Section 3
LOADS

Part A
TYPES OF LOADS

3.1 NOTATIONS

A = maximum expected acceleration of bedrock at the site
a = length of short span of slab (Article 3.24.6)
B = buoyancy (Article 3.22)
b = width of pier or diameter of pile (Article 3.18.2.2.4)
b = length of long span of slab (Article 3.24.6)
C = combined response coefficient
C = stiffness parameter = K(W/L) (Article 3.23.4.3)
C = centrifugal force in percent of live load (Article 3.10.1)
CF = centrifugal force (Article 3.22)
C_n = coefficient for nose inclination (Article 3.18.2.2.1)
C_M = steel bending stress coefficient (Article 3.25.1.5)
C_R = steel shear stress coefficient (Article 3.25.1.5)
D = parameter used in determination of load fraction of wheel load (Article 3.23.4.3)
D = degree of curve (Article 3.10.1)
D = dead load (Article 3.22)
D.F. = fraction of wheel load applied to beam (Article 3.28.1)
DL = contributing dead load
E = width of slab over which a wheel load is distributed (Article 3.24.3)
E = earth pressure (Article 3.22)
EQ = equivalent static horizontal force applied at the center of gravity of the structure
E_c = modulus of elasticity of concrete (Article 3.26.3)
E_s = modulus of elasticity of steel (Article 3.26.3)
E_w = modulus of elasticity of wood (Article 3.26.3)
F = horizontal ice force on pier (Article 3.18.2.2.1)
F = framing factor (Article 3.21.1.1)
F_b = allowable bending stress (Article 3.25.1.3)
F_v = allowable shear stress (Article 3.25.1.3)
g = 32.2 ft./sec.^{2}
I = impact fraction (Article 3.8.2)
I = gross flexural moment of inertia of the precast member (Article 3.23.4.3)
ICE = ice pressure (Article 3.22)
J = gross Saint-Venant torsional constant of the precast member (Article 3.23.4.3)
K = stream flow force constant (Article 3.18.1)
K = stiffness constant (Article 3.23.4)
K = wheel load distribution constant for timber flooring (Article 3.25.1.3)
k = live load distribution constant for spread box girders (Article 3.28.1)
L = loaded length of span (Article 3.8.2)
L = loaded length of sidewalk (Article 3.14.1.1)

$\frac{lb}{ft^3} \times 16 = kg/m^3$

$\frac{lb}{ft^2} \times 5 = kg/m^2$

$\frac{lb}{ft} \times 1.5 = kg/m$

$lb \times 0.4536 = kg$

$ft \times 0.3048 = m$

$\frac{lb}{in^2} \times 0.0703 = kg/cm^2$

$\frac{k}{in^2} \times 70.3 = kg/cm^2$

L = live load (Article 3.22)
L = span length (Article 3.23.4)
LF = longitudinal force from live load (Article 3.22)
M_D = moment capacity of dowel (Article 3.25.1.4)
M_x = primary bending moment (Article 3.25.1.3)
M_y = total transferred secondary moment (Article 3.25.1.4)
N_B = number of beams (Article 3.28.1)
N_L = number of traffic lanes (Article 3.23.4)
n = number of dowels (Article 3.25.1.4)
P = live load on sidewalk (Article 3.14.1.1)
P = stream flow pressure (Article 3.18.1)
P = total uniform force required to cause unit horizontal deflection of whole structure
P = load on one rear wheel of truck (Article 3.24.3)
P = wheel load (Article 3.24.5)
P = design wheel load (Article 3.25.1.3)
P_{15} = 12,000 pounds (Article 3.24.3)
P_{20} = 16,000 pounds (Article 3.24.3)
p = effective ice strength (Article 3.18.2.2.1)
p = proportion of load carried by short span (Article 3.24.6.1)
R = radius of curve (Article 3.10.1)
R = normalized rock response
R = rib shortening (Article 3.22)
R_D = shear capacity of dowel (Article 3.25.1.4)
R_x = primary shear (Article 3.25.1.3)
R_y = total secondary shear transferred (Article 3.25.1.4)
S = design speed (Article 3.10.1)
S = soil amplification spectral ratio
S = shrinkage (Article 3.22)
S = average stringer spacing (Article 3.23.2.3.1)
S = spacing of beams (Article 3.23.3)
S = width of precast member (Article 3.23.4.3)
S = effective span length (Article 3.24.1)
S = span length (Article 3.24.8.2)
S = beam spacing (Article 3.28.1)
s = effective deck span (Article 3.25.1.3)
SF = stream flow (Article 3.22)
T = period of vibration
T = temperature (Article 3.22)
t = thickness of ice (Article 3.18.2.2.4)
t = deck thickness (Article 3.25.1.3)
V = variable spacing of truck axles (Figure 3.7.3A)
V = velocity of water (Article 3.18.1)
W = combined weight on the first two axles of a standard HS Truck (Figure 3.7.7A)
W = width of sidewalk (Article 3.14.1.1)
W = wind load on structure (Article 3.22)
W = total dead weight of the structure
W_e = width of exterior girder (Article 3.23.2.3.2)
W = overall width of bridge (Article 3.23.4.3)
W = roadway width between curbs (Article 3.28.1)
WL = wind load on live load (Article 3.22)
w = width of pier or diameter of circular-shaft pier at the level of ice action (Article 3.18.2.2.1)
X = distance from load to point of support (Article 3.24.5.1)
x = subscript denoting direction perpendicular to longitudinal stringers (Article 3.25.1.3)

Z = reduction for ductility and risk assessment
β = (with appropriate script) coefficient applied to actual loads for service load and load factor designs (Article 3.22)
γ = load factor (Article 3.22)
σ_{PL} = proportional limit stress perpendicular to grain (Article 3.25.1.4)
β_B = load combination coefficient for buoyancy (Article 3.22.1)
β_C = load combination coefficient for centrifugal force (Article 3.22.1)
β_D = load combination coefficient for dead load (Article 3.22.1)
β_E = load combination coefficient for earth pressure (Article 3.22.1)
β_{EQ} = load combination coefficient for earthquake (Article 3.22.1)
β_{ICE} = load combination coefficient for ice (Article 3.22.1)
β_L = load combination coefficient for live load (Article 3.22.1)
β_R = load combination coefficient for rib shortening, shrinkage, and temperature (Article 3.22.1)
β_S = load combination coefficient for stream flow (Article 3.22.1)
β_W = load combination coefficient for wind (Article 3.22.1)
β_{WL} = load combination coefficient for wind on live load (Article 3.22.1)
μ = Poisson's ratio (Article 3.23.4.3)

3.2 GENERAL

3.2.1 Structures shall be designed to carry the following loads and forces:

Dead load.
Live load.
Impact or dynamic effect of the live load.
Wind loads.
Other forces, when they exist, as follows:
Longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses.

