Section 10
STRUCTURAL STEEL

Part A
GENERAL REQUIREMENTS AND MATERIALS

10.1 APPLICATION

10.1.1 Notations

A = area of cross section (Articles 10.37.1.1, 10.34.4, 10.48.1.1, 10.48.2.1, 10.48.4.2, 10.48.5.3, and 10.55.1)
A = bending moment coefficient (Article 10.50.1.1.2)
A_e = effective area of a flange or splice plate with holes or a tension member with holes (Articles 10.12, 10.18.2.2.1, 10.18.2.2.3, 10.18.2.2.4, and 10.18.4.1)
A_F = amplification factor (Articles 10.37.1.1 and 10.55.1)
$(AF_y)_{bf}$ = product of area and yield point for bottom flange of steel section (Article 10.50.1.1.1)
$(AF_y)_c$ = product of area and yield point of that part of reinforcing which lies in the compression zone of the slab (Article 10.50.1.1.1)
$(AF_y)_{tf}$ = product of area and yield point for top flange of steel section (Article 10.50.1.1.1)
$(AF_y)_w$ = product of area and yield point for web of steel section (Article 10.50.1.1.1)
A_f = area of flange (Articles 10.39.4.4.2, 10.48.2.1, 10.53.1.2, and 10.56.3)
A_f = the sum of the area of filler plates on the top and bottom of the connected plate (Article 10.18.1.2.1)
A_{fc} = area of compression flange (Articles 10.48.4.1 and 10.50.1.2.1)
A_g = gross area of a flange, splice plate or tension member (Articles 10.18.2.2.2, 10.18.2.2.4, and 10.18.4.1)
A_n = net section of a tension member (Article 10.18.4.1)
A_p = the smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (Article 10.18.1.2.1)
A_s^r = total area of longitudinal reinforcing steel at the interior support within the effective flange width (Article 10.38.5.1.2)
A_r^s = total area of longitudinal slab reinforcement steel for each beam over interior support (Article 10.38.5.1.3)
A_s = area of steel section (Articles 10.38.5.1.2, 10.54.1.1, and 10.54.2.1)
A_{sc} = cross-sectional area of a stud shear connector (Article 10.38.5.1.2)
A_w = area of web of beam (Article 10.53.1.2)
a = distance from center of bolt under consideration to edge of plate, in. (Articles 10.32.3.3.2 and 10.56.2)
a = spacing of transverse stiffeners (Article 10.39.4.4.2)
a = depth of stress block (Figure 10.50A)
a = ratio of numerically smaller to the larger end moment (Article 10.54.2.2)
B = constant based on the number of stress cycles (Article 10.38.5.1.1)
B = constant for stiffeners (Articles 10.34.4.7 and 10.48.5.3)
b = compression flange width (Table 10.32.1A and Articles 10.34.2.1, 10.48, 10.48.1.1, 10.48.2, 10.48.2.1, and 10.61.4)
b = distance from center of bolt under consideration to toe of fillet of connected part, in. (Articles 10.32.3.3.2 and 10.56.2)
b = effective width of slab (Article 10.50.1.1.1)
b = effective flange width (Articles 10.38.3 and 10.38.5.1.2)
b = widest flange width (Article 10.15.2.1)
b = distance from edge of plate or edge of perforation to the point of support (Article 10.35.2.3)
b = unsupported distance between points of support (Article 10.35.2.7)
b = flange width between webs (Articles 10.37.3.1, 10.39.4.2, 10.51.5.1, and 10.55.3)

b' = width of stiffeners (Articles 10.34.5.2, 10.34.6, 10.37.2.4, 10.39.4.5.1, and 10.55.2)
b' = width of a projecting flange element, angle, or stiffener (Articles 10.34.2.2, 10.34.4.7, 10.37.3.2, 10.39.4.5.1, 10.48.5.3, 10.51.5.5, and 10.55.3)
C = web buckling coefficient (Articles 10.34.4, 10.38.1.7, 10.48.5.3, and 10.48.8)
C = compressive force in the slab (Article 10.50.1.1.1)
C = equivalent moment factor (Article 10.54.2.1)
C' = compressive force in top portion of steel section (Article 10.50.1.1.1)
C_b = bending coefficient (Table 10.32.1A and Articles 10.48.4.1 and 10.50.2.2)
C_c = column slenderness ratio dividing elastic and inelastic buckling (Table 10.32.1A)
C_{mx} = coefficient about X axis (Article 10.36)
C_{my} = coefficient about the Y axis (Article 10.36)
c = buckling stress coefficient (Article 10.51.5.2)
D = clear distance between flanges, in. (Article 10.15.2)
D = clear unsupported distance between flange components (Articles 10.18.2.3.4, 10.18.2.3.7, 10.18.2.3.8, 10.18.2.3.9, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48.1, 10.48.2, 10.48.4, 10.48.5, 10.48.6, 10.48.8, 10.49.2, 10.49.3.2, 10.50.1.1.2, 10.50.2.1, 10.55.2, and 10.61.1)
D' = distance from the top of the slab to the neutral axis at which a composite section in positive bending theoretically reaches its plastic-moment capacity when the maximum strain in the slab is at 0.003 (Article 10.50.1.1.2)
D_c = clear distance between the neutral axis and the compression flange (Articles 10.34.3.2.1, 10.34.5.1, 10.48.4.1, 10.49.2, 10.49.3, 10.50(b), 10.57, and 10.61.1)
D_c = moments caused by dead load acting on composite girder (Article 10.50.1.2.2)
D_{cp} = depth of the web in compression at the plastic moment (Articles 10.50(b), 10.50.1.1.2, and 10.50.2.1)
D_{cs} = depth of the web in compression of the non-composite steel beam or girder (Articles 10.34.5.1 and 10.49.3.2(a))
D_p = distance from the top of the slab to the plastic neutral axis, in. (Article 10.50.1.1.2)
D_s = moments caused by dead load acting on steel girder (Article 10.50.1.2.2)
d = bolt diameter (Table 10.32.3B)
d = diameter of stud, in. (Article 10.38.5.1)
d = depth of beam or girder, in. (Table 10.32.1A and Articles 10.13, 10.48.2, 10.48.4.1, and 10.50.1.1.2)
d = diameter of rocker or roller, in. (Article 10.32.4.2)
d_b = beam depth (Article 10.56.3)
d_c = column depth (Article 10.56.3)
d_o = spacing of intermediate stiffener (Articles 10.34.4, 10.34.5, 10.48.5.3, 10.48.6.3, and 10.48.8)
d_s = distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component (Articles 10.34.3.2.1, 10.34.5.1, 10.48.4.1, 10.49.3.2(a), and 10.61.1)
E = modulus of elasticity of steel, psi (Table 10.32.1A and Articles 10.15.3, 10.36, 10.37, 10.39.4.4.2, 10.54.1, and 10.55.1)
E_c = modulus of elasticity of concrete, psi (Article 10.38.5.1.2)
e = distance from the centerline of a splice to the centroid of the connection on the side of the joint under consideration (Articles 10.18.2.3.3, 10.18.2.3.5, and 10.18.2.3.7)
F = maximum induced stress in the bottom flange (Article 10.20.2.1)
F = maximum compressive stress, psi (Article 10.41.4.6)
F_a = allowable axial unit stress (Table 10.32.1A and Articles 10.36, 10.37.1.2, and 10.55.1)
F_b = allowable bending unit stress (Table 10.32.1A and Articles 10.18.2.2.3, 10.37.1.2, and 10.55.1)
F_{bx} = compressive bending stress permitted about the X axis (Article 10.36)
F_{by} = compressive bending stress permitted about the Y axis (Article 10.36)
F_{cr} = buckling stress of the compression flange plate or column (Articles 10.48.2, 10.50.2.2, 10.51.1, 10.51.5, 10.54.1.1, and 10.54.2.1)
F_{cr} = local buckling stress of a stiffener (Articles 10.34.4.7 and 10.48.5.3)
F_{cf} = design stress for the controlling flange at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8)
F_{cu} = design stress for the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4)
F_D = maximum horizontal force (Article 10.20.2.2)
F_e = Euler buckling stress (Articles 10.37.1, 10.54.2.1, and 10.55.1)

F'_e = Euler stress divided by a factor of safety (Article 10.36)

F_{ncf} = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.3)

F_{ncu} = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.1)

F_p = computed bearing stress due to design load (Table 10.32.3B)

F_s = limiting bending stress (Article 10.34.4)

F_{sr} = allowable range of stress (Table 10.3.1A)

F'_t = reduced allowable tensile stress on rivet or bolt due to the applied shear stress, ksi (Articles 10.32.3.3.4 and 10.56.1.3.3)

F^r_y = specified minimum yield point of the reinforcing steel (Article 10.38.5.1.2)

F.S. = factor of safety (Table 10.32.1A and Articles 10.32.1 and 10.36)

F_u = specified minimum tensile strength (Tables 10.2A, 10.32.1A and 10.32.3B and Article 10.18.4)

F_u = tensile strength of electrode classification (Table 10.56A and Article 10.32.2)

F_u = maximum bending strength of the flange (Articles 10.48.8.2, 10.50.1.2.1, and 10.50.2.2)

F_v = allowable shear stress (Table 10.32.1A and 10.32.3B and Articles 10.18.2.3.6, 10.32.2, 10.32.3, 10.34.4, 10.38.1.7, and 10.40.2.2)

F_v = shear strength of a fastener (Article 10.56.1.3)

F_{vc} = combined tension and shear in bearing-type connections (Article 10.56.1.3)

F_w = design shear stress in the web at a point of splice (Articles 10.18.2.3.6, 10.18.2.3.7, and 10.18.2.3.9)

F_y = specified minimum yield point of steel (Articles 10.15.2.1, 10.15.3, 10.16.11, 10.32.1, 10.32.4, 10.34, 10.35, 10.37.1.3, 10.38.1.7, 10.38.5, 10.39.4, 10.40.2.2, 10.41.4.6, 10.46, 10.48, 10.49, 10.50, 10.51.5, 10.54, and 10.61.4)

F_{yf} = specified minimum yield strength of the flange (Articles 10.18.2.2.1, 10.48.1.1, 10.53.1, 10.57.1, and 10.57.2)

$F_{y\,stiffener}$ = specified minimum yield strength of a transverse stiffener (Articles 10.34.4.7 and 10.48.5.3)

F_{yw} = specified minimum yield strength of the web (Articles 10.18.2.2.1, 10.18.2.2.2, 10.18.2.3.4, 10.53.1, and 10.61.1)

$F_{y\,web}$ = specified mimimum yield strength of the web (Articles 10.34.4.7 and 10.48.5.3)

f = the lesser of (f_b/R_b) or F_y (Articles 10.48.2.1(b), 10.48.2.2, and 10.53)

f_a = computed axial compression stress (Articles 10.35.2.10, 10.36, 10.37, 10.55.2, and 10.55.3)

f_b = computed compressive bending stress (Articles 10.34.2, 10.34.3, 10.34.5.2, 10.37, 10.39, and 10.55)

f_b = factored bending stress in the compression flange (Articles 10.48, 10.48.2.1(b), 10.48.4.1, 10.50.1.2.1, 10.50.2.2, 10.53, and 10.53.1.2)

f_b = maximum factored noncomposite dead load compressive bending stress in the web (Article 10.61.1)

f'_c = unit ultimate compressive strength of concrete as determined by cylinder tests at age of 28 days, psi (Articles 10.38.1, 10.38.5.1.2, 10.45.3, and 10.50.1.1.1)

f_{cf} = maximum flexural stress at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8)

f_{cu} = maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4)

f_{DL} = noncomposite dead load stress in the compression flange (Articles 10.34.5.1 and 10.49.3.2(a))

f_{DL} = top flange compressive stress due to the factored noncomposite dead load divided by the factor R_b (Article 10.61.4)

f_{DL+LL} = total noncomposite and composite dead-load plus composite live-load stress in the compression flange at the most highly stressed section of the web (Articles 10.34.5.1 and 10.49.3.2(a))

$f_{d\ell 1}$ = top flange compressive stress due to noncomposite dead load (Articles 10.34.2.1 and 10.34.2.2)

f_{ncf} = flexural stress at the mid-thickness of the noncontrolling flange concurrent with f_{cf} (Articles 10.18.2.2.3 and 10.18.2.3.8)

f_{ncu} = flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at a point of splice concurrent with f_{cu} (Articles 10.18.2.2.1 and 10.18.2.3.4)

f_o = maximum flexural stress due to $D + \beta_L(L + I)$ at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.2 and 10.18.2.3.5)

f_{of} = flexural stress due to $D + \beta_L(L + I)$ at the mid-thickness of the other flange at a point of splice concurrent with f_o in the flange under consideration (Article 10.18.2.3.5)

f_r = range of stress due to live load plus impact, in the slab reinforcement over the support (Article 10.38.5.1.3)

f_r = modulus of rupture of concrete specified in Article 8.15.2.1.1 (Article 10.50.2.3)

f_s = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener (Article 10.39.4.4)

f_s = factored bending stress in either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_u) (Article 10.48.8.2)

f_t = tensile stress due to applied loads (Articles 10.32.3.3.3 and 10.56.1.3.2)

f_t = allowable tensile stress in the concrete specified in Article 8.15.2.1.1 (Article 10.38.4.3)

f_v = unit shear stress (Articles 10.32.3.2.3, 10.34.4.4, and 10.34.4.7)

f_v = maximum shear stress in the web at a point of splice (Article 10.18.2.3.6)

f_{bx} = computed compressive bending stress about the x axis (Article 10.36)

f_{by} = computed compressive bending stress about the y axis (Article 10.36)

g = gage between fasteners, in. (Articles 10.16.14, 10.24.5, and 10.24.6)

H = height of stud, in. (Article 10.38.5.1.1)

H_w = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.8 and 10.18.2.3.9)

H_{wo} = overload horizontal design force resultant in the web at a point of splice (Article 10.18.2.3.5)

H_{wu} = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.4 and 10.18.2.3.5)

h = average flange thickness of the channel flange, in. (Article 10.38.5.1.2)

I = moment of inertia, in.^{4} (Articles 10.34.4, 10.34.5, 10.38.5.1.1, 10.48.5.3, and 10.48.6.3)

I_s = moment of inertia of stiffener (Articles 10.37.2, 10.39.4.4.1, and 10.51.5.4)

I_t = moment of inertia of transverse stiffeners (Article 10.39.4.4.2)

I_y = moment of inertia of member about the vertical axis in the plane of the web, in^4 (Article 10.48.4.1)

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in^4 (Table 10.32.1A and Article 10.48.4.1)

J = required ratio of rigidity of one transverse stiffener to that of the web plate (Articles 10.34.4.7 and 10.48.5.3)

J = St. Venant torsional constant, in^4 (Table 10.32.1A and Article 10.48.4.1)

K = effective length factor in plane of buckling (Table 10.32.1A and Articles 10.37, 10.54.1, and 10.54.2)

K_b = effective length factor in the plane of bending (Article 10.36)

k = constant: 0.75 for rivets; 0.6 for high-strength bolts with thread excluded from shear plane (Article 10.32.3.3.4)

k = buckling coefficient (Articles 10.34.3.2.1, 10.34.4, 10.39.4.3, 10.48.4.1, 10.48.8, 10.51.5.4, and 10.61.1)

k = distance from outer face of flange to toe of web fillet of member to be stiffened (Article 10.56.3)

k_1 = buckling coefficient (Article 10.39.4.4)

L = distance between bolts in the direction of the applied force (Table 10.32.3B)

L = actual unbraced length (Table 10.32.1A and Articles 10.7.4, 10.15.3, and 10.55.1)

L = 1/2 of the length of the arch rib (Article 10.37.1)

L = distance between transverse beams (Article 10.41.4.6)

L_b = unbraced length (Table 10.48.2.1.A and Articles 10.36, 10.48.1.1, 10.48.2.1, 10.48.4.1, and 10.53.1.3)

L_c = length of member between points of support, in. (Article 10.54.1.1)

L_c = clear distance between the holes, or between the hole and the edge of the material in the direction of the applied bearing force, in. (Table 10.32.3B and Article 10.56.1.3.2)

L_p = limiting unbraced length (Article 10.48.4.1)

L_r = limiting unbraced length (Article 10.48.4.1)

ℓ = member length (Table 10.32.1A and Article 10.35.1)

M = maximum bending moment (Articles 10.48.8, 10.54.2.1, and 10.50.1.1.2)

M_1 = smaller moment at the end of the unbraced length of the member (Article 10.48.1.1(c))

M_1 & M_2 = moments at two adjacent braced points (Tables 10.32.1A and 10.36A and Articles 10.48.4.1 and 10.50.2.2)

M_c = column moment (Article 10.56.3.2)

M_p = full plastic moment of the section (Articles 10.50.1.1.2 and 10.54.2.1)

M_r = lateral torsional buckling moment or yield moment (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, and 10.53.1.3)

M_s = elastic pier moment for loading producing maximum positive moment in adjacent span (Article 10.50.1.1.2)

M_u = maximum bending strength (Articles 10.18.2.2.1, 10.48, 10.49, 10.50.1, 10.50.2, 10.51.1, 10.53.1, 10.54.2.1, and 10.61.3)

M_v = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.7 and 10.18.2.3.9)

M_{vo} = overload design moment due to the eccentricity of the overload design shear at a point of splice (Article 10.18.2.3.5)

M_{vu} = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.3 and 10.18.2.3.5)

M_w = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.8 and 10.18.2.3.9)

M_{wo} = overload design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Article 10.18.2.3.5)

M_{wu} = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.4 and 10.18.2.3.5)

M_y = moment capacity at first yield (Articles 10.18.2.2.1, 10.50.1.1.2, and 10.61.3)

N_1 & N_2 = number of shear connectors (Article 10.38.5.1.2)

N_c = number of additional connectors for each beam at point of contraflexure (Article 10.38.5.1.3)

N_s = number of slip planes in a slip-critical connection (Articles 10.32.3.2.1 and 10.57.3.1)

N_w = number of roadway design lanes (Article 10.39.2)

n = ratio of modulus of elasticity of steel to that of concrete (Article 10.38.1)

n = number of longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4)

P = allowable compressive axial load on members (Article 10.35.1)

P = axial compression on the member (Articles 10.48.1.1, 10.48.2.1, and 10.54.2.1)

P, P_1, P_2, & P_3 = force in the slab (Article 10.38.5.1.2)

P_{cf} = design force in the controlling flange at a point of splice (Article 10.18.2.2.3)

P_{cu} = design force for the controlling flange at a point of splice (Article 10.18.2.2.1)

P_{fo} = overload design force in the flange at a point of splice (Article 10.18.2.2.2)

P_{ncf} = design force for the noncontrolling flange at a point of splice (Article 10.18.2.2.3)

P_{ncu} = design force in the noncontrolling flange at a point of splice (Article 10.18.2.2.1)

P_o = design force for checking slip of a bolted splice in a tension member (Article 10.18.4.2)

P_s = allowable slip resistance (Article 10.32.3.2.1)

P_u = maximum axial compression capacity (Article 10.54.1.1)

P_u = design force for checking the strength of a bolted splice in a tension member (Article 10.18.4.1)

p = allowable bearing (Article 10.32.4.2)

Q = prying tension per bolt (Articles 10.32.3.3.2 and 10.56.2)

Q = statical moment about the neutral axis (Article 10.38.5.1.1)

R = radius (Article 10.15.2.1)

R = number of design lanes per box girder (Article 10.39.2.1)

R = reduction factor for hybrid girders (Articles 10.18.2.2.1, 10.18.2.2.2, 10.18.2.2.3, 10.18.2.3.4, 10.18.2.3.8, 10.40.2.1.1, 10.53.1.2, and 10.53.1.3)

R = reduction factor applied to the design shear strength of fasteners passing through fillers (Article 10.18.1.2.1)

R_b = bending capacity reduction factor (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, 10.53.1.2, 10.53.1.3, and 10.61.4)

R_{cf} = absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8)

R_{cu} = the absolute value of the ratio of F_{cu} to f_{cu} for the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4)

Rev = a range of stress involving both tension and compression during a stress cycle (Table 10.3.1B)

R_s = vertical force at connections of vertical stiffeners to longitudinal stiffeners (Article 10.39.4.4.8)

R_w = vertical web force (Article 10.39.4.4.7)

r = radius of gyration, in (Articles 10.35.1, 10.37.1, 10.41.4.6, 10.48.6.3, 10.54.1.1, 10.54.2.1, and 10.55.1)

r_b = radius of gyration in plane of bending, in. (Article 10.36)

r_y = radius of gyration with respect to the Y-Y axis, in. (Article 10.48.1.1)

r' = radius of gyration of the compression flange about the axis in the plane of the web, in. (Table 10.32.1A and Article 10.48.4.1)

S = allowable rivet or bolt unit stress in shear (Article 10.32.3.3.4)

S = section modulus, $in.^3$ (Articles 10.48.2, 10.51.1, 10.53.1.2, and 10.53.1.3)

S = pitch of any two successive holes in the chain (Article 10.16.14.2)

S_r = range of horizontal shear (Article 10.38.5.1.1)

S_s = section modulus of transverse stiffener, in.^{3} (Articles 10.39.4.4 and 10.48.6.3)

S_t = section modulus of longitudinal or transverse stiffener, in.^{3} (Article 10.48.6.3)

S_u = ultimate strength of the shear connector (Article 10.38.5.1.2)

S_{xc} = section modulus with respect to the compression flange, in.^{3} (Table 10.32.1A and Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2 and 10.53.1.2)

S_{xt} = section modulus with respect to the tension flange, in.^{3} (Articles 10.48.2 and 10.53.1.2)

s = computed rivet or bolt unit stress in shear (Article 10.32.3.3.4)

T = range in tensile stress (Table 10.3.1B)

T = direct tension per bolt due to external load (Articles 10.32.3 and 10.56.2)

T = arch rib thrust at the quarter point from dead+live+impact loading (Articles 10.37.1 and 10.55.1)

t = thickness of the thinner outside plate or shape (Article 10.35.2)

t = thickness of members in compression (Article 10.35.2)

t = thickness of thinnest part connected, in (Articles 10.32.3.3.2 and 10.56.2)

t = computed rivet or bolt unit stress in tension, including any stress due to prying action (Article 10.32.3.3.4)

t = thickness of the wearing surface, in. (Article 10.41.2)

t = flange thickness, in. (Articles 10.18.2.2.4, 10.34.2.1, 10.34.2.2, 10.39.4.2, 10.48, 10.48.1.1, 10.48.2, 10.48.2.1, 10.51.5.1, and 10.61.4)

t = thickness of a flange angle (Article 10.34.2.2)

t = thickness of the web of a channel, in. (Article 10.38.5.1.2)

t = thickness of stiffener (Articles 10.34.4.7 and 10.48.5.3)

t_b = thickness of flange delivering concentrated force (Article 10.56.3.2)

t_c = thickness of flange of member to be stiffened (Article 10.56.3.2)

t_f = thickness of the flange (Articles 10.37.3, 10.55.3, and 10.39.4.3)

t_h = thickness of the concrete haunch above the beam or girder top flange (Article 10.50.1.1.2)

t_s = thickness of stiffener (Article 10.37.2 and 10.55.2)

t_s = slab thickness (Articles 10.38.5.1.2, 10.50.1.1.1, and 10.50.1.1.2)

t_w = web thickness, in. (Articles 10.15.2.1, 10.18.2.3.4, 10.18.2.3.7, 10.18.2.3.8, 10.18.2.3.9, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48, 10.49.2, 10.49.3, 10.55.2, 10.56.3, and 10.61.1)

t_{tf} = thickness of top flange (Article 10.50.1.1.1)

t' = thickness of outstanding stiffener element (Articles 10.39.4.5.1 and 10.51.5.5)

V = shearing force (Articles 10.35.1, 10.48.5.3, 10.48.8, and 10.51.3)

V = maximum shear in the web at a point of splice due to the factored loads (Article 10.18.2.3.2)

V_o = maximum shear in the web at the point of splice due to $D + \beta_L (L + I)$ (Article 10.18.2.3.5)

V_p = shear yielding strength of the web (Articles 10.48.8 and 10.53.1.4)

V_r = range of shear due to live loads and impact, kips (Article 10.38.5.1.1)

V_u = maximum shear force (Articles 10.18.2.3.2, 10.34.4, 10.48.5.3, 10.48.8, and 10.53.3)

V_v = vertical shear (Article 10.39.3.1)

V_w = design shear for a web (Articles 10.39.3.1 and 10.51.3)

V_w = design shear in the web at a point of splice (Articles 10.18.2.3.2, 10.18.2.3.3, and 10.18.2.3.5)

V_{wo} = overload design shear in the web at a point of splice (Article 10.18.2.3.5)

V_{wu} = design shear in the web at a point of splice (Articles 10.18.2.3.2, 10.18.2.3.3, and 10.18.2.3.5)

W = length of a channel shear connector, in. (Article 10.38.5.1.2)

W_c = roadway width between curbs in feet or barriers if curbs are not used (Article 10.39.2.1)

W_n = least net width of a flange (Article 10.18.2.2.4)

W_L = fraction of a wheel load (Article 10.39.2)

w = length of a channel shear connector in inches measured in a transverse direction on the flange of a girder (Article 10.38.5.1.1)

w = unit weight of concrete, lb per cu ft (Article 10.38.5.1.2)

w = width of flange between longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4)

Y_o = distance from the neutral axis to the extreme outer fiber, in. (Article 10.15.3)

$\bar{y}$ = location of steel sections from neutral axis (Article 10.50.1.1.1)

Z = plastic section modulus (Articles 10.48.1, 10.53.1.1, and 10.54.2.1)

Z_r = allowable range of horizontal shear, in pounds on an individual connector (Article 10.38.5.1)

α = constant based on the number of stress cycles (Article 10.38.5.1.1)

α = minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange (Articles 10.40.2 and 10.40.4)
α = factor for flange splice design equal to 1.0, except that a lower value equal to (M_u/M_y) may be used for flanges subject to compression at sections where M_u does not exceed M_y (Article 10.18.2.2.1)
α = constant equal to 1.3 for members without a longitudinal stiffener and 1.0 for members with a longitudinal stiffener (Article 10.61.1)
β = area of the web divided by the area of the tension flange (Articles 10.40.2 and 10.53.1.2)
β = factor applied to gross area of flange, splice plate or tension member in computing the effective area (Articles 10.18.2.2.4 and 10.18.4.1)
γ = the ratio of A_f to A_p (Article 10.18.1.2.1)
γ = load factor equal to 1.3 (Article 10.61)
ρ = F_{yw}/F_{yf} (Article 10.53.1.2)
θ = angle of inclination of the web plate to the vertical (Articles 10.39.3.1 and 10.51.3)
ψ = ratio of total cross-sectional area to the cross-sectional area of both flanges (Article 10.15.2)
ψ = distance from the outer edge of the tension flange to the neutral axis divided by the depth of the steel section (Articles 10.40.2 and 10.53.1.2)
Δ = amount of camber, in. (Article 10.15.3)
Δ_{DL} = dead load camber in inches at any point (Article 10.15.3)
Δ_m = maximum value of Δ_{DL}, in. (Article 10.15.3)
ϕ = reduction factor (Articles 10.38.5.1.2, 10.56.1.1, and 10.56.1.3)
ϕ = longitudinal stiffener coefficient (Articles 10.39.4.3 and 10.51.5.4)
μ = slip coefficient in a slip-critical joint (Article 10.57.3)

10.2 MATERIALS

10.2.1 General

These specifications recognize steels listed in the following subparagraphs. Other steels may be used; however, their properties, strengths, allowable stresses, and workability must be established and specified.

10.2.2 Structural Steels

Structural steels shall conform to the material designated in Table 10.2A. (The stresses in this table are in pounds per square inch.) The modulus of elasticity of all grades of structural steel shall be assumed to be 29,000,000 psi and the coefficient of linear expansion 0.0000065 per degree Fahrenheit.

10.2.3 Steels for Pins, Rollers, and Expansion Rockers

Steels for pins, rollers, and expansion rockers shall conform to one of the designations listed in Tables 10.2A and 10.2B, or shall be stainless steel conforming to ASTM A 240 or ASTM A 276 HNS 21800.

10.2.4 Fasteners—Rivets and Bolts

Fasteners may be carbon steel bolts (ASTM A 307); power-driven rivets, AASHTO M 228 Grades 1 or 2 (ASTM A 502 Grades 1 or 2); or high-strength bolts, AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490).

10.2.5 Weld Metal

Weld metal shall conform to the current requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code.*

10.2.6 Cast Steel, Ductile Iron Castings, Malleable Castings, and Cast Iron

10.2.6.1 Cast Steel and Ductile Iron

Cast steel shall conform to specifications for Steel Castings for Highway Bridges, AASHTO M 192 (ASTM A 486); Mild-to-Medium-Strength Carbon-Steel Castings for General Application, AASHTO M 103 (ASTM A 27); and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and Nickel-Based Alloy Castings for General Application, AASHTO M 163 (ASTM A 743). Ductile iron castings shall conform to ASTM A 536.

10.2.6.2 Malleable Castings

Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47, Grade 35018 (minimum yield point 35,000 psi).

10.2.6.3 Cast Iron

Cast iron castings shall conform to specifications for Gray Iron Castings, AASHTO M 105, Class 30.

TABLE 10.2A

Minimum Material Properties
Structural Steel

AASHTO Designation[a,b]	M 270 Grade 36	M 270 Grade 50	M 270 Grade 50W	M 270 Grade HPS70W[c]	M 270 Grades 100/100W	
Equivalent ASTM Designation[b]	A 709 Grade 36	A 709 Grade 50	A 709 Grade 50W	A 709 Grade HPS70W	A 709 Grades 100/100W[d]	
Thickness of Plates	Up to 4 in. incl.[e]	Up to 4 in. incl.	Up to 4 in. incl.	Up to 4 in. incl.	Up to 2½ in. incl.	Over 2½ in. to 4 in. incl.
Shapes[f]	All groups[e]	All groups	All groups	Not applicable	Not applicable	Not applicable
Minimum Tensile Strength, F_u	58,000	65,000	70,000	90,000	110,000	100,000
Minimum Yield Point or Minimum Yield Strength, F_y	36,000	50,000	50,000	70,000	100,000	90,000

[a] Except for the mandatory notch toughness and weldability requirements, the ASTM designations are similar to the AASHTO designations. Steels meeting the AASHTO requirements are prequalified for use in welded bridges.

[b] M 270 Gr. 36 and A 709 Gr. 36 are equivalent to M 183 and A 36.
M 270 Gr. 50 and A 709 Gr. 50 are equivalent to M 223 Gr. 50 and A 572 Gr. 50.
M 270 Gr. 50W and A 709 Gr. 50W are equivalent to M 222 and A 588.
M 270 Gr. 70W and A 709 Gr. 70W are equivalent to A 852.
M 270 Gr. 100/100W and A 709 Gr. 100/100W are equivalent to M 244 and A 514.

[c] AASHTO M 270 Grade HPS70W replaces AASHTO M 270 Grade 70W. The intent of this replacement is to encourage the use of high-performance steel (HPS) over conventional bridge steels due to its enhanced properties. AASHTO M 270 Grade 70W steel is still available, but should be used only with the owners approval.

[d] Quenched and tempered alloy steel structural shapes and seamless mechanical tubing meeting all mechanical and chemical requirements of A 709 Grades 100/100W, except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechanical tubing, shall be considered as A 709 Grades 100/100W.

[e] For nonstructural applications or bearing assembly components over 4″ thick, use AASHTO M 270 Gr. 36 (ASTM A 709 Gr. 36).

[f] Groups 1 and 2 include all shapes except those in Groups 3, 4, and 5. Group 3 includes L-shapes over 3/4 inch in thickness. HP shapes over 102 pounds/foot, and the following W shapes:
Designation:
W36 × 230 to 300 incl.
W33 × 200 to 240 incl.
W14 × 142 to 211 incl.
W12 × 120 to 190 incl.
Group 4 includes the following W shapes: W14 × 219 to 550 incl.
Group 5 includes the following W shapes: W14 × 605 to 730 incl.
For breakdown of Groups 1 and 2, see ASTM A 6.

TABLE 10.2B

Minimum Material Properties
Pins, Rollers, and Rockers
Expansion Rollers Shall be Not less Than 4 Inches in Diameter

AASHTO Designation with Size Limitations	M 169 4 in. in dia. or less	M 102 to 20 in. in dia.	M 102 to 20 in. in dia.	M 102 to 10 in. in dia.	M 102 to 20 in. in dia.
ASTM Designation Grade or Class	A 108 Grades 1016 to 1030 incl.	A 668 Class C	A 668 Class D	A 668 Class F	A 668[a] Class G
Minimum Yield Point, psi F_y	36,000[b]	33,000	37,500	50,000	50,000

[a] May substitute rolled material of the same properties.

[b] For design purpose only. Not a part of the A 108 specifications. Supplementary material requirements should provide guarantee that material will meet these values.

Part B
DESIGN DETAILS

10.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

10.3.1 Allowable Fatigue Stress Ranges

Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Article 10.32 and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 10.3.1A for the appropriate type and location of material given in Table 10.3.1B and shown in Figure 10.3.1C. For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.38.4.3, the range of stress may be computed using the composite section assuming the concrete deck to be fully effective for both positive and negative moment.

For unpainted weathering steel, A709, all grades, the values of allowable fatigue stress range, Table 10.3.1A, as modified by footnote d, are valid only when the design and details are in accordance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures,* dated October 3, 1989.

Main load carrying components subjected to tensile stresses that may be considered nonredundant load path members—that is, where failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 10.3.1A for Nonredundant Load Path Structures. Examples of nonredundant load path members are flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two-column bents.

10.3.2 Load Cycles

10.3.2.1 The number of cycles of maximum stress range to be considered in the design shall be selected from Table 10.3.2A unless traffic and loadometer surveys or other considerations indicate otherwise. The fatigue live load preferably shall not exceed HS 20 loading.

10.3.2.2 Allowable fatigue stress ranges shall apply to those Group Loadings that include live load or wind load.

10.3.2.3 The number of cycles of stress range to be considered for wind loads in combination with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

10.3.3 Charpy V-Notch Impact Requirements

10.3.3.1 Main load carrying member components subjected to tensile stress require supplemental impact properties as described in the Material Specifications.*

10.3.3.2 These impact requirements vary depending on the type of steel, type of construction, welded or mechanically fastened, and the average minimum service temperature to which the structure may be subjected.** Table 10.3.3A contains the temperature zone designations.

10.3.3.3 Components requiring mandatory impact properties shall be designated on the drawings and the appropriate zone shall be designated in the contract documents.

10.3.4 Shear

10.3.4.1 When longitudinal beam or girder members in bridges designed for Case I roadways are investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge (see footnote c of Table 10.3.2A), the total shear force in the beam or girder under this single-truck loading shall be limited to 0.58 F_yDt_wC. The constant C, the ratio of the buckling shear stress to the shear yield stress is defined in Article 10.34.4.2 or Article 10.48.8.1.

10.4 EFFECTIVE LENGTH OF SPAN

For the calculation of stresses, span lengths shall be assumed as the distance between centers of bearings or other points of support.

*AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing.*

**The basis and philosophy used to develop these requirements are given in a paper entitled "The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels" by John M. Barsom, February 1975, available from the American Iron and Steel Institute, Washington, D.C.

TABLE 10.3.1A Allowable Fatigue Stress Range

Redundant Load Path Structures[a]

Category (See Table 10.3.1B)	Allowable Range of Stress, F_{sr} (ksi)[b] For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	63 (49)[c]	37 (29)[c]	24 (18)[c]	24 (16)[c]
B	49	29	18	16
B′	39	23	14.5	12
C	35.5	21	13	10
				12[d]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8

Nonredundant Load Path Structures

Category (See Table 10.3.1B)	Allowable Range of Stress, F_{sr} (ksi)[b] For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	50 (39)[c]	29 (23)[c]	24 (16)[c]	24 (16)[c]
B	39	23	16	16
B′	31	18	11	11
C	28	16	10	9
			12[d]	11[d]
D	22	13	8	5
E[e]	17	10	6	2.3
E′	12	7	4	1.3
F	12	9	7	6

[a] Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

[b] The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

[c] For unpainted weathering steel, A 709, all grades, when used in conformance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.

[d] For transverse stiffener welds on girder webs or flanges.

[e] Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures.

10.5 DEPTH RATIOS

10.5.1 For beams or girders, the ratio of depth to length of span preferably should not be less than $1/25$.

10.5.2 For composite girders, the ratio of the overall depth of girder (concrete slab plus steel girder) to the length of span preferably should not be less than $1/25$, and the ratio of depth of steel girder alone to length of span preferably should not be less than $1/30$.

10.5.3 For trusses the ratio of depth to length of span preferably should not be less than $1/10$.

10.5.4 For continuous span depth ratios the span length shall be considered as the distance between the dead load points of contraflexure.

10.5.5 The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.*

10.6 DEFLECTION

10.6.1 The term "deflection" as used herein shall be the deflection computed in accordance with the assumption made for loading when computing the stress in the member.

10.6.2 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 $1/1000$. For checking deflection, the service live load preferably shall not exceed HS 20 loading.

10.6.3 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 $1/375$.

10.6.4 When spans have cross-bracing or diaphragms sufficient in depth or strength to ensure lateral distribution of loads, the deflection may be computed for the standard H or HS loading (M or MS) considering all beams or stringers as acting together and having equal deflection.

10.6.5 The moment of inertia of the gross cross-sectional area shall be used for computing the deflections of beams and girders. When the beam or girder is a part of a composite member, the service live load may be considered as acting upon the composite section.

10.6.6 The gross area of each truss member shall be used in computing deflections of trusses. If perforated plates are used, the effective area shall be the net

*For considerations to be taken into account when exceeding these limitations, reference is made to "Bulletin No. 19, Criteria for the Deflection of Steel Bridges," available from the American Iron and Steel Institute, Washington, D.C.

TABLE 10.3.1B

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
Plain Member	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds (with backing bars removed) or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3,4,5,7
	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds with backing bars not removed, or by continuous partial penetration groove welds parallel to the direction of applied stress.	T or Rev	B′	3,4,5,7
	Calculated flexural stress at the toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at ends of partial length welded coverplates with high-strength bolted slip-critical end connections. (See Note[b])	T or Rev	B	22
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends, or wider than flange with welds across the ends:			
	(a) Flange thickness ≤ 0.8 in.	T or Rev	E	7
	(b) Flange thickness > 0.8 in.	T or Rev	E′	7
	Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	T or Rev	E′	7
Groove Welded Connections	Base metal and weld metal in or adjacent to full penetration groove weld splices of rolled or welded sections having similar profiles when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	8,10
	Base metal and weld metal in or adjacent to full penetration groove weld splices with 2 ft radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	13
	Base metal and weld metal in or adjacent to full penetration groove weld splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of the applied stress, and weld soundness established by nondestructive inspection:			
	(a) AASHTO M 270 Grades 100/100W (ASTM A 709) base metal	T or Rev	B′	11,12
	(b) Other base metals	T or Rev	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove weld splices, with or without transitions having slopes no greater than 1 to 2½, when the reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8,10,11,12
Groove Welded Attachments—Longitudinally Loaded[c]	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 in.	T or Rev	C	6,15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15

TABLE 10.3.1B (Continued)

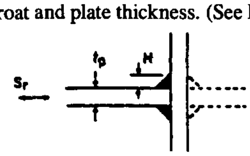
General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	15
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	15
	Base metal adjacent to details attached by full or partial penetration groove welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without end welds ground smooth.	T or Rev	E	16
Groove welded Attachments—Transversely Loaded[c,d]	Detail base metal attached by full penetration groove welds with a transition radius, R, regardless of the detail length and with weld soundness transverse to the direction of stress established by nondestructive inspection:			
	—With equal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—With equal plate thickness and reinforcement not removed	T or Rev		16
	(a) Transition radius ≥ 6 in.		C	
	(b) 6 in. > Transition radius ≥ 2 in.		D	
	(c) 2 in. > Transition radius ≥ 0 in.		E	
	—With unequal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii with unequal plate thickness and reinforcement not removed.	T or Rev	E	16
Fillet Welded Connections	Base metal at details connected with transversely loaded welds, with the welds perpendicular to the direction of stress:			
	(a) Detail thickness ≤ 0.5 in.	T or Rev	C	14
	(b) Detail thickness > 0.5 in.	T or Rev	See Note[e]	
	Base metal at intermittent fillet welds.	T or Rev	E	—
	Shear stress on throat of fillet welds.	Shear	F	9
Fillet Welded Attachments—Longitudinally Loaded[c,d]	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, is less than 2 in. and stud-type shear connectors.	T or Rev	C	15,17,18,20
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15,17
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	7,9,15,17
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	7,9,15

TABLE 10.3.1B (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Fillet Welded Attachments—Transversely Loaded with the Weld in the Direction of Principal Stress[c,f]	Detail base metal attached by fillet welds with a transition radius, R, regardless of the detail length (shear stress on the throat of fillet welds governed by Category F):			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connecting materials.	T or Rev	B	21
	Base metal at net section of high-strength bolted bearing-type connections.	T or Rev	B	21
	Base metal at net section of riveted connections.	T or Rev	D	21
Eyebar or Pin Plates	Base metal at the net section of eyebar head, or pin plate	T	E	23, 24
	Base metal in the shank of eyebars, or through the gross section of pin plates with:			
	(a) rolled or smoothly ground surfaces	T	A	23, 24
	(b) flame-cut edges	T	B	23, 24

[a] "T" signifies range in tensile stress only, "Rev" signifies a range of stress involving both tension and compression during a stress cycle.
[b] See Wattar, Albrecht and Sahli, *Journal of Structural Engineering*, ASCE, Vol. III, No. 6, June 1985, pp. 1235–1249.
[c] "Longitudinally Loaded" signifies direction of applied stress is parallel to the longitudinal axis of the weld. "Transversely Loaded" signifies direction of applied stress is perpendicular to the longitudinal axis of the weld.
[d] Transversely loaded partial penetration groove welds are prohibited.
[e] Allowable fatigue stress range on throat of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, *Journal of the Structural Division*, ASCE, Vol. 105, No. ST9, Sept. 1979.)

$$S_r = S_r^C \left(\frac{0.06 + 0.79H/t_p}{1.1t_p^{1/6}}\right)$$



where S_r^C is equal to the allowable stress range for Category C given in Table 10.3.1A. This assumes no penetration at the weld root.
[f] Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.

volume divided by the length from center to center of perforations.

10.6.7 The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.*

*For considerations to be taken into account when exceeding these limitations, reference is made to "Bulletin No. 19, Criteria for the Deflection of Steel Bridges," available from the American Iron and Steel Institute, Washington, D.C.

10.7 LIMITING LENGTHS OF MEMBERS

10.7.1 For compression members, the slenderness ratio, KL/r, shall not exceed 120 for main members, or those in which the major stresses result from dead or live load, or both; and shall not exceed 140 for secondary members, or those whose primary purpose is to brace the structure against lateral or longitudinal force, or to brace or reduce the unbraced length of other members, main or secondary.

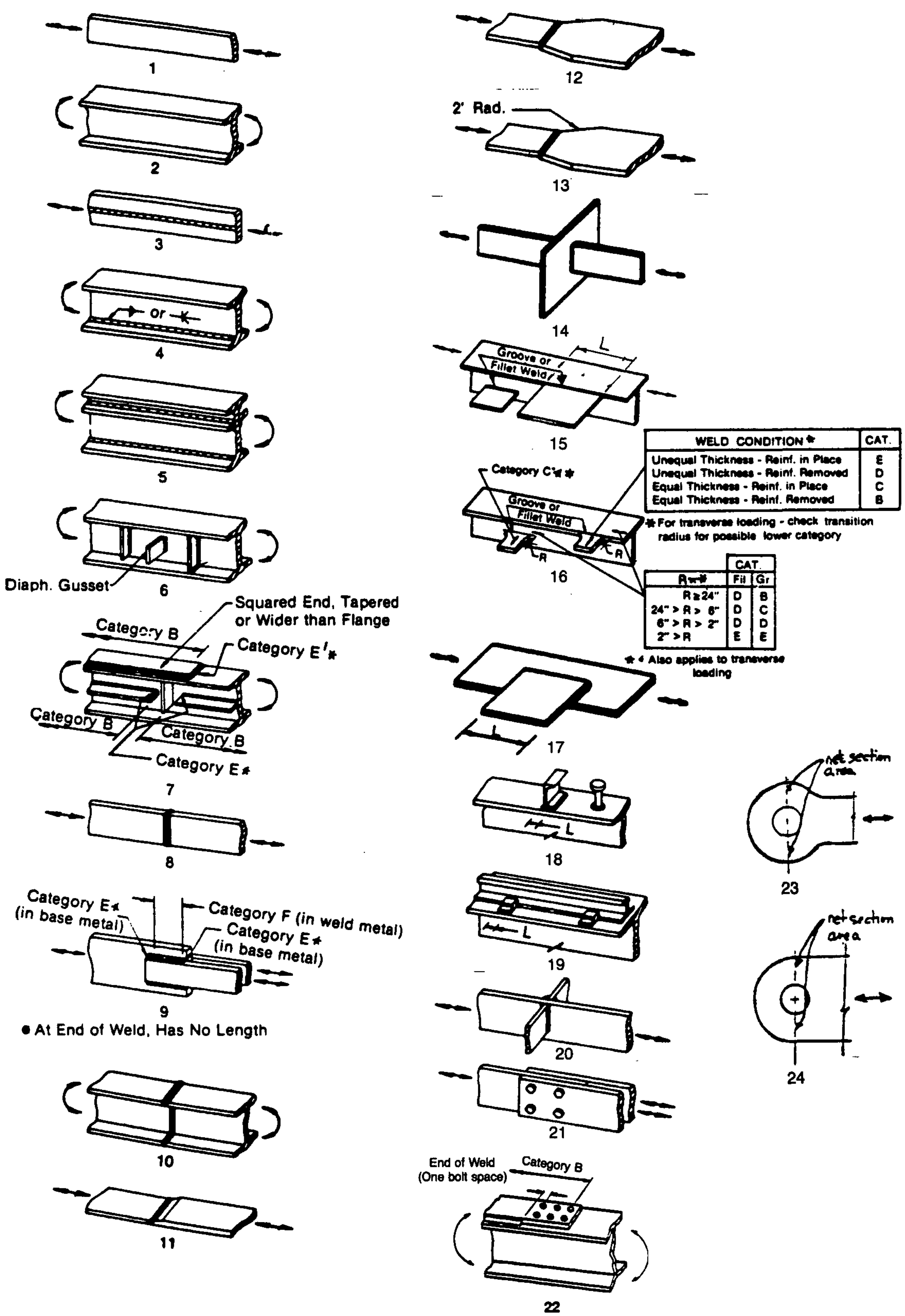


FIGURE 10.3.1C Illustrative Examples

TABLE 10.3.2A Stress Cycles

Type of Road	Case	ADTT[a]	Truck Loading	Lane Loading[b]
Main (Longitudinal) Load Carrying Members				
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	2,000,000[c]	500,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000

Transverse Members and Details Subjected to Wheel Loads

Type of Road	Case	ADTT[a]	Truck Loading
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	over 2,000,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	2,000,000
Other Highways and Streets	III	—	500,000

[a] Average Daily Truck Traffic (one direction).
[b] Longitudinal members should also be checked for truck loading.
[c] Members shall also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading. The shear in steel girder webs shall not exceed 0.58 F_yDt_wC for this single truck loading.

10.7.2 In determining the radius of gyration, r, for the purpose of applying the limitations of the KL/r ratio, the area of any portion of a member may be neglected provided that the strength of the member as calculated without using the area thus neglected and the strength of the member as computed for the entire section with the KL/r ratio applicable thereto, both equal or exceed the computed total force that the member must sustain.

10.7.3 The radius of gyration and the effective area for carrying stress of a member containing perforated cover plates shall be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates, the cross-sectional area of the member shall be considered the same as for a section having perforations in the same transverse plane.

10.7.4 Actual unbraced length, L, shall be assumed as follows:

For the top chords of half-through trusses, the length between panel points laterally supported as indicated under Article 10.16.12; for other main members, the length between panel point intersections or centers of braced points or centers of end connections; for secondary members, the length between the centers of the end connections of such members or centers of braced points.

TABLE 10.3.3A Temperature Zone Designations for Charpy V-Notch Impact Requirements

Minimum Service Temperature	Temperature Zone Designation
0°F and above	1
−1°F to −30°F	2
−31°F to −60°F	3

10.7.5 For tension members, except rods, eyebars, cables, and plates, the ratio of unbraced length to radius of gyration shall not exceed 200 for main members, shall not exceed 240 for bracing members, and shall not exceed 140 for main members subject to a reversal of stress.

10.8 MINIMUM THICKNESS OF METAL

10.8.1 Structural steel (including bracing, cross frames, and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers, and in railings, shall be not less than 5/16 inch in thickness. The web thickness of rolled beams or channels shall not be less than 0.23 inches. The thickness of closed ribs in orthotropic decks shall not be less than 3/16 inch.

10.8.2 Where the metal will be exposed to marked corrosive influences, it shall be increased in thickness or specially protected against corrosion.

10.8.3 It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc. As stated above, fillers need not be 5/16 inch minimum.

10.8.4 For compression members, refer to "Trusses" (Article 10.16).

10.8.5 For stiffeners and other plates, refer to "Plate Girders" (Article 10.34).

10.8.6 For stiffeners and outstanding legs of angles, etc., refer to Article 10.10.

10.9 EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION

10.9.1 The effective area of a single angle tension member, a tee section tension member, or each angle of a double angle tension member in which the shapes are connected back to back on the same side of a gusset plate shall be assumed as the net area of the connected leg or flange plus one-half of the area of the outstanding leg.

10.9.2 If a double angle or tee section tension member is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net area of the shapes shall be considered effective.

10.9.3 When angles connect to separate gusset plates, as in the case of a double-webbed truss, and the angles are connected by stay plates located as near the gusset as practicable, or by other adequate means, the full net area of the angles shall be considered effective. If the angles are not so connected, only 80% of the net areas shall be considered effective.

10.9.4 Lug angles may be considered as effective in transmitting stress, provided they are connected with at least one-third more fasteners than required by the stress to be carried by the lug angle.

10.10 OUTSTANDING LEGS OF ANGLES

The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following:

In main members carrying axial stress, 12 times the thickness.
In bracing and other secondary members, 16 times the thickness.

For other limitations, see Article 10.35.2.

10.11 EXPANSION AND CONTRACTION

In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provisions shall be made for changes in length of span resulting from live load stresses. In spans more than 300 feet long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement.

10.12 FLEXURAL MEMBERS

Flexural members shall be designed using the elastic section modulus except when utilizing compact sections under Strength Design as specified in Articles 10.48.1, 10.50.1.1, and 10.50.2.1. When computing the strength of a flexural member at a section with holes in the tension flange, an effective flange area, A_e, specified by Equation (10-4g) shall be used for that flange in computing the elastic section properties. The diameter of the holes shall be taken as specified in Article 10.16.14.6. In the case of the strength design method, the strength of compact sections with holes in the tension flange shall not be taken greater than the moment capacity at first yield.

10.13 COVER PLATES

10.13.1 The length of any cover plate added to a rolled beam shall be not less than (2d+3) feet, where (d) is the depth of the beam in feet.

10.13.2 Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the member.

10.13.3 The maximum thickness of a single cover plate on a flange shall not be greater than two times the thickness of the flange to which the cover plate is attached. The total thickness of all cover plates should not be greater than 2½ times the flange thickness.

10.13.4 Any partial length welded cover plate shall extend beyond the theoretical end by the terminal distance, and it shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal adjacent to or connected by fillet welds. The theoretical end of the cover plate, when using service load design methods, is the section at which the stress in the flange without that cover plate equals the allowable service load stress, exclusive of fatigue considerations. When using strength design methods, the theoretical end of the cover plate is the section at which the flange strength without that cover plate equals the required strength for the design loads, exclusive of fatigue requirements. The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends, and 1½ times for cover plates welded across their ends. The width at ends of tapered cover plates shall be not less than 3 inches. The weld connecting the cover plate to the flange in its terminal distance shall be continuous and of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end. All welds connecting cover plates to beam flanges shall be continuous and shall not be smaller than the minimum size permitted by Article 10.23.2.

10.13.5 Any partial length end-bolted cover plate shall extend beyond the theoretical end by a terminal distance equal to the length of the end-bolted portion, and the cover plate shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal at ends of partial length welded cover plates with high-strength bolted, slip-critical end connections (Table 10.3.1B). Beams with end-bolted cover plates shall be fabricated in the following sequence: drill holes; clean faying surfaces; install bolts; weld. The theoretical end of the end-bolted cover plate is determined in the same manner as that of a welded cover plate, as is specified in Article 10.13.4. The bolts in the slip-critical connections of the cover plate ends to the flange, shall be of sufficient numbers to develop a total force of not less than the computed force in the cover plate at the theoretical end. The slip resistance of the end-bolted connection shall be determined in accordance with Article 10.32.3.2 for service load design, and Article 10.56.1.4 for load factor design. The longitudinal welds connecting the cover plate to the beam flange shall be continuous and stop a distance equal to one bolt spacing before the first row of bolts in the end-bolted portion.

10.14 CAMBER

Girders should be cambered to compensate for dead load deflections and vertical curvature required by profile grade.

10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

10.15.1 Scope

This section pertains to rolled beams and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified minimum yield point greater than 50,000 psi, except for Grade HPS70W steel, shall not be heat-curved.

10.15.2 Minimum Radius of Curvature

10.15.2.1 For heat-curved beams and girders, the horizontal radius of curvature measured to the center line of the girder web shall not be less than 150 feet and shall not be less than the larger of the values calculated (at any and all cross sections throughout the length of the girder) from the following two equations:

$$R = \frac{14bD}{\sqrt{F_y \psi t_w}} \tag{10-1}$$

$$R = \frac{7{,}500b}{F_y \psi} \tag{10-2}$$

In these equations, F_y is the specified minimum yield point in kips per square inch of steel in the girder web, ψ is the ratio of the total cross-sectional area to the cross-sectional area of both flanges, b is the widest flange width in inches, D is the clear distance between flanges in inches, t_w is the web thickness in inches, and R is the radius in inches.

10.15.2.2 In addition to the above requirements, the radius shall not be less than 1,000 feet when the flange thickness exceeds 3 inches or the flange width exceeds 30 inches.

10.15.3 Camber

To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, the amount of camber in inches, Δ at any section along the length L of the girder shall be equal to:

$$\Delta = \frac{\Delta_{DL}}{\Delta_M}(\Delta_M + \Delta_R) \tag{10-3}$$

$$\Delta_R = \frac{0.02\, L^2 F_y}{EY_o}\left(\frac{1{,}000 - R}{850}\right)$$

$$\Delta_R = 0 \text{ for radii greater than } 1{,}000$$

where Δ_{DL} is the camber in inches at any point along the length L calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads; Δ_M is the maximum value of Δ_{DL} in inches within the length L; E is the modulus of elasticity in ksi; F_y is the specified minimum yield point in ksi of the girder flange; Y_o is the distance from the neutral axis to the extreme outer fiber in inches (maximum distance for nonsymmetrical sections); R is the radius of curvature in feet; and L is the span length for simple spans or for continuous spans, the distance between a simple end support and the dead load contraflexure point, or the distance between points of dead load contraflexure. (L is measured in inches.) Camber loss between dead load contraflexure points adjacent to piers is small and may be neglected.

Note: Part of the camber loss is attributable to construction loads and will occur during construction of the bridge; total camber loss will be complete after several months of in-service loads. Therefore, a portion of the camber increase (approximately 50%) should be included in the bridge profile. Camber losses of this nature (but generally smaller in magnitude) are also known to occur in straight beams and girders.

10.16 TRUSSES

10.16.1 General

10.16.1.1 Component parts of individual truss members may be connected by welds, rivets, or high-strength bolts.

10.16.1.2 Preference should be given to trusses with single intersection web systems. Members shall be symmetrical about the central plane of the truss.

10.16.1.3 Trusses preferably shall have inclined end posts. Laterally unsupported hip joints shall be avoided.

10.16.1.4 Main trusses shall be spaced a sufficient distance apart, center to center, to be secure against overturning by the assumed lateral forces.

10.16.1.5 For the calculation of stresses, effective depths shall be assumed as follows:

Riveted and bolted trusses, distance between centers of gravity of the chords.
Pin-connected trusses, distance between centers of chord pins.

10.16.2 Truss Members

10.16.2.1 Chord and web truss members shall usually be made in the following shapes:

"H" sections, made with two side segments (composed of angles or plates) with solid web, perforated web, or web of stay plates and lacing.
Channel sections, made with two angle segments, with solid web, perforated web, or web of stay plates and lacing.
Single Box sections, made with side channels, beams, angles, and plates or side segments of plates only, connected top and bottom with perforated plates or stay plates and lacing.

Single Box sections, made with side channels, beams, angles and plates only, connected at top with solid cover plates and at the bottom with perforated plates or stay plates and lacing.
Double Box sections, made with side channels, beams, angles and plates or side segments of plates only, connected with a conventional solid web, together with top and bottom perforated cover plates or stay plates and lacing.

10.16.2.2 If the shape of the truss permits, compression chords shall be continuous.

10.16.2.3 In chords composed of angles in channel-shaped members, the vertical legs of the angles preferably shall extend downward.

10.16.2.4 If web members are subject to reversal of stress, their end connections shall not be pinned. Counters preferably shall be rigid. Adjustable counters, if used, shall have open turnbuckles, and in the design of these members an allowance of 10,000 pounds per square inch shall be made for initial stress. Only one set of diagonals in any panel shall be adjustable. Sleeve nuts and loop bars shall not be used.

10.16.3 Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Secondary stresses due to truss distortion or floor beam deflection usually need not be considered in any member, the width of which, measured parallel to the plane of distortion, is less than one-tenth of its length. If the secondary stress exceeds 4,000 pounds per square inch for tension members and 3,000 for compression members, the excess shall be treated as a primary stress. Stresses due to the flexural dead load moment of the member shall be considered as additional secondary stress.

10.16.4 Diaphragms

10.16.4.1 There shall be diaphragms in the trusses at the end connections of floor beams.

10.16.4.2 The gusset plates engaging the pedestal pin at the end of the truss shall be connected by a diaphragm. Similarly, the webs of the pedestal shall, if practicable, be connected by a diaphragm.

10.16.4.3 There shall be a diaphragm between gusset plates engaging main members if the end tie plate is 4 feet or more from the point of intersection of the members.

10.16.5 Camber

The length of the truss members shall be such that the camber will be equal to or greater than the deflection produced by the dead load.

10.16.6 Working Lines and Gravity Axes

10.16.6.1 Main members shall be proportioned so that their gravity axes will be as nearly as practicable in the center of the section.

10.16.6.2 In compression members of unsymmetrical section, such as chord sections formed of side segments and a cover plate, the gravity axis of the section shall coincide as nearly as practicable with the working line, except that eccentricity may be introduced to counteract dead load bending. In two-angle bottom chord or diagonal members, the working line may be taken as the gage line nearest the back of the angle or at the center of gravity for welded trusses.

10.16.7 Portal and Sway Bracing

10.16.7.1 Through truss spans shall have portal bracing, preferably, of the two-plane or box type, rigidly connected to the end post and the top chord flanges, and as deep as the clearance will allow. If a single plane portal is used, it shall be located, preferably, in the central transverse plane of the end posts, with diaphragms between the webs of the posts to provide for a distribution of the portal stresses. The portal bracing shall be designed to take the full end reaction of the top chord lateral system, and the end posts shall be designed to transfer this reaction to the truss bearings.

10.16.7.2 Through truss spans shall have sway bracing 5 feet or more deep at each intermediate panel point. Top lateral struts shall be at least as deep as the top chord.

10.16.7.3 Deck truss spans shall have sway bracing in the plane of the end posts and at all intermediate panel points. This bracing shall extend the full depth of the trusses below the floor system. The end sway bracing shall be proportioned to carry the entire upper lateral stress to the supports through the end posts of the truss.

10.16.8 Perforated Cover Plates

When perforated cover plates are used, the following provisions shall govern their design.

10.16.8.1 The ratio of length, in direction of stress, to width of perforation, shall not exceed two.

10.16.8.2 The clear distance between perforations in the direction of stress shall not be less than the distance between points of support.

10.16.8.3 The clear distance between the end perforation and the end of the cover plate shall not be less than 1.25 times the distance between points of support.

10.16.8.4 The point of support shall be the inner line of fasteners or fillet welds connecting the perforated plate to the flanges. For plates butt welded to the flange edge of rolled segments, the point of support may be taken as the weld whenever the ratio of the outstanding flange width to flange thickness of the rolled segment is less than seven. Otherwise, the point of support shall be the root of the flange of the rolled segment.

10.16.8.5 The periphery of the perforation at all points shall have a minimum radius of 1½ inches.

10.16.8.6 For thickness of metal, see Article 10.35.2.

10.16.9 Stay Plates

10.16.9.1 Where the open sides of compression members are not connected by perforated plates, such members shall be provided with lacing bars and shall have stay plates as near each end as practicable. Stay plates shall be provided at intermediate points where the lacing is interrupted. In main members, the length of the end stay plates between end fasteners shall be not less than 1 ¼ times the distance between points of support and the length of intermediate stay plates not less than ¾ of that distance. In lateral struts and other secondary members, the overall length of end and intermediate stay plates shall be not less than ¾ of the distance between points of support.

10.16.9.2 The point of support shall be the inner line of fasteners or fillet welds connecting the stay plates to the flanges. For stay plates butt welded to the flange edge of rolled segments, the point of support may be taken as the weld whenever the ratio of outstanding flange width to flange thickness of the rolled segment is less than seven. Otherwise, the point of support shall be the root of flange of rolled segment. When stay plates are butt welded to rolled segments of a member, the allowable stress in the member shall be determined in accordance with Article 10.3. Terminations of butt welds shall be ground smooth.

10.16.9.3 The separate segments of tension members composed of shapes may be connected by perforated plates or by stay plates or end stay plates and lacing. End stay plates shall have the same minimum length as specified for end stay plates on main compression members, and intermediate stay plates shall have a minimum length of ¾ of that specified for intermediate stay plates on main compression members. The clear distance between stay plates on tension members shall not exceed 3 feet.

10.16.9.4 The thickness of stay plates shall be not less than 1/50 of the distance between points of support for main members, and 1/60 of that distance for bracing members. Stay plates shall be connected by not less than three fasteners on each side, and in members having lacing bars the last fastener in the stay plates preferably shall also pass through the end of the adjacent bar.

10.16.10 Lacing Bars

When lacing bars are used, the following provisions shall govern their design.

10.16.10.1 Lacing bars of compression members shall be so spaced that the slenderness ratio of the portion of the flange included between the lacing bar connections will be not more than 40 or more than ⅔ of the slenderness ratio of the member.

10.16.10.2 The section of the lacing bars shall be determined by the formula for axial compression in which L is taken as the distance along the bar between its connections to the main segments for single lacing, and as 70% of that distance for double lacing.

10.16.10.3 If the distance across the member between fastener lines in the flanges is more than 15 inches and a bar with a single fastener in the connection is used, the lacing shall be double and fastened at the intersections.

10.16.10.4 The angle between the lacing bars and the axis of the member shall be approximately 45° for double lacing and 60° for single lacing.

10.16.10.5 Lacing bars may be shapes or flat bars. For main members, the minimum thickness of flat bars shall be 1/40 of the distance along the bar between its connections for single lacing and 1/60 for double lacing. For bracing members, the limits shall be 1/50 for single lacing and 1/75 for double lacing.

10.16.10.6 The diameter of fasteners in lacing bars shall not exceed one-third the width of the bar. There shall be at least two fasteners in each end of lacing bars connected to flanges more than 5 inches in width.

10.16.11 Gusset Plates

10.16.11.1 Gusset or connection plates preferably shall be used for connecting main members, except when the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member shall be given consideration. The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure acting on the weakest or critical section of maximum stress.

10.16.11.2 Re-entrant cuts, except curves made for appearance, shall be avoided as far as practicable.

10.16.11.3 If the length of unsupported edge of a gusset plate exceeds the value of the expression $11{,}000/\sqrt{F_y}$ times its thickness, the edge shall be stiffened.

10.16.11.4 Listed below are the values of the expression $11{,}000/\sqrt{F_y}$ for the following grades of steel:

36,000 psi, Y.P. Min 58
50,000 psi, Y.P. Min 49
70,000 psi, Y.P. Min 42
90,000 psi, Y.P. Min 37
100,000 psi, Y.P. Min 35

10.16.12 Half-Through Truss Spans

10.16.12.1 The vertical truss members and the floor beams and their connections in half-through truss spans shall be proportioned to resist a lateral force of not less than 300 pounds per linear foot applied at the top chord panel points of each truss.

10.16.12.2 The top chord shall be considered as a column with elastic lateral supports at the panel points. The critical buckling force of the column, so determined, shall exceed the maximum force from dead load, live load, and impact in any panel of the top chord by not less than 50%.*

*For a discussion of columns with elastic lateral supports, refer to Timoshenko & Gere, ``Theory of Elastic Stability," McGraw-Hill Book Co., First Edition, p. 122.

10.16.13 Fastener Pitch in Ends of Compression Members

In the ends of compression members, the pitch of fasteners connecting the component parts of the member shall not exceed four times the diameter of the fastener for a length equal to 1½ times the maximum width of the member. Beyond this point, the pitch shall be increased gradually for a length equal to 1½ times the maximum width of the member until the maximum pitch is reached.

10.16.14 Net Section of Riveted or High-Strength Bolted Tension Members

10.16.14.1 The net section of a riveted or high-strength bolted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width.

10.16.14.2 The net width for any chain of holes extending progressively across the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding, for each gage space in the chain, the quantity:

$$\frac{S^2}{4g} \qquad (10\text{-}4)$$

where:

S = pitch of any two successive holes in the chain;
g = gage of the same holes.

The net section of the part is obtained from the chain that gives the least net width.

10.16.14.3 For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of gages from back of angle less the thickness.

10.16.14.4 At a splice, the total stress in the member being spliced is transferred by fasteners to the splice material.

10.16.14.5 When determining the unit stress on any least net width of either splice material or member being spliced, the amount of the stress previously transferred by fasteners adjacent to the section being investigated shall be considered in determining the unit stress on the net section.

10.16.14.6 The diameter of the hole shall be taken as ⅛ inch greater than the nominal diameter of the rivet or high-strength bolt, unless larger holes are permitted in accordance with Article 10.24.

10.17 BENTS AND TOWERS

10.17.1 General

Bents preferably shall be composed of two supporting columns, and the bents usually shall be united in pairs to form towers. The design of members for bents and towers is governed by applicable articles.

10.17.2 Single Bents

Single bents shall have hinged ends or else shall be designed to resist bending.

10.17.3 Batter

Bents preferably shall have a sufficient spread at the base to prevent uplift under the assumed lateral loadings. In general, the width of a bent at its base shall be not less than one-third of its height.

10.17.4 Bracing

10.17.4.1 Towers shall be braced, both transversely and longitudinally, with stiff members having either welded, high-strength bolted or riveted connections. The sections of members of longitudinal bracing in each panel shall not be less than those of the members in corresponding panels of the transverse bracing.

10.17.4.2 The bracing of long columns shall be designed to fix the column about both axes at or near the same point.

10.17.4.3 Horizontal diagonal bracing shall be placed in all towers having more than two vertical panels, at alternate intermediate panel points.

10.17.5 Bottom Struts

The bottom struts of towers shall be strong enough to slide the movable shoes with the structure unloaded, the coefficient of friction being assumed at 0.25. Provision for expansion of the tower bracing shall be made in the column bearings.

10.18 SPLICES

10.18.1 General

10.18.1.1 Design Strength

Splices may be made by rivets, by high-strength bolts or by the use of welding. In general, splices whether in tension, compression, bending, or shear, shall be designed in the case of the service load design or strength design methods for a capacity based on not less than the average of the required design strength at the point of splice and the design strength of the member at the same point but, in any event, not less than 75% of the design strength of the member, except as specified herein. Bolted splices in flexural members shall satisfy the requirements of Article 10.18.2. Bolted splices in compression members shall satisfy the requirements of Article 10.18.3. Bolted splices in tension members shall satisfy the requirements of Article 10.18.4. Welded splices shall satisfy the requirements of Article 10.18.5. Where a section changes at a splice, the smaller section is to be used to satisfy the above splice requirements.

10.18.1.2 Fillers

10.18.1.2.1 For fillers ¼ inch and thicker in bolted or riveted axially loaded connections, including girder flange splices, additional fasteners shall be required to distribute the total stress in the member uniformly over the combined section of the member and the filler. The filler shall either be extended beyond the splice material and secured by additional bolts, or as an alternate to extending the filler, an equivalent number of bolts may be included in the connection. Fillers ¼ inch and thicker need not be extended and developed provided that the design shear strength of the fasteners, specified in Article 10.56.1.3.2 in the case of the strength design method and in Table 10.32.3B in the case of the service load design method, is reduced by the following factor R:

$$R = [(1+\gamma)/(1+2\gamma)] \qquad (10\text{-}4a)$$

where: $\gamma = \dfrac{A_f}{A_p}$

A_f = sum of the area of the fillers on the top and bottom of the connected plate

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate

The design slip force, specified in Article 10.57.3.1 in the case of the strength design method and in Article 10.32.3.2.1 in the case of the service load design method, for slip-critical connections shall not be adjusted for the effect of the fillers. Fillers ¼ inch or more in thickness shall consist of not more than two plates, unless special permission is given by the Engineer.

10.18.1.2.2 For bolted web splices with thickness differences of 1/16 inch or less, no filler plates are required.

10.18.1.2.3 Fillers for welded splices shall conform to the requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code.*

10.18.1.3 Design Force for Flange Splice Plates

For a flange splice with inner and outer splice plates, the flange design force may be assumed to be divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than 10%. When the areas of the inner and outer plates differ by more than 10%, the design force in each splice plate and its connection shall be determined by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the shear strength of the connection shall be checked for the maximum calculated splice plate force acting on a single shear plane. The slip resistance of high-strength bolted connections for a flange splice with inner and outer splice plates shall always be checked for the flange design force divided equally to the two slip planes.

