y = 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 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 stiffness 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 the specification Articles 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 non-hybrid girders shall be replaced by the following equations for the allowable average shear stress in the web of transversely stiffened hybrid girders:

$$F_v = \frac{C F_y}{3} \le \frac{F_y}{3} \qquad (10\text{-}90)$$

where:

F_y = specified minimum yield strength of the web (psi)

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, 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.

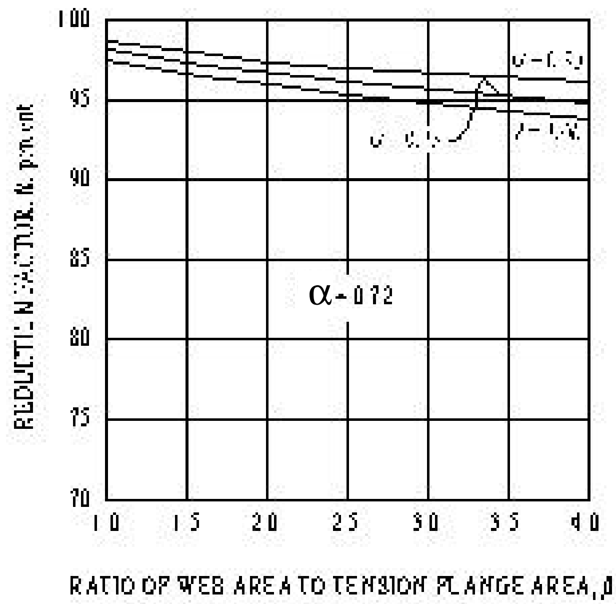

Figure 10.40.2.1A

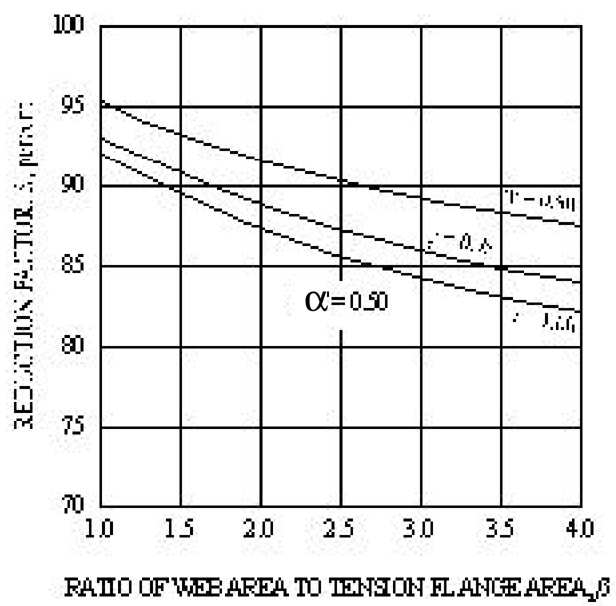

Figure 10.40.2.1B

* Bottom flange of orthotropic deck bridge

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 these Specifications, 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 the preferred 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 (in.)	Length in Direction of Traffic (in.)
8	20 + 2*t*	8 + 2*t*
12	20 + 2*t*	8 + 2*t*
16	24 + 2*t*	8 + 2*t*

In the above table, *t* is the thickness of the wearing surface (in.) |+

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-percent impact shall not exceed 30,000 psi unless a higher allowable stress is justified by a detailed fatigue analysis or 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.

* 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.

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 percent. 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 percent 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:

$$\left(\frac{L}{r}\right)_{\max} = 1{,}000\sqrt{\frac{1{,}500}{F_y} - \frac{2{,}700f}{F_y}} \qquad (10\text{-}91)$$ +

where:

L = distance between transverse beams (in.) +

r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate (in.) +

f = maximum compressive stress 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 (psi) +

F_y = specified minimum yield strength of rib material (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-percent 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 the preferred 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. +

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 – Loads. +

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 3.2.

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 0.85 f'_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.

10.46 DESIGN STRESS FOR STRUCTURAL STEEL

The design stress for structural steel shall be the specified minimum 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 load-carrying member shall be computed from the formulas shown in Article 3.2. 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 2 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 design bending strength shall be computed as:

$$M_u = F_y Z \qquad (10\text{-}92)$$

where:

F_y = specified minimum yield strength of the steel being used (psi)
Z = plastic section modulus* (in.^{3})

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:

$$\frac{b}{t} \le \frac{4{,}110}{\sqrt{F_y}} \qquad (10\text{-}93)$$

where:

b = compression flange width (in.)
t = flange thickness (in.)

(b) Web thickness:

$$\frac{D}{t_w} \le \frac{19{,}230}{\sqrt{F_y}} \qquad (10\text{-}94)$$

where:

D = clear distance between the flanges (in.)
t_w = web thickness (in.)

When both b/t and D/t_w exceed 75% of the above limits, the following interaction equation shall apply

$$\frac{D}{t_w} + 4.68\left(\frac{b}{t}\right) \le \frac{33{,}650}{\sqrt{F_{yf}}} \qquad (10\text{-}95)$$

where:

F_{yf} = specified minimum yield strength of compression flange (psi)

(c) Lateral bracing for compression flange:

$$\frac{L_b}{r_y} \le \frac{\left[3.6 - 2.2\left(M_1 / M_u\right)\right] \times 10^6}{F_y} \qquad (10\text{-}96)$$

where:

L_b = distance between points of bracing of the compression flange(in.)
r_y = radius of gyration of the steel section with respect to the Y-Y axis (in.)
M_1 = smaller moment at the end of the unbraced length of the member (lb-in.)
M_u = design strength 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 (lb-in.). (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:

* Values for rolled sections are listed in the *Manual of Steel Construction*, Ninth Edition, 1989, American Institute of Steel Construction, Appendix D shows the methods of computing Z as presented in the Commentary of AISI Bulletin 15.

$$P \le 0.15 F_y A \quad (10\text{-}97)$$

where:

A = area of the cross section (in.^{2})

Members with axial loads in excess of $0.15F_yA$ should be designed as beam-columns as specified in Article 10.54.2.