Provision shall be made for the transfer of forces between the superstructure and substructure to reflect the effect of friction at expansion bearings or shear resistance at elastomeric bearings.

3.2.2 Members shall be proportioned either with reference to service loads and allowable stresses as provided in Service Load Design (Allowable Stress Design) or, alternatively, with reference to load factors and factored strength as provided in Strength Design (Load Factor Design).

3.2.3 When stress sheets are required, a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately.

3.2.4 Where required by design conditions, the concrete placing sequence shall be indicated on the plans or in the special provisions.

3.2.5 The loading combinations shall be in accordance with Article 3.22.

3.2.6 When a bridge is skewed, the loads and forces carried by the bridge through the deck system to pin connections and hangers should be resolved into vertical, lateral, and longitudinal force components to be considered in the design.

3.3 DEAD LOAD

3.3.1 The dead load shall consist of the weight of the entire structure, including the roadway, sidewalks, car tracks, pipes, conduits, cables, and other public utility services.

3.3.2 The snow and ice load is considered to be offset by an accompanying decrease in live load and impact and shall not be included except under special conditions.

3.3.2.1 If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement.

3.3.3 If a separate wearing surface is to be placed when the bridge is constructed, or is expected to be placed in the future, adequate allowance shall be made for its weight in the design dead load. Otherwise, provision for a future wearing surface is not required.

3.3.4 Special consideration shall be given to the necessity for a separate wearing surface for those regions where the use of chains on tires or studded snow tires can be anticipated.

3.3.5 Where the abrasion of concrete is not expected, the traffic may bear directly on the concrete slab. If considered desirable, ¼ inch or more may be added to the slab for a wearing surface.

3.3.6 The following weights are to be used in computing the dead load:

$\frac{kg}{m^3} = 16 \times$

	#/cu.ft.
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced (2400 kg/m³)	150
Compacted sand, earth, gravel, or ballast	120
Loose sand, earth, and gravel	100
Macadam or gravel, rolled	140
Cinder filling	60
Pavement, other than wood block	150
Railway rails, guardrails, and fastenings (per linear foot of track)	200
Stone masonry	170
Asphalt plank, 1 in. thick	9 lb. sq. ft.

3.4 LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars, and pedestrians.

3.5 OVERLOAD PROVISIONS

گروه 1A

3.5.1 For all loadings less than H 20, provision shall be made for an infrequent heavy load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures.

گروه 1B

3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination Group IB in Article 3.22.

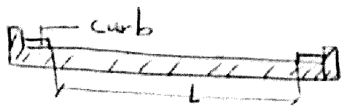
3.6 TRAFFIC LANES

3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet. → 3 m

3.6.2 These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width measured between curbs.

3.6 m

طرح خطوط : DTL = 3.6 m



L را بر 3.6 تقسیم شود، تعداد DTL ها که خارج قسمت صحیح حاصل شود

3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions within their individual traffic lanes, so as to produce the maximum stress in the member under consideration.

3.7 HIGHWAY LOADS

3.7.1 Standard Truck and Lane Loads*

3.7.1.1 The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loading are provided, the H loadings and the HS loadings—the HS loadings being heavier than the corresponding H loadings.

3.7.1.2 Each lane load shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans—see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane.

3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings.

In 1944, the HS series of trucks was developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing three-fourths as much as the basic truck.

3.7.2 Classes of Loading

There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75 percent of loading H 20. Loading HS 15 is 75 percent of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.

3.7.3 Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 Edition in the following manner:

H 15 Loading, 1944 Edition shall be designated..	H 15-44
H 20 Loading, 1944 Edition shall be designated..	H 20-44
H 15-S 12 Loading, 1944 Edition shall be designated..	HS 15-44
H 20-S 16 Loading, 1944 Edition shall be designated..	HS 20-44

The affix shall remain unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loadings previously adopted by the American Association of State Highway and Transportation Officials.

3.7.4 Minimum Loading

Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy truck traffic, shall be designed for HS20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress.

3.7.5 H Loading

The H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

3.7.6 HS Loading

The HS loadings consist of a tractor truck with semi-trailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.6B. The HS loadings are designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments.

3.8 IMPACT

3.8.1 Application

Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground.

3.8.1.1 Group A—Impact shall be included.

(1) Superstructure, including legs of rigid frames.
(2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line.
(3) The portions above the ground line of concrete or steel piles that support the superstructure.

3.8.1.2 Group B—Impact shall not be included.

(1) Abutments, retaining walls, piles except as specified in 3.8.1.1 (3).
(2) Foundation pressures and footings.
(3) Timber structures.
(4) Sidewalk loads.
(5) Culverts and structures having 3 feet or more cover.

3.8.2. Impact Formula

3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula:

$$I = \frac{50}{L+125} \qquad (3\text{-}1)$$

in which,

I = impact fraction (maximum 30 percent);

