} \qquad \text{(C-6)}$$

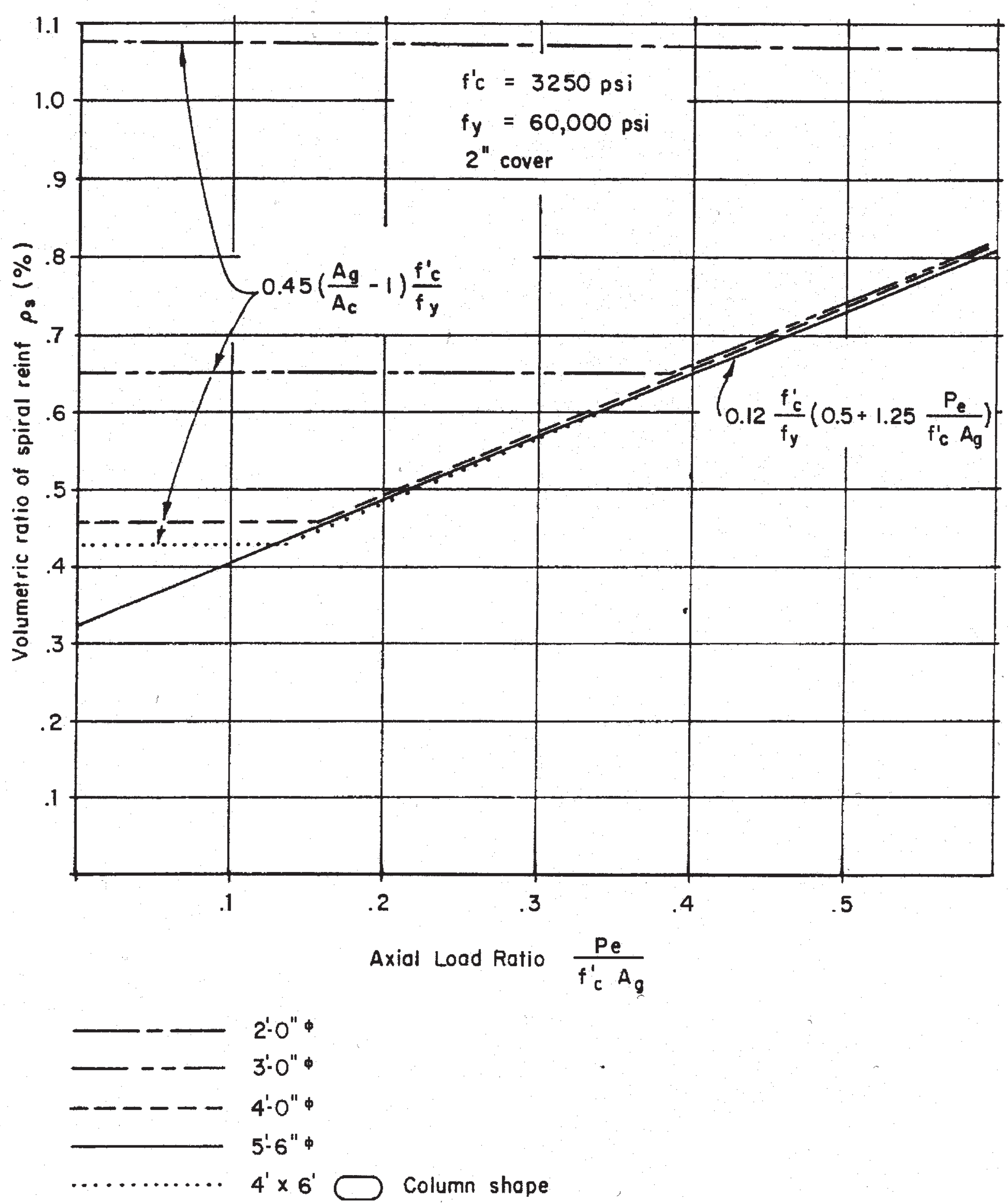


Figure 8.18.2.2 Spiral (or Hoop) Reinforcement

Generally,with $f_c' = 3250$ psi, $f_y = 60{,}000$ psi, 2 inches of cover, and prismatic sections less than 3 feet in diameter, Equation (C-6) will control; between 3 feet and 5 feet - 6 inches, either Equation (C-4) or Equation (C-6) will control depending on the axial load; and for sections greater than 5 feet-6 inches, Equation (C-5) will control. The following plot of ρ_s vs. $P_e /(f_c' A_g)$ shows this relationship (Figure C.8.18.2.2).