10.18.1.4 Truss Chords and Columns

Splices in truss chords and columns shall be located as near to the panel points as practicable and usually on the side where the smaller stress occurs. The arrangement of plates, angles, or other splice elements shall be such as to make proper provision for the stresses, both axial and bending, in the component parts of the members spliced.

10.18.2 Flexural Members

10.18.2.1 General

10.18.2.1.1 In continuous spans, splices shall preferably be made at or near points of dead-load contraflexure.

10.18.2.1.2 In both flange and web splices, there shall be not less than two rows of bolts on each side of the joint.

10.18.2.1.3 Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices.

10.18.2.1.4 In both flange and web splices, high-strength bolted connections shall be proportioned to prevent slip during erection of the steel and during the casting or placing of the deck.

10.18.2.1.5 In the case of the strength design method, the strength of compact sections at the point of splice shall not be taken greater than the moment capacity at first yield, computed by accounting for the holes in the tension flange as specified in Article 10.12.

10.18.2.1.6 Flange and web splices in areas of stress reversal shall be checked for both positive and negative flexure.

10.18.2.1.7 Riveted and bolted flange angle splices shall include two angles, one on each side of the flexural member.

10.18.2.2 Flange Splices

10.18.2.2.1 As a minimum, in the case of the strength design method, the splice plates on the controlling flange shall be proportioned for a design force, P_{cu}. The controlling flange shall be taken as the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness due to the factored loads to its maximum strength. P_{cu} shall be taken equal to a design stress, F_{cu}, times the smaller effective flange area, A_e, on either side of the splice. A_e is defined in Article 10.18.2.2.4 and F_{cu} is defined as follows:

$$F_{cu} = \frac{\left(|f_{cu}/R| + \alpha F_{yf}\right)}{2} \geq 0.75\alpha F_{yf} \qquad (10\text{-}4b)$$

where:

α = 1.0 except that a lower value equal to (M_u/M_y) may be used for flanges in compression at sections where M_u is less than M_y.

M_u = maximum bending strength of the section in positive or negative flexure at the point of splice, whichever causes the maximum compressive stress due to the factored loads at the mid-thickness of the flange under consideration

M_y = moment capacity at first yield for the section at the point of splice used to compute M_u. For composite sections, M_y shall be calculated in accordance with Article 10.50(c). For hybrid sections, M_y shall be computed in accordance with Article 10.53.

f_{cu} = maximum elastic flexural stress due to the factored loads at the mid-thickness of the controlling flange at the point of splice.

R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when f_{cu} is less than or equal to F_{yw}, where F_{yw} is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to 1.0.

F_{yf} = specified minimum yield strength of the flange

As a minimum, the splice plates for the noncontrolling flange shall be proportioned for a design force, P_{ncu}. P_{ncu} shall be taken equal to a design stress, F_{ncu}, times the smaller effective flange area, A_e, on either side of the splice. F_{ncu} is defined as follows:

$$F_{ncu} = R_{cu}\left(|f_{ncu} / R|\right) \geq 0.75\alpha F_{yf} \qquad (10\text{-}4c)$$

where:

R_{cu} = the absolute value of the ratio of F_{cu} to f_{cu} for the controlling flange.

f_{ncu} = flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at the point of splice concurrent with f_{cu}

In calculating f_{cu}, f_{ncu}, M_u, M_y and R, holes in the flange subject to tension shall be accounted for as specified in Article 10.12. For a flange splice with inner and outer splice plates, the flange design force shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, A_e, of each splice plate shall be sufficient to prevent yielding of the splice plate under its calculated portion of the design force. A_e of each splice plate shall be taken as defined in Article 10.18.2.2.4. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied.

10.18.2.2.2 As a minimum, in the case of the strength design method, high-strength bolted connections for both top and bottom flange splices shall be proportioned to prevent slip at an overload design force, P_{fo}. For the flange under consideration, P_{fo} shall be computed as follows:

$$P_{fo} = |f_o/R|A_g \qquad (10\text{-}4d)$$

where:

- f_o = maximum flexural stress due to D + β_L(L + I) at the mid-thickness of the flange under consideration for the smaller section at the point of splice, where β_L is defined in Article 3.22
- R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when f_o is less than or equal to F_{yw}, where F_{yw} is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to 1.0.
- A_g = smaller gross flange area on either side of the splice

f_o and R shall be computed using the gross section of the member. The slip resistance of the connection shall be computed from Equation (10-172).

10.18.2.2.3 As a minimum, in the case of the service load design method, the splice plates on the controlling flange shall be proportioned for a design force, P_{cf}. The controlling flange shall be taken as the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness to its allowable stress. P_{cf} shall be taken equal to a design stress, F_{cf}, times the smaller effective flange area, A_e, on either side of the splice. A_e is defined in Article 10.18.2.2.4 and F_{cf} is defined as follows:

$$F_{cf} = \frac{(|f_{cf}/R| + F_b)}{2} \geq 0.75F_b \qquad (10\text{-}4e)$$

where:

- f_{cf} = maximum elastic flexural stress at the mid-thickness of the controlling flange at the point of splice.
- F_b = allowable flexural stress for the flange under consideration at the point of splice
- R = reduction factor for hybrid girders specified in Article 10.40.2.1. R shall be taken equal to 1.0 when f_{cf} is less than or equal to the allowable flexural stress for the web steel. For homogeneous girders, R shall always be taken equal to 1.0.

As a minimum, the splice plates for the noncontrolling flange shall be proportioned for a design force, P_{ncf}. P_{ncf} shall be taken equal to a design stress, F_{ncf}, times the smaller effective flange area, A_e, on either side of the splice. F_{ncf} is defined as follows:

$$F_{ncf} = R_{cf}(|f_{ncf}/R|) \geq 0.75F_b \qquad (10\text{-}4f)$$

where:

- R_{cf} = the absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange
- f_{ncf} = flexural stress at the mid-thickness of the noncontrolling flange at the point of splice concurrent with f_{cf}

In calculating F_{cf}, f_{ncf} and R, holes in the flange subject to tension shall be accounted for as specified in Article 10.12. For a flange splice with inner and outer splice plates, the flange design force shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, A_e, of each splice plate shall be sufficient to ensure that the stress in the splice plate does not exceed the allowable flexural stress under its calculated portion of the design force. A_e of each splice plate shall be taken as defined in Article 10.18.2.2.4. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Table 10.32.3B. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied. As a minimum, high-strength bolted connections shall also be proportioned to prevent slip at a force equal to the maximum elastic flexural stress due to D + (L + I) at the mid-thickness of the flange under consideration for the smaller section at the point of splice times the smaller value of the gross flange area on either side of the splice. The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1.

10.18.2.2.4 For checking the strength of flange splices, an effective area, A_e, shall be used for the flange and for the individual splice plates as follows:

For flanges and their splice plates subject to tension:

$$A_e = W_n t + \beta A_g \leq A_g \qquad (10\text{-}4g)$$

where:

- W_n = least net width of the flange or splice plate computed as specified in Article 10.16.14

t = flange or splice plate thickness
A_g = gross area of the flange or splice plate
β = 0.0 for M 270 Grade 100/100W steels, or when holes exceed 1¼ inch in diameter.
= 0.15 for all other steels and when holes are less than or equal to 1¼ inch in diameter.

The diameter of the holes shall be taken as specified in Article 10.16.14.6.

For the flanges and their splice plates subject to compression:

$$A_e = A_g \tag{10-4h}$$

10.18.2.3 Web Splices

10.18.2.3.1 In general, web splice plates and their connections shall be proportioned for shear, a moment due to the eccentricity of the shear at the point of splice, and a portion of the flexural moment that is assumed to be resisted by the web at the point of splice.* Webs shall be spliced symmetrically by plates on each side. The web splice plates shall extend as near as practical for the full depth between flanges.

10.18.2.3.2 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned for a design shear in the web at the point of splice, V_{wu}, defined as follows:

For $V < 0.5V_u$:

$$V_{wu} = 1.5V \tag{10-4i}$$

For $V \geq 0.5V_u$:

$$V_{wu} = \frac{[V + V_u]}{2} \tag{10-4j}$$

where:

V = maximum shear in the web at the point of splice due to the factored loads
V_u = shear capacity of the web at the point of splice

10.18.2.3.3 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned for a design moment, M_{vu}, due to the eccentricity of the design shear at the point of splice defined as follows:

$$M_{vu} = V_{wu}e \tag{10-4k}$$

where:

V_{wu} = design shear in the web at the point of splice defined in Article 10.18.2.3.2
e = distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration

10.18.2.3.4 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned for a design moment at the point of splice, M_{wu}, representing the portion of the flexural moment that is assumed to be resisted by the web. M_{wu} shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H_{wu}, shall also be applied at the mid-depth of the web. M_{wu} and H_{wu} may be computed as follows:

$$M_{wu} = \frac{t_w D^2}{12} \left| RF_{cu} - R_{cu} f_{ncu} \right| \tag{10-4l}$$

$$H_{wu} = \frac{t_w D}{2} (RF_{cu} + R_{cu} f_{ncu}) \tag{10-4m}$$

where:

F_{cu} = design stress for the controlling flange at the point of splice defined in Article 10.18.2.2.1 (positive for tension; negative for compression)
R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when f_{cu} is less than or equal to F_{yw}, where F_{yw} is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to 1.0.
R_{cu} = the absolute value of the ratio of F_{cu} to f_{cu} for the controlling flange
f_{ncu} = flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at the point of splice concurrent with f_{cu} (positive for tension; negative for compression)

10.18.2.3.5 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned to develop the most critical combi-

*For an alternative approach for compact steel sections, reference is made to Firas I. Sheikh-Ibrahim and Karl H. Frank, "The Ultimate Strength of Symmetric Beam Bolted Splices," *AISC Engineering Journal*, 3rd Quarter, 1998, and "The Ultimate Strength of Unsymmetric Beam Bolted Splices," *AISC Engineering Journal*, 2nd Quarter, 2001.

nation of V_{wu}, M_{vu}, M_{wu} and H_{wu}. The connections shall be proportioned as eccentrically loaded connections to develop the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of: 1) an overload design shear, V_{wo}, 2) an overload design moment, M_{vo}, due to the eccentricity of the overload design shear, 3) an overload design moment, M_{wo}, applied at mid-depth of the web representing the portion of the flexural moment that is assumed to be resisted by the web, and 4) for sections where the neutral axis is not located at the mid-depth of the web, an overload horizontal design force resultant, H_{wo}, applied at mid-depth of the web, as follows:

$$V_{wo} = V_o \qquad (10\text{-}4n)$$

where:

V_o = maximum shear in the web due to D + β_L(L+I) at the point of the splice, where β_L is defined in Article 3.22

$$M_{vo} = V_{wo}e \qquad (10\text{-}4o)$$

M_{wo} and H_{wo} may be computed as follows:

$$M_{wo} = \frac{t_w D^2}{12}|f_o - f_{of}| \qquad (10\text{-}4p)$$

$$H_{wo} = \frac{t_w D}{2}(f_o + f_{of}) \qquad (10\text{-}4q)$$

where:

f_o = maximum flexural stress due to D + β_L(L+I) at the mid-thickness of the flange under consideration for the smaller section at the point of splice (positive for tension; negative for compression)

f_{of} = flexural stress due to D + β_L(L+I) at the mid-thickness of the other flange at the point of splice concurrent with f_o in the flange under consideration (positive for tension; negative for compression)

f_o and f_{of} shall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Equation (10-172) with N_b taken equal to 1.0.

10.18.2.3.6 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned for a design shear stress in the web at the point of splice, F_w, defined as follows:

For $f_v < 0.5F_V$:

$$F_w = 1.5f_v \qquad (10\text{-}4r)$$

For $f_v \geq 0.5F_V$:

$$F_w = \frac{(f_v + F_v)}{2} \qquad (10\text{-}4s)$$

where:

f_v = maximum shear stress in the web at the point of splice

F_v = allowable shear stress in the web at the point of splice

10.18.2.3.7 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned for a design moment, M_v, due to the eccentricity of the design shear at the point of splice defined as follows:

$$M_v = F_w D t_w e \qquad (10\text{-}4t)$$

where:

F_w = design shear stress in the web at the point of splice defined in Article 10.18.2.3.6

D = web depth

t_w = web thickness

10.18.2.3.8 As a minimum, in cases of the service load design method, web splice plates and their connections shall be proportioned for a design moment at the point of splice, M_w, representing the portion of the flexural moment that is assumed to be resisted by the web. M_w shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at the mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H_w, shall also be applied at the mid-depth of the web. M_w and H_w may be computed as follows:

$$M_w = \frac{t_w D^2}{12}|RF_{cf} - R_{cf}f_{ncf}| \qquad (10\text{-}4u)$$

$$H_w = \frac{t_w D}{2}(RF_{cf} + R_{cf}f_{ncf}) \qquad (10\text{-}4v)$$

where:

F_{cf} = design stress at the point of splice for the controlling flange defined in Article 10.18.2.2.3 (positive for tension; negative for compression)

R = reduction factor for hybrid girders specified in Article 10.40.2.1. R shall be taken equal to 1.0 when F_{cf} is less than or equal to the allowable flexural stress for the web steel. For homogeneous girders, R shall always be taken equal to 1.0.

R_{cf} = the absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange

f_{ncf} = flexural stress at the mid-thickness of the non-controlling flange at the point of splice concurrent with f_{cf} (positive for tension; negative for compression)

10.18.2.3.9 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned to develop the most critical combination of F_wDt_w, M_v, M_w and H_w. The connections shall be proportioned as eccentrically loaded connections to develop the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Table 10.32.3B. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of shear, moment, and horizontal force due to D + (L + I) at the point of splice. The portion of the flexural moment that is assumed to be resisted by the web and the horizontal force resultant shall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Article 10.32.3.2.1 with N_b taken to equal 1.0.

10.18.3 Compression Members

Compression members such as columns and chords shall have ends in close contact at riveted and bolted splices. Splices of such members which will be fabricated and erected with close inspection and detailed with milled ends in full contact bearing at the splices may be held in place by means of splice plates and rivets or high-strength bolts proportioned for not less than 50% of the lower allowable design strength of the sections spliced. The strength of compression members connected by high-strength bolts or rivets shall be determined using the gross section.

10.18.4 Tension Members

10.18.4.1. As a minimum, splices in tension members shall be proportioned for a design force, P_u, equal to the allowable design strength specified in Article 10.18.1.1 times the effective area of the member, A_e, defined as follows:

$$A_e = A_n + \beta A_g \leq A_g \qquad (10\text{-}4w)$$

where:

A_n = net section of the member computed as specified in Article 10.16.14

β = 0.0 for AASHTO M 270 Grade 100/100W (ASTM A 709 Grade 100/100W) steels, or when holes exceed 1¼ inch in diameter
= 0.15 for all other steels and when holes are less than or equal to 1¼ inch in diameter.

A_g = gross area of the member

The diameter of the holes shall be taken as specified in Article 10.16.14.6. As a minimum, the connection shall be proportioned to develop the design force through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2 in the case of the strength design method and in Table 10.32.3B in the case of the service load design method.

10.18.4.2 As a minimum, in the case of the strength design method, high-strength bolted connections for splices in tension members shall be proportioned to prevent slip at an overload design force, P_o, equal to the maximum tensile stress in the member due to D + β_L (L + I) times the gross area of the member, where β_L is defined in Article 3.22. The slip resistance of the connection shall be computed from Equation (10-172). In the case of the service load design method, high-strength bolted connections shall be proportioned to prevent slip at a force equal to the maximum tensile stress in the member due to D + (L + I) times the gross area of the member. The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1.

10.18.5 Welded Splices

10.18.5.1 Tension and compression members may be spliced by means of full penetration butt welds, preferably without the use of splice plates.

10.18.5.2 Welded field splices preferably should be arranged to minimize overhead welding.

10.18.5.3 Material of different widths spliced by butt welds shall have transitions conforming to Figure 10.18.5A. The type transition selected shall be consistent with the Fatigue Stress Category from Table 10.3.1B for the Groove Welded Connection used in the design of the member. At butt-welded splices joining pieces of different thicknesses, there shall be a uniform slope between the offset surfaces, including the weld, of not more than 1 in 2½.

10.19 STRENGTH OF CONNECTIONS

10.19.1 General

10.19.1.1 Except as otherwise provided herein, connections for main members shall be designed in the case of service load design for a capacity based on not less than the average of the calculated design stress in the member at the point of connection and the allowable stress of the member at the same point, but, in any event, not less than 75% of the allowable stress in the member. Connections for main members in the case of load factor design shall be designed for not less than the average of the required strength at the point of connection and the strength of the member at the same point, but, in any event, not less than 75% of the strength of the member.

10.19.1.2 Connections shall be made symmetrical about the axis of the members insofar as practicable. Connections, except for lacing bars and handrails, shall contain not less than two fasteners or equivalent weld.

10.19.1.3 Members, including bracing, preferably shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided, if practicable, but if unavoidable the members shall be so proportioned that the combined fiber stresses will not exceed the allowed axial design stress.

10.19.1.4 In the case of connections which transfer total member shear at the end of the member, the gross section shall be taken as the gross section of the connected elements.

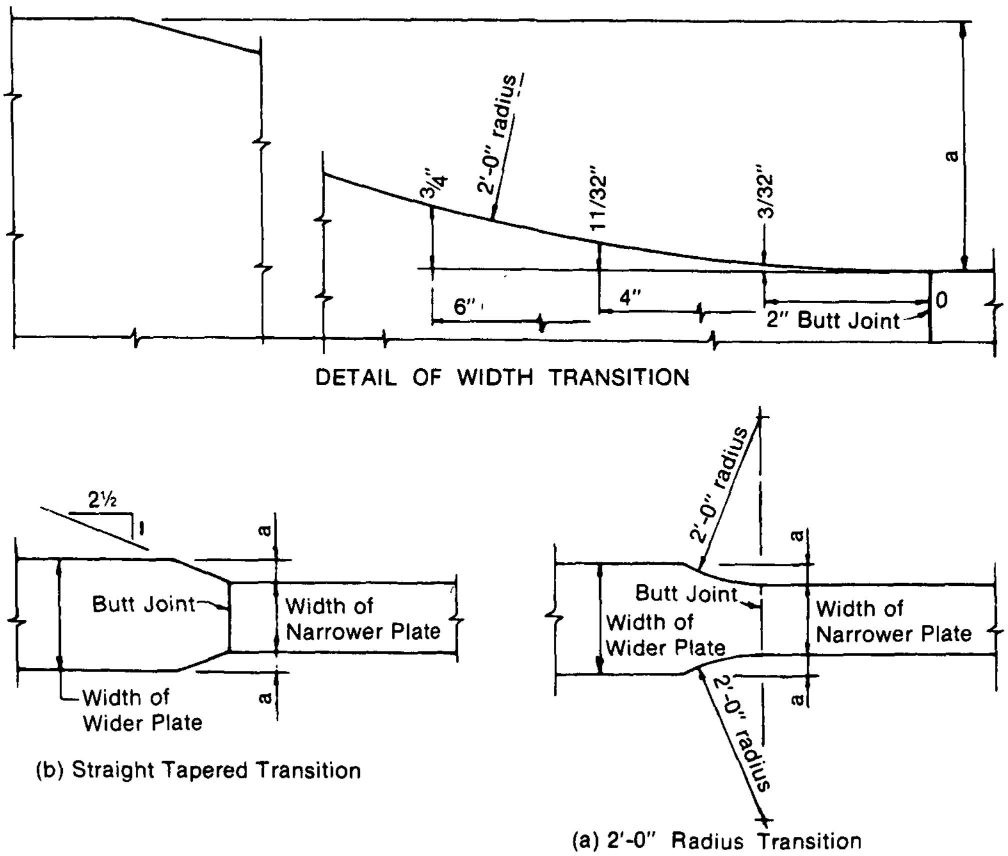


FIGURE 10.18.5A Splice Details

10.19.2 End Connections of Floor Beams and Stringers

10.19.2.1 The end connection shall be designed for the calculated member loads. The end connection angles of floor beams and stringers shall be not less than ⅜ inch in finished thickness. Except in cases of special end floor beam details, each end connection for floor beams and stringers shall be made with two angles. The length of these angles shall be as great as the flanges will permit. Bracket or shelf angles which may be used to furnish support during erection shall not be considered in determining the number of fasteners required to transmit end shear.

10.19.2.2 End-connection details shall be designed with special care to provide clearance for making the field connection.

10.19.2.3 End connections of stringers and floor beams preferably shall be bolted with high-strength bolts; however, they may be riveted or welded. In the case of welded end connections, they shall be designed for the vertical loads and the end-bending moment resulting from the deflection of the members.

10.19.2.4 Where timber stringers frame into steel floor beams, shelf angles with stiffeners shall be provided to carry the total reaction. Shelf angles shall be not less than 7/16 inch thick.

10.19.3 End Connections of Diaphragms and Cross Frames

10.19.3.1 The end connections for diaphragms or cross frames in straight rolled-beam and plate-girder bridges shall be designed for the calculated member loads.

10.19.3.2 Vertical connection plates such as transverse stiffeners which connect diaphragms or cross frames to the beam or girder shall be rigidly connected to both top and bottom flanges.

10.20 DIAPHRAGMS AND CROSS FRAMES

10.20.1 General

Rolled beam and plate girder spans shall be provided with cross frames or diaphragms at each support and with intermediate cross frames or diaphragms placed in all bays and spaced at intervals not to exceed 25 feet. Diaphragms for rolled beams shall be at least ⅓ and preferably ½ the beam depth and for plate girders shall be at least ½ and preferably ¾ the girder depth. Cross frames shall be as deep as practicable. Intermediate cross frames shall preferably be of the cross type or vee type. End cross frames or diaphragms shall be proportioned to adequately transmit all the lateral forces to the bearings. Intermediate cross frames shall be normal to the main members when the supports are skewed more than 20°. Cross frames on horizontally curved steel girder bridges shall be designed as main members with adequate provisions for transfer of lateral forces from the girder flanges. Cross frames and diaphragms shall be designed for horizontal wind forces as described in Article 10.21.2.

10.20.2 Stresses Due to Wind Loading When Top Flanges Are Continuously Supported

10.20.2.1 Flanges

The maximum induced stresses, F, in the bottom flange of each girder in the system can be computed from the following:

$$F = RF_{cb} \qquad (10\text{-}5)$$

where:

$$R = [0.2272L - 11]\, S_d^{-2/3} \quad \begin{pmatrix}\text{when no bottom lateral}\\ \text{bracing is provided}\end{pmatrix} \qquad (10\text{-}6)$$

$$R = [0.059L - 0.64]\, S_d^{-1/2} \quad \begin{pmatrix}\text{when bottom lateral}\\ \text{bracing is provided}\end{pmatrix} \qquad (10\text{-}7)$$

$$F_{cb} = \frac{72M_{cb}}{t_f b_f^2}\,(\text{psi}) \qquad (10\text{-}8)$$

$$M_{cb} = .08WS_d^2\,(\text{ft-lb}) \qquad (10\text{-}9)$$

W = wind loading along the exterior flange (lb/ft)
S_d = diaphragm spacing (ft)
L = span length (ft)
t_f = thickness of flange (in.)
b_f = width of flange (in.)

10.20.2.2 Diaphragms and Cross Frames

The maximum horizontal force (F_D) in the transverse diaphragms and cross frames is obtained from the following:

$$F_D = 1.14WS_d \quad \text{with or without bracing} \qquad (10\text{-}10)$$

10.20.3 Stresses Due to Wind Load When Top Flanges Are Not Continuously Supported

The stress shall be computed using the structural system in the plane of the flanges under consideration.

10.21 LATERAL BRACING

10.21.1 The need for lateral bracing shall be investigated. Flanges attached to concrete decks or other decks of comparable rigidity will not require lateral bracing.

10.21.2 A horizontal wind force of 50 pounds per square foot shall be applied to the area of the superstructure exposed in elevation. Half of this force shall be applied in the plane of each flange. The stress induced shall be computed in accordance with Article 10.20.2.1. The allowable stress shall be factored in accordance with Article 3.22.

10.21.3 When required, lateral bracing preferably shall be placed in the exterior bays between diaphragms or cross-frames. All required lateral bracing shall be placed in or near the plane of the flange being braced.

10.21.4 Where beams or girders comprise the main members of through spans, such members shall be stiffened against lateral deformation by means of gusset plates or knee braces with solid webs which shall be connected to the stiffeners on the main members and the floor beams. If the unsupported length of the edge of the gusset plate (or solid web) exceeds 60 times its thickness, the plate or web shall have a stiffening plate or angles connected along its unsupported edge.

10.21.5 Through truss spans, deck truss spans, and spandrel braced arches shall have top and bottom lateral bracing.

10.21.6 Bracing shall be composed of angles, other shapes, or welded sections. The smallest angle used in bracing shall be 3 by 2½ inches. There shall be not less than two fasteners or equivalent weld in each end connection of the angles.

10.21.7 If a double system of bracing is used, both systems may be considered effective simultaneously if the members meet the requirements both as tension and compression members. The members shall be connected at their intersections.

10.21.8 The lateral bracing of compression chords preferably shall be as deep as the chords and effectively connected to both flanges.

10.22 CLOSED SECTIONS AND POCKETS

10.22.1 Closed sections and pockets or depressions that will retain water, shall be avoided where practicable. Pockets shall be provided with effective drain holes or be filled with waterproofing material.

10.22.2 Details shall be so arranged that the destructive effects of bird life and the retention of dirt, leaves, and other foreign matter will be reduced to a minimum. Where angles are used, either singly or in pairs, they preferably shall be placed with the vertical legs extending downward. Structural tees preferably shall have the web extending downward.

10.23 WELDING

10.23.1 General

10.23.1.1 Steel base to be welded, weld metal, and welding design details shall conform to the requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code.*

10.23.1.2 Welding symbols shall conform with the latest edition of the American Welding Society Publication AWS A2.4

10.23.1.3 Fabrication shall conform to Article 11.4—Division II.

10.23.2 Effective Size of Fillet Welds

10.23.2.1 Maximum Size of Fillet Welds

The maximum size of a fillet weld that may be assumed in the design of a connection shall be such that the stresses in the adjacent base material do not exceed the values allowed in Article 10.32. The maximum size that may be used along edges of connected parts shall be:

(1) Along edges of material less than ¼ inch thick, the maximum size may be equal to the thickness of the material.
(2) Along edges of material ¼ inch or more in thickness, the maximum size shall be ⅟₁₆ inch less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

10.23.2.2 Minimum Size of Fillet Welds

The minimum fillet weld size shall be as shown in the following table.

Base Metal Thickness of Thicker Part Jointed (T)		Minimum Size of Fillet Weld[a,b]		
in.	mm	in.	mm	
T≤3/4	T≤20.0	1/4	6	Single-pass welds must be used
3/4<T	20.0<T	5/16	8	

[a] Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, particular care should be taken to provide sufficient preheat to ensure weld soundness.
[b] Smaller fillet welds may be approved by the Engineer based upon applied stress and the use of appropriate preheat.

10.23.3 Minimum Effective Length of Fillet Welds

The minimum effective length of a fillet weld shall be four times its size and in no case less than 1½ inches.

10.23.4 Fillet Weld End Returns

Fillet welds which support a tensile force that is not parallel to the axis of the weld, or which are proportioned to withstand repeated stress, shall not terminate at corners of parts or members but shall be returned continuously, full size, around the corner for a length equal to twice the weld size where such return can be made in the same plane. End returns shall be indicated on design and detail drawings.

10.23.5 Seal Welds

Seal welding shall preferably be accomplished by a continuous weld combining the functions of sealing and strength, changing section only as the required strength or the requirements of minimum size fillet weld, based on material thickness, may necessitate.

10.24 FASTENERS (RIVETS AND BOLTS)

10.24.1 General

10.24.1.1 In proportioning fasteners, for shear and tension the cross-sectional area based upon the nominal diameter shall be used.

10.24.1.2 High-strength bolts may be substituted for Grade 1 rivets (ASTM A 502) or ASTM A307 bolts. When AASHTO M 164 (ASTM A 325) high-strength bolts are substituted for ASTM A 307 bolts they need not be installed to the requirements of Article 11.5.6.4, Division II, nor inspected to the requirements of Article 11.5.6.4.9, Division II, but shall be tightened to the full effort of a man using an ordinary spud wrench.

10.24.1.3 All bolts, except high-strength bolts tensioned to the requirements of Table 11.5A or Table 11.5B, Division II, shall have single self-locking nuts or double nuts.

10.24.1.4 Joints required to resist shear between their connected parts are designated as either slip-critical or bearing-type connections. Slip-critical joints are defined as joints subject to stress reversal, heavy impact loads, severe vibration or where stress and strain due to joint slippage would be detrimental to the serviceability of the structure. They include:

(1) Joints subject to fatigue loading.
(2) Joints with bolts installed in oversized holes.
(3) Except where the Engineer intends otherwise and so indicates in the contract documents, joints with bolts installed in slotted holes where the force on the joint is in a direction other than normal (between approximately 80 and 100°) to the axis of the slot.
(4) Joints subject to significant load reversal.
(5) Joints in which welds and bolts share in transmitting load at a common faying surface.
(6) Joints in which, in the judgment of the Engineer, any slip would be critical to the performance of the joint or the structure and so designated on the contract plans and specifications.

10.24.1.5 High-strength bolted connections subject to computed tension or combined shear and computed tension shall be slip-critical connections.

10.24.1.6 Bolted bearing-type connections using high-strength bolts shall be limited to members in compression and secondary members.

10.24.1.7 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than ⅜ inch thick, countersunk fasteners shall not be assumed to carry stress. In metal ⅜ inch thick and over, one-half the depth of countersink shall be omitted in calculating the bearing area.

10.24.1.8 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread runout.

10.24.1.9 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote h).

10.24.2 Hole Types

Hole types for high-strength bolted connections are standard holes, oversize holes, short slotted holes and long slotted holes. The nominal dimensions for each type hole shall be not greater than those shown in Table 10.24.2, except as may be permitted under Division II, Article 11.4.8.1.4.

10.24.2.1 In the absence of approval by the Engineer for use of other hole types, standard holes shall be used in high-strength bolted connections.

10.24.2.2 When approved by the Engineer, oversize, short slotted holes or long slotted holes may be used subject to the following joint detail requirements.

10.24.2.2.1 Oversize holes may be used in all plies of connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3, as applicable. Oversize holes shall not be used in bearing-type connections.

10.24.2.2.2 Short slotted holes may be used in any or all plies of high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100°) to the axis of the slot. Short slotted holes may be used without regard for the direction of applied load in any or all plies of connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable.

10.24.2.2.3 Long slotted holes may be used in one of the connected parts at any individual faying surface in high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100°) to the axis of the slot. Long slotted holes may be used in one of the connected parts at any individual faying surface without regard for the direction of applied load on connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable.

TABLE 10.24.2 Nominal Hole Dimension

	Hole Dimensions			
Bolt (Dia.)	Standard (Dia.)	Oversize (Dia.)	Short Slot (Width × Length)	Long Slot (Width × Length)
$\frac{5}{8}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + \frac{1}{16}$	$d + \frac{5}{16}$	$(d + \frac{1}{16}) \times (d + \frac{3}{8})$	$(d + \frac{1}{16}) \times (2.5 \times d)$

10.24.3 Washer Requirements

Design details shall provide for washers in high-strength bolted connections as follows:

10.24.3.1 Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism.

10.24.3.2 Hardened washers are not required for connections using AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) bolts except as required in Articles 10.24.3.3 through 10.24.3.7.

10.24.3.3 Hardened washers shall be used under the element turned in tightening when the tightening is to be performed by calibrated wrench method.

10.24.3.4 Irrespective of the tightening method, hardened washers shall be used under both the head and the nut when AASHTO M 253 (ASTM A 490) bolts are to be installed in material having a specified yield point less than 40 ksi.

10.24.3.5 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in an oversize or short slotted hole in an outer ply, a hardened washer conforming to ASTM F 436 shall be used.

10.24.3.6 When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be installed in an oversize or short slotted hole in an outer ply, hardened washers conforming to ASTM F 436 except with $\frac{5}{16}$ inch minimum thickness shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than $\frac{5}{16}$ inch do not satisfy this requirement.

10.24.3.7 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least $\frac{5}{16}$ inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened except as follows. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to ASTM F 436 but with 5/16 inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement.

10.24.4 Size of Fasteners (Rivets or High-Strength Bolts)

10.24.4.1 Fasteners shall be of the size shown on the drawings, but generally shall be 3/4 inch or 7/8 inch in diameter. Fasteners 5/8 inch in diameter shall not be used in members carrying calculated stress except in 2 1/2-inch legs of angles and in flanges of sections requiring 5/8-inch fasteners.

10.24.4.2 The diameter of fasteners in angles carrying calculated stress shall not exceed one-fourth the width of the leg in which they are placed.

10.24.4.3 In angles whose size is not determined by calculated stress, 5/8-inch fasteners may be used in 2-inch legs, 3/4-inch fasteners in 2 1/2-inch legs, 7/8-inch fasteners in 3-inch legs, and 1-inch fasteners in 3 1/2-inch legs.

10.24.4.4 Structural shapes which do not admit the use of 5/8-inch diameter fasteners shall not be used except in handrails.

10.24.5 Spacing of Fasteners

10.24.5.1 Pitch and Gage of Fasteners

The pitch of fasteners is the distance along the line of principal stress, in inches, between centers of adjacent fasteners, measured along one or more fastener lines. The gage of fasteners is the distance in inches between adjacent lines of fasteners or the distance from the back of angle or other shape to the first line of fasteners.

10.24.5.2 Minimum Spacing of Fasteners

The minimum distance between centers of fasteners in standard holes shall be three times the diameter of the fastener but, preferably, shall not be less than the following:

For 1-inch fasteners, 3 1/2 inches
For 7/8-inch fasteners, 3 inches
For 3/4-inch fasteners, 2 1/2 inches
For 5/8-inch fasteners, 2 1/4 inches

10.24.5.3 Minimum Clear Distance Between Holes

When oversize or slotted holes are used, the minimum clear distance between the edges of adjacent bolt holes in the direction of the force and transverse to the direction of the force shall not be less than twice the diameter of the bolt.

10.24.5.4 Maximum Spacing of Fasteners

The maximum spacing of fasteners shall be in accordance with the provisions of Article 10.24.6, as applicable.

10.24.6 Maximum Spacing of Sealing and Stitch Fasteners

10.24.6.1 Sealing Fasteners

For sealing against the penetration of moisture in joints, the fastener spacing along a single line of fasteners adjacent to a free edge of an outside plate or shape shall not exceed 4 inches + 4t or 7 inches. If there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge, at a gage "g" less than 1 1/2 inches + 4t therefrom, the staggered spacing in two such lines, considered together, shall not exceed 4 inches + 4t − 3g/4 or 7 inches, but need not be less than one-half the requirement for a single line, t = the thickness in inches of the thinner outside plate or shape, and g = gage between fasteners in inches.