$I_{max} = 0.3$

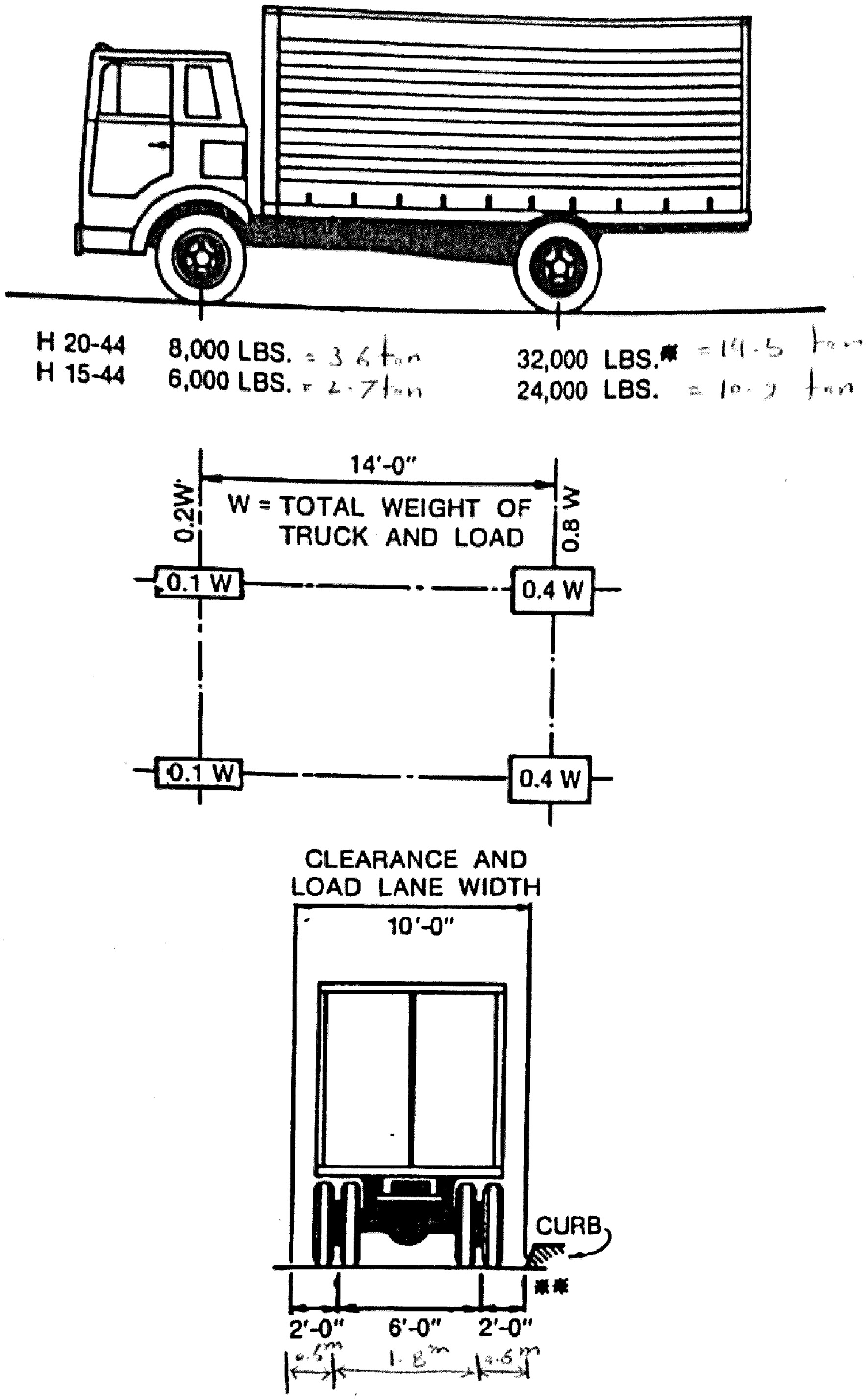
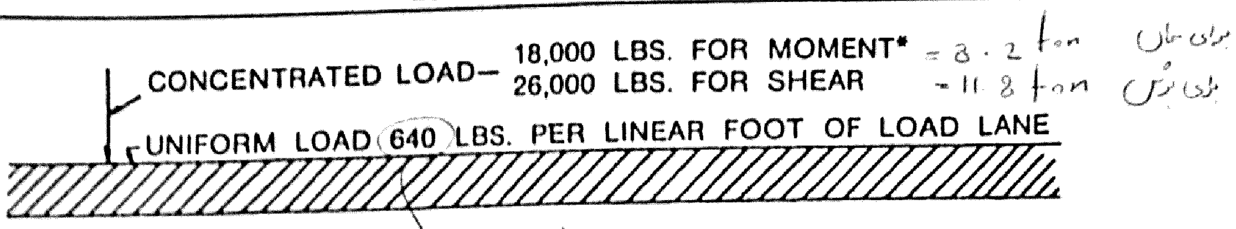


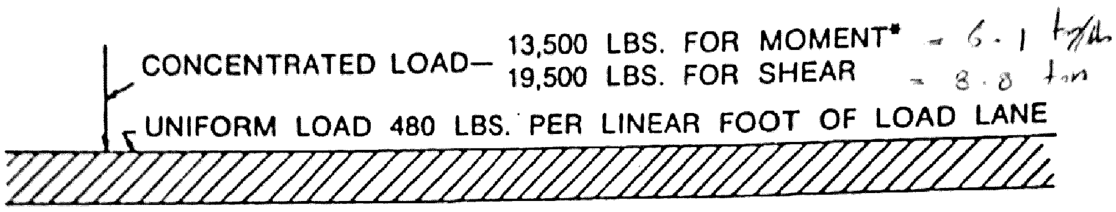
FIGURE 3.7.6A Standard H Trucks

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.)



H20-44 LOADING
HS20-44 LOADING



H15-44 LOADING
HS15-44 LOADING

FIGURE 3.7.6B. **Lane Loading**

*For the loading of continuous spans involving lane loading refer to Article 3.11.3 which provides for an additional concentrated load.

L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

3.8.2.2 For uniformity of application, in this formula, the loaded length, L, shall be as follows:

(a) For roadway floors: the design span length.
(b) For transverse members, such as floor beams: the span length of member center to center of supports.
(c) For computing truck load moments: the span length, or for cantilever arms the length from the moment center to the farthermost axle.
(d) For shear due to truck loads: the length of the loaded portion of span from the point under consideration to the far reaction; except, for cantilever arms, use a 30 percent impact factor.
(e) For continuous spans: the length of span under consideration for positive moment, and the average of two adjacent loaded spans for negative moment.

3.8.2.3 For culverts with cover
0′0″ to 1′-0″ inc. I = 30%
1′-1″ to 2′-0″ inc. I = 20%
2′-1″ to 2′-11″ inc. I = 10%

3.9 LONGITUDINAL FORCES

نیروهای طولی

Provision shall be made for the effect of a longitudinal force of 5 percent of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 3.7, with reduction for multiple-loaded lanes as specified in Article 3.12. The center of gravity of the longitudinal force shall be assumed to be located 6 feet above the floor slab and to be transmitted to the substructure through the superstructure.