10.48.1.2 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 ASTM A 709, Grade HPS 70W meet these requirements. The limitations set forth in Article 10.48.1 are given in Table 10.48.1.2A.

TABLE 10.48.1.2A Limitations for Compact Sections

F_y (psi)	36,000	50,000	70,000
b/t	21.6	18.4	15.0
D/t_w	101	86	70
$L_b/r_y(M_1/M_u = 0^*)$	100	72	51
$L_b/r_y(M_1/M_u = 1^*)$	39	28	20

For values of M_1/M_u other than 0 and 1, use Equation (10-96).

10.48.1.3 In the design of a continuous beam with compact negative-moment support sections of AASHTO M 270 Grade 36, 50 and 50W (ASTM A 709 Grade 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 percent. Such reduction shall be accompanied by an increase in moments throughout adjacent spans statically equivalent and opposite in sign to the decrease of the 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-percent redistribution of moment shall not apply to compact sections of ASTM A 709 Grade HPS 70W or AASHTO M 270, Grade 70W steel.

10.48.2 Braced Non-Compact Sections

For sections of rolled or fabricated flexural members not meeting the requirements of Article 10.48.1.1 but meeting the requirement of Article 10.48.2.1 below, the design strength shall be computed as the lesser of

$$M_u = F_y S_{xt} \quad (10\text{-}98)$$

or

$$M_u = F_{cr} S_{xc} R_b \quad (10\text{-}99)$$

subject to the requirement of Article 10.48.2.1 (c) where

$$F_{cr} = \left(4{,}400\frac{t}{b}\right)^2 \le F_y$$

b = compression flange width (in.)
t = compression flange thickness (in.)
S_{xt} = elastic section modulus with respect to tension flange (in.^{3})
S_{xc} = elastic 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 (psi), but not to exceed F_y

10.48.2.1 The above equations are applicable to sections meeting the following requirements:

(a) Compression flange:

$$\frac{b}{t} \le 24 \quad (10\text{-}100)$$

(b) Web thickness:

The web thickness shall not exceed the requirement given by Equation (10-104) or Equation (10-109), as applicable, subject to the corresponding requirements as shown in Table 10.48.5A of Article 10.49.2 or 10.49.3. For unstiffened web, the web thickness shall not be less than $D/150$.

(c) Spacing of lateral bracing for compression flange:

$$L_b \leq \frac{20{,}000{,}000 A_f}{F_y d} \qquad (10\text{-}101)$$

where:

d = depth of beam or girder (in.)
A_f = flange area (in.^{2})

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 $0.15F_yA$ should be designed as beam-columns as specified in Article 10.54.2.

TABLE 10.48.2.1A Limitations for Braced Non-Compact 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 article 10.48.2.1(a)

10.48.2.2 The limitations set forth in Article 10.48.2.1 above are given Table 10.48.2.1A .

10.48.3 Transitions

The design strength of sections with geometric properties falling between the limits of Articles 10.48.1 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 Sections

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 design bending 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 design bending 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:

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

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

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 (in.)
f_b = factored bending stress in compression flange (psi)

Otherwise, for girders with or without longitudinal stiffeners, R_b shall be calculated as

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_{fc}}\right)\left[\frac{D_c}{t_w} - \frac{l}{\sqrt{\frac{M_r}{S_{xc}}}}\right] \leq 1.0 \qquad \text{(10-103b)}$$

D_c = depth of 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 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 $D/2$

For sections with $\frac{D_c}{t_w} \leq \frac{l}{\sqrt{F_y}}$ or with longitudinally stiffened webs:

$$M_r = 91 \times 10^6 C_b \left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \leq M_y \qquad \text{(10-103c)}$$

For sections with $\frac{l}{\sqrt{F_y}} < \frac{D_c}{t_w}$

for $L_b \leq L_p$

$$M_r = M_y \qquad \text{(10-103d)}$$

$$L_p = \frac{9{,}500\,r'}{\sqrt{F_y}} \qquad \text{(10-103d1)}$$

for $L_r \geq L_b > L_p$

$$M_r = C_b F_y S_{xc}\left[1 - 0.5\left(\frac{L_b - L_p}{L_r - L_b}\right)\right] \leq M_y \qquad \text{(10-103e)}$$

$$L_r = \left(\frac{572 \times 10^6 I_{yc} d}{F_y S_{sc}}\right)^{1/2} \qquad \text{(10-103f)}$$

for $L_b > L_r$

$$M_r = C_b \frac{F_y S_{xc}}{2}\left(\frac{L_r}{L_b}\right)^2 \leq M_y \qquad \text{(10-103g)}$$

where:

L_b = unbraced length of the compression flange (in.)
L_p = limiting unbraced length for the yield moment capacity (in.)
L_r = limiting unbraced length for elastic lateral torsional buckling moment capacity (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 the thickness of the compression and tension flange, respectively (in.^{4})

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_c}$$

where:

M_{max} = absolute value of maximum moment in the unbraced beam segment (lb-in.)
M_A = absolute value of moment at quarter point of the unbraced beam segment (lb-in.)

M_B = absolute value of moment at midpoint of the unbraced beam segment (lb-in.)

M_c = absolute value of moment at three- quarter point of the unbraced segment (lb-in.)

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.[a]

The compression flange shall satisfy the requirement of Article 19.48.2.1(a). The web thickness shall not exceed the requirement given by Equation (10-104) or Equation (10-109), as applicable, subject to the corresponding requirements of Articles 10.49.2 or 10.49.3. For unstiifened web, the web thickness shall not be less than $D/150$.