10.24.6.2 Stitch Fasteners

In built-up members where two or more plates or shapes are in contact, stitch fasteners shall be used to ensure that the parts act as a unit and, in compression members, to prevent buckling. In compression members the pitch of stitch fasteners on any single line in the direction of stress shall not exceed 12t, except that, if the fasteners on adjacent lines are staggered and the gage, g, between the line under consideration and the farther adjacent line (if there are more than two lines) is less than 24t, the staggered pitch in the two lines, considered together, shall not exceed 12t or 15t − 3g/8. The gage between adjacent lines of fasteners shall not exceed 24t; t = the thickness, in inches, of the thinner outside plate or shape. In tension members the pitch shall not exceed twice that specified for compression members and the gage shall not exceed that specified for compression members.

The maximum pitch of fasteners in built-up members shall be governed by the requirements for sealing or stitch fasteners, whichever is the minimum.

For pitch of fasteners in the ends of compression members, see Article 10.16.13.

10.24.7 Edge Distance of Fasteners

10.24.7.1 General

The minimum distance from the center of any fastener in a standard hole to a sheared or thermally cut edge shall be:

For 1-inch fasteners, 1¾ inches
For ⅞-inch fasteners, 1½ inches
For ¾-inch fasteners, 1¼ inches
For ⅝-inch fasteners, 1⅛ inches

The minimum distance from the center of any fastener in a standard hole to a rolled or planed edge, except in flanges of beams and channels, shall be:

For 1-inch fasteners, 1½ inches
For ⅞-inch fasteners, 1¼ inches
For ¾-inch fasteners, 1⅛ inches
For ⅝-inch fasteners, 1 inch

In the flanges of beams and channels the minimum distance from the center of a standard hole to the edge of the flange shall be:

For 1-inch fasteners, 1¼ inches
For ⅞-inch fasteners, 1⅛ inches
For ¾-inch fasteners, 1 inch
For ⅝-inch fasteners, ⅞ inch

The maximum distance from the center of any fastener to any edge shall be eight times the thickness of the thinnest outside plate, but shall not exceed 5 inches.

10.24.7.2 When there is only a single transverse fastener in the direction of the line of force in a standard or short slotted hole, the distance from the center of the hole to the edge of the connected part shall not be less than 1½ times the diameter of the fastener, unless accounted for by the bearing provisions of Table 10.32.3B or Article 10.56.1.3.2.

10.24.7.3 When oversize or slotted holes are used, the clear distance between edges of holes and edges of members shall not be less than the diameter of the bolt.

10.24.8 Long Rivets

Rivets subjected to calculated stress and having a grip in excess of 4½ diameters shall be increased in number at least 1% for each additional ¹⁄₁₆ inch of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

10.25 LINKS AND HANGERS

10.25.1 Net Section

In pin-connected tension members other than eyebars, the net section across the pin hole shall be not less than 140%, and the net section back of the pin hole not less than 100% of the required net section of the body of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than 8. Flanges not bearing on the pin shall not be considered in the net section across the pin.

10.25.2 Location of Pins

Pins shall be so located with respect to the gravity axis of the members as to reduce to a minimum the stresses due to bending.

10.25.3 Size of Pins

Pins shall be proportioned for the maximum shears and bending moments produced by the stresses in the members connected. If there are eyebars among the parts connected, the diameter of the pin shall be not less than

$$\left[\frac{3}{4} + \frac{\text{(yield point of steel)}}{400{,}000}\right] \text{times the width of the body of the eyebar in inches} \qquad (10\text{-}11)$$

10.25.4 Pin Plates

When necessary for the required section or bearing area, the section at the pin holes shall be increased on each segment by plates so arranged as to reduce to a minimum the eccentricity of the segment. One plate on each side shall be as wide as the outstanding flanges will allow. At least one full-width plate on each segment shall extend to the far edge of the stay plate and the others not less than 6 inches beyond the near edge. These plates shall be connected by enough rivets, bolts, or fillet and plug welds to transmit the bearing pressure, and so arranged as to distribute it uniformly over the full section.

10.25.5 Pins and Pin Nuts

10.25.5.1 Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers. If the pins are bored, through rods with cap washers may be used. Pin nuts shall be malleable castings or steel. They shall be secured by cotter pins in the screw ends or else the screw ends shall be long enough to permit burring the threads.

10.25.5.2 Members shall be restrained against lateral movement on the pins and against lateral distortion due to the skew of the bridge.

10.26 UPSET ENDS

Bars and rods with screw ends, where specified, shall be upset to provide a section at the root of the thread, which will exceed the net section of the body of the member by at least 15%.

10.27 EYEBARS

10.27.1 Thickness and Net Section

Eyebars shall be of a uniform thickness without reinforcement at the pin holes. The thickness of eyebars shall be not less than ⅛ of the width, nor less than ½ inch, and not greater than 2 inches. The section of the head through the center of the pin hole shall exceed the required section of the body of the bar by at least 35%. The net section back of the pin hole shall not be less than 75% of the required net section of the body of the member. The radius of transition between the head and body of the eyebar shall be equal to or greater than the width of the head through the center line of the pin hole.

10.27.2 Packing of Eyebars

10.27.2.1 The eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as practicable. Bars shall be as close together as practicable and held against lateral movement, but they shall be so arranged that adjacent bars in the same panel will be separated by at least ½ inch.

10.27.2.2 Intersecting diagonal bars not far enough apart to clear each other at all times shall be clamped together at the intersection.

10.27.2.3 Steel filling rings shall be provided, if needed, to prevent lateral movement of eyebars or other members connected on the pin.

10.28 FORKED ENDS

Forked ends will be permitted only where unavoidable. There shall be enough pin plates on forked ends to make the section of each jaw equal to that of the member. The pin plates shall be long enough to develop the pin plate beyond the near edge of the stay plate, but not less than the length required by Article 10.25.4.

10.29 FIXED AND EXPANSION BEARINGS

10.29.1 General

10.29.1.1 Fixed ends shall be firmly anchored. Bearings for spans less than 50 feet need have no provision for deflection. Spans of 50 feet or greater shall be provided with a type of bearing employing a hinge, curved bearing plates, elastomeric pads, or pin arrangement for deflection purposes.

10.29.1.2 Spans of less than 50 feet may be arranged to slide upon metal plates with smooth surfaces and no provisions for deflection of the spans need be made. Spans of 50 feet and greater shall be provided with rollers, rockers, or sliding plates for expansion purposes and shall also be provided with a type of bearing employing a hinge, curved bearing plates, or pin arrangement for deflection purposes.

10.29.1.3 In lieu of the above requirements, elastomeric bearings may be used. See Section 14 of this specification.

10.29.2 Bronze or Copper-Alloy Sliding Expansion Bearings

Bronze or copper-alloy sliding plates shall be chamfered at the ends. They shall be held securely in position, usually by being inset into the metal of the pedestals or sole plates. Provisions shall be made against any accumulation of dirt which will obstruct free movement of the span.

10.29.3 Rollers

Expansion rollers shall be connected by substantial side bars and shall be guided by gearing or other effectual means to prevent lateral movement, skewing, and creeping. The rollers and bearing plates shall be protected from dirt and water as far as practicable, and the design shall be such that water will not be retained and that the roller nests may be inspected and clean easily.

10.29.4 Sole Plates and Masonry Plates

10.29.4.1 Sole plates and masonry plates shall have a minimum thickness of ¾ inch.

10.29.4.2 For spans on inclined grades greater than 1% without hinged bearings, the sole plates shall be beveled so that the bottom of the sole plate is level, unless the bottom of the sole plate is radially curved.

10.29.5 Masonry Bearings

Beams, girders, or trusses on masonry shall be so supported that the bottom chords or flanges will be above the bridge seat, preferably not less than 6 inches.

10.29.6 Anchor Bolts

10.29.6.1 Trusses, girders, and rolled beam spans preferably shall be securely anchored to the substructure. Anchor bolts shall be swedged or threaded to secure a satisfactory grip upon the material used to embed them in the holes.

10.29.6.2 The following are the minimum requirements for each bearing:

For rolled beam spans the outer beams shall be anchored at each end with 2 bolts, 1 inch in diameter, set 10 inches in the masonry.

For trusses and girders:

Spans 50 feet in length or less; 2 bolts, 1 inch in diameter, set 10 inches in the masonry.

Spans 51 to 100 feet; 2 bolts, 1¼ inches in diameter, set 12 inches in the masonry.

Spans 101 to 150 feet; 2 bolts, 1½ inches in diameter, set 15 inches in the masonry.

Spans greater than 150 feet; 4 bolts, 1½ inches in diameter, set 15 inches in the masonry.

10.29.6.3 Anchor bolts shall be designed to resist uplift as specified in Article 3.17.

10.29.7 Pedestals and Shoes

10.29.7.1 Pedestals and shoes preferably shall be made of cast steel or structural steel. The difference in width between the top and bottom bearing surfaces shall not exceed twice the distance between them. For hinged bearings, this distance shall be measured from the center of the pin. In built-up pedestals and shoes, the web plates and angles connecting them to the base plate shall be not less than ⅝ inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast steel pedestals shall be 1 inch. Pedestals and shoes shall be so designed that the load will be distributed uniformly over the entire bearing.

10.29.7.2 Webs and pin holes in the webs shall be arranged to keep any eccentricity to a minimum. The net section through the hole shall provide 140% of the net section required for the actual stress transmitted through the pedestal or shoe. Pins shall be of sufficient length to secure a full bearing. Pins shall be secured in position by appropriate nuts with washers. All portions of pedestals and shoes shall be held against lateral movement of the pins.

10.30 FLOOR SYSTEM

10.30.1 Stringers

Stringers preferably shall be framed into floor beams. Stringers supported on the top flanges of floor beams preferably shall be continuous over two or more panels.

10.30.2 Floor Beams

Floor beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Floor beam connections preferably shall be located so the lateral bracing system will engage both the floor beam and the main supporting member. In pin-connected trusses, if the floor beams are located below the bottom chord pins, the vertical posts shall be extended sufficiently below the pins to make a rigid connection to the floor beam.

10.30.3 Cross Frames

In bridges with wooden floors and steel stringers, intermediate cross frames (or diaphragms) shall be placed between stringers more than 20 feet long.

10.30.4 Expansion Joints

10.30.4.1 To provide for expansion and contraction movement, floor expansion joints shall be provided at all expansion ends of spans and at other points where they may be necessary.

10.30.4.2 Apron plates, when used, shall be designed to bridge the joint and to prevent, so far as practicable, the accumulation of roadway debris upon the bridge seats. Preferably, they shall be connected rigidly to the end floor beam.

10.30.5 End Floor Beams

There shall be end floor beams in all square-ended trusses and girder spans and preferably in skew spans. End floor beams for truss spans preferably shall be designed to permit the use of jacks for lifting the superstructure. For this case, the allowable stresses may be increased 50%.

10.30.6 End Panel of Skewed Bridges

In skew bridges without end floor beams, the end panel stringers shall be secured in correct position by end struts connected to the stringers and to the main truss or girder. The end panel lateral bracing shall be attached to the main trusses or girders and also to the end struts. Adequate provisions shall be made for the expansion movement of stringers.

10.30.7 Sidewalk Brackets

Sidewalk brackets shall be connected in such a way that the bending stresses will be transferred directly to the floor beams.

10.30.8 Stay-in-Place Deck Forms

10.30.8.1 Concrete Deck Panels

When precast prestressed deck panels are used as permanent forms spanning between beams, stringers, or girders, the requirements of Article 9.12, Deck Panels, and Article 9.23, Deck Panels, shall be met.

10.30.8.2 Metal Stay-in-Place Forms

When metal stay-in-place forms are used as permanent forms spanning between beams, stringers, or girders, the forms shall be designed to support, as a minimum, the weight of the concrete (including that in the corrugations, if applicable), a construction load of 50 psf, and the weight of the form. The forms shall be designed to be elastic under construction loads. The elastic deformation caused by the dead load of the forms, plastic concrete and reinforcement shall not exceed a deflection of greater than L/180 or ½ inch for form work spans (L) of 10 feet or less, or a deflection of L/240 or ¾ inch for form work spans (L) over 10 feet.

Part C
SERVICE LOAD DESIGN METHOD

ALLOWABLE STRESS DESIGN

10.31 SCOPE

Allowable stress design is a method for proportioning structural members using design loads and forces, allowable stresses, and design limitations for the appropriate material under service conditions. See Part D—Strength Design Method—Load Factor Design for an alternate design procedure.

10.32 ALLOWABLE STRESSES

10.32.1 Steel

Allowable stresses for steel shall be as specified in Table 10.32.1A.

10.32.2 Weld Metal

Unless otherwise specified, the yield point and ultimate strength of weld metal shall be equal to or greater than minimum specified value of the base metal. Allowable stresses on the effective areas of weld metal shall be as follows:

Butt Welds:

The same as the base metal joined, except in the case of joining metals of different yields when the lower yield material shall govern.

Fillet Welds:

$$F_v = 0.27\ F_u \qquad (10\text{-}12)$$

where,

F_v = allowable basic shear stress;
F_u = tensile strength of the electrode classification

When detailing fillet welds for quenched and tempered steels—the designer may use electrode classifications with strengths less than the base metal provided that this requirement is clearly specified on the plans.

Plug Welds:

F_v = 12,400 psi for resistance to shear stresses only, where,
F_v = allowable basic shear stress.

TABLE 10.32.1A Allowable Stresses—Structural Steel (In pounds per square inch)

Type		Structural Carbon Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High-Yield Strength Quenched and Tempered Alloy Steel[a]	
AASHTO Designation[b,c]		M 270 Grade 36	M 270 Grade 50	M 270 Grade 50W	M 270 Grade HPS70W & Grade 70W	M 270 Grades 100/100W	
Equivalent ASTM Designation[c]		A 709 Grade 36	A 709 Grade 50	A 709 Grade 50W	A 709 Grade HPS70W & Grade 70W	A 709 Grades 100/100W	
Thickness of Plates		Up to 4 in. incl.	Up to 4 in. incl.	Up to 4 in. incl.	Up to 4 in. incl.	Up to 2½ in. incl.	Over 2½ in. to 4 in. incl.
Shapes		All Groups	All Groups	All Groups	Not Applicable	Not Applicable	
Axial tension in members with no holes	$0.55F_y$	20,000	27,000	27,000	38,000	Not Applicable	
	$0.46F_u$		Not Applicable			50,600	46,000
Axial tension in members with holes and tension in extreme fiber of rolled shapes, girders, and built-up sections with holes subject to bending. Satisfy both Gross and Net Section criterion.	Gross Section $0.55F_y$	20,000	27,000	27,000	38,000	Not Applicable	
	Net Section $0.46F_u$	26,700	29,900	32,200	41,400	50,600	46,000
Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section		20,000	27,000	27,000	38,000	55,000	49,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections subject to bending. Gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete	$0.55F_y$	20,000	27,000	27,000	38,000	55,000	49,000
(B) Partially supported or is unsupported[d,e]							

$$F_b = \frac{50 \times 10^6 C_b}{S_{xc}}\left(\frac{I_{yc}}{\ell}\right)\sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{\ell}\right)^2} \leq 0.55F_y$$

$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

$C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

Type	Structural Carbon Steel	HSLA Grade 50	HSLA Grade 50W	Q&T Low-Alloy	HY Q&T up to 2½ in.	HY Q&T over 2½ in.
Compression in concentrically loaded columns[f]						
with $C_c = (2\pi^2 E/F_y)^{1/2} =$	126.1	107.0	107.0	90.4	75.7	79.8
when $KL/r \leq C_c$						
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2 F_y}{4\pi^2 E}\right] =$	$16{,}980 - 0.53(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$	$33{,}020 - 2.02(KL/r)^2$	$47{,}170 - 4.12(KL/r)^2$	$42{,}450 - 3.33(KL/r)^2$

TABLE 10.32.1A Allowable Stresses—Structural Steel (In pounds per square inch) (Continued)

Type		Structural Carbon Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High-Yield Strength Quenched and Tempered Alloy Steel[a]	
when $KL/r > C_c$							
$F_a = \dfrac{\pi^2 E}{F.S.(KL/r)^2} =$			$\dfrac{135{,}000{,}740}{(KL/r)^2}$				
with F.S. = 2.12							
Shear in girder webs, gross section	$F_v = 0.33F_y$	12,000	17,000	17,000	23,000	33,000	30,000
Bearing on milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Stress in extreme fiber of pins[g]	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Shear in pins	$F_v = 0.40F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on pins not subject to rotation[h]	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Bearing on pins subject to rotation (such as used in rockers and hinges)	$0.40F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on connected material at Low Carbon Steel Bolts (ASTM A 307), Turned Bolts, Ribbed Bolts, and Rivets (ASTM A 502 Grades 1 and 2)—Governed by Table 10.32.3A							

[a] Quenched and tempered alloy steel structural shapes and seamless mechanical tubing meeting all mechanical and chemical requirements of A 709 Grades 100/100W except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechanical tubing, shall be considered as A 709 Grades 100/100W steel.

[b] Except for the mandatory notch toughness and weldability requirements, the ASTM designations are similar to the AASHTO designations. Steels meeting the AASHTO requirements are prequalified for use in welded bridges.

[c] M 270 Gr. 36 and A 709 Gr. 36 are equivalent to M 183 and A 36
M 270 Gr. 50 and A 709 Gr. 50 are equivalent to M 223 Gr. 50 and A 572 Gr. 50
M 270 Gr. 50W and A 709 Gr. 50W are equivalent to M 222 and A 588
M 270 Gr. 70W and A 709 Gr. 70W are equivalent to A 852
M 270 Gr. 100/100W and A 709 Gr. 100/100W are equivalent to M 244 and A 514

[d] For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., p. 135.
If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

[e] ℓ = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support.
I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web in.^{4}
d = depth of girder, in.
$J = \dfrac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, respectively (in.^{4}).
S_{xc} = section modulus with respect to compression flange (in.^{3}).

[f] E = modulus of elasticity of steel
r = governing radius of gyration
L = actual unbraced length
K = effective length factor (see Appendix C.)
F.S. = factor of safety = 2.12
For graphic representation of these formulas, see Appendix C.
The formulas do not apply to members with variable moment of inertia. Procedures for designing members with variable moments of inertia can be found in the following references: "Engineering Journal," American Institute of Steel Construction, January 1969, Volume 6, No. 1, and October 1972, Volume 9, No. 4; and "Steel Structures," by William McGuire, 1968, Prentice-Hall, Inc., Englewood Cliffs, New Jersey. For members with eccentric loading, see Article 10.36. Singly symmetric and unsymmetric compression members, such as angles or tees, and doubly symmetric compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckling. Refer to the *Manual of Steel Construction*, Ninth Edition, 1989, American Institute of Steel Construction.

[g] See also Article 10.32.4.

[h] This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

TABLE 10.32.3A Allowable Stresses for Low-Carbon Steel Bolts and Power Driven Rivets (in psi)

Type of Fastener	Tension[a]	Bearing[b]	Shear Bearing-Type Connection[a]
(A) Low-Carbon Steel Bolts[c] Turned Bolts (ASTM A 307) Ribbed Bolts	18,000	20,000	11,000[d]
(B) Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven)			
Structural Steel Rivet Grade 1 (ASTM A 502 Grade 1)	—	40,000	13,500
Structural Steel Rivet (high-strength) Grade 2 (ASTM A 502 Grade 2)	—	40,000	20,000

[a] Applies to fastener cross-sectional area based upon nominal body diameter.

[b] Applies to nominal diameter of fastener multiplied by the thickness of the metal.

[c] ASTM A 307 bolts shall not be used in connections subject to fatigue.

[d] In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 inches, the tabulated value shall be reduced 20 percent.

10.32.3 Fasteners (Rivets and Bolts)

Allowable stresses for fasteners shall be as listed in Tables 10.32.3.A and 10.32.3.B, and the allowable force on a slip-critical connection shall be as provided by Article 10.32.3.2.1.

10.32.3.1 General

10.32.3.1.1 In proportioning fasteners for shear or tension, the cross-sectional area based upon the nominal diameter shall be used except as otherwise noted.

10.32.3.1.2 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than 3/8 inch thick, countersunk fasteners shall not be assumed to carry stress. In metal 3/8 inch thick and over, one-half of the depth of the countersink shall be omitted in calculating the bearing area.

10.32.3.1.3 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread runout.

10.32.3.1.4 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote g.)

10.32.3.1.5 All bolts except high-strength bolts, tensioned to the requirements of Division II. Table 11.5A or Table 11.5B, shall have single self-locking nuts or double nuts.

10.32.3.1.6 Joints, utilizing high-strength bolts, required to resist shear between their connected parts are designated as either slip-critical (See Article 10.24.1.4) or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be slip-critical connections. Potential slip

TABLE 10.32.3B Allowable Stresses on High-Strength Bolts or Connected Material (ksi)

Load Condition	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)[a]
Applied Static Tension[b,c]	38[d]	47
Shear, F_v, on bolt with threads in shear plane[e,f]	19[d]	24
Bearing, F_p, on connected material in standard, oversize, short-slotted holes loaded in any direction, or long-slotted holes parallel to the applied bearing force	$\frac{0.5L_cF_u}{d} \leq F_u$[g,h,i,j]	
Bearing, F_p, on connected material in long-slotted holes perpendicular to the applied bearing force	$\frac{0.4L_cF_u}{d} \leq 0.8F_u$[g,h,i,j]	

[a] AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) high-strength bolts are available in three types, designated as types 1, 2, or 3. Type 3 shall be required on the plans when using unpainted AASHTO M 270 Grade 50W (ASTM A 709 Grade 50W).

[b] Bolts must be tensioned to requirements of Table 11.5A, Div II.

[c] See Article 10.32.3.4 for bolts subject to tensile fatigue.

[d] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1 inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.

[e] In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 inches, tabulated values shall be reduced 20 percent. For flange splices, the 50-inch length is to be measured between the extreme bolts on only one side of the connection.

[f] If material thickness or joint details preclude threads in the shear plane, multiply tabulated values by 1.25.

[g] F_u = specified minimum tensile strength of connected material.

[h] Connections using high-strength bolts in slotted holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversized holes shall be designed for resistance against slip in accordance with Article 10.32.3.2.1.

[i] L_c is equal to the clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force, in. and d is the nominal diameter of the bolt, in.

[j] The allowable bearing force for the connection is equal to the sum of the allowable bearing forces for the individual bolts in the connection.

of joints should be investigated at intermediate load stages especially those joints located in composite regions.

10.32.3.1.7 The percentage of unit stress increase shown in Article 3.22, Combination of Loads, shall apply to allowable stresses in bolted slip-critical connections using high-strength bolts, except that in no case shall the percentage of allowable stress exceed 133%, and the requirements of Article 10.32.3.3 shall not be exceeded.

10.32.3.1.8 Bolted bearing-type connections shall be limited to members in compression and secondary members.

10.32.3.2 The allowable stress in shear, bearing and tension for AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) bolts shall be as listed in Table 10.32.3B.

10.32.3.2.1 In addition to the allowable stress requirements of Article 10.32.3.2 the force on a slip-critical connection as defined in Article 10.24.1.4 shall not exceed the allowable slip force (P_s) of the connection according to

$$P_s = F_s A_b N_b N_s \qquad (10\text{-}13)$$

Where

F_s = nominal slip resistance per unit of bolt area from Table 10.32.3C, ksi.
A_b = area corresponding to the nominal body area of the bolt sq in.
N_b = number of bolts in the joint.
N_s = number of slip planes.

Class A, B, or C surface conditions of the bolted parts as defined in Table 10.32.3C shall be used in joints designated as slip-critical except as permitted in Article 10.32.3.2.2.

10.32.3.2.2 Subject to the approval of the Engineer, coatings providing a slip coefficient less than 0.33 may be used provided the mean slip coefficient is established by test in accordance with the requirements of Article 10.32.3.2.3, and the slip resistance per unit area are established. The slip resistance per unit area shall be taken as equal to the slip resistance per unit area from Table 10.32.3C for Class A coatings as appropriate for the hole type and bolt type times the slip coefficient determined by test divided by 0.33.

10.32.3.2.3 Paint, used on the faying surfaces of connections specified to be slip-critical, shall be qualified by test in accordance with "Test Method to Determine the

TABLE 10.32.3C Nominal Slip Resistance for Slip-Critical Connections (Slip Resistance per Unit of Bolt Area, F_s, ksi)

	Hole Type and Direction of Load Application							
	Any Direction				Transverse		Parallel	
	Standard		Oversized and Short Slot		Long Slots		Long Slots	
Contact Surface of Bolted Parts	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)
Class A (Slip Coefficient 0.33) Clean mill scale and blast-cleaned surfaces with Class A coatings[b]	15	19	13	16	11	13	9	11
Class B (Slip Coefficient 0.50) Blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings[b]	23	29	19	24	16	20	14	17
Class C (Slip Coefficient 0.33) Hot-dip galvanized surfaces and roughened by wire brushing after galvanizing	15	19	13	16	11	13	9	11

[a] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1 inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.

[b] Coatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.50, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints. See Article 10.32.3.2.3.

Slip Coefficient for Coatings Used in Bolted Joints" as adopted by the Research Council on Structural Connections. See Appendix A of Allowable Stress Design Specification for Structural Joints Using ASTM A 325 or A 490 Bolts published by the Research Council on Structural Connections.

10.32.3.3 Applied Tension, Combined Tension, and Shear

10.32.3.3.1 High-strength bolts preferably shall be used for fasteners subject to tension or combined tension and shear.

10.32.3.3.2 Bolts required to support applied load by means of direct tension shall be so proportioned that their average tensile stress computed on the basis of nominal bolt area will not exceed the appropriate stress in Table 10.32.3B. The applied load shall be the sum of the external load and any tension resulting from prying action. The tension due to the prying action shall be

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right]T \qquad (10\text{-}14)$$

where

- Q = the prying tension per bolt (taken as zero when negative);
- T = the direct tension per bolt due to external load;
- a = distance from center of bolt to edge of plate in inches;
- b = distance from center of bolt under consideration to toe of fillet of connected part in inches;
- t = thickness of thinnest part connected in inches.

10.32.3.3.3 For combined shear and tension in slip-critical joints using high-strength bolts where applied forces reduce the total clamping force on the friction plane, the slip resistance per unit area of bolt, f_v, shall not exceed the value obtained from the following equation:

$$f_v = F_s(1 - 1.88f_t/F_u) \qquad (10\text{-}15)$$

where:

- f_t = computed tensile stress in the bolt due to applied loads including any stress due to prying action, ksi;
- F_s = nominal slip resistance per unit of bolt area from Table 10.32.3C, ksi;
- F_u = 120 ksi for M 164 (A 325) bolts up to 1-inch diameter;
  = 105 ksi for M 164 (A 325) bolts over 1-inch diameter;
  = 150 ksi for M 253 (A 490) bolts.

10.32.3.3.4 Where rivets or high-strength bolts are subject to both shear and tension, the tensile stress shall not exceed the value obtained from the following equations:

for $f_v/F_v \leq 0.33$

$$F_t' = F_t \qquad (10\text{-}16)$$

for $f_v/F_v > 0.33$

$$F_t' = F_t\sqrt{1-(f_v/F_v)^2} \qquad (10\text{-}17)$$

where

- f_v = computed rivet or bolt shear stress in shear, ksi;
- F_v = allowable shear stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B, ksi;
- F_t = allowable tensile stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B, ksi;
- F_t' = reduced allowable tensile stress on rivet or bolt due to the applied shear stress, ksi.

Note: Equation (10-18) has been removed.

10.32.3.4 Fatigue

When subject to tensile fatigue loading, the tensile stress in the bolt due to the service load plus the prying force resulting from application of service load shall not exceed the following design stresses in kips per square inch. The nominal diameter of the bolt shall be used in calculating the bolt stress. The prying force shall not exceed 60% of the externally applied load.

Number of Cycles	AASHTO M 164 (ASTM A 325)	AASHTO M 253 (ASTM A 490)
Not more than 20,000	38	47
From 20,000 to 500,000	35.5	44.0
More than 500,000	27.5	34.0

10.32.4 Pins, Rollers, and Expansion Rockers

10.32.4.1 The effective bearing area of a pin shall be its diameter multiplied by the thickness of the material on

TABLE 10.32.4.3A Allowable Stresses—Steel Bars and Steel Forgings

AASHTO Designation with Size Limitations	—	M 169 4 in. in dia. or less	M 102 To 20 in. in dia.	M 102 To 20 in. in dia.	M 102 To 10 in. in dia.	M 102 To 20 in. in dia.
ASTM Designation Grade or Class	—	A 108 Grades 1016 1030 incl.	A 668 Class C	A 668 Class D	A 668 Class F	A 668[a] Class G
Minimum Yield Point, psi	F_y	36,000[b]	33,000	37,500	50,000	50,000
Stress in Extreme Fiber, psi	$0.80F_y$	29,000[b]	26,000	30,000	40,000	40,000
Shear, psi	$0.40F_y$	14,000[b]	13,000	15,000	20,000	20,000
Bearing on Pins not Subject to Rotation, psi[c]	$0.80F_y$	29,000[b]	26,000	30,000	40,000	40,000
Bearing on Pins Subject to Rotation, psi (such as used in rockers and hinges)	$0.40F_y$	14,000[b]	13,000	15,000	20,000	20,000

[a] May substitute rolled material of the same properties.
[b] For design purposes only. Not a part of the A 108 specifications. Supplementary material requirements should provide guarantee that material will meet these values.
[c] This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

which it bears. When parts in contact have different yield points, F_y shall be the smaller value.

10.32.4.2 Design stresses for Steel Bars, Carbon Cold Finished Standard Quality, AASHTO M 169 (ASTM A 108), and Steel Forgings, Carbon and Alloy, for General Industrial Use, AASHTO M 102 (ASTM A 668), are given in Table 10.32.4.3A.

10.32.5 Cast Steel, Ductile Iron Castings, Malleable Castings, and Cast Iron

10.32.5.1 Cast Steel and Ductile Iron

10.32.5.1.1 For cast steel conforming to specifications for Steel Castings for Highway Bridges, AASHTO M 192 (ASTM A 486), Mild-to-Medium-Strength Carbon-Steel Castings for General Application, AASHTO M 103 (ASTM A 27), and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and Nickel-Based Alloy Castings for General Application, AASHTO M 163 (ASTM A 743), and for Ductile Iron Castings (ASTM A 536), the allowable stresses in pounds per square inch shall be in accordance with Table 10.32.5.1A.

10.32.5.1.2 When in contact with castings or steel of a different yield point, the allowable unit bearing stress of the material with the lower yield point shall govern. For riveted or bolted connections, Article 10.32.3 shall govern.

10.32.5.2 Malleable Castings

Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47 Grade 35018. The following allowable stresses in pounds per square inch shall be used:

Tension 18,000
Bending in Extreme Fiber 18,000
Modulus of Elasticity 25,000,000

10.32.5.3 Cast Iron

Cast iron castings shall conform to specifications for Gray Iron Castings, AASHTO M 105 (ASTM A 48), Class 30B. The following allowable stresses in pounds per square inch shall be used:

Bending in Extreme Fiber 3,000
Shear 3,000
Direct Compression, Short Columns 12,000

10.32.5.4 Bronze or Copper-Alloy

10.32.5.4.1 Bronze castings, AASHTO M 107 (ASTM B 22), Copper Alloys 913 or 911, or Copper-Alloy Plates, AASHTO M 108 (ASTM B 100), shall be specified.

10.32.5.4.2 The allowable unit-bearing stress in pounds per square inch on bronze castings or copper-alloy plates shall be 2,000.

TABLE 10.32.5.1A Allowable Stresses—Cast Steel and Ductile Iron

<table>
<tr><td>AASHTO Designation</td><td>M 103</td><td>M 192</td><td colspan="2">M 192</td><td>M 163</td><td>None</td></tr>
<tr><td>ASTM Designation</td><td>A 27</td><td>A 486</td><td colspan="2">A 486</td><td>A 743</td><td>A 536</td></tr>
<tr><td>Class or Grade</td><td>70-36</td><td>70</td><td>90</td><td>120</td><td>CA-15</td><td>60-40-18</td></tr>
<tr><td>Minimum Yield Point, F_y</td><td colspan="2">36,000</td><td>60,000</td><td>95,000</td><td>65,000</td><td>40,000</td></tr>
<tr><td>Axial Tension</td><td colspan="2">14,500</td><td>22,500</td><td>34,000</td><td>24,000</td><td>16,000</td></tr>
<tr><td>Tension in Extreme Fibers</td><td colspan="2">14,500</td><td>22,500</td><td>34,000</td><td>24,000</td><td>16,000</td></tr>
<tr><td>Axial Compression, Short Columns</td><td colspan="2">20,000</td><td>30,000</td><td>45,000</td><td>32,000</td><td>22,000</td></tr>
<tr><td>Compression in Extreme Fibers</td><td colspan="2">20,000</td><td>30,000</td><td>45,000</td><td>32,000</td><td>22,000</td></tr>
<tr><td>Shear</td><td colspan="2">9,000</td><td>13,500</td><td>21,000</td><td>14,000</td><td>10,000</td></tr>
<tr><td>Bearing, Steel Parts in Contact</td><td colspan="2">30,000</td><td>45,000</td><td>68,000</td><td>48,000</td><td>33,000</td></tr>
<tr><td>Bearing on Pins not Subject to Rotation</td><td colspan="2">26,000</td><td>40,000</td><td>60,000</td><td>43,000</td><td>28,000</td></tr>
<tr><td>Bearing on Pins Subject to Rotation (such as used in rockers and hinges)</td><td colspan="2">13,000</td><td>20,000</td><td>30,000</td><td>21,500</td><td>14,000</td></tr>
</table>

10.32.6 Bearing on Masonry

10.32.6.1 The allowable unit-bearing stress in pounds per square inch on the following types of masonry shall be:

Granite 800
Sandstone and Limestone 400

10.32.6.2 The above bridge seat unit stress will apply only where the edge of the bridge seat projects at least 3 inches (average) beyond the edge of shoe or plate. Otherwise, the unit stresses permitted will be 75% of the above amounts.

10.32.6.3 For allowable unit-bearing stress on concrete masonry, refer to Article 8.15.2.1.3.

10.33 ROLLED BEAMS

10.33.1 General

10.33.1.1 Rolled beams, including those with welded cover plates, shall be designed by the moment of inertia method. Rolled beams with riveted cover plates shall be designed on the same basis as riveted plate girders.

10.33.1.2 The compression flanges of rolled beams supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support.

10.33.2 Bearing Stiffeners

Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings when the unit shear in the web adjacent to the bearing exceeds 75% of the allowable shear for girder webs. See the related provisions of Article 10.34.6.

10.34 PLATE GIRDERS

10.34.1 General

10.34.1.1 Girders shall be proportioned by the moment of inertia method. For members primarily in bending, the entire gross section shall be used when calculating tensile and compressive stresses. Holes for high-strength bolts or rivets and/or open holes not exceeding 1¼ inches, may be neglected provided the area removed from each flange does not exceed 15% of that flange. That area in excess of 15% shall be deducted from the gross area.