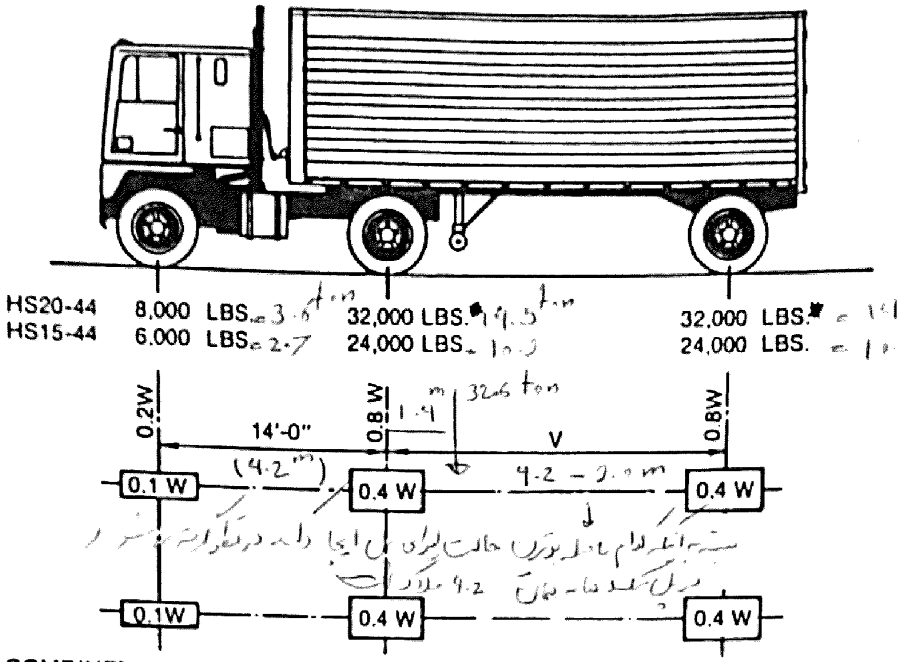
$F = 0.05\,[0.962\,L + 8.2](DTL)$

0.05 [lane load + load conc. mom]

@ 1.8 m above

32.6 ton اکسل

AASHTO



W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.
V = VARIABLE SPACING — 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES.

HS 20-44

1.8 t 7.25 t 7.25

4.2 m 4.2m-9m

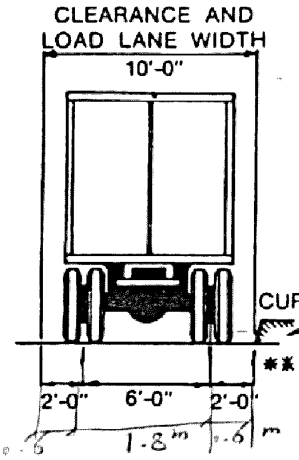


FIGURE 3.7.7A. Standard HS Trucks

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.)

کامیون HS-20 در هر DTL یک عدد و مرکز شدیدترین حالت ها را در نظر بگیرد

3.10 CENTRIFUGAL FORCES

3.10.1 Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes:

$$C = 0.00117S^2D = \frac{6.68S^2}{R} \quad (3\text{-}2)$$

where,

- C = the centrifugal force in percent of the live load, without impact;
- S = the design speed in miles per hour;
- D = the degree of curve;
- R = the radius of the curve in feet.

3.10.2 The effects of superelevation shall be taken into account.

3.10.3 The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 3.7 with one standard truck on each design traffic lane placed in position for maximum loading.

3.10.4 Lane loads shall not be used in the computation of centrifugal forces.

3.10.5 When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load.

3.11 APPLICATION OF LIVE LOAD

3.11.1 Traffic Lane Units

In computing stresses, each 10-foot lane load or single standard truck shall be considered as a unit, and fractions of load lane widths or trucks shall not be used.

3.11.2 Number and Position of Traffic Lane Units

The number and position of the lane load or truck loads shall be as specified in Article 3.7 and, whether lane or truck loads, shall be such as to produce maximum stress, subject to the reduction specified in Article 3.12.

3.11.3 Lane Loads on Continuous Spans

For the determination of maximum negative moment in the design of continuous spans, the lane load shown in Figure 3.7.6B shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment.

3.11.4 Loading for Maximum Stress

3.11.4.1 On both simple and continuous spans, the type of loading, whether lane load or truck load, to be used shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which types of loading controls for simple spans.

3.11.4.2 For continuous spans, the lane loading shall be continuous or discontinuous; only one standard H or HS truck per lane shall be considered on the structure.

3.12 REDUCTION IN LOAD INTENSITY

3.12.1 Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, the following percentages of the live loads shall be used in view of the improbability of coincident maximum loading:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.12.2 The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam.

3.13 ELECTRIC RAILWAY LOADS

If highway bridges carry electric railway traffic, the railway loads shall be determined from the class of traffic which the bridge may be expected to carry. The possibility that the bridge may be required to carry railroad freight cars shall be given consideration.

3.14 SIDEWALK, CURB, AND RAILING LOADING

3.14.1 Sidewalk Loading

3.14.1.1 Sidewalk floors, stringers, and their immediate supports shall be designed for a live load of 85 pounds per square foot of sidewalk area. Girders, trusses, arches, and other members shall be designed for the following sidewalk live loads:

Spans 0 to 25 feet in length . . . . .85 lb./ft.^{2}
Spans 26 to 100 feet in length . . . . .60 lb./ft.^{2}
Spans over 100 feet in length according to the formula

$$P = \left(30 + \frac{3{,}000}{L}\right)\left(\frac{55 - W}{50}\right) \qquad (3\text{-}3)$$

in which

P = live load per square foot, max. 60-lb. per sq. ft.
L = loaded length of sidewalk in feet.
W = width of sidewalk in feet.

3.14.1.2 In calculating stresses in structures that support cantilevered sidewalks, the sidewalk shall be fully loaded on only one side of the structure if this condition produces maximum stress.