The revised specifications are a compromise between the New Zealand recommendation and AASHTO/SEAOC; where the New Zealand recommendation is below AASHTO, the AASHTO spec is retained. For low axial load ratio on large columns, the New Zealand recommendation usually governs. The Caltrans specifications ensure that the volumetric ratio of spiral reinforcement shall not be less than that required by AASHTO in any case.

8.18.2.3 Ties

Specifications for ties now include confinement requirements similar to spirals. These requirements were originally taken from the 1983 AASHTO Seismic Design guidelines and amended for column axial load in accordance with the New Zealand code. The AASHTO cross tie specifications were modified to generally conform to ACI guidelines. The following sketches (Figure 8.18.2.3) identify some acceptable tie arrangements. It is strongly recommended for confinement and construction reasons that spirals be used in lieu of ties wherever possible. The labor requirements to assemble ties for such a column is enormous. In addition, access for inspection is almost impossible. Since spirals/hoops and interlocking spirals/hoops are more economical, and provide a more effective confinement, these are the typical types of transverse column reinforcement.

The sentence on the use of deformed wire or welded wire fabric instead of bars has been removed from the corresponding AASHTO article. Typically, such wires do not perform adequately from fatigue considerations and their use in structural concrete is not recommended.

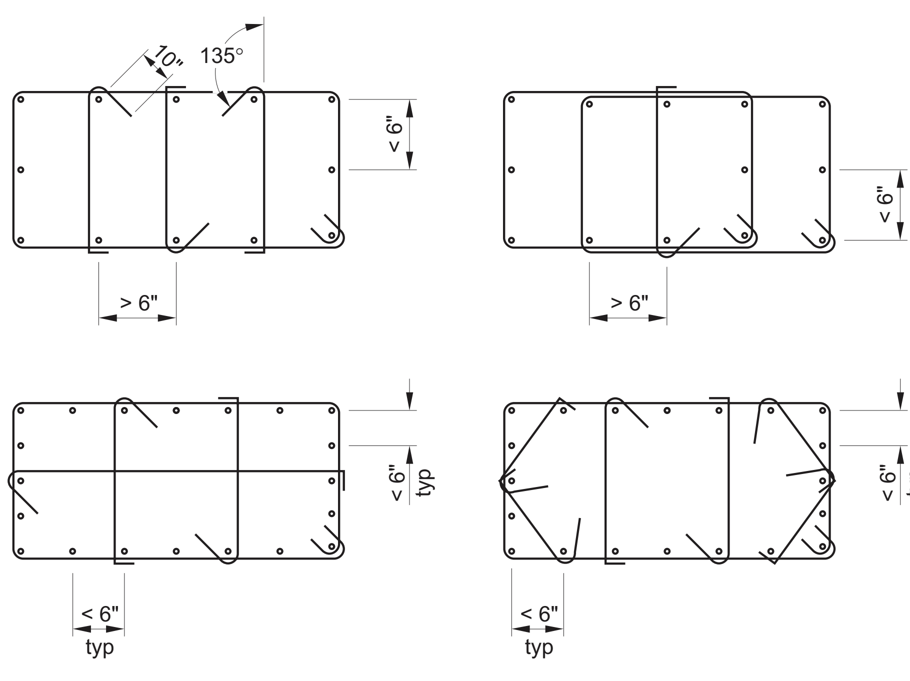


FIGURE C.8.18.2.3 Ties

8.21 SPACING LIMITS FOR REINFORCEMENT

8.21.1.1 This recommendation is based on the guidelines in the New Zealand code for the Design of Concrete Structures as well as SEAOC. The spacing limitation on transverse reinforcement ensures adequate confinement of core concrete in potential plastic hinge zones and provides for restraint against buckling of longitudinal bars.

8.21.1.2 Limitations are introduced for minimum bar sizes and maximum bar spacings of longitudinal reinforcing bars to help retain the shape of the lateral reinforcement and to confine the concrete core (Figure C.8.21.1.2). In addition, this spacing requirement ensures that the bars are distributed reasonably uniformly around the perimeter of a column in potential plastic hinge zone.