10.34.1.2 The compression flanges of plate girders supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide support.

10.34.2 Flanges

10.34.2.1 Welded Girders

10.34.2.1.1 Each flange may comprise a series of plates joined end to end by full penetration butt welds. Changes in flange areas may be accomplished by varying the thickness and/or width of the flange plate, or by adding cover plates. Where plates of varying thicknesses or widths are connected, the splice shall be made in accordance with Article 10.18 and welds ground smooth before attaching to the web. The compression-flange width, b, on fabricated I-shaped girders preferably shall not be less than 0.2 times the web depth, but in no case shall it be less than 0.15 times the web depth. If the area of the compression flange is less than the area of the tension flange, the minimum flange width may be based on two times the depth of the web in compression rather than the web depth. The compression-flange thickness, t, preferably shall not be less than 1.5 times the web thickness. The width-to-thickness ratio, b/t, of flanges subject to tension shall not exceed 24.

10.34.2.1.2 When cover plates are used, they shall be designed in accordance with Article 10.13.

10.34.2.1.3 The ratio of compression flange plate width to thickness shall not exceed the value determined by the formula

$$\frac{b}{t} = \frac{3{,}250}{\sqrt{f_b}} \quad \text{but in no case shall } b/t \text{ exceed } 24 \qquad (10\text{-}19)$$

10.34.2.1.4 Where the calculated compressive bending stress equals .55 F_y the (b/t) ratios for the various grades of steel shall not exceed the following:

36,000 psi, Y.P. Min. b/t = 23
50,000 psi, Y.P. Min. b/t = 20
70,000 psi, Y.P. Min. b/t = 17
90,000 psi, Y.P. Min. b/t = 15
100,000 psi, Y.P. Min. b/t = 14

In the above b is the flange plate width, t is the thickness, and f_b is the calculated maximum compressive bending stress. (See Article 10.40.3 for Hybrid Girders.)

10.34.2.1.5 In the case of a composite girder the ratio of the top compression flange plate width to thickness shall not exceed the value determined by the formula

$$\frac{b}{t} = \frac{3{,}860}{\sqrt{f_{d\ell 1}}} \quad \text{but in no case shall } b/t \text{ exceed } 24 \qquad (10\text{-}20)$$

where $f_{d\ell 1}$ is the top flange compressive stress due to non-composite dead load.

10.34.2.2 Riveted or Bolted Girders

10.34.2.2.1 Flange angles shall form as large a part of the area of the flange as practicable. Side plates shall not be used except where flange angles exceeding ⅞ inch in thickness otherwise would be required.

10.34.2.2.2 Width of outstanding legs of flange angles in compression, except those reinforced by plates, shall not exceed the value determined by the formula

$$\frac{b'}{t} = \frac{1{,}625}{\sqrt{f_b}} \quad \text{but in no case shall } b'/t \text{ exceed } 12 \qquad (10\text{-}21)$$

10.34.2.2.3 Where the calculated compressive bending stress equals 0.55 F_y, the b′/t ratios for the various grades of steel shall not exceed the following:

36,000 psi, Y.P. Min. b′/t = 11
50,000 psi, Y.P. Min. b′/t = 10
70,000 psi, Y.P. Min. b′/t = 8.5
90,000 psi, Y.P. Min. b′/t = 7.5
100,000 psi, Y.P. Min. b′/t = 7

10.34.2.2.4 In the case of a composite girder the width of outstanding legs of top flange angles in compression, except those reinforced by plates, shall not exceed the value determined by the following formula

$$\frac{b'}{t} = \frac{1{,}930}{\sqrt{f_{d\ell 1}}} \quad \text{but in no case shall } b'/t \text{ exceed } 12 \qquad (10\text{-}22)$$

In the above b′ is the width of a flange angle, t is the thickness, f_b is the calculated maximum compressive stress, and $f_{d\ell 1}$ is the top flange compressive stress due to non-composite dead load.

10.34.2.2.5 The gross area of the compression flange, except for composite design, shall be not less than the gross area of the tension flange.

10.34.2.2.6 Flange plates shall be of equal thickness, or shall decrease in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

10.34.2.2.7 At least one cover plate of the top flange shall extend the full length of the girder except when the flange is covered with concrete. Any cover plate that is not full length shall extend beyond the theoretical cutoff point far enough to develop the capacity of the plate or shall extend to a section where the stress in the remainder of the girder flange is equal to the allowable fatigue stress, whichever is greater. The theoretical cutoff point of the cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations.

10.34.2.2.8 The number of fasteners connecting the flange angles to the web plate shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange.

10.34.2.2.9 Legs of angles 6 inches or greater in width, connected to web plates, shall have two lines of fasteners. Cover plates over 14 inches wide shall have four lines of fasteners.

10.34.3 Thickness of Web Plates

10.34.3.1 Girders Not Stiffened Longitudinally

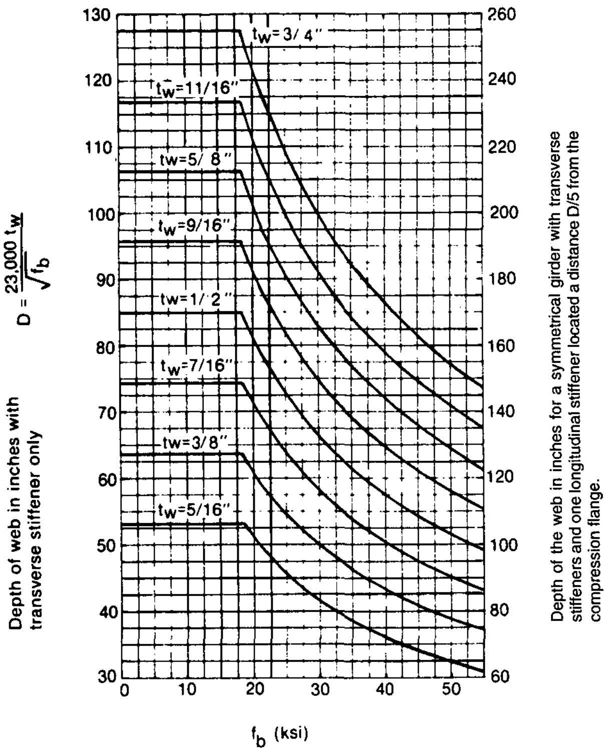
10.34.3.1.1 The web plate thickness of plate girders without longitudinal stiffeners shall not be less than that determined by the formula

$$t_w = \frac{D\sqrt{f_b}}{23{,}000} \text{ (See Figure 10.34.3.1A.)} \quad (10-23)$$

but in no case shall the thickness be less than D/170.

10.34.3.1.2 Where the calculated compressive bending stress in the flange equals the allowable bending stress, the thickness of the web plate (with the web stiffened or not stiffened, depending on the requirements for transverse stiffeners) shall not be less than (where the Y.P. is for the flange material)

36,000 psi, Y.P. Min. D/165
50,000 psi, Y.P. Min. D/140
70,000 psi, Y.P. Min. D/115
90,000 psi, Y.P. Min. D/105
100,000 psi, Y.P. Min. D/100



WEB THICKNESS AND GIRDER DEPTH
(a function of bending stress)

D = depth of web
t_w = thickness of web
f_b = calculated compressive bending stress in flange

FIGURE 10.34.3.1A Web Thickness vs. Girder Depth for Noncomposite Symmetrical Sections

10.34.3.2 Girders Stiffened Longitudinally

10.34.3.2.1 The web plate thickness of plate girders equipped with longitudinal stiffeners shall not be less than that determined by the formula

$$t_w = \frac{D\sqrt{f_b}}{4{,}050\sqrt{k}} \quad (10-24)$$

$$\text{for } \frac{d_s}{D_c} \geq 0.4 \quad k = 5.17\left(\frac{D}{d_s}\right)^2 \geq 9\left(\frac{D}{D_c}\right)^2$$

$$\text{for } \frac{d_s}{D_c} < 0.4 \quad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

but in no case shall the thickness be less than D/340. For symmetrical girders see Figure 10.34.3.1.A.

In the above, D (depth of the web) is the clear unsupported distance in inches between the flange components, t_w is the web thickness, k is the buckling coefficient, d_s is the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component, D_c is the depth of the web in compression calculated by summing the stresses from the applicable stages of loading, and f_b is the calculated flange bending stress in the compression flange. The depth of web in compression, D_c, in composite sections subjected to negative bending may be taken as the depth of the web in compression of the composite section without summing the stresses from the various stages of loading. When both edges of the web are in compression, k shall be taken equal to 7.2.

10.34.3.2.2 Where the calculated bending stress in the flange equals the allowable bending stress, the thickness of the web plate in a symmetrical girder stiffened with transverse stiffeners in combination with one longitudinal stiffener located a distance D/5 from the compression flange shall not be less than (where the Y.P. is for the flange material)

36,000 psi, Y.P. Min. D/327
50,000 psi, Y.P. Min. D/278
70,000 psi, Y.P. Min. D/235

90,000 psi, Y.P. Min. D/207
100,000 psi, Y.P. Min. D/196

In the above, D (depth of web) is the clear unsupported distance in inches between flange components.

10.34.4 Transverse Intermediate Stiffeners

10.34.4.1 Transverse intermediate stiffeners may be omitted if the average calculated unit-shearing stress in the gross section of the web plate at the point considered, f_v, is less than the value given by the following equation:

$$F_v = \frac{7.33 \times 10^7}{(D/t_w)^2} \leq \frac{F_y}{3} \qquad (10\text{-}25)$$

where

D = unsupported depth of web plate between flanges in inches;
t_w = thickness of the web plate in inches;
F_v = allowable shear stress in psi.

10.34.4.2 Where transverse intermediate stiffeners are required, the spacing of the transverse intermediate stiffener shall be such that the actual shearing stress will not exceed the value given by the following equation; the maximum spacing is further limited to 3D and is subject to the handling requirement below:

$$F_v = \frac{F_y}{3}\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \qquad (10\text{-}26)$$

The constant C is equal to the buckling shear stress divided by the shear yield stress, and is determined as follows:

for $\dfrac{D}{t_w} < \dfrac{6{,}000\sqrt{k}}{\sqrt{F_y}}$

$$C = 1.0$$

for

$$\frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \leq (D/t_w) \leq \frac{7{,}500\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}27)$$

$$C = \frac{6{,}000\sqrt{k}}{(D/t_w)\sqrt{F_y}}$$

for

$$D/t_w > \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{4.5 \times 10^7 k}{(D/t_w)^2 F_y} \qquad (10\text{-}28)$$

where

$$k = 5 + \frac{5}{(d_o/D)^2}$$

d_o = spacing of intermediate stiffener
F_y = yield strength of the web plate

$(F_y/3)$ in Equation (10-26) can be replaced by the allowable shearing stress given in Table 10.32.1A.

Transverse stiffeners shall be required if D/t_w is greater than 150. The spacing of these stiffeners shall not exceed the handling requirement $D[260/(D/t_w)]^2$.

10.34.4.3 The spacing of the first intermediate stiffener at the simple support end of a girder shall be such that the shearing stress in the end panel shall not exceed the value given by the following equation (the maximum spacing is limited to 1.5D):

$$F_v = CF_y/3 \leq F_y/3 \qquad (10\text{-}29)$$

10.34.4.4 If a girder panel is subjected to simultaneous action of shear and bending moment with the magnitude of the shear stress higher than 0.6 F_v, the bending tensile stress, F_s, shall be limited to

$$F_s = (.754 - .34f_v/F_v)F_y \qquad (10\text{-}30)$$

where

f_v = average calculated unit-shearing stress at the section; live load shall be the load to produce maximum moment at the section under consideration
F_v = value obtained from Equation (10-26).

10.34.4.5 Where the calculated shear stress equals the allowable shear stress, transverse intermediate stiffeners may be omitted if the thickness of the web is not less than

36,000 psi, Y.P. Min. D/78
50,000 psi, Y.P. Min. D/66
70,000 psi, Y.P. Min. D/56
90,000 psi, Y.P. Min. D/50
100,000 psi, Y.P. Min. D/47

10.34.4.6 Intermediate stiffeners preferably shall be made of plates for welded plate girders and shall be made of angles for riveted plate girders. They may be in pairs, one stiffener fastened on each side of the web plate, with a tight fit at the compression flange. They may, however, be made of a single stiffener fastened to one side of the web plate. Stiffeners provided on only one side of the web must be in bearing against, but need not be attached to, the compression flange for the stiffener to be effective. However, transverse stiffeners which connect diaphragms or crossframes to the beam or girder shall be rigidly connected to both the top and bottom flanges.

10.34.4.7 The moment of inertia of any type of transverse stiffener with reference to the plane defined in Article 10.34.4.8 shall not be less than

$$I = d_o t_w^3 J \qquad (10\text{-}31)$$

where

$$J = 2.5\,(D/d_o)^2 - 2, \text{ but not less than } 0.5 \qquad (10\text{-}32)$$

In these expressions,

- I = minimum permissible moment of inertia of any type of transverse intermediate stiffener in inches^4;
- J = required ratio of rigidity of one transverse stiffener to that of the web plate;
- d_o = distance between stiffeners in inches;
- D = unsupported depth of web plate between flange components in inches;
- t_w = thickness of the web plate in inches.

The gross cross-sectional area of intermediate transverse stiffeners shall be greater than

$$A = \left[0.15B\frac{D}{t_w}(1-C)\left(\frac{f_v}{F_v}\right) - 18\right]\frac{F_{y\,web}}{F_{cr}}t_w^2 \qquad (10-32a)$$

$$\text{where } F_{cr} = \frac{9{,}025{,}000}{\left(\frac{b'}{t}\right)^2} \le F_{y\,stiffener} \qquad (10-32b)$$

where $F_{y\,stiffener}$ is the yield strength of the stiffener; $B = 1.0$ for stiffener pairs, 1.8 for single angles, and 2.4 for single plates; and C is computed by Article 10.34.4.2. When values computed by Equation (10-32a) approach zero or are negative, then transverse stiffeners need only meet the requirements of Equation (10-31), and the requirements of Article 10.34.4.10.

10.34.4.8 When stiffeners are in pairs, the moment of inertia shall be taken about the center line of the web plate. When single stiffeners are used, the moment of inertia shall be taken about the face in contact with the web plate.

10.34.4.9 Transverse intermediate stiffeners need not be in bearing with the tension flange. The distance between the end of the stiffener weld and the near edge of the web-to-flange fillet welds shall not be less than $4t_w$ or more than $6t_w$. Stiffeners at points of concentrated loading shall be placed in pairs and should be designed in accordance with Article 10.34.6. However, transverse stiffeners which connect diaphragms or crossframes to the beam or girder shall be rigidly connected to both the top and bottom flanges.

10.34.4.10 The width of a plate or the outstanding leg of an angle intermediate stiffener shall not be less than 2 inches plus 1/30 the depth of the girder, and it shall preferably not be less than 1/4 the full width of the girder flange. The thickness of a plate or the outstanding leg of an angle intermediate stiffener shall not be less than 1/16 its width. Intermediate stiffeners may be AASHTO M 270 Grade 36 steel.

10.34.5 Longitudinal Stiffeners

10.34.5.1 The optimum distance, d_s, of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener from the inner surface or the leg of the compression flange component is D/5 for a symmetrical girder. The optimum distance, d_s, for an unsymmetrical composite girder in positive-moment regions may be determined from the equation given below:

$$\frac{d_s}{D_{cs}} = \frac{1}{1+1.5\sqrt{\frac{f_{DL+LL}}{f_{DL}}}} \qquad (10\text{-}32b)$$

where D_{cs} is the depth of the web in compression of the noncomposite steel beam or girder, f_{DL} is the noncomposite dead-load stress in the compression flange, and f_{DL+LL} is the total noncomposite and composite dead-load plus the composite live-load stress in the compression flange at the most highly stressed section of the web. The optimum distance, d_s, of the stiffener in negative-moment regions of composite sections is $2D_c/5$, where D_c is the depth of the web in compression of the composite section at the most highly stressed section of the web.

The longitudinal stiffener shall be proportioned so that

$$I = Dt_w^3\left(2.4\frac{d_o^2}{D^2} - 0.13\right) \qquad (10\text{-}33)$$

where

- I = minimum moment of inertia of the longitudinal stiffener about its edge in contact with the web plate in inches^4;

D = unsupported distance between flange components in inches;
t_w = thickness of the web plate in inches;
d_o = actual distance between transverse stiffeners in inches.

10.34.5.2 The thickness of the longitudinal stiffener t_s shall not be less than

$$\frac{b'\sqrt{F_y}}{2,600} \quad (10\text{-}34)$$

where

b' = width of stiffener
F_y = yield strength of the longitudinal stiffener

10.34.5.3 The stress in the stiffener shall not be greater than the basic allowable bending stress for the material used in the stiffener.

10.34.5.4 Longitudinal stiffeners are usually placed on one side only of the web plate. They need not be continuous and may be cut at their intersections with the transverse stiffeners.

10.34.5.5 For longitudinally stiffened girders, transverse stiffeners shall be spaced a distance, d_o, according to shear capacity as specified in Article 10.34.4.2, but not more than 1.5 times the web depth. The handling requirement given in Article 10.34.4.2 shall not apply to longitudinally stiffened girders. The spacing of the first transverse stiffener at the simple support end of a longitudinally stiffened girder shall be such that the shearing stress in the end panel does not exceed the value given in Article 10.34.4.3. The maximum spacing of the first transverse stiffener at the simple support end of a longitudinally stiffened girder is limited to 1.5 times the web depth. The total web depth D shall be used in determining the shear capacity of longitudinally stiffened girders in Articles 10.34.4.2 and 10.34.4.3.

10.34.5.6 Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 10.34.4.7.

10.34.6 Bearing Stiffeners

10.34.6.1 Welded Girders

Over the end bearings of welded plate girders and over the intermediate bearings of continuous welded plate girders there shall be stiffeners. They shall extend as nearly as practicable to the outer edges of the flange plates. They preferably shall be made of plates placed on both sides of the web plate. Bearing stiffeners shall be designed as columns, and their connection to the web shall be designed to transmit the entire end reaction to the bearings. For stiffeners consisting of two plates, the column section shall be assumed to comprise the two plates and a centrally located strip of the web plate whose width is equal to not more than 18 times its thickness. For stiffeners consisting of four or more plates, the column section shall be assumed to comprise the four or more plates and a centrally located strip of the web plate whose width is equal to that enclosed by the four or more plates plus a width of not more than 18 times the web plate thickness. (See Article 10.40 for Hybrid Girders.) The radius of gyration shall be computed about the axis through the center line of the web plate. The stiffeners shall be ground to fit against the flange through which they receive their reaction, or attached to the flange by full penetration groove welds. Only the portions of the stiffeners outside the flange-to-web plate welds shall be considered effective in bearing. The thickness of the bearing stiffener plates shall not be less than

$$\frac{b'}{12}\sqrt{\frac{F_y}{33,000}} \quad (10\text{-}35)$$

The allowable compressive stress and the bearing pressure on the stiffeners shall not exceed the values specified in Article 10.32.

10.34.6.2 Riveted or Bolted Girders

Over the end bearings of riveted or bolted plate girders there shall be stiffener angles, the outstanding legs of which shall extend as nearly as practicable to the outer edge on the flange angle. Bearing stiffener angles shall be proportioned for bearing on the outstanding legs of flange angles, no allowance being made for the portions of the legs being fitted to the fillets of the flange angles. Bearing stiffeners shall be arranged, and their connections to the web shall be designed to transmit the entire end reaction to the bearings. They shall not be crimped. The thickness of the bearing stiffener angles shall not be less than

$$\frac{b'}{12}\sqrt{\frac{F_y}{33,000}} \quad (10\text{-}36)$$

The allowable compressive stress and the bearing pressure on the stiffeners shall not exceed the values specified in Article 10.32.

10.35 TRUSSES

10.35.1 Perforated Cover Plates and Lacing Bars

The shearing force normal to the member in the planes of lacing or continuous perforated plates shall be assumed divided equally between all such parallel planes. The shearing force shall include that due to the weight of the member plus any other external force. For compression members, an additional force shall be added as obtained by the following formula:

$$V = \frac{P}{100}\left[\frac{100}{\ell / r + 10} + \frac{(\ell / r)F_y}{3,300,000}\right] \qquad (10\text{-}37)$$

In the above expression

- V = normal shearing force in pounds;
- P = allowable compressive axial load on members in pounds;
- ℓ = length of member in inches;
- r = radius of gyration of section about the axis perpendicular to plane of lacing or perforated plate in inches;
- F_y = specified minimum yield point of type of steel being used.

10.35.2 Compression Members—Thickness of Metal

10.35.2.1 Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

10.35.2.2 The center of gravity of a built-up section shall coincide as nearly as practicable with the center of the section. Preferably, segments shall be connected by solid webs or perforated cover plates.

10.35.2.3 Plates supported on one side, outstanding legs of angles and perforated plates—for outstanding plates, outstanding legs of angles, and perforated plates at the perforations, the b/t ratio of the plates or angle segments when used in compression shall not be greater than the value obtained by use of the formula

$$\frac{b}{t} = \frac{1,625}{\sqrt{f_a}} \qquad (10\text{-}38)$$

but in no case shall b/t be greater than 12 for main members and 16 for secondary members.

(Note: b is the distance from the edge of plate or edge of perforation to the point of support.)

10.35.2.4 When the compressive stress equals the limiting factor of 0.44 F_y, the b/t ratio of the segments indicated above shall not be greater than the ratios shown for the following grades of steel:

36,000 psi, Y.P. Min. b/t = 12
50,000 psi, Y.P. Min. b/t = 11
70,000 psi, Y.P. Min. b/t = 9
90,000 psi, Y.P. Min. b/t = 8
100,000 psi, Y.P. Min. b/t = 7.5

10.35.2.5 Plates supported on two edges or webs of main component segments—for members of box shape consisting of main plates, rolled sections, or made up component segments with cover plates, the b/t ratio of the main plates or webs of the segments when used in compression shall not be greater than the value obtained by use of the formula

$$\frac{b}{t} = \frac{4,000}{\sqrt{f_a}} \qquad (10\text{-}39)$$

but in no case shall b/t be greater than 45.
(Note: b is the distance between points of support for the plate and between roots of flanges for the webs of rolled segments.)

10.35.2.6 When the compressive stresses equal the limiting factor of 0.44 F_y, the b/t ratio of the plates and segments indicated above shall not be greater than the ratios shown for the following grades of steel:

36,000 psi, Y.P. Min. b/t = 32
50,000 psi, Y.P. Min. b/t = 27
70,000 psi, Y.P. Min. b/t = 23
90,000 psi, Y.P. Min. b/t = 20
100,000 psi, Y.P. Min. b/t = 19

10.35.2.7 Solid cover plates supported on two edges or webs connecting main members or segments—for members of H or box shapes consisting of solid cover plates or solid webs connecting main plates or segments, the b/t ratio of the solid cover plates or webs when used in compression shall not be greater than the value obtained by use of the formula

$$\frac{b}{t} = \frac{5,000}{\sqrt{f_a}} \qquad (10\text{-}40)$$

but in no case shall b/t be greater than 50.

(Note: b is the unsupported distance between points of support.)

10.35.2.8 When the compressive stresses equal the limiting factor of 0.44 F_y, the b/t ratio of the cover plate and webs indicated above shall not be greater than the ratios shown for the following grades of steel:

36,000 psi, Y.P. Min. b/t = 40
50,000 psi, Y.P. Min. b/t = 34
70,000 psi, Y.P. Min. b/t = 28
90,000 psi, Y.P. Min. b/t = 25
100,000 psi, Y.P. Min. b/t = 24

10.35.2.9 Perforated cover plates supported on two edges—for members of box shapes consisting of perforated cover plates connecting main plates or segments, the b/t ratio of the perforated cover plates when used in compression shall not be greater than the value obtained by use of the formula

$$\frac{b}{t} = \frac{6{,}000}{\sqrt{f_a}} \qquad (10\text{-}41)$$

but in no case shall b/t be greater than 55.
(Note: b is the distance between points of support. Attention is directed to requirements for plate thickness at perforations, namely, plate supported on one side, which also shall be satisfied.)

10.35.2.10 When the compressive stresses equal the limiting factor of 0.44 F_y, the b/t ratio of the perforated cover plates shall not be greater than the ratios shown for the following grades of steel:

36,000 psi, Y.P. Min. b/t = 48
50,000 psi, Y.P. Min. b/t = 41
70,000 psi, Y.P. Min. b/t = 34
90,000 psi, Y.P. Min. b/t = 30
100,000 psi, Y.P. Min. b/t = 29

In the above expressions—

f_a = computed compressive stress;
b = width (defined as indicated for each expression);
t = plate or web thickness.

10.35.2.11 The point of support shall be the inner line of fasteners or fillet welds connecting the plate to the main segment. For plates butt welded to the flange edge of rolled segments the point of support may be taken as the weld whenever the ratio of outstanding flange width to flange thickness of the rolled segment is less than seven. Otherwise, point of support shall be the root of flange of rolled segment. Terminations of the butt welds are to be ground smooth.

10.36 COMBINED STRESSES

All members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \qquad (10\text{-}42)$$

and

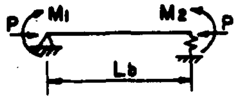
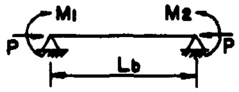
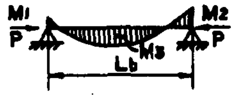
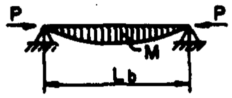
$$\frac{f_a}{0.472 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \text{ (at points of support)} \qquad (10\text{-}43)$$

where

$$F'_e = \frac{\pi^2 E}{F.S.\ (K_b L_b / r_b)^2} \qquad (10\text{-}44)$$

f_a = computed axial stress;
f_{bx} or f_{by} = computed compressive bending stress about the x axis and y axis, respectively;
F_a = axial stress that would be permitted if axial force alone existed, regardless of the plane of bending;
F_{bx}, F_{by} = compressive bending stress that would be permitted if bending moment alone existed about the x axis and the y axis, respectively, as evaluated according to Table 10.32.1A;
F'_e = Euler buckling stress divided by a factor of safety;
E = modulus of elasticity of steel;
K_b = effective length factor in the plane of bending (see Appendix C);
L_b = actual unbraced length in the plane of bending;
r_b = radius of gyration in the plane of bending;
C_{mx}, C_{my} = coefficient about the x axis and y axis, respectively, whose value is taken from Table 10.36A;
F.S. = factor of safety = 2.12.

TABLE 10.36A Bending-Compression Interaction Coefficients

Loading Conditions	Remarks	C_m
Computed moments maximum at end; joint translation not prevented		0.85
Computed moments maximum at end; no transverse loading, joint translation prevented		$\left[(0.4)\frac{M_1}{M_2} + 0.6\right]$
Transverse loading; joint translation prevented		0.85
Transverse loading; joint translation prevented		1.0

M_1 = smaller end moment.
M_1/M_2 is positive when member is bent in single curvature.
M_1/M_2 is negative when member is bent in reverse curvature.
In all cases C_m may be conservatively taken equal to 1.0.

10.37 SOLID RIB ARCHES

10.37.1 Moment Amplification and Allowable Stress

10.37.1.1 Live load plus impact moments that are determined by an analysis which neglects arch rib deflection shall be increased by an amplification factor A_F

$$A_F = \frac{1}{1 - \dfrac{1.70T}{AF_e}} \qquad (10\text{-}45)$$

where

T = arch rib thrust at the quarter point from dead plus live plus impact loading;

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \text{(Euler buckling stress)} \qquad (10\text{-}46)$$

L = one-half of the length of the arch rib;
A = area of cross section;
r = radius of gyration;
K = factor to account for effective length.

K Values for Use in Calculating F_e and F_a

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

10.37.1.2 The arch rib shall be proportioned to satisfy the following requirement:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1 \qquad (10\text{-}47)$$

where

f_a = the computed axial stress;
f_b = the calculated bending stress, including moment amplification, at the extreme fiber;
F_a = the allowable axial unit stress;
F_b = the allowable bending unit stress.

10.37.1.3 For buckling in the vertical plane

$$F_a = \frac{F_y}{2.12}\left[1 - \frac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right] \qquad (10\text{-}48)$$

where KL is as defined above.

10.37.1.4 The effects of lateral slenderness should be investigated. Tied arch ribs, with the tie and roadway suspended from the rib, are not subject to moment amplification, and F_a shall be based on an effective length equal to the distance along the arch axis between suspenders, for buckling in the vertical plane. However, the smaller cross-sectional area of cable suspenders may result in an effective length slightly longer than the distance between suspenders.

10.37.2 Web Plates

10.37.2.1 The depth to thickness ratio D/t_w of the web plates, having no longitudinal stiffeners, shall not be greater than the following:

$$\frac{D}{t_w} = \frac{5{,}000}{\sqrt{f_a}}, \quad \text{maximum } D/t_w = 60 \qquad (10\text{-}49)$$

where t_w = web thickness.

10.37.2.2 If one longitudinal stiffener is used at mid-depth of the web, maximum D/t_w shall be as follows:

$$\frac{D}{t_w} = \frac{7{,}500}{\sqrt{f_a}}, \quad \text{maximum } D/t_w = 90 \qquad (10\text{-}50)$$

and the moment of inertia of the stiffener about an axis parallel to the web and at the base of the stiffener shall be equal to

$$I_s = 0.75\,Dt_w^3 \qquad (10\text{-}51)$$

10.37.2.3 If two longitudinal stiffeners are used at the one-third points of the web depth D, maximum D/t_w shall be as follows:

$$\frac{D}{t_w} = \frac{10{,}000}{\sqrt{f_a}}, \quad \text{maximum } D/t_w = 120 \qquad (10\text{-}52)$$

and the moment of inertia of each stiffener shall be

$$I_s = 2.2\,Dt_w^3 \qquad (10\text{-}53)$$

10.37.2.4 The width to thickness ratio b'/t_s of any outstanding element of the web stiffeners shall not exceed the following:

$$\frac{b'}{t_s} = \frac{1{,}625}{\sqrt{f_a + \frac{f_b}{3}}}, \quad \text{maximum } b'/t_s = 12 \qquad (10\text{-}54)$$

10.37.2.5 Web plate equations apply between limits

$$0.2 \le \frac{f_b}{f_a + f_b} \le 0.7 \qquad (10\text{-}55)$$

10.37.3 Flange Plates

10.37.3.1 The b/t_f ratio for the width of flange plates between webs shall be not greater than

$$\frac{b'}{t_f} = \frac{4{,}250}{\sqrt{f_a + f_b}}, \quad \text{maximum } b/t_f = 47 \qquad (10\text{-}56)$$

10.37.3.2 The b'/t_f ratio for the overhang width of flange plates shall be not greater than

$$\frac{b'}{t_f} = \frac{1{,}625}{\sqrt{f_a + f_b}}, \quad \text{maximum } b'/t_f = 12 \qquad (10\text{-}57)$$

10.38 COMPOSITE GIRDERS

10.38.1 General

10.38.1.1 This section pertains to structures composed of steel girders with concrete slabs connected by shear connectors.

10.38.1.2 General specifications pertaining to the design of concrete and steel structures shall apply to structures utilizing composite girders where such specifications are applicable. Composite girders and slabs shall be designed and the stresses computed by the composite moment of inertia method and shall be consistent with the predetermined properties of the various materials used.

10.38.1.3 The ratio of the moduli of elasticity of steel (29,000,000 psi) to those of normal weight concrete (W = 145 pcf) of various design strengths shall be as follows:

f'_c = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete. The value of n, as a function of the ultimate cylinder strength of concrete, shall be assumed as follows:

f'_c = 2,000–2,300	n = 11
2,400–2,800	= 10
2,900–3,500	= 9
3,600–4,500	= 8
4,600–5,900	= 7
6,000 or more	= 6

10.38.1.4 The effect of creep shall be considered in the design of composite girders which have dead loads acting on the composite section. In such structures, stresses and horizontal shears produced by dead loads acting on the composite section shall be computed for n as given above or for this value multiplied by 3, whichever gives the higher stresses and shears.

10.38.1.5 If concrete with expansive characteristics is used, composite design should be used with caution and provision must be made in the design to accommodate the expansion.

10.38.1.6 Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflection calculations, for determining stiffness factors used in calculating moments and shears, and for computing fatigue stress ranges and fatigue shear ranges as permitted under the provisions of Articles 10.3.1 and 10.38.5.1.

10.38.1.7 The steel beams or girders, especially if not supported by intermediate falsework, shall be investigated for stability and strength for the loading applied during the time the concrete is in place and before it has hardened. The casting or placing sequence specified in the plans for the composite concrete deck shall be considered when calculating the moments and shears on the steel section. The maximum flange compression stress shall not exceed the value specified in Table 10.32.1A for partially supported or unsupported compression flanges multiplied by a factor of 1.4, but not to exceed $0.55F_y$. The sum of the noncomposite and composite dead-load shears in the web shall not exceed the shear-buckling capacity of the web multiplied by 1.35, nor the allowable shear stress, as follows:

$$F_v = 0.45CF_y \leq 0.33F_y \qquad (10\text{-}57a)$$

where C is specified in Article 10.34.4.2.

10.38.2 Shear Connectors

10.38.2.1 The mechanical means used at the junction of the girder and slab for the purpose of developing the shear resistance necessary to produce composite action shall conform to the specifications of the respective materials as provided in Division II. The shear connectors shall be of types that permit a thorough compaction of the concrete in order to ensure that their entire surfaces are in contact with the concrete. They shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

10.38.2.2 The capacity of stud and channel shear connectors welded to the girders is given in Article 10.38.5. Channel shear connectors shall have at least $^3/_{16}$-inch fillet welds placed along the heel and toe of the channel.

10.38.2.3 The clear depth of concrete cover over the tops of the shear connectors shall be not less than 2 inches. Shear connectors shall penetrate at least 2 inches above bottom of slab.

10.38.2.4 The clear distance between the edge of a girder flange and the edge of the shear connectors shall be not less than 1 inch. Adjacent stud shear connectors shall not be closer than 4 diameters center to center.

10.38.3 Effective Flange Width

10.38.3.1 In composite girder construction the assumed effective width of the slab as a T-beam flange shall not exceed the following:

(1) One-fourth of the span length of the girder.
(2) The distance center to center of girders.
(3) Twelve times the least thickness of the slab.

10.38.3.2 For girders having a flange on one side only, the effective flange width shall not exceed $^1/_{12}$ of the span length of the girder, or six times the thickness of the slab, or one-half the distance center to center of the next girder.

10.38.4 Stresses

10.38.4.1 Maximum compressive and tensile stresses in girders that are not provided with temporary supports during the placing of the permanent dead load shall be the sum of the stresses produced by the dead loads acting on the steel girders alone and the stresses produced by the superimposed loads acting on the composite girder. When girders are provided with effective intermediate supports that are kept in place until the concrete has attained 75% of its required 28-day strength, the dead and live load stresses shall be computed on the basis of the composite section.