3.14.1.3 Bridges for pedestrian and/or bicycle traffic shall be designed for a live load of 85 PSF.

425 kg/m²

3.14.1.4 Where bicycle or pedestrian bridges are expected to be used by maintenance vehicles, special design consideration should be made for these loads.

3.14.2 Curb Loading

750 kg/m

3.14.2.1 Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, or at an elevation 10 inches above the floor if the curb is higher than 10 inches.

25 cm

3.14.2.2 Where sidewalk, curb, and traffic rail form an integral system, the traffic railing loading shall be applied and stresses in curbs computed accordingly.

3.14.3 Railing Loading

For Railing Loads, see Article 2.7.

3.15 WIND LOADS

The wind load shall consist of moving uniformly distributed loads applied to the exposed area of the structure. The exposed area shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a base wind velocity of 100 miles per hour. For Group II and Group V loadings, but not for Group III and Group VI loadings, they may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain which make such changes safe and advisable. If a change in the design wind velocity is made, the design wind velocity shall be shown on the plans.

3.15.1 Superstructure Design

3.15.1.1 Group II and Group V Loadings

3.15.1.1.1 A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure:

For trusses and arches ....75 pounds per square foot 375 kg/m²
For girders and beams ....50 pounds per square foot 250 kg/m²

3.15.1.1.2 The total force shall not be less than 300 pounds per linear foot in the plane of the windward chord and 150 pounds per linear foot in the plane of the leeward chord on truss spans, and not less than 300 pounds per linear foot on girder spans. 450 kg/m

3.15.1.2 Group III and Group VI Loadings

150 kg/m

Group III and Group VI loadings shall comprise the loads used for Group II and Group V loadings reduced by 70 percent and a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on a moving live load. When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the wind load on the moving live load.

3.15.2 Substructure Design

Forces transmitted to the substructure by the superstructure and forces applied directly to the substructure by wind loads shall be as follows:

3.15.2.1 Forces from Superstructure

3.15.2.1.1 The transverse and longitudinal forces transmitted by the superstructure to the substructure for various angles of wind direction shall be as set forth in the following table. The skew angle is measured from the perpendicular to the longitudinal axis and the assumed wind direction shall be that which produces the maximum stress in the substructure. The transverse and longitudinal forces shall be applied simultaneously at the elevation of the center of gravity of the exposed area of the superstructure.

	Trusses		Girders	
Skew Angle of Wind	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
Degrees	PSF	PSF	PSF	PSF
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	24	50	17	19

The loads listed above shall be used in Group II and Group V loadings as given in Article 3.22.

3.15.2.1.2 For Group III and Group VI loadings, these loads may be reduced by 70 percent and a load per linear foot added as a wind load on a moving live load, as given in the following table:

Skew Angle of Wind	Lateral Load	Longitudinal Load
Degrees	lb./ft.	lb./ft.
0	100	0
15	88	12
30	82	24
45	66	32
60	34	38

This load shall be applied at a point 6 feet above the deck.

3.15.2.1.3 For the usual girder and slab bridges having maximum span lengths of 125 feet, the following wind loading may be used in lieu of the more precise loading specified above:

W (wind load on structure)
50 pounds per square foot, transverse
12 pounds per square foot, longitudinal
Both forces shall be applied simultaneously.

WL (wind load on live load)
100 pounds per linear foot, transverse
40 pounds per linear foot, longitudinal
Both forces shall be applied simultaneously.

3.15.2.2 Forces Applied Directly to the Substructure

The transverse and longitudinal forces to be applied directly to the substructure for a 100-mile per hour wind shall be calculated from an assumed wind force of 40 pounds per square foot. For wind directions assumed skewed to the substructure, this force shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation and the component perpendicular to the front elevation shall act on the exposed areas and shall be applied simultaneously with the wind loads from the superstructure. The above loads are for Group II and Group V loadings and may be reduced by 70 percent for Group III and Group VI loadings, as indicated in Article 3.22.

3.15.3 Overturning Forces

The effect of forces tending to overturn structures shall be calculated under Groups II, III, V, and VI of Article 3.22 assuming that the wind direction is at right angles to the longitudinal axis of the structure. In addition, an upward force shall be applied at the windward quarter point of the transverse superstructure width. This force shall be 20 pounds per square foot of deck and sidewalk plan area for Group II and Group V combinations and 6 pounds per square foot for Group III and Group VI combinations.

3.16 THERMAL FORCES

Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.

The range of temperature shall generally be as follows:

Metal structures:
Moderate climate, from 0 to 120°F. −18 to 49 °C
Cold climate, from −30 to 120°F. −34 to 49 °C

	Temperature Rise	Temperature Fall
Concrete structures:		
Moderate climate..........	30°F (17 °C)	40°F (23 °C)
Cold climate..........	35°F (20 °C)	45°F (25 °C)

3.17 UPLIFT

3.17.1 Provision shall be made for adequate attachment of the superstructure to the substructure by ensuring that the calculated uplift at any support is resisted by tension members engaging a mass of masonry equal to the largest force obtained under one of the following conditions:

(a) 100 percent of the calculated uplift caused by any loading or combination of loadings in which the live plus impact loading is increased by 100 percent.
(b) 150 percent of the calculated uplift at working load level.

3.17.2 Anchor bolts subject to tension or other elements of the structure stressed under the above conditions shall be designed at 150 percent of the allowable basic stress.

3.18 FORCES FROM STREAM CURRENT AND FLOATING ICE, AND DRIFT CONDITIONS

All piers and other portions of structures that are subject to the force of flowing water, floating ice, or drift shall be designed to resist the maximum stresses induced thereby.

3.18.1 Force of Stream Current on Piers

3.18.1.1 Stream Pressure

3.18.1.1.1 The effect of flowing water on piers and drift build-up, assuming a second-degree parabolic velocity distribution and thus a triangular pressure distribution, shall be calculated by the formula:

$$P_{avg} = K(V_{avg})^2 \quad (3\text{-}4)$$

kg/m²

where,

P_{avg} = average stream pressure, in pounds per square foot.
V_{avg} = average velocity of water in feet per second, computed by dividing the flow rate by the flow area.
K = a constant, being 1.4 for all piers subjected to drift build-up and square-ended piers, 0.7 for circular piers, and 0.5 for angle-ended piers where the angle is 30 degrees or less.