10.48.4.2 Members with axial loads in excess of 0.15 $F_y A$ 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 width-thickness ratio (D/t_w) of the web shall not exceed the limiting values specified in Table 10.48.5A subject to the web thickness requirement of Article 10.49.2. 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 design 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.

TABLE 10.48.5A Limiting Width-Thickness Ratios for Web Plates and Stiffeners

Description of Component		Width-Thickness Ratio	Limiting Width-Thickness Ratio	F_y (psi)	Limiting Width-Thickness Ratios
Web Plates	With transverse stiffeners only	D/t_w	$\dfrac{36{,}500}{\sqrt{F_y}}$ (10-104)	36,000	192
				50,000	163
				70,000	138
				90,000	122
				100,000	115
	With transverse stiffeners and one longitudinal stiffener		$\dfrac{73{,}000}{\sqrt{F_y}}$ (10-109)	36,000	385
				50,000	326
				70,000	276
				90,000	243
				100,000	231
Stiffeners	Longitudinal stiffeners	b'/t_s	$\dfrac{2{,}600}{\sqrt{F_y}}$ (10-144)		
	Transverse stiffeners		16 (10-105)		

b' = width of flange plate or width of outstanding element of web stiffeners (in.)
D = clear distance between flanges (in.)
F_y = specified minimum yield strength of the component under consideration (psi)
t_s = web stiffener outstanding element thickness (in.)
t_w = web plate thickness (in.)

[a] For the use of larger C_b values, see Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures, 4th Ed., pg. 157.

10.48.5.3 The shear capacity of transversely stiffened girders shall be computed by Article 10.48.8. The width-to-thickness ratio (b'/t_s) of transverse stiffeners shall not exceed the limiting values specified in Table 10.48.5A.

The gross cross-sectional area of intermediate transverse stiffeners, A (in.^{2}) shall meet the following requirement:

$$A \geq \left[0.15B\frac{D}{t_w}(1-C)\left(\frac{V}{V_u}\right)-18\right]\frac{F_{yweb}}{F_{cr}}t_w^2 \quad (10\text{-}106a)$$

where:

$$F_{cr} = \frac{9{,}025{,}000}{\left(\frac{b'}{t_s}\right)^2} \leq F_{ystiffener} \quad (10\text{-}106b)$$

b' = projecting width of the stiffener (in.)
t_s = thickness of the stiffener (in.)
F_{yweb} = specified minimum yield strength of the web (psi)
$F_{ystiffener}$ = specified minimum yield strength of the stiffener (psi)
B = 1.0 for stiffener pairs, 1.8 for single angles, and 2.4 for single plates;
C = constant 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 (10-107), (10-105) and Article 10.34.4.10.

The moment of inertia of transverse stiffeners with reference to the plane defined below shall meet the following requirement:

$$I \geq d_o t_w^3 J \quad (10\text{-}107)$$

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \geq 0.5 \quad (10\text{-}108)$$

d_o = distance between transverse stiffeners

When stiffeners are in pairs, the moment of inertia shall be taken about the centerline 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 intermediate stiffeners shall be preferably fitted tightly to the tension flange. If the intermediate stiffener is used for attaching a cross frame or diaphragm, a positive connection using either bolts or welds must be made to the tension flange. The distance between the end of the vertical weld on the stiffener to the web-to-flange weld shall be $4t_w$ but not less than $1\,^1/_2$ inches. Transverse stiffeners provided only on one side of the web must be welded to the compression flange and fitted tightly to the tension flange for the stiffener to be effective. Stiffener properties shall be as covered in Article 10.34.4.10.

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 width-thickness ratio (D/t_w) of the web of plate girders with transverse stiffeners and one longitudinal stiffener shall not exceed the limiting values specified in Table 10.48.5A.

Singly symmetric girders are subject to the requirements of Article 10.49.3

10.48.6.2 The design bending strength of longitudinal 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 width-to-thickness ratio meets the requirement given in Table 10.48.5A. The factored bending stress in the longitudinal stiffener is not greater than the yield strength of the longitudinal stiffener.

(b) the moment of inertia of the stiffener meets the following requirement:

$$I \geq D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \qquad (10\text{-}110)$$

(c) the radius of gyration of the stiffener meets the following requirement:

$$r \geq \frac{d_o \sqrt{F_y}}{23{,}000} \qquad (10\text{-}111)$$

In computing I and r values 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 meet the following requirement:

$$S_s \geq \frac{1}{3} \left(\frac{D}{d_o} \right) S_t \qquad (10\text{-}112)$$

where:

D = total panel depth (clear distance between flange components) (in.)

S_t = section modulus of the longitudinal stiffener. (in^3.)

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.

Axial compression strength shall be computed as specified in Article 10.54.1. Bearing strength shall be taken at 1.35 F_y times the bearing area of bearing stiffners.

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 design shear strength shall be limited to the plastic or buckling shear strength as follows:

$$V_u = C V_p \qquad (10\text{-}113)$$

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:

$$V_u = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1 + \left(\frac{d_o}{D} \right)^2}} \right] \qquad (10\text{-}114)$$

V_p is equal to the plastic shear strength and is determined as follows:

$$V_p = 0.58\, F_y D t_w \qquad (10\text{-}115)$$

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

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

$$C = 1.0$$

for $\dfrac{6{,}000\sqrt{k}}{\sqrt{F_y}} \leq \dfrac{D}{t_w} \leq \dfrac{7{,}500\sqrt{k}}{\sqrt{F_y}}$

$$C = \frac{6{,}000\sqrt{k}}{\left(\frac{D}{t_w} \right) \sqrt{F_y}} \qquad (10\text{-}116)$$

for $\dfrac{D}{t_w} > \dfrac{7{,}500\sqrt{k}}{\sqrt{F_y}}$

$$C = \frac{4.5 \times 10^7\, k}{\left(\frac{D}{t_w} \right)^2 F_y} \qquad (10\text{-}117)$$

where the buckling coefficient, $k = 5 + 5 / (d_o/D)^2$, except k shall be taken as 5 for unstiffened beams and girders.