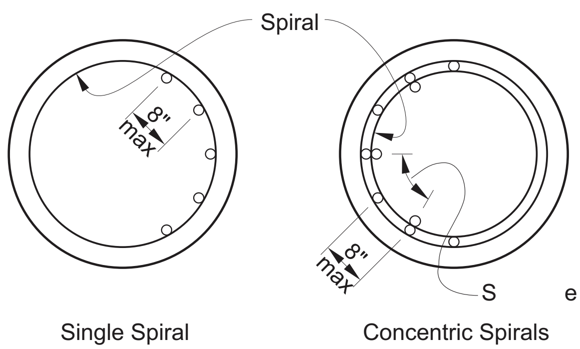


FIGURE C.8.21.1.2 Spacing of Reinforcement

Note: The maximum spacing between longitudinal reinforcement in the inner circle equals twice that in the outer circle. If the inner circle is required for confinement, the spacing between longitudinal bars of the inner circle should not exceed 8 inches. It is a better practice to provide an equal number of bars in each circle.

8.21.7 The Design Specification is required to conform to the SSP for Cast-in-Place concrete piles (SSP 49-310). This SSP permits the contractor to construct cast-in-drilled-hole (CIDH) piles by water or slurry displacement methods for piles with a diameter greater than or equal to 24 inches, when caving and water cannot be controlled by temporary casing. CIDH piles with a diameter greater than or equal to 24 inches require vibration only in the upper 6 feet of the pile when constructed in wet conditions. CIDH piles with a diameter less 24 inches and CIP concrete piles in steel shells require vibration in the upper 15 feet of the pile.

The specifications require an increase in the clear distance between the reinforcement to permit free flow of concrete around the reinforcing bars, and against the steel shell or earth in areas where the concrete is not vibrated. The range of allowable nominal penetration for the concrete has been increased to achieve this free flow of concrete. The Standard Specifications require the minimum concrete strength to be 3600 psi, and this is considered as concrete designated by compressive strength (trail batch required). For additional information refer to the following specifications; 49-310, 49CISS, 49CEND and 49SLUR.

8.22 PROTECTION AGAINST CORROSION

The table for minimum concrete cover for protection against corrosion has been developed for a 75-year design life. However, the service life of bridge decks and barrier rails are typically less than 75 years. Therefore, the concrete mix design and cover requirements for corrosion protection of decks and barrier rails have incorporated these aspects.

Environmental conditions such as proximity to corrosive atmosphere, marine environment, wave action, water table elevation and chloride content have been incorporated in determining the cover requirements.

Corrosion protection can be improved by increasing concrete denseness or imperviousness to water, as well as by furnishing other protection methods. Such methods include:

a) a reduction in water-to-cementitious material ratio;
b) use of 25% mineral admixture conforming to ASTM Designation C618 Type F or N;
c) use of 5% mineral admixture conforming to ASTM Designation C 1240 with 20% mineral admixture conforming to ASTM Designation C618 Type F or N, in lieu of 25% mineral admixture conforming to ASTM Designation C618 Type F or N.
d) use of different kinds of epoxy coatings for reinforcing bars;
e) protective concrete coatings;
f) use of chemical admixtures;
g) cathodic protection, and,
h) use of alternate materials.

The minimum concrete cover, concrete mix and epoxy-coated reinforcement requirements for structural elements exposed to deicing salt, snow run-off or snow blower spray shall be adopted only if the Engineer determines that the structural elements are directly exposed to these corrosive conditions. For example, when the deck is subjected to de-icing salt, snow run-off or snow blower spray, it is unlikely that the girders or bent caps will be exposed to the same harsh conditions, particularly when there are no deck-joints. Therefore, the girders and the bent caps may be designed for a non-corrosive exposure condition.

If other considerations, such as a need to reduce the dead load of a structure, require a further reduction in concrete cover than those specified in Table 8.22.1, then a reduction in cover should only be done after a thorough investigation and research into existing state-of-practice.

8.29.3.6 Per ACI-318 (1995), the 20 % increase in development length is provided to account for reduced bond when reinforcement is epoxy-coated.