10.38.4.2 A continuous composite bridge may be built with shear connectors either in the positive moment regions or throughout the length of the bridge. The positive moment regions may be designed with composite sections as in simple spans. Shear connectors shall be provided in the negative moment portion in which the reinforcement steel embedded in the concrete is considered a part of the composite section. In case the reinforcement steel embedded in the concrete is not used in computing section properties for negative moments, shear connectors need not be provided in these portions of the spans, but additional anchorage connectors shall be placed in the region of the point of dead load contra-flexure in accordance with Article 10.38.5.1.3. Shear connectors shall be provided in accordance with Article 10.38.5.

10.38.4.3 The minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1% of the cross-sectional area of the concrete slab whenever the longitudinal tensile stress in the concrete slab due to either the construction loads or the design loads exceeds f_t specified in Article 8.15.2.1.1. The area of the concrete slab shall be taken equal to the structural thickness times the entire width of the bridge deck. The required reinforcement shall be No. 6 bars or smaller spaced at not more than 12 inches. Two-thirds of this required reinforcement is to be placed in the top layer of slab. Placement of distribution steel as specified in Article 3.24.10 is waived.

10.38.4.4 When shear connectors are omitted from the negative moment region, the longitudinal reinforcement shall be extended into the positive moment region beyond the anchorage connectors at least 40 times the reinforcement diameter. For epoxy-coated bars, the length to be extended into the positive moment region beyond the anchorage connectors should be modified to comply with Article 8.25.2.3.

10.38.5 Shear

10.38.5.1 Horizontal Shear

The maximum pitch of shear connectors shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange.

Resistance to horizontal shear shall be provided by mechanical shear connectors at the junction of the concrete slab and the steel girder. The shear connectors shall be mechanical devices placed transversely across the flange of the girder spaced at regular or variable intervals. The shear connectors shall be designed for fatigue* and checked for ultimate strength.

10.38.5.1.1 Fatigue

The range of horizontal shear shall be computed by the formula

$$S_r = \frac{V_r Q}{I} \qquad (10\text{-}58)$$

where

S_r = range of horizontal shear, in kips per inch, at the junction of the slab and girder at the point in the span under consideration;

V_r = range of shear due to live loads and impact in kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads);

Q = statical moment about the neutral axis of the composite section of the transformed concrete area, in^3. Between points of dead-load contraflexure, the statical moment about the neutral axis of the composite section of the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for negative moment in computing the longitudinal range of stress;

I = moment of inertia of the transformed composite section, in^4. Between points of dead-load contraflexure, the moment of inertia of the steel girder including the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for negative moment in computing the longitudinal range of stress.

(In the formula, the concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio, n.)

The allowable range of horizontal shear, Z_r, in pounds on an individual connector is as follows:

Channels

$$Z_r = Bw \qquad (10\text{-}59)$$

Welded studs (for $H/d \geq 4$)

$$Z_r = \alpha d^2 \qquad (10\text{-}60)$$

where

w = length of a channel shear connector, in inches, measured in a transverse direction on the flange of a girder;

d = diameter of stud in inches;

α = 13,000 for 100,000 cycles
10,600 for 500,000 cycles
7,850 for 2,000,000 cycles
5,500 for over 2,000,000 cycles;

*Reference is made to the paper titled "Fatigue Strength of Shear Connectors," by Roger G. Slutter and John W. Fisher, in *Highway Research Record, No. 147,* published by the Highway Research Board, Washington, D.C., 1966.

B = 4,000 for 100,000 cycles
3,000 for 500,000 cycles
2,400 for 2,000,000 cycles
2,100 for over 2,000,000 cycles;
H = height of stud in inches.

The required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one transverse girder cross-section (ΣZ_r) by the horizontal range of shear S_r, but not to exceed the maximum pitch specified in Article 10.38.5.1. Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged.

10.38.5.1.2 Ultimate Strength

The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength.

The number of shear connectors required shall equal or exceed the number given by the formula

$$N_1 = \frac{P}{\phi S_u} \qquad (10\text{-}61)$$

where

N_1 = number of connectors between points of maximum positive moment and adjacent end supports;
S_u = ultimate strength of the shear connector as given below;
ϕ = reduction factor = 0.85;
P = force in the slab as defined hereafter as P_1 or P_2.

At points of maximum positive moment, the force in the slab is taken as the smaller value of the formulas

$$P_1 = A_s F_y \qquad (10\text{-}62)$$

or

$$P_2 = 0.85 f_c' b t_s \qquad (10\text{-}63)$$

where

A_s = total area of the steel section including coverplates;
F_y = specified minimum yield point of the steel being used;
f_c' = compressive strength of concrete at age of 28 days;
b = effective flange width given in Article 10.38.3;
t_s = thickness of the concrete slab.

The number of connectors, N_2, required between the points of maximum positive moment and points of adjacent maximum negative moment shall equal or exceed the number given by the formula

$$N_2 = \frac{P + P_3}{\phi S_u} \qquad (10\text{-}64)$$

At points of maximum negative moment the force in the slab is taken as

$$P_3 = A_s^r F_y^{r*} \qquad (10\text{-}65)$$

where

A_s^r = total area of longitudinal reinforcing steel at the interior support within the effective flange width;
F_y^{r*} = specified minimum yield point of the reinforcing steel.

The ultimate strength of the shear connector is given as follows:

Channels

$$S_u = 550\left(h + \frac{t}{2}\right) W \sqrt{f_c'} \qquad (10\text{-}66)$$

Welded studs (for H/d > 4)

$$S_u = 0.4 d^2 \sqrt{f_c' E_c} \le 60{,}000 A_{sc} \qquad (10\text{-}67)$$

where

E_c = modulus of elasticity of the concrete in pounds per square inch;

$$E_c = w^{3/2} 33 \sqrt{f_c'} \qquad (10\text{-}68)$$

S_u = ultimate strength of individual shear connector in pounds;
A_{sc} = cross-sectional area of a stud shear connector in square inches;
h = average flange thickness of the channel flange in inches;
t = thickness of the web of a channel in inches;
W = length of a channel shear connector in inches;
f_c' = compressive strength of the concrete in 28 days in pounds per square inch;
d = diameter of stud in inches;
w = unit weight of concrete in pounds per cubic foot.

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P_3 is equal to zero.

10.38.5.1.3 Additional Connectors to Develop Slab Stresses

The number of additional connectors required at points of contraflexure when reinforcing steel embedded in the concrete is not used in computing section properties for negative moments shall be computed by the formula

$$N_c = A_r^s f_r / Z_r \qquad (10\text{-}69)$$

where

N_c = number of additional connectors for each beam at point of contraflexure;
A_r^s = total area of longitudinal slab reinforcing steel for each beam over interior support;
f_r = range of stress due to live load plus impact in the slab reinforcement over the support (in lieu of more accurate computations, f_r may be taken as equal to 10,000 psi);
Z_r = allowable range of horizontal shear on an individual shear connector.

The additional connectors, N_c, shall be placed adjacent to the point of dead load contraflexure within a distance equal to one-third the effective slab width, i.e., placed either side of this point or centered about it. It is preferable to locate field splices so that they clear the connectors.

10.38.5.2 Vertical Shear

The intensity of unit-shearing stress in a composite girder may be determined on the basis that the web of the steel girder carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformly distributed throughout the gross area of the web.

10.38.6 Deflection

10.38.6.1 The provisions of Article 10.6 in regard to deflections from live load plus impact also shall be applicable to composite girders.

10.38.6.2 When the girders are not provided with falsework or other effective intermediate support during the placing of the concrete slab, the deflection due to the weight of the slab and other permanent dead loads added before the concrete has attained 75% of its required 28-day strength shall be computed on the basis of noncomposite action.

10.39 COMPOSITE BOX GIRDERS

10.39.1 General

10.39.1.1 This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single cell composite box girders. The distance center-to-center of flanges of each box should be the same and the average distance center-to-center of flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of flanges of each box. In addition to the above, when nonparallel girders are used, the distance center-to-center of adjacent flanges at supports shall be not greater than 1.35 times and not less than 0.65 times the distance center-to-center of flanges of each box. The cantilever overhang of the deck slab, including curbs and parapets, shall be limited to 60% of the average distance center-to-center of flanges of adjacent boxes, but shall in no case exceed 6 feet.

10.39.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.39.1 through 10.39.8.

10.39.2 Lateral Distribution of Loads for Bending Moment

10.39.2.1 The live load bending moment for each box girder shall be determined by applying to the girder, the fraction W_L of a wheel load (both front and rear), determined by the following equation:

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} \qquad (10\text{-}70)$$

where

$$R = \frac{N_w}{\text{Number of Box Girders}} \qquad (10\text{-}71)$$

N_w = $W_c/12$ reduced to the nearest whole number;
W_c = roadway width between curbs in feet, or barriers if curbs are not used. R shall not be less than 0.5 or greater than 1.5.

10.39.2.2 The provision of Article 3.12, Reduction of Load Intensity, shall not apply in the design of box girders when using the design load W_L given by the above equation.

10.39.3 Design of Web Plates

10.39.3.1 Vertical Shear

The design shear V_w for a web shall be calculated using the following equation:

$$V_w = V_v / \cos\theta \qquad (10\text{-}72)$$

where

V_v = vertical shear;
θ = angle of inclination of the web plate to the vertical.

10.39.3.2 Secondary Bending Stresses

10.39.3.2.1 Web plates may be plumb (90° to bottom of flange) or inclined. If the inclination of the web plates to a plane normal to the bottom flange is no greater than 1 to 4, and the width of the bottom flange is no greater than 20% of the span, then the transverse bending stresses resulting from distortion of the span, and the transverse bending stresses resulting from distortion of the girder cross section and from vibrations of the bottom plate need not be considered. For structures in this category transverse bending stresses due to supplementary loadings, such as utilities, shall not exceed 5,000 psi.

10.39.3.2.2 For structures exceeding these limits, a detailed evaluation of the transverse bending stresses due to all causes shall be made. These stresses shall be limited to a maximum stress or range of stress of 20,000 psi.

10.39.4 Design of Bottom Flange Plates

10.39.4.1 Tension Flanges

10.39.4.1.1 In cases of simply supported spans, the bottom flange shall be considered completely effective in resisting bending if its width does not exceed one-fifth the span length. If the flange plate width exceeds one-fifth of the span, an amount equal to one-fifth of the span only shall be considered effective.

10.39.4.1.2 For continuous spans, the criteria above shall be applied to the lengths between points of contraflexure.

10.39.4.2 Compression Flanges Unstiffened

10.39.4.2.1 Unstiffened compression flanges designed for the basic allowable stress of 0.55 F_y shall have a width to thickness ratio equal to or less than the value obtained by the use of the formula

$$\frac{b}{t} = \frac{6,140}{\sqrt{F_y}} \tag{10-73}$$

where

b = flange width between webs in inches;
t = flange thickness in inches.

10.39.4.2.2 For greater b/t ratios, but not exceeding $13,300/\sqrt{F_y}$, the stress in an unstiffened bottom flange shall not exceed the value determined by the use of the formula

$$f_b = 0.55F_y - 0.224F_y \times \left[1 - \sin\left(\frac{\pi}{2} \times \frac{13,300 - \frac{b\sqrt{F_y}}{t}}{7,160}\right)\right] \tag{10-74}$$

10.39.4.2.3 For values of b/t exceeding $13,300/\sqrt{F_y}$, the stress in the flange shall not exceed the value given by the formula

$$f_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^6 \tag{10-75}$$

10.39.4.2.4 The b/t ratio preferably should not exceed 60 except in areas of low stress near points of dead load contraflexure.

10.39.4.2.5 Should the b/t ratio exceed 45, longitudinal stiffeners should be considered.

10.39.4.2.6 Unstiffened compression flanges shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress.

10.39.4.3 Compression Flanges Stiffened Longitudinally*

10.39.4.3.1 Longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to

$$I_s = \phi\, t_f^3 w \tag{10-76}$$

where

ϕ = 0.07 k^3n^4 for values of n greater than 1;
ϕ = 0.125 k^3 for a value of n = 1;
t_f = thickness of the flange;
w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener;
n = number of longitudinal stiffeners;
k = buckling coefficient which shall not exceed 4.

*In solving these equations a value of k between 2 and 4 generally should be assumed.

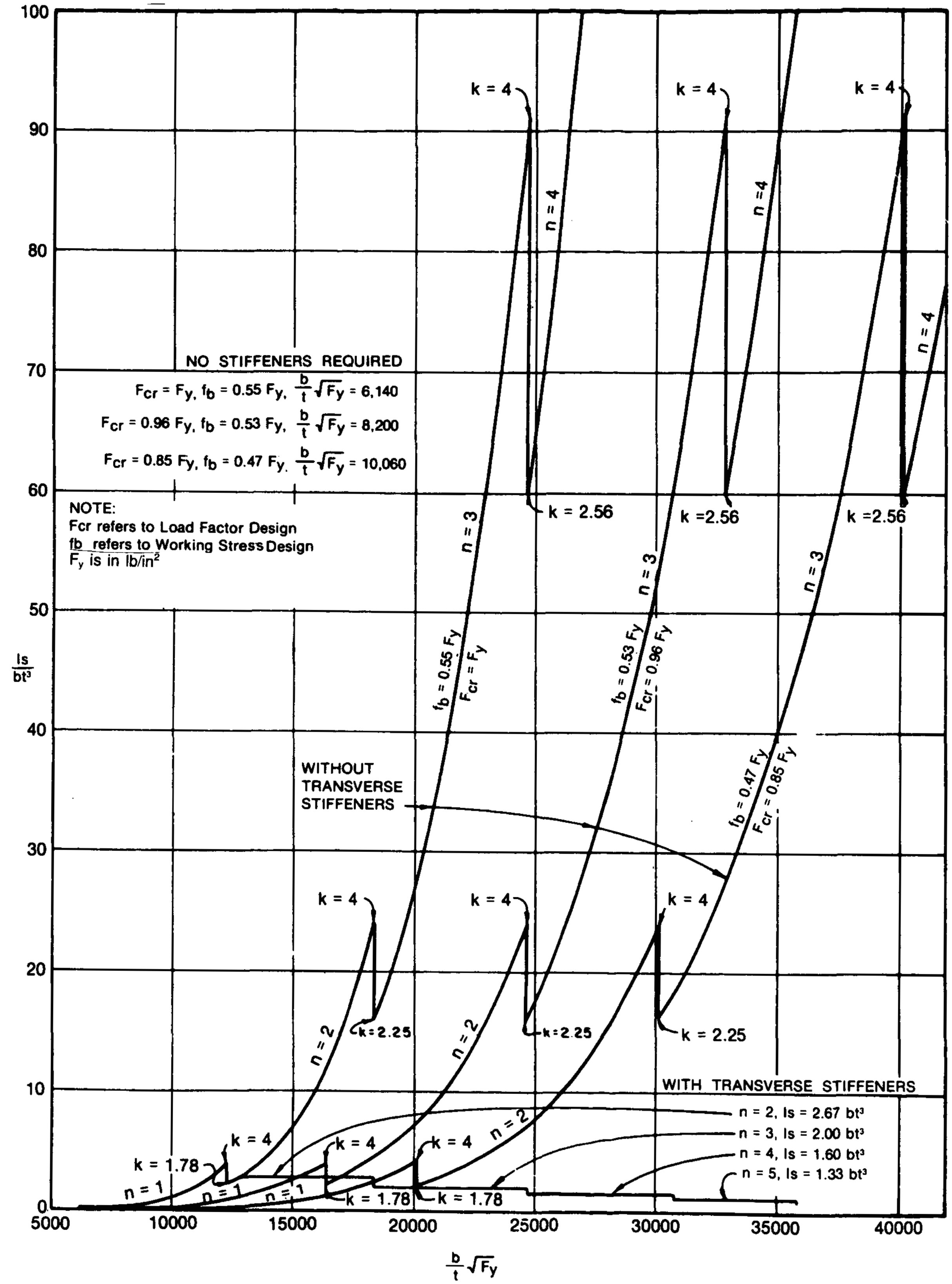


FIGURE 10.39.4.3A. Longitudinal Stiffeners—Box Girder Compression Flange

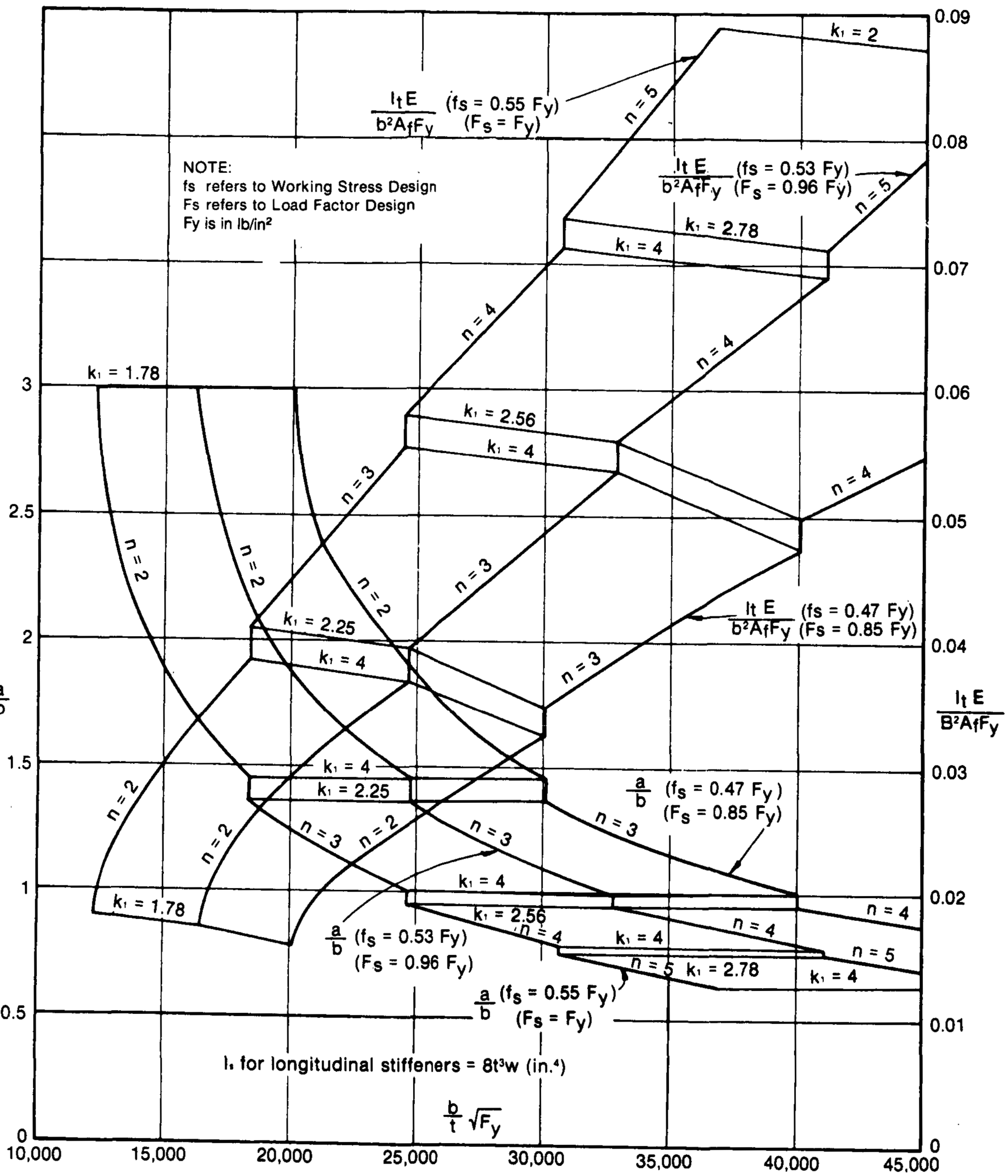


FIGURE 10.39.4.3B Spacing and Size of Transverse Stiffeners (for Flange Stiffened Longitudinally and Transversely)

10.39.4.3.2 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 F_y, the ratio w/t shall not exceed the value given by the formula

$$\frac{w}{t} = \frac{3{,}070\sqrt{k}}{\sqrt{F_y}} \quad (10\text{-}77)$$

10.39.4.3.3 For greater values of w/t but not exceeding 60 or $(6{,}650\sqrt{k})/\sqrt{F_y}$, whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula

$$f_b = 0.55F_y - 0.224F_y \times \left[1 - \sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k} - \frac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k}}\right)\right] \quad (10\text{–}78)$$

10.39.4.3.4 For values of w/t exceeding $(6{,}650\sqrt{k})/\sqrt{F_y}$ but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula

$$f_b = 14.4\ k(t/w)^2 \times 10^6 \quad (10\text{-}79)$$

10.39.4.3.5 When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener.

10.39.4.3.6 If the longitudinal stiffeners are placed at their maximum w/t ratio to be designed for the basic allowable design stresses of 0.55 F_y and the number of longitudinal stiffeners exceeds 2, then transverse stiffeners should be considered.

10.39.4.3.7 Compression flanges stiffened longitudinally shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress.

10.39.4.4 Compression Flanges Stiffened Longitudinally and Transversely

10.39.4.4.1 The longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to

$$I_s = 8\ t_f^3 w \quad (10\text{-}80)$$

10.39.4.4.2 The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge is at least equal to

$$I_t = 0.10(n+1)^3 w^3 \frac{f_s}{E}\frac{A_f}{a} \quad (10\text{-}81)$$

where

A_f = area of bottom flange including longitudinal stiffeners;
a = spacing of transverse stiffeners;
f_s = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener;
E = modulus of elasticity of steel.

10.39.4.4.3 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 F_y, the ratio w/t for the longitudinal stiffeners shall not exceed the value given by the formula

$$\frac{w}{t} = \frac{3{,}070\sqrt{k_1}}{\sqrt{F_y}} \quad (10\text{–}82)$$

where

$$k_1 = \frac{[1+(a/b)^2]^2 + 87.3}{(n+1)^2(a/b)^2[1+0.1(n+1)]} \quad (10\text{–}83)$$

10.39.4.4.4 For greater values of w/t, but not exceeding 60 or $(6{,}650\sqrt{k_1})/\sqrt{F_y}$, whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula

$$f_b = 0.55F_y - 0.224F_y \times \left[1 - \sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k_1} - \frac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k_1}}\right)\right] \quad (10\text{–}84)$$

10.39.4.4.5 For values of w/t exceeding $(6{,}650\sqrt{k_1})/\sqrt{F_y}$ but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula

$$f_b = 14.4k_1\left(\frac{t}{w}\right)^2 \times 10^6 \tag{10–85}$$

10.39.4.4.6 The maximum value of the buckling coefficient, k_1, shall be 4. When k_1 has its maximum value, the transverse stiffeners shall have a spacing, a, equal to or less than 4w. If the ratio a/b exceeds 3, transverse stiffeners are not necessary.

10.39.4.4.7 The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of the box and to each longitudinal stiffener. The connection to the web shall be designed to resist the vertical force determined by the formula

$$R_w = \frac{F_y S_s}{2b} \tag{10–86}$$

where S_s = section modulus of the transverse stiffener.

10.39.4.4.8 The connection to each longitudinal stiffener shall be designed to resist the vertical force determined by the formula

$$R_s = \frac{F_y S_s}{nb} \tag{10–87}$$

10.39.4.4.9 Compression flanges stiffened longitudinally and transversely shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress.

10.39.4.5 Compression Flange Stiffeners, General

10.39.4.5.1 The width to thickness ratio of any outstanding element of the flange stiffeners shall not exceed the value determined by the formula

$$\frac{b'}{t'} = \frac{2,600}{\sqrt{F_y}} \tag{10–88}$$

where

b' = width of any outstanding stiffener element
t' = thickness of outstanding stiffener element
F_y = yield strength of outstanding stiffener element.

10.39.4.5.2 Longitudinal stiffeners shall be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds.

10.39.5 Design of Flange to Web Welds

The total effective thickness of the web-flange welds shall be not less than the thickness of the web, except, when two or more interior intermediate diaphragms per span are provided, the minimum size fillet welds specified in Article 10.23.2.2 may be used. Regardless of the type weld used, welds shall be deposited on both sides of the connecting flange or web plate.

10.39.6 Diaphragms

10.39.6.1 Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion.

10.39.6.2 Intermediate diaphragms or cross-frames are not required for steel box girder bridges designed in accordance with this specification.

10.39.7 Lateral Bracing

Generally, no lateral bracing system is required between box girders. A horizontal wind load of 50 pounds per square foot shall be applied to the area of the superstructure exposed in elevation. Half of the resulting force shall be applied in the plane of the bottom flange. The section assumed to resist the horizontal load shall consist of the bottom flange acting as a web and 12 times the thickness of the webs acting as flanges. A lateral bracing system shall be provided if the combined stresses due to the specified horizontal force and dead load of steel and deck exceed 150% of the allowable design stress.

10.39.8 Access and Drainage

Consistent with climate, location, and materials, consideration shall be given to the providing of manholes, or other openings, either in the deck slab or in the steel box for form removal, inspection, maintenance, drainage, etc.

10.40 HYBRID GIRDERS

10.40.1 General

10.40.1.1 This section pertains to the design of girders that utilize a lower strength steel in the web

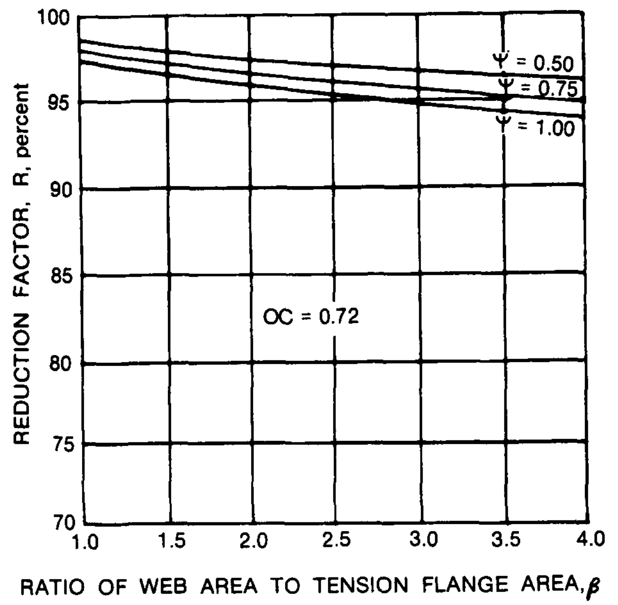


FIGURE 10.40.2.1A

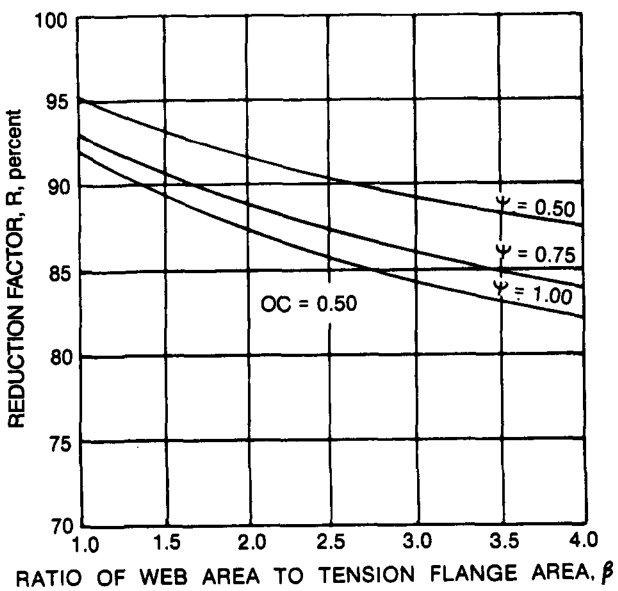


FIGURE 10.40.2.1B

than in one or both of the flanges. It applies to composite and noncomposite plate girders, and composite box girders. At any cross section where the bending stress in either flange exceeds 55% of the minimum specified yield strength of the web steel, the compression-flange area shall not be less than the tension-flange area. The top-flange area shall include the transformed area of any portion of the slab or reinforcing steel that is considered to act compositely with the steel girder.

10.40.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.40.1 through 10.40.4.

10.40.2 Allowable Stresses

10.40.2.1 Bending

10.40.2.1.1 The bending stress in the web may exceed the allowable stress for the web steel provided that the stress in each flange does not exceed the allowable stress from Articles 10.3 or 10.32 for the steel in that flange multiplied by the reduction factor, R.

$$R = 1 - \frac{\beta\psi(1-\alpha)^2(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)} \qquad (10\text{-}89)$$

(See Figure 10.40.2.1A and 10.40.2.1B.)

where:

α = minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange;*

β = area of the web divided by the area of the tension flange;*

ψ = distance from the outer edge of the tension flange* to the neutral axis (of the transformed section for composite girders) divided by the depth of the steel section.

10.40.2.1.2 The bending stress in the concrete slab in composite girders shall not exceed the allowable stress for the concrete multiplied by R.

10.40.2.1.3 R shall be taken as 1.0 at sections where the bending stress in both flanges does not exceed the allowable stress for the web.

10.40.2.1.4 Longitudinal web stiffeners preferably shall not be located in yielded portions of the web.

10.40.2.2 Shear

The design of the web for a hybrid girder shall be in compliance with Article 10.34.3 except that Equation (10-26) of Article 10.34.4.2 for the allowable average shear stress in the web of transversely stiffened nonhybrid girders shall be replaced by the following equation for the allowable average shear stress in the web of transversely stiffened hybrid girders:

*Bottom flange of orthotropic deck bridges.

$$F_v = CF_y/3 \leq F_y/3 \qquad (10\text{-}90)$$

where F_y is equal to the specified minimum yield strength of the web. The provisions of Article 10.34.4.4, and the equation for A in Article 10.34.4.7 are not applicable to hybrid girders.

10.40.2.3 Fatigue

Hybrid girders shall be designed for the allowable fatigue stress range given in Article 10.3 and Table 10.3.1A.

10.40.3 Plate Thickness Requirements

In calculating the maximum width-to-thickness ratio of the flange plate according to Article 10.34.2, f_b shall be taken as the lesser of the calculated bending stress in the compression flange divided by the reduction factor, R, or the allowable bending stress for the compression flange.

10.40.4 Bearing Stiffener Requirements

In designing bearing stiffeners at interior supports of continuous hybrid girders for which α is less than 0.7, no part of the web shall be assumed to act in bearing.

10.41 ORTHOTROPIC-DECK SUPERSTRUCTURES

10.41.1 General

10.41.1.1 This section pertains to the design of steel bridges that utilize a stiffened steel plate as a deck. Usually the deck plate is stiffened by longitudinal ribs and transverse beams; effective widths of deck plate act as the top flanges of these ribs and beams. Usually the deck including longitudinal ribs, acts as the top flange of the main box or plate girders. As used in Articles 10.41.1 through 10.41.4.10, the terms rib and beam refer to sections that include an effective width of deck plate.

10.41.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.41.1 through 10.41.4.10.

An appropriate method of elastic analysis, such as the equivalent-orthotropic-slab method or the equivalent-grid method, shall be used in designing the deck. The equivalent stiffness properties shall be selected to correctly simulate the actual deck. An appropriate method of elastic analysis, such as the thin-walled-beam method, that accounts for the effects of torsional distortions of the cross-sectional shape shall be used in designing the girders of orthotropic-deck box-girder bridges. The box-girder design shall be checked for lane or truck loading arrangements that produce maximum distortional (torsional) effects.

10.41.1.3 For an alternate design method (Strength Design), see Article 10.60.

10.41.2 Wheel Load Contact Area

The wheel loads specified in Article 3.7 shall be uniformly distributed to the deck plate over the rectangular area defined below:

Wheel Load (kip)	Width Perpendicular to Traffic (inches)	Length in Direction of Traffic (inches)
8	20 + 2t	8 + 2t
12	20 + 2t	8 + 2t
16	24 + 2t	8 + 2t

In the above table, t is the thickness of the wearing surface in inches.

10.41.3 Effective Width of Deck Plate

10.41.3.1 Ribs and Beams

The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam may be calculated by accepted approximate methods.*

10.41.3.2 Girders

10.41.3.2.1 The full width of deck plate may be considered effective in acting as the top flange of the girders if the effective span of the girders is not less than: (1) 5 times the maximum distance between girder webs and (2) 10 times the maximum distance from edge of the deck to the nearest girder web. The effective span shall be taken as the actual span for simple spans and the distance between points of contraflexure for continuous spans. Alternatively, the effective width may be determined by accepted analytical methods.

10.41.3.2.2 The effective width of the bottom flange of a box girder shall be determined according to the provisions of Article 10.39.4.1.

10.41.4 Allowable Stresses

10.41.4.1 Local Bending Stresses in Deck Plate

The term local bending stresses refers to the stresses caused in the deck plate as it carries a wheel load to the ribs and beams. The local transverse bending stresses caused in the deck plate by the specified wheel load plus 30% impact shall not exceed 30,000 psi unless a higher allowable stress is justified by a detailed fatigue analysis or

*"Design Manual for Orthotropic Steel Plate Deck Bridges," AISC, 1963, or "Orthotropic Bridges, Theory and Design," by M.S. Troitsky, Lincoln Arc Welding Foundation, 1967.

by applicable fatigue-test results. For deck configurations in which the spacing of transverse beams is at least 3 times the spacing of longitudinal-rib webs, the local longitudinal and transverse bending stresses in the deck plate need not be combined with the other bending stresses covered in Articles 10.41.4.2 and 10.41.4.3.

10.41.4.2 Bending Stresses in Longitudinal Ribs

The total bending stresses in longitudinal ribs due to a combination of (1) bending of the rib and (2) bending of the girders may exceed the allowable bending stresses in Article 10.32 by 25%. The bending stress due to each of the two individual modes shall not exceed the allowable bending stresses in Article 10.32.

10.41.4.3 Bending Stresses in Transverse Beams

The bending stresses in transverse beams shall not exceed the allowable bending stresses in Article 10.32.

10.41.4.4 Intersections of Ribs, Beams, and Girders

Connections between ribs and the webs of beams, holes in the webs of beams to permit passage of ribs, connections of beams to the webs of girders, and rib splices may affect the fatigue life of the bridge when they occur in regions of tensile stress. Where applicable, the number of cycles of maximum stress and the allowable fatigue stresses given in Article 10.3 shall be applied in designing these details; elsewhere, a rational fatigue analysis shall be made in designing the details. Connections between webs of longitudinal ribs and the deck plate shall be designed to sustain the transverse bending fatigue stresses caused in the webs by wheel loads.

10.41.4.5 Thickness of Plate Elements

10.41.4.5.1 Longitudinal Ribs and Deck Plate

Plate elements comprising longitudinal ribs, and deck-plate elements between webs of these ribs, shall meet the minimum thickness requirements of Article 10.35.2. The quantity f_a may be taken as 75% of the sum of the compressive stresses due to (1) bending of the rib and (2) bending of the girder, but not less than the compressive stress due to either of these two individual bending modes.

10.41.4.5.2 Girders and Transverse Beams

Plate elements of box girders, plate girders, and transverse beams shall meet the requirements of Articles 10.34.2 to 10.34.6 and 10.39.4.

10.41.4.6 Maximum Slenderness of Longitudinal Ribs

The slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following formula unless it can be shown by a detailed analysis that overall buckling of the deck will not occur as a result of compressive stress induced by bending of the girders

$$\left(\frac{L}{r}\right)_{max} = 1{,}000\sqrt{\frac{1{,}500}{F_y} - \frac{2{,}700F}{F_y^2}} \qquad (10\text{-}91)$$

where

- L = distance between transverse beams;
- r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate;
- F = maximum compressive stress in psi in the deck plate as a result of the deck acting as the top flange of the girders; this stress shall be taken as positive;
- F_y = yield strength of rib material in psi.