D = clear, unsupported distance between flange components (in.)
d_o = distance between transverse stiffeners (in.)
F_y = specified minimum yield strength of the web plate (psi)

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 0.75 M_u, the shear shall meet the following requirement:

$$\frac{V}{V_u} \leq 2.2 - \left(\frac{1.6\,M}{M_u}\right) \qquad (10\text{-}118)$$

If a girder panel adjacent to 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 the factored bending stress f_s greater than $0.75F_u$, the shear shall meet the following requirement:

$$\frac{V}{V_u} \leq 2.2 - \left(\frac{1.6 f_s}{F_{um}}\right) \qquad (10\text{-}118a)$$

where:

f_s = factored bending stress in either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_{um})
F_{um} = maximum bending strength of either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_{um})

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 requirements 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 $D\left[\frac{260}{D/t_w}\right]^2$.

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 strength 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.

10.49 FLEXURAL MEMBERS WITH 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 Transversely Stiffened Sections

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:

$$\frac{D_c}{t_w} \leq \frac{18{,}250}{\sqrt{F_y}} \qquad (10\text{-}120)$$

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 Sections

10.49.3.1 Longitudinal stiffeners shall be required on singly symmetric sections when the web thickness does not meet the requirement specified by Articles 10.48.5.1 or 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:

$$\frac{d_s}{D_{cs}} = \frac{1}{1+1.5\sqrt{\frac{f_{DL+LL}}{f_{DL}}}} \tag{10-121}$$

where:

D_{cs} = depth of the web in compression of the non-composite steel beam or girder (in.)

f_{DL} = non-composite dead-load stress in the compression flange (psi)

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

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:

$$\frac{D_c}{t_w} \leq \frac{36{,}500}{\sqrt{F_y}} \tag{10-122}$$

10.49.4 Braced Non-Compact Sections

Singly symmetric braced non-compact 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 Sections

The design 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 design 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 section 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 full plastic 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) steel beam or girder shall satisfy the constructibility requirements of Article 10.61.

(e) The stress in the top flange of a composite girder shall be limited to $0.6F_y$ under dead load if no calculations are made for the construction loading stage of the concrete deck. The concrete deck is assumed to be placed instantaneously.

10.50.1 Positive Moment Sections

10.50.1.1 Compact Sections

The design 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. Steel such as AASHTO M 270 Grades 26, 50 and 50W (ASTM A 709 Grades 36, 50 and 50W), and ASTM A 709 Grade HPS 70W meet these requirements.

10.50.1.1.1 The resultant moment of the fully plastic stress distribution (Figure 10.50A) 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:

(1) $$C = 0.85 f'_c b t_s + (AF_y)_c \qquad (10\text{-}123)$$

(2) $$C = (AF_y)_{bf} + (AF_y)_{tf} + (AF_y)_w \qquad (10\text{-}124)$$

where:

b = effective width of slab specified in Article 38.3 (in.)

t_s = the slab thickness (in.)

$(AF_y)_c$ = product of the area and yield strength of that part of reinforcement which lies in the compression zone of the slab (lb.)

$(AF_y)_{bf}$ = product of area and yield strength for bottom flange of steel section (including cover plate if any) (lb.)

$(AF_y)_{tf}$ = product of area and yield strength for top flange of steel section (lb.)

$(AF_y)_w$ = product of area and yield strength for web of steel section (lb.)

(b) The depth of the stress block is computed from the compressive force in the slab.

$$a = \frac{C - (AF_y)_c}{0.85 f'_c b} \qquad (10\text{-}125)$$

(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 following compressive force:

$$C' = \frac{\sum (AF_y) - C}{2} \qquad (10\text{-}126)$$

where:

(AF_y) = product of the area and yield strength of steel girder section (lb.)

(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 plastic moment capacity, M_p 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.

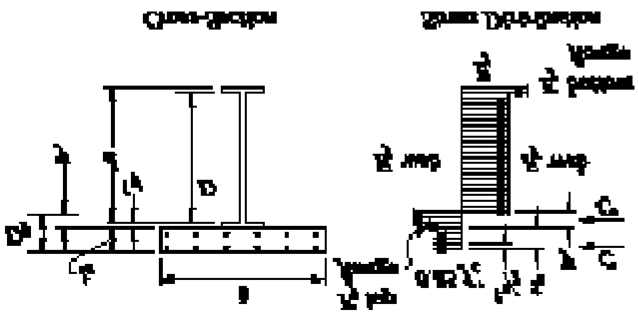

FIGURE 10.50A Plastic Stress Distribution

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:

$$\frac{2D_{cp}}{t_w} \le \frac{19{,}230}{\sqrt{F_y}} \qquad (10\text{-}129)$$

where:

D_{cp} = depth of the web in compression at the plastic moment calculated in accordance with Article 10.50.1.1.1 (in.)
t_w = web thickness (in.)

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 slab to the neutral axis at the plastic moment, D_p, shall satisfy:

$$\left(\frac{D_p}{D'}\right) \le 5 \qquad (10\text{-}129a)$$

where:

$$D' = b\,\frac{(d + t_s + t_h)}{7.5}$$

b = 0.9 for F_y = 36,000 psi
= 0.7 for F_y = 50,000 psi and 70,000 psi
d = depth of the steel beam or girder (in.)
t_s = thickness of the slab (in.)
t_h = thickness of the concrete haunch above the beam or girder top flange (in.)

Equation (10-129a) need not be checked for sections where the maximum flange stress does not exceed the specified minimum flange yield strength.