The maximum stream flow pressure, P_{max}, shall be equal to twice the average stream flow pressure, P_{avg}, computed by Equation 3-4. Stream flow pressure shall be a triangular distribution with P_{max} located at the top of water elevation and a zero pressure located at the flow line.

3.18.1.1.2 The stream flow forces shall be computed by the product of the stream flow pressure, taking into account the pressure distribution, and the exposed pier area. In cases where the corresponding top of water elevation is above the low beam elevation, stream flow loading on the superstructure shall be investigated. The stream flow pressure acting on the superstructure may be taken as P_{max} with a uniform distribution.

3.18.1.2 Pressure Components

When the direction of stream flow is other than normal to the exposed surface area, or when bank migration or a change of stream bed meander is anticipated, the effects of the directional components of stream flow pressure shall be investigated.

3.18.1.3 Drift Lodged Against Pier

Where a significant amount of drift lodged against a pier is anticipated, the effects of this drift buildup shall be considered in the design of the bridge opening and the bridge components. The overall dimensions of the drift buildup shall reflect the selected pier locations, site conditions, and known drift supply upstream. When it is anticipated that the flow area will be significantly blocked by drift buildup, increases in high water elevations, stream velocities, stream flow pressures, and the potential increases in scour depths shall be investigated.

3.18.2 Force of Ice on Piers

3.18.2.1 General

Ice forces on piers shall be selected, having regard to site conditions and the mode of ice action to be expected. Consideration shall be given to the following modes:

(a) Dynamic ice pressure due to moving ice-sheets and ice-floes carried by streamflow, wind, or currents.
(b) Static ice pressure due to thermal movements of

LRFD

11 ft

$$k = \begin{cases} 1\frac{1}{3} \longrightarrow 71 \\ \frac{1}{2} \longrightarrow 27 \\ \frac{2}{3} \longrightarrow 36 \end{cases}$$

continuous stationary ice-sheets on large bodies of water.

(c) Static pressure resulting from ice-jams.

(d) Static uplift or vertical loads resulting from adhering ice in waters of fluctuating level.

3.18.2.2 Dynamic Ice Force

3.18.2.2.1 Horizontal forces resulting from the pressure of moving ice shall be calculated by the formula:

$$F = C_n p \cdot t \cdot w \qquad (3\text{-}5)$$

where,

F = horizontal ice force on pier in pounds;
C_n = coefficient for nose inclination from table;
p = effective ice strength in pounds per square inch;
t = thickness of ice in contact with pier in inches;
w = width of pier or diameter of circular-shaft pier at the level of ice action in inches.

Inclination of Nose to vertical	C_n
0° to 15°	1.00
15° to 30°	0.75
30° to 45°	0.50

3.18.2.2.2 The effective ice strength p shall normally be taken in the range of 100 to 400 pounds per square inch on the assumption that crushing or splitting of the ice takes place on contact with the pier. The value used shall be based on an assessment of the probable condition of the ice at time of movement, on previous local experience, and on assessment of existing structure performance. Relevant ice conditions include the expected temperature of the ice at time of movement, the size of moving sheets and floes, and the velocity at contact. Due consideration shall be given to the probability of extreme rather than average conditions at the site in question.

3.18.2.2.3 The following values of effective ice strength appropriate to various situations may be used as a guide.

(a) In the order of 100 psi where breakup occurs at melting temperatures and where the ice runs as small "cakes" and is substantially disintegrated in its structure.

(b) In the order of 200 psi where breakup occurs at melting temperatures, but the ice moves in large pieces and is internally sound.

(c) In the order of 300 psi where at breakup there is an initial movement of the ice sheet as a whole or where large sheets of sound ice may strike the piers.

(d) In the order of 400 psi where breakup or major ice movement may occur with ice temperatures significantly below the melting point.

3.18.2.2.4 The preceding values for effective ice strength are intended for use with piers of substantial mass and dimensions. The values shall be modified as necessary for variations in pier width or pile diameter, and design ice thickness by multiplying by the appropriate coefficient obtained from the following table:

b/t	Coefficient
0.5	1.8
1.0	1.3
1.5	1.1
2.0	1.0
3.0	0.9
4.0 or greater	0.8

where,

b = width of pier or diameter of pile;
t = design ice thickness.

3.18.2.2.5 Piers should be placed with their longitudinal axis parallel to the principal direction of ice action. The force calculated by the formula shall then be taken to act along the direction of the longitudinal axis. A force transverse to the longitudinal axis and amounting to not less than 15 percent of the longitudinal force shall be considered to act simultaneously.

3.18.2.2.6 Where the longitudinal axis of a pier cannot be placed parallel to the principal direction of ice action, or where the direction of ice action may shift, the total force on the pier shall be computed by the formula and resolved into vector components. In such conditions, forces transverse to the longitudinal axis shall in no case be taken as less than 20 percent of the total force.

3.18.2.2.7 In the case of slender and flexible piers, consideration should be given to the vibrating nature of dynamic ice forces and to the possibility of high momentary pressures and structural resonance.

3.18.2.3 Static Ice Pressure

Ice pressure on piers frozen into ice sheets on large bodies of water shall receive special consideration where

TABLE 3.22.1A Table of Coefficients γ and β

جدول بسیار مهم

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
GROUP		γ	β FACTORS													%	
			D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE		
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

بالای اطلاعات ← $(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L+I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

1.25 ← 1.67

1.67 → 1

** Percentage = $\dfrac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.00 for vertical and lateral loads on all other structures.

$\beta_E = 1$ برای اغلب سازه‌ها

For culvert loading specifications, see Article 6.2.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

For Load Factor Design

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts. For lateral at-rest earth pressures, β_E = 1.15

β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20.

β_E = 1.0 for vertical earth pressure

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity ..... For Column Design

β_D = 1.0 when checking member for maximum axial load and minimum moment ....

β_D = 1.0 for flexural and tension members

β_E = 1.0 for Rigid Culverts

β_E = 1.5 for Flexible Culverts

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

there is reason to believe that the ice sheets are subject to significant thermal movements relative to the piers.

3.19 BUOYANCY

Buoyancy shall be considered where it affects the design of either substructure, including piling, or the superstructure.