10.41.4.7 Diaphragms

Diaphragms, cross frames, or other means shall be provided at each support to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders. The stiffness and strength of the intermediate and support diaphragms or cross frames shall be consistent with the analysis of the girders.

10.41.4.8 Stiffness Requirements

10.41.4.8.1 Deflections

The deflections of ribs, beams, and girders due to live load plus impact may exceed the limitations in Article 10.6 but preferably shall not exceed 1/500 of their span. The calculation of the deflections shall be consistent with the analysis used to calculate the stresses.

To prevent excessive deterioration of the wearing surface, the deflection of the deck plate due to the specified wheel load plus 30% impact preferably shall be less than 1/300 of the distance between webs of ribs. The stiffening effect of the wearing surface shall not be included in calculating the deflection of the deck plate.

10.41.4.8.2 Vibrations

The vibrational characteristics of the bridge shall be considered in arriving at a proper design.

10.41.4.9 Wearing Surface

A suitable wearing surface shall be adequately bonded to the top of the deck plate to provide a smooth, nonskid riding surface and to protect the top of the plate against corrosion and abrasion. The wearing surface material shall provide (1) sufficient ductility to accommodate, without cracking or debonding, expansion and contraction imposed by the deck plate, (2) sufficient fatigue strength to withstand flexural cracking due to deck-plate deflections, (3) sufficient durability to resist rutting, shoving, and wearing, (4) imperviousness to water and motor-vehicle fuels and oils, and (5) resistance to deterioration from deicing salts, oils, gasolines, diesel fuels, and kerosenes.

10.41.4.10 Closed Ribs

Closed ribs without access holes for inspection, cleaning, and painting are permitted. Such ribs shall be sealed against the entrance of moisture by continuously welding (1) the rib webs to the deck plate, (2) splices in the ribs, and (3) diaphragms, or transverse beam webs, to the ends of the ribs.

Part D
STRENGTH DESIGN METHOD
LOAD FACTOR DESIGN

10.42 SCOPE

Load factor design is a method of proportioning structural members for multiples of the design loads. To ensure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings, and to the control of live load deflections under service loadings. See Part C—Service Load Design Method—Allowable Stress Design for an alternate design procedure.

10.43 LOADS

10.43.1 Service live loads are vehicles which may operate on a highway legally without special load permit.

10.43.2 For design purposes, the service loads are taken as the dead, live, and impact loadings described in Section 3.

10.43.3 Overloads are the live loads that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes, the maximum overload is taken as $5(L + I)/3$.

10.43.4 The maximum loads are the loadings specified in Article 10.47.

10.44 DESIGN THEORY

10.44.1 The moments, shears, and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 10.48.1.3.

10.44.2 The members shall be proportioned by the methods specified in Articles 10.48 through 10.56 so that their computed maximum strengths shall be at least equal to the total effects of design loads multiplied by their respective load factors specified in Article 3.22.

10.44.3 Service behavior shall be investigated as specified in Articles 10.57 through 10.59.

10.45 ASSUMPTIONS

10.45.1 Strain in flexural members shall be assumed directly proportional to the distance from the neutral axis.

10.45.2 Stress in steel below the yield strength, F_y, of the grade of steel used shall be taken as 29,000,000 psi times the steel strain. For strain greater than that corresponding to the yield strength, F_y, the stress shall be considered independent of strain and equal to the yield strength, F_y. This assumption shall apply also to the longitudinal reinforcement in the concrete floor slab in the region of negative moment when shear connectors are provided to ensure composite action in this region.

10.45.3 At maximum strength the compressive stress in the concrete slab of a composite beam shall be assumed independent of strain and equal to $0.85f'_c$.

10.45.4 Tensile strength of concrete shall be neglected in flexural calculations, except as permitted under the provisions of Articles 10.57.2, 10.58.1, and 10.58.2.2.

10.46 DESIGN STRESS FOR STRUCTURAL STEEL

The design stress for structural steel shall be the specified minimum yield point or yield strength, F_y, of the steel used as set forth in Article 10.2.

10.47 MAXIMUM DESIGN LOADS

The maximum moments, shears, or forces to be sustained by a stress-carrying member shall be computed for the load combinations specified in Article 3.22. Each part of the structure shall be proportioned for the group loads that are applicable and the maximum design required by the group loading combinations shall be used.

10.48 FLEXURAL MEMBERS

Flexural members are subject to the following requirements in this article in addition to any applicable requirements from Articles 10.49 through 10.61 that may supersede these requirements. The compression-flange width, b, on fabricated I-shaped girders preferably shall not be less than 0.2 times the web depth, but in no case shall it be less than 0.15 times the web depth. If the area of the compression flange is less than the area of the tension flange, the minimum flange width may be based on two times the depth of the web in compression rather than the web depth. The compression-flange thickness, t, preferably shall not be less than 1.5 times the web thickness. The width-to-thickness ratio, b/t, of flanges subject to tension shall not exceed 24.

10.48.1 Compact Sections

Sections of properly braced constant-depth flexural members without longitudinal web stiffeners, without holes in the tension flange and with high resistance to local buckling qualify as compact sections.

Sections of rolled or fabricated flexural members meeting the requirements of Article 10.48.1.1 below shall be considered compact sections and the maximum strength shall be computed as

$$M_u = F_y Z \qquad (10\text{-}92)$$

where F_y is the specified yield point of the steel being used, and Z is the plastic section modulus.*

10.48.1.1 Compact sections shall meet the following requirements: (For certain frequently used steels these requirements are listed in Table 10.48.1.2A.)

(a) Compression flange

$$\frac{b}{t} \leq \frac{4,110}{\sqrt{F_y}} \qquad (10\text{-}93)$$

where b is the flange width and t is the flange thickness.

(b) Web thickness

$$\frac{D}{t_w} \leq \frac{19,230}{\sqrt{F_y}} \qquad (10\text{-}94)$$

where D is the clear distance between the flanges and t_w is the web thickness.

When both b/t and D/t_w exceed 75% of the above limits, the following interaction equation shall apply

$$\frac{D}{t_w} + 4.68\left(\frac{b}{t}\right) \leq \frac{33,650}{\sqrt{F_{yf}}} \qquad (10\text{-}95)$$

where F_{yf} is the yield strength of the compression flange.

(c) Spacing of lateral bracing for compression flange

$$\frac{L_b}{r_y} \leq \frac{[3.6 - 2.2(M_1/M_u)] \times 10^6}{F_y} \qquad (10\text{-}96)$$

where L_b is the distance between points of bracing of the compression flange, r_y is the radius of gyration of the steel section with respect to the Y-Y axis, M_1 is the smaller moment at the end of the unbraced length of the member, and M_u is the ultimate moment from Equation (10-92) at the other end of the unbraced length: (M_1/M_u) is positive when moments cause single curvature between brace points. (M_1/M_u) is negative when moments cause reverse curvature between brace points.

The required lateral bracing shall be provided by braces capable of preventing lateral displacement and twisting of the main members or by embedment of the top and sides of the compression flange in concrete.

(d) Maximum axial compression

$$P \leq 0.15\, F_y A \qquad (10\text{-}97)$$

where A is the area of the cross section. Members with axial loads in excess of $0.15F_yA$ should be designed as beam-columns as specified in Article 10.54.2.

10.48.1.2 Article 10.48.1 is applicable to steels with a demonstrated ability to reach M_p. Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A 709 Grades 36, 50, and 50W), and AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) meet these requirements. The limitations set forth in Article 10.48.1 are given in Table 10.48.1.2A.

10.48.1.3 In the design of a continuous beam with compact negative-moment support sections of AASHTO

*Values for rolled sections are listed in the *Manual of Steel Construction,* Ninth Edition, 1989, American Institute of Steel Construction. Appendix D shows the method of computing Z as presented in the Commentary of AISI Bulletin 15.

TABLE 10.48.1.2A Limitations for Compact Sections

F_y (psi)	36,000	50,000	70,000
b/t	21.7	18.4	15.5
D/t_w	101	86	72
L_b/r_y ($M_l/M_u = 0$*)	100	72	51
L_b/r_y ($M_l/M_u = 1$*)	39	28	20

* For values of M_l/M_u other than 0 and 1, use Equation (10-96).

M 270 Grades 36, 50 and 50W (ASTM A 709 Grades 36, 50, and 50W) steel complying with the provision of Article 10.48.1.1, negative moments over such supports at Overload and Maximum Load determined by elastic analysis may be reduced by a maximum of 10%. Such reductions shall be accompanied by an increase in moments throughout adjacent spans statically equivalent and opposite in sign to the decrease of negative moments at the adjacent supports. For example, the increase in moment at the center of the span shall equal the average decrease of the moments at the two adjacent supports. The reduction shall not apply to the negative moment of a cantilever.

This 10% redistribution of moment shall not apply to compact sections of AASHTO M 270 Grade HPS70W or Grade 70W (ASTM A 709 Grade HPS70W or Grade 70W) steel.

10.48.2 Braced Noncompact Sections

For sections of rolled or fabricated flexural members not meeting the requirements of Article 10.48.1.1 but meeting the requirements of Article 10.48.2.1 below, the maximum strength shall be computed as the lesser of

$$M_u = F_y S_{xt} \qquad (10\text{-}98)$$

or

$$M_u = F_{cr} S_{xc} R_b \qquad (10\text{-}99)$$

subject to the requirement of Article 10.48.2.1(c) where

$$F_{cr} = \left(4{,}400\frac{t}{b}\right)^2 \leq F_y$$

b = compression flange width

t = compression flange thickness

S_{xt} = section modulus with respect to tension flange (in.^{3})

S_{xc} = section modulus with respect to compression flange (in.^{3})

R_b = flange-stress reduction factor determined from the provisions of Article 10.48.4.1, with f_b substituted for the term M_r/S_{xc} when Equation (10-103b) applies

f_b = factored bending stress in the compression flange, but not to exceed F_y

10.48.2.1 The above equations are applicable to sections meeting the following requirements:

(a) Compression flange

$$\frac{b}{t} \leq 24 \qquad (10\text{-}100)$$

(b) Web thickness

The web thickness shall meet the requirement given by Equation (10-104) or Equation (10-109), as applicable, subject to the corresponding requirements of Article 10.49.2 or 10.49.3. For unstiffened webs, the web thickness shall not be less than D/150.

(c) Spacing of lateral bracing for compression flange

$$L_b \leq \frac{20{,}000{,}000A_f}{F_y d} \qquad (10\text{-}101)$$

where d is the depth of beam or girder, and A_f is the flange area. If Equation (10-101) is not satisfied, M_u calculated from Equation (10-99) shall not exceed M_u calculated from the provisions of Article 10.48.4.1.

(d) Maximum axial compression

$$P \leq 0.15\, F_y A. \qquad (10\text{-}102)$$

Members with axial loads in excess of 0.15 F_yA should be designed as beam-columns as specified in Article 10.54.2.

10.48.2.2 The limitations set forth in Article 10.48.2.1 above are given in Table 10.48.2.1A.

10.48.3 Transitions

The maximum strength of sections with geometric properties falling between the limits of Articles 10.48.1

TABLE 10.48.2.1A Limitations for Braced Noncompact Sections

F_y (psi)	36,000	50,000	70,000	90,000	100,000
b/t *	23.2	19.7	16.6	14.7	13.9
$\frac{L_b d}{A_f}$	556	400	286	222	200
D/t_w	Refer to Articles 10.48.5.1, 10.48.6.1, 10.49.2, or 10.49.3, as applicable. For unstiffened webs, the limit is 150.				

* Limits shown are for $F_{cr} = F_y$. Refer also to Articles 10.48.2 and 10.48.2.1(a).

and 10.48.2 may be computed by straight-line interpolation, except that the web thickness must always satisfy Article 10.48.1.1(b).

10.48.4 Partially Braced Members

Members not meeting the lateral bracing requirement of Article 10.48.2.1(c) shall be braced at discrete locations spaced at a distance, L_b, such that the maximum strength of the section under consideration satisfies the requirements of Article 10.48.4.1. Bracing shall be provided such that lateral deflection of the compression flange is restrained and the entire section is restrained against twisting.

10.48.4.1 If the lateral bracing requirement of Article 10.48.2.1(c) is not satisfied and the ratio of the moment of inertia of the compression flange to the moment of inertia of the member about the vertical axis of the web, I_{yc}/I_y, is within the limits of $0.1 \leq I_{yc}/I_y \leq 0.9$, the maximum strength for the limit state of lateral-torsional buckling shall be computed as

$$M_u = M_r R_b \qquad (10\text{-}103a)$$

R_b = 1 for longitudinally stiffened girders if the web slenderness satisfies the following requirement:

$$\frac{D}{t_w} \leq 5{,}460\sqrt{\frac{k}{f_b}}$$

where

$$\text{for } \frac{d_s}{D_c} \geq 0.4 \quad k = 5.17\left(\frac{D}{d_s}\right)^2 \geq 9\left(\frac{D}{D_c}\right)^2$$

$$\text{for } \frac{d_s}{D_c} < 0.4 \quad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

d_s = the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component.

f_b = factored bending stress in the compression flange

When both edges of the web are in compression, k shall be taken equal to 7.2.

Otherwise, for girders with or without longitudinal stiffeners, R_b shall be calculated as

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_{fc}}\right)\left[\frac{D_c}{t_w} - \frac{\lambda}{\sqrt{\frac{M_r}{S_{xc}}}}\right] \leq 1.0 \qquad (10\text{-}103b)$$

D_c = depth of the web in compression (in.). For composite beams and girders, D_c shall be calculated in accordance with the provisions specified in Article 10.50(b).

t_w = thickness of web (in.);

A_{fc} = area of compression flange (in.^{2});

M_r = lateral torsional buckling moment, or yield moment, defined below (lb-in.);

S_{xc} = section modulus with respect to compression flange (in.^{3}). Use S_{xc} for live load for a composite section;

λ = 15,400 for all sections where D_c is less than or equal to D/2;

= 12,500 for sections where D_c is greater than D/2.

The moment capacity, M_r, cannot exceed the yield moment, M_y. In addition M_r cannot exceed the lateral torsional buckling moment given below:

For sections with $\frac{D_c}{t_w} \leq \frac{\lambda}{\sqrt{F_y}}$ or with longitudinally stiffened webs

$$M_r = 91 \times 10^6 C_b\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \leq M_y \qquad (10\text{-}103c)$$

For sections with $\frac{\lambda}{\sqrt{F_y}} < \left(\frac{D_c}{t_w}\right)$

for $L_b \leq L_p$

$$M_r = M_y \qquad (10\text{-}103d)$$

for $L_r \geq L_b > L_p$

$$M_r = C_b F_y S_{xc}\left[1 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \qquad (10\text{-}103e)$$

$$L_r = \left(\frac{572 \times 10^6 I_{yc} d}{F_y S_{xc}}\right)^{1/2} \qquad (10\text{-}103f)$$

for $L_b > L_r$

$$M_r = C_b\frac{F_y S_{xc}}{2}\left(\frac{L_r}{L_b}\right)^2 \qquad (10\text{-}103g)$$

L_b = unbraced length of the compression flange, in.

L_p = $9{,}500r'/\sqrt{F_y}$, in.

r' = radius of gyration of compression flange about the vertical axis in the plane of the web, in.

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

$J = \frac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, respectively, $in.^4$

$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

$C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.*

The compression flange shall satisfy the requirement of Article 10.48.2.1(a). The web thickness shall meet the requirement given by Equation (10-104) or Equation (10-109), as applicable, subject to the corresponding requirements of Article 10.49.2 or 10.49.3. For unstiffened webs, the web thickness shall not be less than D/150.

10.48.4.2 Members with axial loads in excess of $0.15F_yA$ should be designed as beam-columns as specified in Article 10.54.2.

10.48.5 Transversely Stiffened Girders

10.48.5.1 For girders not meeting the shear requirements of Article 10.48.8.1 (Equation 10-113) transverse stiffeners are required for the web. For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement:

$$\frac{D}{t_w} \leq \frac{36,500}{\sqrt{F_y}} \qquad (10\text{-}104)$$

subject to the web thickness requirement of Article 10.49.2. For different grades of steel this limit is

D/t_w	F_y(psi)
192	36,000
163	50,000
138	70,000
122	90,000
115	100,000

* For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures,* 4th Ed., pg. 135.

If the web slenderness D/t_w exceeds the upper limit, either the section shall be modified to comply with the limit, or a longitudinal stiffener shall be provided.

10.48.5.2 The maximum bending strength of transversely stiffened girders meeting the requirements of Article 10.48.5.1 shall be computed by Articles 10.48.1, 10.48.2, 10.48.4.1, 10.50, 10.51, or 10.53, as applicable, subject to the requirements of Article 10.48.8.2.

10.48.5.3 The shear capacity of transversely stiffened girders shall be computed by Article 10.48.8. The width-to-thickness ratio of transverse stiffeners shall be such that

$$\frac{b'}{t} \leq 16 \qquad (10\text{-}105)$$

where b' is the projecting width of the stiffener.

The gross cross-sectional area of intermediate transverse stiffeners shall not be less than

$$A = \left[0.15B\frac{D}{t_w}(1-C)\left(\frac{V}{V_u}\right) - 18\right]\frac{F_{y\,web}}{F_{cr}}t_w^2 \qquad (10-106a)$$

$$\text{where } F_{cr} = \frac{9,025,000}{\left(\frac{b'}{t}\right)^2} \leq F_{y\,stiffener} \qquad (10-106b)$$

where $F_{y\ stiffener}$ is the yield strength of the stiffener; B = 1.0 for stiffener pairs, 1.8 for single angles, and 2.4 for single plates; and C is computed by Article 10.48.8.1. When values computed by Equation (10-106a) approach zero or are negative, then transverse stiffeners need only meet the requirements of Equations (10-105) and (10-107), and Article 10.34.4.10.

The moment of inertia of transverse stiffeners with reference to the plane defined below shall be not less than

$$I = d_o t_w^3 J \qquad (10\text{-}107)$$

where

$J = 2.5(D/d_o)^2 - 2$, but not less than 0.5 (10-108)

d_o = distance between transverse stiffeners

When stiffeners are in pairs, the moment of inertia shall be taken about the center line of the web plate. When single stiffeners are used, the moment of inertia shall be taken about the face in contact with the web plate.

Transverse stiffeners need not be in bearing with the tension flange. The distance between the end of the stiffener weld and the near edge of the web-to-flange fillet weld shall not be less than $4t_w$ or more than $6t_w$. Stiffeners provided on only one side of the web must be in bearing against, but need not be attached to, the compression flange for the stiffener to be effective. However, transverse stiffeners which connect diaphragms or crossframes to the beam or girder shall be rigidly connected to both the top and bottom flanges.

10.48.6 Longitudinally Stiffened Girders

10.48.6.1 Longitudinal stiffeners shall be required on symmetrical girders when the web thickness is less than that specified by Article 10.48.5.1 and shall be placed at a distance D/5 from the inner surface of the compression flange.

The web thickness of plate girders with transverse stiffeners and one longitudinal stiffener shall meet the requirement:

$$\frac{D}{t_w} \le \frac{73{,}000}{\sqrt{F_y}} \qquad (10\text{-}109)$$

For different grades of steel, this limit is

D/t_w	F_y(psi)
385	36,000
326	50,000
276	70,000
243	90,000
231	100,000

Singly symmetric girders are subject to the requirements of Article 10.49.3.

10.48.6.2 The maximum bending strength of longitudinally stiffened girders meeting the requirements of Article 10.48.6.1 shall be computed by Articles 10.48.2, 10.48.4.1, 10.50.1.2, 10.50.2.2, 10.51, or 10.53, as applicable, subject to the requirements of Article 10.48.8.2.

10.48.6.3 The shear capacity of girders with one longitudinal stiffener shall be computed by Article 10.48.8.

The dimensions of the longitudinal stiffener shall be such that

(a) the thickness of the longitudinal stiffener is not less than that given by Article 10.34.5.2, and the factored bending stress in the longitudinal stiffener is not greater than the yield strength of the longitudinal stiffener.

(b) the rigidity of the stiffener is not less than:

$$I \ge Dt_w^3\left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13\right] \qquad (10\text{-}110)$$

where:

I = moment of inertia of the longitudinal stiffener about its edge in contact with the web plate, in^4.

(c) the radius of gyration of the stiffener is not less than

$$r \ge \frac{d_o\sqrt{F_y}}{23{,}000} \qquad (10\text{-}111)$$

In computing the r value above, a centrally located web strip not more than $18t_w$ in width shall be considered as a part of the longitudinal stiffener. Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 10.48.5.3. In addition, the section modulus of the transverse stiffener shall be not less than

$$S_s = \frac{1}{3}(D/d_o)S_t \qquad (10\text{-}112)$$

where D is the total panel depth (clear distance between flange components) and S_t is the section modulus of the longitudinal stiffener.

10.48.7 Bearing Stiffeners

Bearing stiffeners shall be designed for beams and girders as specified in Articles 10.33.2 and 10.34.6.

10.48.8 Shear

10.48.8.1 The shear capacity of webs of rolled or fabricated flexural members shall be computed as follows:

For unstiffened webs, the shear capacity shall be limited to the plastic or buckling shear force as follows:

$$V_u = CV_p \qquad (10\text{-}113)$$

For stiffened web panels complying with the provisions of Article 10.48.8.3, the shear capacity shall be determined by including post-buckling resistance due to tension-field action as follows:

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \qquad (10\text{-}114)$$

V_p is equal to the plastic shear force and is determined as follows:

$$V_p = 0.58F_yDt_w \qquad (10\text{-}115)$$

The constant C is equal to the buckling shear stress divided by the shear yield stress, and is determined as follows:

$$\text{for } \frac{D}{t_w} < \frac{6{,}000\sqrt{k}}{\sqrt{F_y}}$$

$$C = 1.0$$

$$\text{for } \frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \le \frac{D}{t_w} \le \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{6{,}000\sqrt{k}}{\left(\frac{D}{t_w}\right)\sqrt{F_y}} \qquad (10\text{-}116)$$

$$\text{for } \frac{D}{t_w} > \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{4.5 \times 10^7 k}{\left(\frac{D}{t_w}\right)^2 F_y} \qquad (10\text{-}117)$$

where the buckling coefficient, $k = 5 + [5 \div (d_o/D)^2]$, except k shall be taken as 5 for unstiffened beams and girders.

D = clear, unsupported distance between flange components;
d_o = distance between transverse stiffeners;
F_y = yield strength of the web plate.

10.48.8.2 If a girder panel is controlled by Equation (10-114) and is subjected to the simultaneous action of shear and bending moment with the magnitude of the moment greater than $0.75M_u$, the shear shall be limited to not more than

$$V/V_u = 2.2 - (1.6M/M_u) \qquad (10\text{-}118)$$

If girder panel of a composite noncompact section is controlled by Equation (10-114) and is subjected to the simultaneous action of shear and bending moment with the magnitude of the factored bending stress f_s greater than $0.75\ F_u$, the shear shall instead be limited to not more than:

$$V/V_u = 2.2 - (1.6 f_s/F_u) \qquad (10\text{-}118a)$$

where f_s = factored bending stress in either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_u)
F_u = maximum bending strength of either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_u)

10.48.8.3 Where transverse intermediate stiffeners are required, transverse stiffeners shall be spaced at a distance, d_o, according to shear capacity as specified in Article 10.48.8.1, but not more than 3D. Transverse stiffeners may be omitted in those portions of the girders where the maximum shear force is less than the value given by Article 10.48.8.1, Equation (10-113), subject to the handling requirement below.

Transverse stiffeners shall be required if D/t_w is greater than 150. The spacing of these stiffeners shall not exceed the handling requirement $D[260/(D/t_w)]^2$.

For longitudinally stiffened girders, transverse stiffeners shall be spaced a distance, d_o, according to shear capacity as specified in Article 10.48.8.1, but not more than 1.5 times the web depth. The handling requirement given above shall not apply to longitudinally stiffened girders. The total web depth D shall be used in determining the shear capacity of longitudinally stiffened girders in Article 10.48.8.1 and in Equation (10-119).

The first stiffener space at the simple support end of a transversely or longitudinally stiffened girder shall be such that the shear force in the end panel will not exceed the plastic or buckling shear force given by the following equation

$$V_u = CV_p \qquad (10\text{-}119)$$

For transversely stiffened girders, the maximum spacing of the first transverse stiffener is limited to 1.5D. For longitudinally stiffened girders, the maximum spacing of the first transverse stiffener is also limited to 1.5D.

10.49 SINGLY SYMMETRIC SECTIONS

10.49.1 General

For sections symmetric about the vertical axis but unsymmetric with respect to the horizontal centroidal axis, the provisions of Articles 10.48.1 through 10.48.4 shall be applicable.

10.49.2 Singly Symmetric Sections with Transverse Stiffeners

Girders with transverse stiffeners shall be designed and evaluated by the provisions of Article 10.48.5 except that when D_c, the clear distance between the neutral axis and the compression flange, exceeds D/2 the web thickness, t_w, shall meet the requirement

$$\frac{D_c}{t_w} \le \frac{18{,}250}{\sqrt{F_y}} \qquad (10\text{-}120)$$

If the web slenderness D_c/t_w exceeds the upper limit, either the section shall be modified to comply with the limit, or a longitudinal stiffener shall be provided.

10.49.3 Longitudinally Stiffened Singly Symmetric Sections

10.49.3.1 Longitudinal stiffeners shall be required on singly symmetric sections when the web thickness is less than that specified by Article 10.49.2.

10.49.3.2 For girders with one longitudinal stiffener and transverse stiffeners, the provisions of Article 10.48.6 for symmetrical sections shall be applicable in addition to the following:

(a) The optimum distance, d_s, of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener from the inner surface or the leg of the compression flange component is D/5 for a symmetrical girder. The optimum distance, d_s, for a singly symmetric composite girder in positive-moment regions may be determined from the equation given below

$$\frac{d_s}{D_{cs}} = \frac{1}{1 + 1.5\sqrt{\frac{f_{DL+LL}}{f_{DL}}}} \qquad (10\text{-}121)$$

where D_{cs} is the depth of the web in compression of the noncomposite steel beam or girder, f_{DL} is the noncomposite dead-load stress in the compression flange, and f_{DL+LL} is the total noncomposite and composite dead-load plus the composite live-load stress in the compression flange at the most highly stressed section of the web. The optimum distance, d_s, of the stiffener in negative-moment regions of composite sections is $2D_c/5$, where D_c is the depth of the web in compression of the composite section at the most highly stressed section of the web.

(b) When D_c exceeds D/2, the web thickness, t_w, shall meet the requirement

$$\frac{D_c}{t_w} \le \frac{36{,}500}{\sqrt{F_y}} \qquad (10\text{-}122)$$

10.49.4 Singly Symmetric Braced Noncompact Sections

Singly symmetric braced, noncompact sections of rolled or fabricated flexural members shall be designed and evaluated by the provisions of Article 10.48.2.

10.49.5 Partially Braced Members with Singly Symmetric Sections

The maximum strength of singly symmetric sections meeting all requirements of Article 10.48.2.1, except for the lateral bracing requirement given by Equation (10-101), shall be computed as the lesser of M_u calculated from Equation (10-98) or M_u calculated from Equation (10-99), with M_u calculated from Equation (10-99) not to exceed M_u calculated from the provisions of Article 10.48.4.1.

10.50 COMPOSITE SECTIONS

Composite sections shall be so proportioned that the following criteria are satisfied.

(a) The maximum strength of any section shall not be less than the sum of the computed moments at that section multiplied by the appropriate load factors.

(b) The web of the steel section shall be designed to carry the total external shear and must satisfy the applicable provisions of Articles 10.48 and 10.49. The value of D_c shall be taken as the clear distance between the neutral axis and the compression flange. In positive-moment regions, the value of D_c shall be calculated by summing the stresses due to the appropriate loadings acting on the respective cross sections supporting the loading. The depth of web in compression, D_c, in composite sections subjected to negative bending may be taken as the depth of the web in compression of the composite section without summing the stresses from the various stages of loading. The web depth in compression, D_{cp}, of sections meeting the web compactness and ductility requirements of Article 10.50.1.1.2 under the maximum design loads shall be calculated from the fully plastic cross section ignoring the sequence of load application. Girders with a web slenderness exceeding the limits of Article 10.48.5.1 or 10.49.2 shall either be modified to comply with these limits or else shall be stiffened by one longitudinal stiffener.

(c) The moment capacity at first yield shall be computed considering the application of the dead and live loads to the steel and composite sections.

(d) The steel beam or girder shall satisfy the constructibility requirements of Article 10.61.

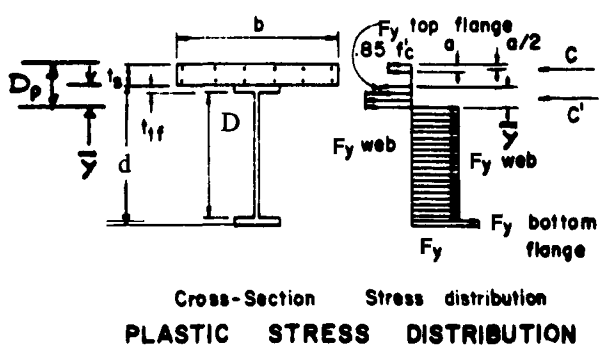


FIGURE 10.50A

10.50.1 Positive Moment Sections

10.50.1.1 Compact Sections

The maximum strength, M_u, of compact composite sections in positive-moment regions shall be computed in accordance with Article 10.50.1.1.2. The steel shall have the demonstrated ability to reach M_p. Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A 709 Grades 36, 50, and 50W), and AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) meet these requirements.

10.50.1.1.1 The resultant moment of the fully plastic stress distribution may be computed as follows:

(a) The compressive force in the slab, C, is equal to the smallest of the values given by the following Equations:

$$C = 0.85f'_c\, bt_s + (AF_y)_c \qquad (10\text{-}123)$$

where b is the effective width of slab, specified in Article 10.38.3, t_s is the slab thickness, and $(AF_y)_c$ is the product of the area and yield point of that part of reinforcement which lies in the compression zone of the slab.

$$C = (AF_y)_{bf} + (AF_y)_{tf} + (AF_y)_w \qquad (10\text{-}124)$$

where $(AF_y)_{bf}$ is the product of area and yield point for bottom flange of steel section (including cover plate if any), $(AF_y)_{tf}$ is the product of area and yield point for top flange of steel section, and $(AF_y)_w$ is the product of area and yield point for web of steel section.

(b) The depth of the stress block is computed from the compressive force in the slab.

$$a = \frac{C - (AF_y)_c}{0.85f'_c b} \qquad (10\text{-}125)$$

(c) When the compressive force in the slab is less than the value given by Equation (10-124), the top portion of the steel section will be subjected to the compressive force C′ (Figure 10.50A) given by the following equation:

$$C' = \frac{\Sigma(AF_y) - C}{2} \qquad (10\text{-}126)$$

(d) The location of the neutral axis within the steel section measured from the top of the steel section may be determined as follows:

for $C' < (AF_y)_{tf}$

$$\bar{y} = \frac{C'}{(AF_y)_{tf}} t_{tf} \qquad (10\text{-}127)$$

for $C' \geq (AF_y)_{tf}$

$$\bar{y} = t_{tf} + \frac{C' - (AF_y)_{tf}}{(AF_y)_w} D \qquad (10\text{-}128)$$

(e) The maximum strength of the section in bending is the first moment of all forces about the neutral axis, taking all forces and moment arms as positive quantities.

10.50.1.1.2 Composite sections of constant-depth members in positive-moment regions without longitudinal web stiffeners and without holes in the tension flange shall qualify as compact when the web of the steel section satisfies the following requirement:

$$\frac{2D_{cp}}{t_w} \leq \frac{19{,}230}{\sqrt{F_y}} \qquad (10\text{-}129)$$

where D_{cp} is the depth of the web in compression at the plastic moment calculated in accordance with Article 10.50.1.1.1, and t_w is the web thickness. Equation (10-129) is satisfied if the neutral axis at the plastic moment is located above the web; otherwise D_{cp} shall be computed as $\bar{y}$ from Equation (10-128) minus t_{tf}. Also, the distance from the top of the slab to the neutral axis at the plastic moment, D_p, shall satisfy

$$\left(\frac{D_p}{D'}\right) \leq 5 \qquad (10\text{-}129a)$$

where

$D' = \beta \dfrac{(d + t_s + t_h)}{7.5}$;

β = 0.9 for F_y = 36,000 psi;
= 0.7 for F_y = 50,000 and 70,000 psi;

d = depth of the steel beam or girder;

t_s = thickness of the slab;

t_h = thickness of the concrete haunch above the beam or girder top flange.

Equation (10-129a) need not be checked for sections where the maximum flange stress does not exceed the specified minimum flange yield stress.

The maximum bending strength, M_u, of compact composite sections in simple spans or in the positive-moment regions of continuous spans with compact noncomposite or composite negative-moment pier sections shall be taken as for $D_p \le D'$

$$M_u = M_p \qquad (10\text{-}129b)$$

for $D' < D_p \le 5D'$

$$M_u = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \qquad (10\text{-}129c)$$

where

M_p = plastic moment capacity of the composite positive moment section calculated in accordance with Article 10.50.1.1.1;

M_y = moment capacity at first yield of the composite positive moment section calculated as F_y times the section modulus with respect to the tension flange. The modular ratio, n, shall be used to compute the transformed section properties.

In continuous spans with compact composite positive-moment sections, but with noncompact noncomposite or composite negative-moment pier sections, the maximum bending strength, M_u, of the composite positive-moment sections shall be taken as either the moment capacity at first yield determined as specified in Article 10.50(c), or as

$$M_u = M_y + A(M_u - M_s)_{pier} \qquad (10\text{-}129d)$$

where

M_y = the moment capacity at first yield of the compact positive moment section calculated in accordance with Article 10.50(c);

$(M_u - M_s)_{Pier}$ = moment capacity of the noncompact section at the pier, M_u, given by Article 10.48.2 or Article 10.48.4, minus the elastic moment at the pier, M_s, for the loading producing maximum positive bending in the span. Use the smaller value of the difference for the two-pier sections for interior spans;

A = 1 for interior spans;
= distance from end support to the location of maximum positive moment divided by the span length for end spans.

M_u computed from Equation (10-129d) shall not exceed the applicable value of M_u computed from either Equation (10-129b) or Equation (10-129c).

For continuous spans where the maximum bending strength of the positive-moment sections is determined from Equation (10-129d), the maximum positive moment in the span shall not exceed M_y, for the loading which produces the maximum negative moment at the adjacent pier(s).

For composite sections in positive-moment regions not satisfying the requirements of Equation (10-129) or Equation (10-129a), or of variable-depth members or with longitudinal web stiffeners, or with holes in the tension flange, the maximum bending strength shall be determined as specified in Article 10.50.1.2.