The design bending strength, M_u of compact composite sections in simple spans or in the positive-moment regions of continuous spans with compact non-composite 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 (lb-in.)
M_y = moment capacity at first yield of the composite moment section calculated as F_y times the section modulus with respect to the tension flange (lb-in.). 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 the 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 at first yield of the compact positive moment section calculated in accordance with Article 10.50(c) (lb-in.)

+ $(M_u - M_s)_{pier}$ = moment capacity of 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 (lb-in.)

+ 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 design bending strength shall be determined as specified in Article 10.50.1.2.

10.50.1.2 Non-Compact 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 bending stresses due to the appropriate loadings acting on the respective cross sections supporting the loadings shall not exceed F_y for the tension flange, and F_yR_b for compression flange, 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 (psi), 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 $1.30M_{sD}$ acting on the steel girder alone with $1.30M_{cD}$ and the appropriate factored live loading according to Table 3.22.1A acting on the composite girder shall not exceed yield strength at any point, where M_{sD} and M_{cD} 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 percent of its required 28-day strength, stresses produced by the factored dead load plus live loading, acting on the composite girder, shall not exceed yield strength at any point. +

10.50.2 Negative Moment Sections

The design strength of composite sections in negative moment regions shall be computed in accordance with Articles 10.50.2.1 or 10.50.2.2, as applicable. It shall be assumed that the concrete slab does not carry tensile forces. 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 without longitudinal web stiffeners and without holes in the tension flange in negative bending 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 Grade 26, 50 and 50W (ASTM A 709, Grade 36, 50 and 50W), and ASTM A 709 Grade HPS 70W 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 slab reinforcement. +

If the distance from the neutral axis to the compression flange exceeds $D/2$, the compact section requirements given by Equation (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 Non-Compact Sections

When the steel section does not satisfy the compactness requirements of Article 10.50.2.1, but does satisfy the requirement of Article 10.48.2.1, the sum of bending stresses due to the appropriate loadings acting on the respective cross sections supporting the loadings shall not exceed F_y for the tension flange, and $F_{cr}R_b$ for compression flange, 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 (psi), but not to exceed F_y. When all requirements of Article 10.48.2.1 are satisfied, except for lateral bracing requirement given by Equation (10-101), the design strength of the compression flange shall be taken to be $F_{cr}R_b$ 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.

10.50.2.3 The minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1 percent 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 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 the 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 of the 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 percent 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 Design Bending Strength

The design bending strength of box girders shall be determined according to the applicable provisions of Articles 10.48, 10.49, and 10.50. In addition, the design strength of the negative moment sections shall be limited by:

$$M_u = F_{cr}S \qquad (10\text{-}130)$$

where:

F_{cr} = critical stress of the bottom flange plate as given in Article 10.51.5 (psi)

10.51.2 Live Load Moment 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:

$$V_w = \frac{V}{\cos q} \qquad (10\text{-}131)$$

where:

V = one-half of the total vertical shear force on one box girder (lb.)
q = 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.

* For information regarding the design of long-span steel box girder bridge, Report No. FHWA-TS-80-205, "Proposed Design Specifications for Steel Box Girder Bridges" is available from the Federal Highway Administration.

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 For unstiffened compression flanges, the critical stress, F_{cr} , shall be computed as:

for
$$\frac{b}{t} \le \frac{6{,}140}{\sqrt{F_y}} \qquad \text{(10-132)}$$

$$F_{cr} = F_y \qquad \text{(10-132a)}$$

for
$$\frac{6{,}140}{\sqrt{F_y}} < \frac{b}{t} \le \frac{13{,}300}{\sqrt{F_y}} \qquad \text{(10-133)}$$

$$F_{cr} = 0.592\,F_y\left(1 + 0.687\sin\frac{cp}{2}\right) \qquad \text{(10-134)}$$

$$c = \frac{13{,}300 - \frac{b}{t}\sqrt{F_y}}{7{,}160} \qquad \text{(10-135)}$$

for
$$\frac{b}{t} > \frac{13{,}300}{\sqrt{F_y}} \qquad \text{(10-136)}$$

$$F_{cr} = 105\left(\frac{t}{b}\right)^2 \times 10^6 \qquad \text{(10-137)}$$

where:

b = flange width between webs (in.)
t = flange thickness (in.)

10.51.5.2 *Deleted*

10.51.5.3 *Deleted*

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 meet the following requirement:

$$I_s \ge f\,t^3 w \qquad \text{(10-138)}$$

where:

f = 0.07 $k^3 n^4$ when n equals 2, 3, 4, or 5;
f = 0.125 k^3 when n equals 1;
w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener (in.)
n = number of longitudinal stiffeners;
k = buckling coefficient which shall not exceed 4.

10.51.5.4.1 For longitudinally stiffened flanges, the critical stress, F_{cr} , shall be computed as:

for
$$\frac{w}{t} \le \frac{3{,}070\sqrt{k}}{\sqrt{F_y}} \qquad \text{(10-139)}$$

$$F_{cr} = F_y \qquad \text{(10-139a)}$$

for
$$\frac{3{,}070\sqrt{k}}{\sqrt{F_y}} < \frac{w}{t} \le \frac{6{,}650\sqrt{k}}{\sqrt{F_y}} \qquad \text{(10-140)}$$

$$F_{cr} = 0.592\,F_y\left(1 + 0.687\sin\frac{cp}{2}\right) \qquad \text{(10-141)}$$

$$c = \frac{6{,}650\sqrt{k} - \frac{w}{t}\sqrt{F_y}}{3{,}580} \qquad \text{(10-141a)}$$

+ for $$\frac{b}{t} > \frac{6{,}650\sqrt{k}}{\sqrt{F_y}} \qquad \text{(10-142)}$$

+ $$F_{cr} = 26.2k\left(\frac{t}{w}\right)^2 \times 10^6 \qquad \text{(10-143)}$$

+ *10.51.5.4.2 Deleted*

+ *10.51.5.4.3 Deleted*

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 shall be considered.