3.20 EARTH PRESSURE

$\frac{1}{2} K \gamma H^2$

3.20.1 Structures which retain fills shall be proportioned to withstand pressure as given by Rankine's formula or by other expressions given in Section 5, "Retaining Walls"; provided, however, that no structure shall be designed for less than an equivalent fluid weight (mass) of 30 pounds per cubic foot.

باید $K \gamma > 480\ kg/m^3$

3.20.2 For rigid frames a maximum of one-half of the moment caused by earth pressure (lateral) may be used to reduce the positive moment in the beams, in the top slab, or in the top and bottom slab, as the case may be.

3.20.3 When highway traffic can come within a horizontal distance from the top of the structure equal to one-half its height, the pressure shall have added to it a live load surcharge pressure equal to not less than 2 feet of earth.

60 cm

سربار زنده معادل ۶۰ سانت خاک

3.20.4 Where an adequately designed reinforced concrete approach slab supported at one end by the bridge is provided, no live load surcharge need be considered.

3.20.5 All designs shall provide for the thorough drainage of the back-filling material by means of weep holes and crushed rock, pipe drains or gravel drains, or by perforated drains.

3.21 EARTHQUAKES

In regions where earthquakes may be anticipated, structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with $\boxed{\text{Division I-A—Seismic Design.}}$

طرح لرزه‌ای پل

Part B
COMBINATIONS OF LOADS

3.22 COMBINATIONS OF LOADS

3.22.1 The following Groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for Service Load Design and Load Factor Design are given by:

$$\begin{aligned} \text{Group (N)} = \gamma[&\beta_D \cdot D + \beta_L (L + I) + \beta_C CF + \beta_E E \\ &+ \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_L \cdot LF + \beta_R (R + S + T) \\ &+ \beta_{EQ} EQ + \beta_{ICE} ICE] \end{aligned} \tag{3-10}$$

where,

- N = group number;
- γ = load factor, see Table 3.22.1A;
- β = coefficient, see Table 3.22.1A;
- D = dead load;
- L = live load;
- I = live load impact;
- E = earth pressure;
- B = buoyancy;
- W = wind load on structure;
- WL = wind load on live load—100 pounds per linear foot;
- LF = longitudinal force from live load;
- CF = centrifugal force;
- R = rib shortening;
- S = shrinkage;
- T = temperature;
- EQ = earthquake;
- SF = stream flow pressure;
- ICE = ice pressure.

3.22.2 For service load design, the percentage of the basic unit stress for the various groups is given in Table 3.22.1A.

The loads and forces in each group shall be taken as appropriate from Articles 3.3 to 3.21. The maximum section required shall be used.

3.22.3 For load factor design, the gamma and beta factors given in Table 3.22.1A shall be used for designing structural members and foundations by the load factor concept.

TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
1. Timber:[a]		
a. Plank[b]	S/4.0	S/3.75
b. Nail laminated[c] 4" thick or multiple layer[d] floors over 5" thick	S/4.5	S/4.0
c. Nail laminated[c] 6" or more thick	S/5.0 If S exceeds 5' use footnote f.	S/4.25 If S exceeds 6.5' use footnote f.
d. Glued laminated[e] Panels on glued laminated stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 7.5' use footnote f.
e. On steel stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/5.25 If S exceeds 5.5' use footnote f.	S/4.5 If S exceeds 7' use footnote f.
2. Concrete:		
a. On steel I-Beam stringers[g] and prestressed concrete girders	S/7.0 (S/2.13) If S exceeds 10' (S<3.0 m) use footnote f.	S/5.5 (S/1.68 m) If S exceeds 14' (=4.27 m) use footnote f.
b. On concrete T-Beams	S/6.5 (S/1.98) If S exceeds 6' (S<1.83) use footnote f.	S/6.0 (S/1.83) If S exceeds 10' (=3.0 m) use footnote f.
c. On timber stringers	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10' use footnote f.
d. Concrete box girders[h]	S/8.0 If S exceeds 12' use footnote f. See Article 10.39.2.	S/7.0 If S exceeds 16' use footnote f.
e. On steel box girders	See Article 10.39.2.	
On prestressed concrete spread box Beams	See Article 3.28.	
3. Steel grid:		
(Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10.5' use footnote f.
4. Steel bridge Corrugated plank[i] (2" min depth)	S/5.5	S/4.5

S = average stringer spacing in feet.
[a]Timber dimensions shown are for nominal thickness.
[b]Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11—Division II).
[c]Nail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed to the preceding piece (see Article 16.3.12—Division II).
[d]Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (see Article 16.3.11—Division II).
[e]Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13—Division II).
[f]In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.
[g]"Design of I-Beam Bridges" by N. M. Newmark—Proceedings, ASCE, March 1948.
[h]The sidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.
[i]Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:

Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland.

These distribution factors were developed based on studies using 6" × 2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" × 2" corrugated plank used in the studies.

3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of construction are different from those anticipated by the specifications.

3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H 20, Group IA loading combination shall be used (see Article 3.5).

Part C
DISTRIBUTION OF LOADS

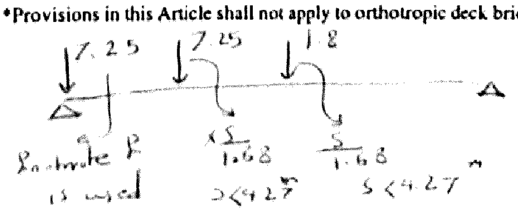
3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS*

3.23.1 Position of Loads for Shear

3.23.1.1 In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the transverse floor beam or the end of the longitudinal beam or stringer at which the stress is being determined.

3.23.1.2 Lateral distribution of the wheel loads at ends of the beams or stringers shall be that produced by

*Provisions in this Article shall not apply to orthotropic deck bridges.



assuming the flooring to act as a simple span between stringers or beams. For wheels or axles in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the calculations of horizontal shear in rectangular timber beams shall be in accordance with Article 13.3.

3.23.2 Bending Moments in Stringers and Longitudinal Beams**

3.23.2.1 General

In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.

3.23.2.2 Interior Stringers and Beams

The live load bending moment for each interior stringer shall be determined by applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1.

3.23.2.3 Outside Roadway Stringers and Beams

3.23.2.3.1 Steel-Timber-Concrete T-Beams

3.23.2.3.1.1 The dead load supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surface, if placed after the slab has cured, may be distributed equally to all roadway stringers or beams.