10.50.1.2 Noncompact Sections

10.50.1.2.1 When the steel section does not satisfy the compactness requirements of Article 10.50.1.1.2, the sum of the bending stresses due to the appropriate loadings acting on the respective cross sections supporting the loadings shall not exceed the maximum strength, F_u, of the tension flange taken equal to F_y or the maximum strength, F_u, of the compression flange taken equal to F_yR_b, where R_b is the flange-stress reduction factor determined from the provisions of Article 10.48.4.1. When R_b is determined from Equation (10-103b), f_b shall be substituted for the term M_r/S_{xc} and A_{fc} shall be taken as the effective combined transformed area of the top flange and concrete deck that yields D_c calculated in accordance with Article 10.50(b). f_b is equal to the factored bending stress in the compression flange, but not to exceed F_y. The resulting R_b factor shall be distributed to the top flange and concrete deck in proportion to their relative stiffness. The provisions of Article 10.48.2.1(b) shall apply.

10.50.1.2.2 When the girders are not provided with temporary supports during the placing of dead loads, the sum of the stresses produced by $1.30D_s$ acting on the steel girder alone with $1.30(D_c + 5(L + I)/3)$ acting on the composite girder shall not exceed yield stress at any point, where D_s and D_c are the moments caused by the dead load acting on the steel girder and composite girder, respectively.

10.50.1.2.3 When the girders are provided with effective intermediate supports that are kept in place until the concrete has attained 75% of its required 28-day strength, stresses produced by the loading, $1.30(D + 5(L + I)/3)$, acting on the composite girder, shall not exceed yield stress at any point.

10.50.2 Negative Moment Sections

The maximum bending strength of composite sections in negative moment regions shall be computed in accordance with Article 10.50.2.1 or 10.50.2.2, as applicable.

It shall be assumed that the concrete slab does not carry tensile stresses. In cases where the slab reinforcement is continuous over interior supports, the reinforcement may be considered to act compositely with the steel section.

10.50.2.1 Compact Sections

Composite sections of constant-depth members in negative bending without longitudinal web stiffeners and without holes in the tension flange qualify as compact when their steel section meets the requirements of Article 10.48.1.1, and has the demonstrated ability to reach M_p. Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A 709 Grades 36, 50, and 50W), and AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) meet these requirements. M_u shall be computed as the resultant moment of the fully plastic stress distribution acting on the section including any composite rebars.

If the distance from the neutral axis to the compression flange exceeds D/2, the compact section requirements given by Equations (10-94) and (10-95) must be modified by replacing D with the quantity $2D_{cp}$, where D_{cp} is the depth of the web in compression at the plastic moment.

10.50.2.2 Noncompact Sections

When the steel section does not satisfy the compactness requirements of Article 10.50.2.1 but does satisfy all the requirements of Article 10.48.2.1, the sum of the bending stresses due to the appropriate loadings acting on the respective cross sections supporting the loadings shall not exceed the maximum strength, F_u, of the tension flange taken equal to F_y or the maximum strength, F_u, of the compression flange taken equal to $F_{cr}R_b$, where F_{cr} is the critical compression flange stress specified in Article 10.48.2 and R_b is the flange-stress reduction factor determined from the provisions of Article 10.48.4.1. When R_b is determined from Equation (10-103b), f_b shall be substituted for the term M_r/S_{xc}. f_b is equal to the factored bending stress in the compression flange, but not to exceed F_y. When all requirements of Article 10.48.2.1 are satisfied, except for the lateral bracing requirement given by Equation (10-101), F_u of the compression flange shall be taken equal to $F_{cr}R_b$, but not to exceed M_u/S_{xc}, where M_u and S_{xc} are determined according to the provisions of Article 10.48.4.1. In determining the factor C_b in Article 10.48.4.1, the smaller and larger values of f_b at each end of the unbraced segment of the girder shall be substituted for the smaller and larger end moments, M_1 and M_2, respectively.

10.50.2.3

The minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1% of the cross-sectional area of the concrete slab whenever the longitudinal tensile stress in the concrete slab due to either the factored construction loads or the overload specified in Article 10.57 exceeds $0.9f_r$, where f_r is the modulus of rupture specified in Article 8.15.2.1.1. The area of the concrete slab shall be taken equal to the structural thickness times the entire width of the bridge deck. The required reinforcement shall be No. 6 bars or smaller spaced at not more than 12 inches. Two-thirds of this required reinforcement is to be placed in the top layer of slab. Placement of distribution steel as specified in Article 3.24.10 is waived.

10.50.2.4

When shear connectors are omitted from the negative moment region, the longitudinal reinforcement shall be extended into the positive moment region beyond the anchorage connectors at least 40 times the reinforcement diameter.

10.51 COMPOSITE BOX GIRDERS*

This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single-cell composite box girders. The distance center-to-center flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of the flanges of each box. In addition to the above, when nonparallel girders are used the distance center-to-center of adjacent flanges at supports shall be not greater than 1.35 times and not less than 0.65 times the distance center-to-center of the flanges of each box. The cantilever overhang of the deck slab, including curbs and parapet, shall be limited to 60% of the distance between the centers of adjacent top steel flanges of adjacent box girders, but in no case greater than 6 feet.

10.51.1 Maximum Strength

The maximum strength of box girders shall be determined according to the applicable provisions of Articles 10.48, 10.49, and 10.50. In addition, the maximum strength of the negative moment sections shall be limited by

$$M_u = F_{cr}S \tag{10-130}$$

where F_{cr} is the buckling stress of the bottom flange plate as given in Article 10.51.5.

*For information regarding the design of long-span steel box girder bridges, Report No. FHWA-TS-80-205, "Proposed Design Specifications for Steel Box Girder Bridges" is available from the Federal Highway Administration.

10.51.2 Lateral Distribution

The live-load bending moment for each box girder shall be determined in accordance with Article 10.39.2.

10.51.3 Web Plates

The design shear V_w for a web shall be calculated using the following equation

$$V_w = V/\cos\theta \qquad (10\text{-}131)$$

where V = one-half of the total vertical shear force on one box girder, and θ = the angle of inclination of the web plate to the vertical.

The inclination of the web plates to the vertical shall not exceed 1 to 4.

10.51.4 Tension Flanges

In the case of simply supported spans, the bottom flange shall be considered fully effective in resisting bending if its width does not exceed one-fifth the span length. If the flange plate width exceeds one-fifth of the span, only an amount equal to one-fifth of the span shall be considered effective.

For continuous spans, the requirements above shall be applied to the distance between points of contraflexure.

10.51.5 Compression Flanges

10.51.5.1 Unstiffened compression flanges designed for the yield stress, F_y, shall have a width-to-thickness ratio equal to or less than the value obtained from the formula

$$\frac{b}{t} = \frac{6{,}140}{\sqrt{F_y}} \qquad (10\text{-}132)$$

where b = flange width between webs in inches, and t = flange thickness in inches.

10.51.5.2 For greater b/t ratios,

$$\frac{6{,}140}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{13{,}300}{\sqrt{F_y}} \qquad (10\text{-}133)$$

the buckling stress of an unstiffened bottom flange is given by the formula

$$F_{cr} = 0.592F_y\left(1 + 0.687\sin\frac{c\pi}{2}\right) \qquad (10\text{-}134)$$

in which c shall be taken as

$$c = \frac{13{,}300 - \frac{b}{t}\sqrt{F_y}}{7{,}160} \qquad (10\text{-}135)$$

10.51.5.3 For values of

$$\frac{b}{t} > \frac{13{,}300}{\sqrt{F_y}} \qquad (10\text{-}136)$$

the buckling stress of the flange is given by the formula

$$F_{cr} = 105(t/b)^2 \times 10^6 \qquad (10\text{-}137)$$

10.51.5.4 If longitudinal stiffeners are used, they shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to

$$I_s = \phi\, t^3 w \qquad (10\text{-}138)$$

where

- ϕ = $0.07k^3n^4$ when n equals 2, 3, 4, or 5;
- ϕ = $0.125k^3$ when n = 1;
- w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener;
- n = number of longitudinal stiffeners;
- k = buckling coefficient which shall not exceed 4.

10.51.5.4.1

For a longitudinally stiffened flange designed for the yield stress F_y, the ratio w/t shall not exceed the value given by the formula

$$\frac{w}{t} = \frac{3{,}070\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}139)$$

10.51.5.4.2 For greater values of w/t

$$\frac{3{,}070\sqrt{k}}{\sqrt{F_y}} < \frac{w}{t} \leq \frac{6{,}650\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}140)$$

the buckling stress of the flange, including stiffeners, is given by Article 10.51.5.2 in which c shall be taken as

$$c = \frac{6{,}650\sqrt{k} - \frac{w}{t}\sqrt{F_y}}{3{,}580\sqrt{k}} \qquad (10\text{-}141)$$

10.51.5.4.3 For values of

$$\frac{w}{t} > \frac{6{,}650\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}142)$$

the buckling stress of the flange, including stiffeners, is given by the formula

$$F_{cr} = 26.2k(t/w)^2 \times 10^6 \qquad (10\text{-}143)$$

10.51.5.4.4 When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener. The number of longitudinal stiffeners preferably shall not exceed 2. If the number of longitudinal stiffeners exceeds 2, then the use of additional transverse stiffeners should be considered.

10.51.5.5 The width-to-thickness ratio of any outstanding element of the flange stiffeners shall not exceed the value determined by the formula

$$\frac{b'}{t'} = \frac{2{,}600}{\sqrt{F_y}} \qquad (10\text{-}144)$$

where

b' = width of any outstanding stiffener element, and;
t' = thickness of outstanding stiffener element;
F_y = yield strength of outstanding stiffener element.

10.51.5.6 Compression flanges shall also satisfy the provisions of Article 10.51.4. The effective flange plate width shall be used to calculate the factored flange bending stress. The full flange plate width shall be used to calculate the buckling stress of the flange.

10.51.6 Diaphragms

Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion.

Intermediate diaphragms or cross-frames are not required for box girder bridges designed in accordance with this specification.

10.51.7 Design of Flange to Web Welds

The total effective thickness of the web-flange welds shall not be less than the thickness of the web, except, when two or more interior intermediate diaphragms per span are provided, the minimum size fillet welds specified in Article 10.23.2.2 may be used. Regardless of the type weld used, welds shall be deposited on both sides of the connecting flange or web plate.

10.52 SHEAR CONNECTORS

10.52.1 General

The horizontal shear at the interface between the concrete slab and the steel girder shall be provided for by mechanical shear connectors throughout the simple spans and the positive moment regions of continuous spans. In the negative moment regions, shear connectors shall be provided when the reinforcing steel embedded in the concrete is considered a part of the composite section. In case the reinforcing steel embedded in the concrete is not considered in computing section properties of negative moment sections, shear connectors need not be provided in these portions of the span, but additional connectors shall be placed in the region of the points of dead load contraflexure as specified in Article 10.38.5.1.3.

10.52.2 Design of Connectors

The number of shear connectors shall be determined in accordance with Article 10.38.5.1.2 and checked for fatigue in accordance with Articles 10.38.5.1.1 and 10.38.5.1.3.

10.52.3 Maximum Spacing

The maximum pitch shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange.

10.53 HYBRID GIRDERS

This section pertains to the design of girders that utilize a lower strength steel in the web than in one or both of the flanges. It applies to composite and noncomposite plate girders and to composite box girders. At any cross section where the bending stress in either flange caused by the maximum design load exceeds the minimum specified yield strength of the web steel, the compression-flange area shall not be less than the tension-flange area. The top-flange area shall include the transformed area of any portion of the slab or reinforcing steel that is considered to act compositely with the steel girder.

The provisions of Articles 10.48 through 10.52, 10.57.1, and 10.57.2 shall apply to hybrid beams and girders except as modified below. In all equations of these articles, F_y shall be taken as the minimum specified yield strength of the steel of the element under consideration with the following exceptions

(1) In Articles 10.48.1.1(b), 10.48.4.1, 10.48.5.1, 10.48.6.1, 10.49.2, 10.49.3.2(b), and 10.50.1.1.2, use F_y of the compression flange.
(2) Articles 10.57.1 and 10.57.2 shall apply to the flanges, but not to the web of hybrid girders.

The provision specified in Article 10.40.4 shall also apply. Longitudinal web stiffeners preferably shall not be located in yielded portions of the web.

10.53.1 Noncomposite Hybrid Sections

10.53.1.1 Compact Sections

The equation of Article 10.48.1 for the maximum strength of compact sections shall be replaced by the expression

$$M_u = F_{yf}Z \tag{10-145}$$

where F_{yf} is the specified minimum yield strength of the flange, and Z is the plastic section modulus.

In computing Z, the web thickness shall be multiplied by the ratio of the minimum specified yield strength of the web, F_{yw}, to the minimum specified yield strength of the flange, F_{yf}.

10.53.1.2 Braced Noncompact Sections

The equations of Article 10.48.2 for the maximum strength of braced noncompact sections shall be replaced by the expressions

$$M_u = F_{yf}S_{xt}R \tag{10-146}$$

$$M_u = F_{cr}S_{xc}R_bR \tag{10-146a}$$

For symmetrical sections

$$R = \frac{12 + \beta(3\rho - \rho^3)}{12 + 2\beta} \tag{10-147}$$

where

$\rho = F_{yw}/F_{yf}$

$\beta = A_w/A_f$

For unsymmetrical sections

$$R = 1 - \left[\frac{\beta\psi(1-\rho)^2(3 - \psi + \rho\psi)}{6 + \beta\psi(3 - \psi)}\right] \tag{10-148}$$

where ψ is the distance from the outer fiber of the tension flange to the neutral axis divided by the depth of the steel section. R shall be taken as 1.0 at sections where the stress in both flanges caused by the maximum design loads does not exceed the specified minimum yield strength of the web.

10.53.1.3 Partially Braced Members

The strength of noncompact hybrid sections of partially braced members not satisfying the lateral bracing requirement given by Equation (10-101) shall be calculated as the lesser of M_u calculated from Equation (10-146) or M_u calculated from Equation (10-146a). M_u calculated from Equation (10-146a) is not to exceed M_u calculated from the provisions of Article 10.48.4.1 with Equation (10-103a) replaced by the expression

$$M_u = M_rR_bR \tag{10-148a}$$

and the yield moment calculated as

$$M_y = F_{yf}S\,R \tag{10-148b}$$

where the appropriate R is determined from Article 10.53.1.2 above, and R_b is determined by Equation (10-103b).

10.53.2 Composite Hybrid Sections

The maximum strength of a compact composite section shall be computed as specified in Article 10.50.1.1.2 or Article 10.50.2.1, as applicable, using the specified minimum yield strength of the element under consideration to compute the plastic moment capacity. The yield moment in Article 10.50.1.1.2 shall be multiplied by R (for unsymmetrical sections) from Article 10.53.1.2, with Ψ calculated as specified below for noncompact composite sections.

The maximum strength of a noncompact composite section shall be taken as the maximum strength computed from Article 10.50.1.2 or Article 10.50.2.2, as applicable, times R (for unsymmetrical sections) from Article 10.53.1.2, in which ψ is the distance from the outer fiber of the tension flange to the neutral axis of the transformed section divided by the depth of the steel section.

10.53.3 Shear

Equation (10-114) of Article 10.48.8.1 for the shear capacity of transversely stiffened girders shall be replaced by the expression

$$V_u = V_pC \qquad (10\text{-}149)$$

The provisions of Article 10.48.8.2, and the equation for A in Article 10.48.5.3 are not applicable to hybrid girders.

10.54 COMPRESSION MEMBERS

10.54.1 Axial Loading

10.54.1.1 Maximum Capacity

The maximum strength of concentrically loaded columns shall be computed as

$$P_u = 0.85A_sF_{cr} \qquad (10\text{-}150)$$

where A_s is the gross effective area of the column cross section and F_{cr} is determined by one of the following two formulas*:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2E}\left(\frac{KL_c}{r}\right)^2\right] \qquad (10\text{-}151)$$

$$\text{for } \frac{KL_c}{r} \le \sqrt{\frac{2\pi^2E}{F_y}} \qquad (10\text{-}152)$$

$$F_{cr} = \frac{\pi^2E}{\left(\frac{KL_c}{r}\right)^2} \qquad (10\text{-}153)$$

$$\text{for } \frac{KL_c}{r} > \sqrt{\frac{2\pi^2E}{F_y}} \qquad (10\text{-}154)$$

where

K = effective length factor in the plane of buckling;
L_c = length of the member between points of support in inches;
r = radius of gyration in the plane of buckling in inches;
F_y = yield stress of the steel in pounds per square inch;
E = 29,000,000 pounds per square inch;
F_{cr} = buckling stress in pounds per square inch.

*Singly symmetric and unsymmetric compression members, such as angles or tees, and doubly symmetric compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckling. Refer to the *Manual of Steel Construction*, Ninth Edition, 1989, American Institute of Steel Construction.

10.54.1.2 Effective Length

The effective length factor K shall be determined as follows

(a) For members having lateral support in both directions at its ends

K = 0.75 for riveted, bolted, or welded end connections;
K = 0.875 for pinned ends.

(b) For members having ends not fully supported laterally by diagonal bracing or an attachment to an adjacent structure, the effective length factor shall be determined by a rational procedure.**

10.54.2 Combined Axial Load and Bending

10.54.2.1 Maximum Capacity

The combined maximum axial force P and the maximum bending moment M acting on a beam-column subjected to eccentric loading shall satisfy the following equations:

$$\frac{P}{0.85A_sF_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_sF_e}\right)} \le 1.0 \qquad (10\text{-}155)$$

$$\frac{P}{0.85A_sF_y} + \frac{M}{M_p} \le 1.0 \qquad (10\text{-}156)$$

where:

F_{cr} = buckling stress as determined by the equations of Article 10.54.1.1;
M_u = maximum strength as determined by Articles 10.48.1, 10.48.2, or 10.48.4;

$$F_e = \frac{E\pi^2}{\left(\frac{KL_c}{r}\right)^2} = \text{the Euler Buckling stress in the plane of bending;} \qquad (10\text{-}157)$$

C = equivalent moment factor, as defined below;
M_p = F_yZ, the full plastic moment of the section;
Z = plastic section modulus;
$\frac{KL_c}{r}$ = effective slenderness ratio in the plane of bending.

**B. G. Johnston, *Guide to Stability Design Criteria for Metal Structures,* John Wiley and Sons, Inc., New York, 1976.

10.54.2.2 Equivalent Moment Factor C

If the ends of the beam-column are restrained from sidesway in the plane of bending by diagonal bracing or attachment to an adjacent laterally braced structure, then the value of equivalent moment factor, C, may be computed by the formula

$$C = 0.6 + 0.4a \qquad (10\text{-}158)$$

where a is the ratio of the numerically smaller to the larger end moment. The ratio a is positive when the two end moments act in an opposing sense (i.e., one acts clockwise and the other acts counterclockwise) and negative when they act in the same sense. In all cases, factor C may be taken conservatively as unity.

10.55 SOLID RIB ARCHES

See Article 3.2 for load factors and combinations. Use Service Load Design Method for factored loads and the formulas changed as follows:

10.55.1 Moment Amplification and Allowable Stresses

$$A_F = \frac{1}{1 - \dfrac{1.18T}{AF_e}} \qquad (10\text{-}159)$$

$$F_a = \frac{F_y}{1.18}\left[1 - \frac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right] \text{ and } F_b = F_y \qquad (10\text{-}160)$$

10.55.2 Web Plates

No longitudinal stiffener

$$D/t_w = \frac{6,750}{\sqrt{f_a}} \qquad (10\text{-}161)$$

One longitudinal stiffener

$$D/t_w = \frac{10,150}{\sqrt{f_a}} \qquad (10\text{-}162)$$

Two longitudinal stiffeners

$$D/t_w = \frac{13,500}{\sqrt{f_a}} \qquad (10\text{-}163)$$

The b'/t_s ratio for the stiffeners shall be

$$\frac{b'}{t_s} = \frac{2,200}{\sqrt{f_a + \dfrac{f_b}{3}}} \quad \text{maximum } \frac{b'}{t_s} = 12 \qquad (10\text{-}164)$$

10.55.3 Flange Plates

$$\frac{b'}{t_f} = \frac{5,700}{\sqrt{f_a + f_b}} \text{ for width between webs} \qquad (10\text{-}165)$$

$$\frac{b'}{t_f} = \frac{2,200}{\sqrt{f_a + f_b}} \text{ for overhang widths, maximum } b'/t_f = 12 \qquad (10\text{-}166)$$

10.56 SPLICES, CONNECTIONS, AND DETAILS

10.56.1 Connectors

10.56.1.1 General

Connectors and connections shall be proportioned so that their design resistance, ϕR, (maximum strength multiplied by a resistance factor) as given in this Article, as applicable, shall be at least equal to the effects of service loads multiplied by their respective load factors as specified in Article 3.22.

10.56.1.2 Welds

The ultimate strength of the weld metal in groove and fillet welds shall be equal to or greater than that of the base metal, except that the designer may use electrode classifications with strengths less than the base metal when detailing fillet welds for quenched and tempered steels. However, the welding procedure and weld metal shall be selected to ensure sound welds. The effective weld area shall be taken as defined in *ANSI/AASHTO/AWS D1.5 Bridge Welding Code,* Article 2.3.

10.56.1.3 Bolts and Rivets

10.56.1.3.1 In proportioning fasteners, the cross sectional area based upon nominal diameter shall be used.

10.56.1.3.2 The design force, ϕR, in kips, for AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) high-strength bolts subject to applied axial tension or shear is given by

$$\phi R = \phi F A_b \qquad (10\text{-}166a)$$

TABLE 10.56A Design Strength of Connectors

Type of Fastener	Strength (ϕF)
Groove Weld[a]	$1.00\ F_y$
Fillet Weld[b]	$0.45\ F_u$
Low-Carbon Steel Bolts	
ASTM A 307	
Tension	30 ksi
Shear on Bolt with Threads in Shear Plane[d]	18 ksi
Power-Driven Rivets	
ASTM A 502	
Shear—Grade 1	25 ksi
Shear—Grade 2	30 ksi
High-Strength Bolts	
AASHTO M 164	
(ASTM A 325)	
Applied Static Tension[c]	68 ksi
Shear on Bolt with Threads in Shear Plane[c,d,e]	35 ksi
AASHTO M 253	
(ASTM A 490)	
Applied Static Tension	85 ksi
Shear on Bolt with Threads in Shear Plane[d,e]	43 ksi

[a] F_y = yield point of connected material.
[b] F_u = minimum strength of the welding rod metal.
[c] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1-inch in diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.
[d] Tabulated values shall be reduced by 20 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 inches. For flange splices, the 50-inch length is to be measured between the extreme bolts on only one side of the connection.
[e] If material thickness or joint details preclude threads in the shear plane, multiply values by 1.25.

where

ϕF = design strength per bolt area as given in Table 10.56A for appropriate kind of load, ksi;
A_b = area of bolt corresponding to nominal diameter, sq in.

The design bearing force, ϕR, on the connected material in standard, oversized, short-slotted holes loaded in any direction, or long-slotted holes parallel to the applied bearing force shall be taken as

$$\phi R = 0.9 L_c t F_u \leq 1.8 d t F_u \qquad (10\text{-}166b)$$

The design bearing force, ϕR, on the connected material in long-slotted holes perpendicular to the applied bearing force shall be taken as

$$\phi R = 0.75 L_c t F_u \leq 1.5 d t F_u \qquad (10\text{-}166c)$$

The design bearing force for the connection is equal to the sum of the design bearing forces for the individual bolts in the connection.

In the foregoing

ϕR = design bearing force, kips.
F_u = specified minimum tensile strength of the connected material, ksi.
L_c = clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force, in.
d = nominal diameter of bolt, in.
t = thickness of connected material, in.

10.56.1.3.3 High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension.

For combined tension and shear, bolts and rivets shall be proportioned so that the tensile stress does not exceed for $f_v/F_v \le 0.33$

$$F_t' = F_t \quad (10\text{-}167)$$

for $f_v/F_v > 0.33$

$$F_t' = F_t\sqrt{1-(f_v/F_v)^2} \quad (10\text{-}167a)$$

where

f_v = computed rivet or bolt stress in shear, ksi;
F_v = design shear strength of rivet or bolt from Table 10.56A, ksi;
F_t = design tensile strength of rivet or bolt from Table 10.56A, ksi;
F_t' = reduced design tensile strength of rivet or bolt due to the applied shear stress, ksi.

10.56.1.4 Slip-Critical Joints

Slip-critical joints shall be designed to prevent slip at the overload in accordance with Article 10.57.3, but as a minimum the bolts shall be capable of developing the minimum strength requirements in shear and bearing of Article 10.56.1.3 under the maximum design loads.

Potential slip of joints should be investigated at intermediate load stages especially those joints located in composite regions.

10.56.2 Bolts Subjected to Prying Action by Connected Parts

Bolts required to support applied load by means of direct tension shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. The total tension should not exceed the values given in Table 10.56A.

The tension due to prying actions shall be computed as

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right]T \quad (10\text{-}168)$$

where

Q = prying tension per bolt (taken as zero when negative);
T = direct tension per bolt due to external load;
a = distance from center of bolt to edge of plate;
b = distance from center of bolt to toe of fillet of connected part;
t = thickness of thinnest part connected in inches.

10.56.3 Rigid Connections

10.56.3.1 All rigid frame connections, the rigidity of which is essential to the continuity assumed as the basis of design, shall be capable of resisting the moments, shears, and axial loads to which they are subjected by maximum loads.

10.56.3.2 The beam web shall equal or exceed the thickness given by

$$t_w \ge \sqrt{3}\left(\frac{M_c}{F_y d_b d_c}\right) \quad (10\text{-}169)$$

where

M_c = column moment;
d_b = beam depth;
d_c = column depth.

When the thickness of the connection web is less than that given by the above formula, the web shall be strengthened by diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area.

At joints where the flanges of one member are rigidly framed into one flange of another member, the thickness of the web, t_w, supporting the latter flange and the thickness of the latter flange, t_c, shall be checked by the formulas below. Stiffeners are required on the web of the second member opposite the compression flange of the first member when

$$t_w < \frac{A_f}{t_b + 5k} \quad (10\text{-}170)$$

and opposite the tension flange of the first member when

$$t_c < 0.4\sqrt{A_f} \quad (10\text{-}171)$$

where

t_w = thickness of web to be stiffened;
k = distance from outer face of flange to toe of web fillet of member to be stiffened;
t_b = thickness of flange delivering concentrated force;
t_c = thickness of flange of member to be stiffened;
A_f = area of flange delivering concentrated load.

10.57 OVERLOAD

For AASHTO H or HS loadings, the overload is defined as D + 5(L+I)/3, except for beams and girders designed

TABLE 10.57A **Design Slip Resistance for Slip-Critical Connections (Slip Resistance per Unit of Bolt Area, $\phi F_s = \phi T_b \mu$, ksi)**

	Hole Type and Direction of Load Application							
	Any Direction				Transverse		Parallel	
	Standard		Oversize and Short Slot		Long Slots		Long Slots	
Contact Surface of Bolted Parts	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)
Class A (Slip Coefficient 0.33) Clean mill scale and blast-cleaned surfaces with Class A coatings[b]	21	26	18	22	15	18	13	16
Class B (Slip Coefficient 0.50) Blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings[b]	32	40	27	34	22	28	19	24
Class C (Slip Coefficient 0.33) Hot-dip galvanized surfaces roughened by wire brushing after galvanizing	21	26	18	22	15	18	13	16

[a] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1-inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.

[b] Coatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.50, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints. See Article 10.32.3.2.3.

for the Group IA load combination specified in Article 3.5.1 for which overload is defined as D + 2.2(L+I) with (L+I) assumed to occupy a single lane without concurrent loading in any other lane. For beams and girders designed for an overload vehicle selected by the operating agency in accordance with the Group IB load combination, the overload is defined as D + (L+I). If moment redistribution is permitted under the provisions of Article 10.48.1.3, the limitations specified in Articles 10.57.1 and 10.57.2 shall apply to the modified moments, but not to the original moments. Web bend-buckling shall be checked for the overload according to Equation (10-173). For composite sections, D_c shall be calculated in accordance with Article 10.50(b). Sections that do not satisfy Equation (10-173) shall be modified to comply with the requirement.

10.57.1 Noncomposite Sections

At noncomposite sections, the maximum overload flange stress shall not exceed $0.8F_y$.

10.57.2 Composite Sections

At composite sections, the maximum overload flange stress shall not exceed $0.95F_y$. In computing dead load stresses, the presence or absence of temporary supports during the construction shall be considered. For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.50.2.3, the overload flange stresses caused by loads acting on the appropriate composite section may be computed assuming the concrete deck to be fully effective for both positive and negative moment. For this case, the resulting stresses shall be combined with the stresses due to loads acting on the noncomposite section to calculate D_c for checking web bend buckling.

10.57.3 Slip-Critical Joints

10.57.3.1 In addition to the requirements of Articles 10.56.1.3.1 and 10.56.1.3.2 for fasteners, the force caused by D + 5(L + I)/3 on a slip-critical joint shall not exceed the design slip force (ϕR_s) given by

$$\phi R_s = \phi F_s A_b N_b N_s \qquad (10\text{-}172a)$$

where

ϕF_s = $\phi T_b \mu$, design slip resistance per unit of bolt area given in Table 10.57A, ksi;

A_b = area corresponding to the nominal body area of the bolt, sq in.;

N_b = number of bolts in the joint;
N_s = number of slip planes;
T_b = specified tension in the bolt;
μ = slip coefficient;
= 0.33 for clean mill scale and Class A coatings
= 0.50 for blast-cleaned surfaces and Class B coatings;
= 0.33 for hot-dip galvanized and roughened surfaces;
ϕ = 1.0 for standard holes;
= 0.85 for oversized and short slotted holes;
= 0.70 for long slotted holes loaded transversely;
= 0.60 for long slotted holes loaded longitudinally.

Class A, B, or C surface conditions of the bolted parts as defined in Table 10.57A shall be used in joints designated as slip-critical except as permitted in Article 10.57.3.2.

10.57.3.2 Subject to the approval of the Engineer, coatings providing a slip coefficient less than 0.33 may be used provided the mean slip coefficient is established by test in accordance with the requirements of Article 10.57.3.3, and the slip resistance per unit area established. The slip resistance per unit area shall be taken as equal to the slip resistance per unit area from Table 10.57A for Class A coatings as appropriate for the hole type and bolt type times the slip coefficient determined by test divided by 0.33.

10.57.3.3 Paint, used on the faying surfaces of connections specified to be slip critical, shall be qualified by test in accordance with "Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints" as adopted by the Research Council on Structural Connections. See Appendix A of *Allowable Stress Design Specification for Structural Joints Using ASTM A 325 or A 490 Bolts*, published by the Research Council on Structural Connections.

10.57.3.4 For combined shear and tension in slip critical joints where applied forces reduce the total clamping force on the friction plane, the design slip force shall not exceed the value $\phi R_s'$ obtained from the following equation:

$$\phi R_s' = \phi R_s (1 - 1.88 f_t / F_u) \qquad (10\text{-}172b)$$

where

f_t = computed tensile stress in the bolt due to applied loads including any stress due to prying action, ksi;
ϕR_s = design slip force specified in Equation (10-172a), kips;
F_u = 120 ksi for M 164 (A 325) bolts up to 1-inch diameter;
= 105 ksi for M 164 (A 325) bolts over 1-inch diameter;
= 150 ksi for M 253 (A 490) bolts.

10.58 FATIGUE

10.58.1 General

The analysis of the probability of fatigue of steel members or connections under service loads and the allowable range of stress for fatigue shall conform to Article 10.3, except that the limitation imposed by the basic criteria given in Article 10.3.1 shall not apply. For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.50.2.3, the range of stress may be computed using the composite section assuming the concrete deck to be fully effective for both positive and negative moment.

10.58.2 Composite Construction

10.58.2.1 Slab Reinforcement

When composite action is provided in the negative moment region, the range of stress in slab reinforcement shall be limited to 20,000 psi.

10.58.2.2 Shear Connectors

The shear connectors shall be designed for fatigue in accordance with Article 10.38.5.1.

10.58.3 Hybrid Beams and Girders

Hybrid girders shall be designed for fatigue in accordance with Article 10.3.

10.59 DEFLECTION

The control of deflection of steel or of composite steel and concrete structures shall conform to the provision of Article 10.6.

10.60 ORTHOTROPIC SUPERSTRUCTURES

A rational analysis based on the Strength Design Method, in accordance with the specifications, will be considered as compliance with the specifications.

10.61 CONSTRUCTIBILITY

The moment and shear capacity of a steel beam or girder shall meet the requirements specified below to control local buckling of the web and compression flange, and to prevent lateral torsional buckling of the cross section under the noncomposite dead load prior to hardening of the deck slab. The casting or placing sequence of the concrete deck specified in the plans shall be considered in determining the applied moments and shears. A load factor of $\gamma = 1.3$ shall be used in calculating the applied moments and shears.

10.61.1 Web Bend Buckling

The maximum factored noncomposite dead load compressive bending stress in the web shall not exceed the value given below:

$$f_b \le \frac{26,200,000\alpha k}{\left(\frac{D}{t_w}\right)^2} \le F_{yw} \qquad (10\text{-}173)$$

where

F_{yw} = specified minimum yield strength of the web
D_c = depth of the web of the steel beam or girder in compression
D = web depth
t_w = thickness of web
k = $9(D/D_c)^2$ for members without a longitudinal stiffener
α = 1.3 for members without a longitudinal stiffener
α = 1.0 for members with a longitudinal stiffener

Sections without longitudinal stiffeners that do not satisfy Equation (10-173) shall either be modified to comply with the requirement or a longitudinal stiffener shall be added to the web at a location on the web that satisfies both Equation (10-173) and all strength requirements, which may or may not correspond to the optimum location of the longitudinal stiffener specified in Article 10.49.3.2(a). For longitudinally stiffened girders, the buckling coefficient, k, is calculated as

$$\text{for } \frac{d_s}{D_c} \ge 0.4 \quad k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2$$

$$\text{for } \frac{d_s}{D_c} < 0.4 \quad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

where

d_s = the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component.

For members with or without a longitudinal stiffener, k shall be taken equal to 7.2 when both edges of the web are in compression.

The web thickness requirements specified in Articles 10.48.5.1, 10.48.6.1, 10.49.2, and 10.49.3.2(b) shall not be applied to the constructibility load case.

10.61.2 Web Shear Buckling

The sum of the factored noncomposite and composite dead-load shears shall not exceed the shear buckling capacity of the web specified in Article 10.48.8.1 (Equation 10-113).

10.61.3 Lateral-Torsional Buckling of the Cross Section

The maximum factored non-composite dead-load moment shall not exceed the value of M_u calculated for the steel beam or girder using the equations specified in Article 10.48.4.1, nor M_y.

10.61.4 Compression Flange Local Buckling

The ratio of the top compression flange width to thickness in positive-moment regions shall not exceed the value determined by the formula

$$\frac{b}{t} = \frac{4,400}{\sqrt{f_{d\ell}}} \le 24 \qquad (10\text{-}174)$$

where $f_{d\ell}$ is the top-flange compressive stress due to the factored noncomposite dead load divided by the factor R_b specified in Article 10.48.4.1, but not to exceed F_y.