10.51.5.5 The width-to-thickness ratio of any outstanding element of the flange stiffeners shall not exceed the limiting values specified in Table 10.48.5A. +

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 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 Number 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 non-composite 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 Article 10.48 through 10.52 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 provisions specified in Article 10.40.4 shall also apply. Longitudinal web stiffeners preferably shall not be located in yielded portion of the web.

10.53.1 Non-Composite 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:

$$M_u = F_{yf} Z \qquad (10\text{-}145)$$

where:

F_{yf} = specified minimum yield strength of the flange (psi)
Z = plastic section modulus (in.³)

In computing Z, the web thickness shall be multiplied by the ratio of the specified minimum yield strength of the web, F_{yw}, to the specified minimum yield strength of the flange F_{yf}.

10.53.1.2 Braced Non-Compact Sections

The equations of Article 10.48.2 for the design bending strength of braced non-compact sections shall be replaced by the expressions

$$M_u = F_{yf} S_{xt} R \qquad (10\text{-}146)$$

$$M_u = F_{cr} S_{xc} R_b R \qquad (10\text{-}146a)$$

where:

S_{xt} = section modulus with respect to the tension flange (in.³)

For symmetrical sections,

$$R = \frac{12 + b\left(3a - a^3\right)}{12 + 2b} \qquad (10\text{-}147)$$

where:

$$a = \frac{F_{yw}}{F_{yf}}$$

$$b = \frac{A_w}{A_f}$$

for unsymmetrical sections,

$$R = 1 - \frac{b\,y\,(1-a)^2\,(3 - y + a\,y)}{6 + b\,y\,(3 - y)} \qquad (10\text{-}148)$$

where:

y = distance from the outer fiber of the tension flange to the neutral axis divided by depth of the steel section

R shall be taken 1.0 at sections where the bending stress in both flanges caused by factored loads does not exceed the specified minimum yield strength of the web.

10.53.1.3 Partially Braced Sections

The design bending strength of non-compact hybrid sections of partially braced members not satisfying the lateral bracing requirement given by Equation (10-101) shall be calculated as the lesser of the 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

$$M_u = M_r\, R_b\, R \qquad (10\text{-}148a)$$

and the yield moment calculated as

$$M_y = F_{yf}\, S\, R \qquad (10\text{-}148b)$$

where:

R = Reduction factor for hybrid girders determined from Article 10.53.1.2
R_b = bending reduction factor determined by Equation (10-103)

10.53.1.4 Transversely Stiffened Girders

Equation (10-114) of Article 10.48.8.1 for the shear capacity of transversely stiffened girders shall be replaced by the expression

$$V_u = V_p C \qquad (10\text{-}149)$$

The provisions of Article 10.48.8.2 and equation for A in Article 10.48.5.3 are not applicable to hybrid girders.

10.53.2 Composite Sections

The design strength of a compact composite section shall be computed as specified in Article 10.50.1.1.2 or Article10.50.2.1, 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 singly symmetrical sections) from Article 10.53.1.2, where y is calculated as specified below for non-compact composite sections.

The design bending strength of a non-compact composite section shall be taken as the design strength computed from Article10.50.1.2 or Article 10.50.2.2, as applicable times R (for unsymmetrical sections) from Article 10.53.1.2, in which y 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.

10.54 COMPRESSION MEMBERS

10.54.1 Axial Loading

10.54.1.1 Design Axial Strength

The design axial compression strength of concentrically loaded columns shall be computed as:

$$P_u = 0.85\, A_s\, F_{cr} \qquad (10\text{-}150)$$

where:

A_s = gross effective area of the column cross section (in.^{2})
F_{cr} = critical stress determined by one of the following two formulas* (psi)

for
$$\frac{KL_c}{r} \le \sqrt{\frac{2p^2 E}{F_y}} \qquad (10\text{-}151)$$

$$F_{cr} = F_y \left[1 - \frac{F_y}{4p^2 E}\left(\frac{KL_c}{r}\right)^2\right] \qquad (10\text{-}152)$$

for
$$\frac{KL_c}{r} > \sqrt{\frac{2p^2 E}{F_y}} \qquad (10\text{-}153)$$

$$F_{cr} = \frac{p^2 E}{\left(\dfrac{KL_c}{r}\right)^2} \qquad (10\text{-}154)$$

where:

K = effective length factor in the plane of buckling;
L_c = length of the member between points of support (in.)
r = radius of gyration in the plane of buckling (in.)
F_y = yield stress of the steel (psi)
E = 29,000,000 (psi)

*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 end:

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

The combined maximum axial force P and the maximum bending moment M acting on a beam-column shall satisfy the following equations:

$$\frac{P}{0.85 A_s F_{cr}} + \frac{M_x C_{mx}}{M_{ux}\left(1-\frac{P}{A_s F_{ex}}\right)} + \frac{M_y C_{my}}{M_{uy}\left(1-\frac{P}{A_s F_{ey}}\right)} \le 1.0 \qquad (10\text{-}155)$$

$$\frac{P}{0.85 A_s F_y} + \frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \le 1.0 \qquad (10\text{-}156)$$

where:

F_{cr} = critical stress as determined by the equations of Article 10.54.1.1(psi)
M_u = design bending strength as determined by Articles 10.48.1, 10.48.2, or 10.48.4;
F_e = $\dfrac{E\pi^2}{\left(\frac{KL_c}{r}\right)^2}$ the Euler buckling stress in the plane of bending; (10-157)
C_{mx}, C_{my} = coefficients applied to bending terms in interaction formula for prismatic members; dependent upon member curvature caused by applied moments about the x-axis and y- axis, respectively, as determined from Table 10.36A
M_p = F_yZ, the full plastic moment of the section (lb-in.)
Z = plastic section modulus (in.^{3})
$\frac{KL_c}{r}$ = effective slenderness ratio in the plane of bending;
x = subscript; represents the x axis
y = subscript; represents the y axis

10.54.2.1 *Deleted*

10.54.2.2 *Deleted*

10.55 SOLID RIB ARCHES

10.55.1 Moment Amplification and Allowable Stresses

$$A_F = \frac{1}{1-\frac{1.18\,T}{AF_e}} \qquad (10\text{-}159)$$

$$F_a = \frac{F_y}{1.18}\left[1-\frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right] \text{ and } F_b = F_y \qquad (10\text{-}160)$$

10.55.2 Web Plates

10.55.2.1 The width-thickness ratio (D/t_w) of the web plates shall not exceed the limiting values specified in Table 10.55.2A.

* B.G. Johnson, *Guide to Stability Design Criteria for Metal Structures*, John Wiley and Sons, Inc., New York, 1976.

TABLE 10.55.2A Limiting Width-Thickness Ratios for Solid Rib Arches

Description of Component		Width-Thickness ratio	Limiting Width-Thickness Ratio
Web Plates	Without longitudinal stiffeners	D/t_w	$\dfrac{6{,}750}{\sqrt{f_a}}$ (10-161)
	With two longitudinal stiffeners at the one-third points of the web depth		$\dfrac{10{,}150}{\sqrt{f_a}}$ (10-162)
			$\dfrac{13{,}500}{\sqrt{f_a}}$ (10-163)
	Outstanding element of stiffeners	b'/t_s	$\dfrac{2{,}200}{\sqrt{f_a + f_b/3}} \leq 12$ (10-164)
Flange Plates	Plates between webs	b'/t_f	$\dfrac{5{,}700}{\sqrt{f_a + f_b}}$ (10-165)
	Overhang plates		$\dfrac{2{,}200}{\sqrt{f_a + f_b}} \leq 12$ (10-166)

b' = width of flange plate or width of outstanding element of web stiffeners (in.)
D = clear distance between flanges (in.)
f_a = calculated axial compressive stress in the component under consideration (psi)
f_b = calculated compressive bending stress in the component under consideration (psi)
t_f = flange plate thickness (in.)
t_s = web stiffener outstanding element thickness (in.)
t_w = web plate thickness (in.)

10.55.2.2 The width-thickness ratio (b'/t_s) of any outstanding element of the web stiffeners shall not exceed the limiting values specified in Table 10.55.2A.

10.55.3 Flange Plates

The width-thickness ratio (b'/t_f) of flange plates shall not exceed the limiting values specified in Table 10.55.2.A.

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 strength 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 design 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 and the Standard Specifications of the California Department of Transportation.

The design strength of the weld metal, F (psi), shall be taken as:

Groove Welds:

$$F = 1.00\,F_y \qquad \text{(10-166a)}$$

Fillet Welds:

$$F = 0.45\,F_u \qquad \text{(10-166b)}$$

where:

F_y = specified minimum yield strength of connected material (psi)
F_u = specified minimum tensile strength of the welding rod but not greater than the tensile stress of the connected parts (psi)

10.56.1.3 Fasteners

10.56.1.3.1 In proportioning fasteners (rivets, bolts, threaded studs and threaded rods), the cross sectional area based upon nominal diameter shall be used.

10.56.1.3.2 Design Shear Strength

The design shear strength of a fastener, R_v (lb.), shall be taken as:

For fasteners in joints whose length between extreme fasteners measured parallel to the line of action of the force is less than 50 inches:

$$R_v = A_b F_v N_s \qquad \text{(10-166c)}$$

where:

A_b = area of the fastener corresponding to the nominal diameter (in.^{2})
F_v = design shear strength of fastener material specified in Table 10.56A (psi)
N_s = number of shear planes of the fastener

The design shear strength of a fastener in connections greater than 50 inches shall be taken as 0.8 times the value given by Equation (10-166c).

10.56.1.3.3 Design Tension Strength

The design tension strength of a fastener, R_t (lb.), shall be taken as:

$$R_t = A_b F_t \qquad \text{(10-166d)}$$

where:

F_t = design tension strength of fastener material specified in Table 10.56A (psi)

TABLE 10.56A Design Strength of and Fastener Materials

Type of Fasteners	Design Shear Strength $F_v = fF_{nv}$ (psi)		Design Tension Strength $F_t = fF_{nt}$ (psi)	
	f	F_{nv} (psi)	f	F_{nt} (psi)
Rivets	0.65	$0.58F_u$	0.65	F_u
Bolts Threads are excluded from shear plane	0.78	$0.48F_u$	0.65 (for A307, A36, A588, A572) 0.75 (for others)	$0.75F_u$
Bolts Threads are included in shear plane		$0.38F_u$		

F_u = specified minimum tensile strength of the fastener given in Table 10.2C (psi)

10.56.1.3.4 The design bearing strength, R (lb.), 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:

$$R = 0.9 L_c t F_u \leq 1.8 d\, t F_u \tag{10-166e}$$

The design bearing strength, R (lb.), on the connected material in long-slotted holes perpendicular to the applied bearing force shall be taken as:

$$R = 0.75 L_c t F_u \leq 1.5 d\, t F_u \tag{10-166f}$$

The design bearing strength for the connection is equal to the sum of the design bearing force strength for the individual bolts in the connection.

where:

- R = design bearing strength (lb.)
- F_u = specified minimum tensile strength of the connected part (psi)
- 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.)

10.56.1.3.5 High-strength bolts preferably shall be used for fasteners subject 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 $f_v / F_v \leq 0.33$

$$F_t' = F_t \tag{10-167}$$

for $f_v / F_v > 0.33$

$$F_t' = F_t \sqrt{1 - \left(f_v / F_v\right)^2} \tag{10-167a}$$

where:

- f_v = calculated rivet or bolt stress in shear (psi)
- F_v = design shear strength of rivet or bolt from Table 10.56A or equal to $K_h m T_b$ as specified in Article 10.57.3.1 (psi)
- F_t = design tensile strength of rivet or bolt from Table 10.56A (psi)
- F_t' = reduced design tensile strength of rivet or bolt due to the applied shear stress (psi)

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 bearing of Articles 10.18 and 10.19.

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 carry 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:

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right]T \qquad (10\text{-}168)$$

where:

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

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 thickness of beam web shall meet the following requirement:

$$t_w \geq \sqrt{3}\left(\frac{M_c}{F_y d_b d_c}\right) \qquad (10\text{-}169)$$

where:

M_c = column moment (lb-in.)
d_b = beam depth (in.)
d_c = column depth (in.)

When the thickness of the connection web does not satisfy the above requirement, 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

$$t_w < \frac{A_f}{t_b + 5k} \qquad (10\text{-}170)$$

and opposite the tension flange of the first member when

$$t_c < 0.4\sqrt{A_f} \qquad (10\text{-}171)$$

where:

t_w = thickness of web to be stiffened (in.)
k = distance from outer face of flange to toe of web fillet of member to be stiffened (in.)
t_b = thickness of flange delivering concentrated force (in.)
t_c = thickness of flange of member to be stiffened (in.)
A_f = area of flange delivering concentrated load (in.^{2})

10.57 OVERLOAD

The overload is defined as Group 1 loading divided by 1.3. If moment distribution 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 bending-buckling shall be checked at 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 Non-Composite Sections

At non-composite 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 non-composite 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 the overload on a slip-critical joint shall not exceed the design slip strength, R_s (lb.), given by:

$$R_s = K_h \, m \, T_b A_n N_b N_s \qquad (10\text{-}172)$$

where:

A_n = net cross section area of the bolt (in.^{2})
N_b = number of bolts in the joint
N_s = number of slip planes
T_b = required minimum bolt tension stress specified in the Standard Specifications of California Department of Transportation or equal to 70% of specified minimum tensile strength of bolts given in Table 10.2C (psi)
m = slip coefficient specified in Table10.57A
K_h = hole size factor specified in Table 10.57B

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.

High strength bolts done according to the Standard Specifications of the California Department of Transportation, Section 55, will be tensioned and the contact surface condition of the assembly will be Class B.

TABLE 10.57A Slip Coefficient *m*

Class Types	Contact Surface of Bolted Parts	*m*
Class A	Clean mill scale and blast-cleaned surfaces with Class A coating	0.33
Class B	Blast-cleaned surfaces and blast-cleaned surfaces with Class B coating	0.5
Class C	Hot-dip galvanized surfaces roughened by wire brushing after galvanizing	0.33

Note: Coatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.5, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in the Bolted Joints. See Article 10.32.3.2.3.

TABLE 10.57B Hole Size Factor Slip K_h

Hole Types	K_h
Standard	1.0
Oversize and Short-slotted	0.85
Long-slotted holes with the slot perpendicular to the direction of the force	0.70
Long-slotted holes with the slot parallel to the direction of the force	0.60

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 slip force shall not exceed the design slip strength given by:

$$R_s' = R_s\left(1 - 1.88\, f_t / F_u\right) \qquad \text{(10-172b)}$$

where:

f_t = calculated tensile stress in the bolt due to applied loads including any stress due to prying actions (psi)
R_s = design slip strength specified in Equation (10-172) (lb.)
F_u = specified minimum tensile strength of the bolt from Table 10.2C (psi)

10.58 FATIGUE

10.58.1 General

The analysis of the probability of fatigue of steel members or connection 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 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 non-composite dead load prior to hardening of the deck slab. The casting or placing sequence of the concrete deck specified in plans shall be considered in determining the applied moments and shears. A load factor g = 1.3 shall be used in calculating the applied moments and shears.

10.61.1 Web

The maximum factored non-composite dead load compressive bending stress in the web shall not exceed the allowable design bending stress given below:

$$F_b = \frac{26,200,000\,a\,k}{\left(\frac{D}{t_w}\right)^2} \le F_{yw} \qquad (10\text{-}173)$$

where:

F_{yw} = specified minimum yield strength of the web (psi)
D_c = depth of the web of the steel beam or girder in compression (in.)
D = web depth (in.)
t_w = thickness of web (in.)
k = $9(D/D_c)^2$ for members without a longitudinal stiffener
a = 1.3 for members without a longitudinal stiffener
a = 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 stiffeners specified in Article 10.49.3.2(a). For longitudinally stiffened girders, the buckling coefficient, k is calculated as

for

$$\frac{d_s}{D_c} \ge 0.4 \qquad k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2$$

for

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

where:

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 (in.)

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.

The sum of the factored non-composite and composite dead load shears shall not exceed design shear strength of the web specified in Article 10.48.8.1 (Equation 10-113).

10.61.2 *Deleted*

10.61.3 Cross Section

The maximum factored non-composite dead-load moment shall not exceed the value of M_u calculated for the steel beams or girder using the equations specified in Article 10.48.4.1.

10.61.4 Compression Flange

The ratio of the compression flange width to thickness in positive-moment regions shall meet the following requirement:

$$\frac{b}{t} \le \frac{4,400}{\sqrt{f_{dl}}} \le 24 \qquad (10\text{-}174)$$

where:

f_{dl} = top-flange compressive stress due to the factored non-composite dead load divided by the factor R_b specified in Article 10.48.4.1, but not to exceed F_y. (psi)