3.23.2.3.1.2 The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

3.23.2.3.1.3 When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact and the structure is to be designed by the service load method, the allowable stress in the beam or stringer may be increased by 25 percent for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When the combination of sidewalk live load and traffic live load plus impact governs the design and the structure is to be designed by the load factor method, 1.25 may be used as the beta factor in place of 1.67.

**In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges.

3.23.2.3.1.4 In no case shall an exterior stringer have less carrying capacity than an interior stringer.

3.23.2.3.1.5 In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than:

$$S \leq 1.8 \qquad \frac{S}{5.5} \qquad \boxed{\frac{S}{1.68}}$$

where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and

$$4.27 > S > 1.8 \qquad \frac{S}{4.0+0.25S} \qquad \frac{S}{1.2+0.25S}$$

where, S is more than 6 feet and less than 14 feet. When S is 14 feet or more, use footnote f, Table 3.23.1.

3.23.2.3.2 Concrete Box Girders

3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1.

3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be $W_e/7$, where W_e is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.

3.23.2.3.3 Total Capacity of Stringers and Beams

The combined design load capacity of all the beams and stringers in a span shall not be less than required to support the total live and dead load in the span.

3.23.3 Bending Moments in Floor Beams (Transverse)

3.23.3.1 In calculating bending moments in floor beams, no transverse distribution of the wheel loads shall be assumed.

3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1.

TABLE 3.23.3.1 Distribution of Wheel Loads in Transverse Beams

Kind of Floor	Fraction of Wheel Load to Each Floor Beam	
Plank[a,b]	$\frac{S}{4}$	
Nail laminated[c] or glued laminated[e], 4 inches in thickness, or multiple layer[d] floors more than 5 inches thick	$\frac{S}{4.5}$	
Nail laminated[c] or glued laminated[e], 6 inches or more in thickness	$\frac{S^f}{5}$	
Concrete	$\frac{S^f}{6}$	$\frac{S}{1.68}$
Steel grid (less than 4 inches thick)	$\frac{S}{4.5}$	
Steel grid (4 inches or more)	$\frac{S^f}{6}$	
Steel bridge corrugated plank (2 inches minimum depth)	$\frac{S}{5.5}$	$\simeq \frac{S}{1.68}$

Note:
S = spacing of floor beams in feet.

[a–e] For footnotes a through e, see Table 3.23.1.

[f] If S exceeds denominator, the load on the beam shall be the reaction of the wheels loads assuming the flooring between beams to act as a simple beam.

3.23.4 Precast Concrete Beams Used in Multi-Beam Decks

3.23.4.1 A multi-beam bridge is constructed with precast reinforced or prestressed concrete beams that are placed side by side on the supports. The interaction between the beams is developed by continuous longitudinal shear keys used in combination with transverse tie assemblies which may, or may not, be prestressed, such as bolts, rods, or prestressing strands, or other mechanical means. Full-depth rigid end diaphragms are needed to ensure proper load distribution for channel, single- and multi-stemmed tee beams.

3.23.4.2 In calculating bending moments in multi-beam precast concrete bridges, conventional or prestressed, no longitudinal distribution of wheel load shall be assumed.

3.23.4.3 The live load bending moment for each section shall be determined by applying to the beam the fraction of a wheel load (both front and rear) determined by the following equation:

$$\text{Load Fraction} = \frac{S}{D} \qquad (3\text{-}11)$$

where,

S = width of precast member;
$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2$ when $C \leq 5$ (3-12)
$D = (5.75 - 0.5N_L)$ when $C > 5$ (3-13)
N_L = number of traffic lanes from Article 3.6;
$C = K(W/L)$ (3-14)

where,

W = overall width of bridge measured perpendicular to the longitudinal girders in feet;
L = span length measured parallel to longitudinal girders in feet; for girders with cast-in-place end diaphragms, use the length between end diaphragms;
$K = \{(1 + \mu)\, I/J\}^{1/2}$

If the value of $\sqrt{I/J}$ exceeds 5.0, the live load distribution should be determined using a more precise method, such as the Articulated Plate Theory or Grillage Analysis.

where,

I = moment of inertia;
J = Saint-Venant torsion constant;
μ = Poisson's ratio for girders.

In lieu of more exact methods, "J" may be estimated using the following equations:

For Non-voided Rectangular Beams, Channels, Tee Beams:

$$J = \Sigma\{(1/3)bt^3(1 - 0.630t/b)\}$$

where,

b = the length of each rectangular component within the section,
t = the thickness of each rectangular component within the section.

The flanges and stems of stemmed or channel sections are considered as separate rectangular components whose values are summed together to calculate "J". Note that for "Rectangular Beams with Circular Voids" the value of "J" can usually be approximated by using the equation above for rectangular sections and neglecting the voids.

For Box-Section Beams:

$$J = \frac{2tt_f(b-t)^2(d-t_f)^2}{bt + dt_f - t^2 - t_f^2}$$

where

b = the overall width of the box,
d = the overall depth of the box,
t = the thickness of either web,
t_f = the thickness of either flange.

The formula assumes that both flanges are the same thickness and uses the thickness of only one flange. The same is true of the webs.

For preliminary design, the following values of K may be used:

Bridge Type	Beam Type	K
Multi-beam	Non-voided rectangular beams	0.7
	Rectangular beams with circular voids	0.8
	Box section beams	1.0
	Channel, single- and multi-stemmed tee beams	2.2

3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS*

3.24.1 Span Lengths (See Article 8.8)

3.24.1.1 For simple spans the span length shall be the distance center to center of supports but need not exceed clear span plus thickness of slab.

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3.24.1.2 The following effective span lengths shall be used in calculating the distribution of loads and bending moments for slabs continuous over more than two supports:

(a) Slabs monolithic with beams or slabs monolithic with walls without haunches and rigid top flange prestressed beams with top flange width to minimum thickness ratio less than 4.0. "S" shall be the clear span.

(b) Slabs supported on steel stringers, or slabs supported on thin top flange prestressed beams with top flange width to minimum thickness ratio equal to or greater than 4.0. "S" shall be the distance between edges of top flange plus one-half of stringer top flange width.

(c) Slabs supported on timber stringers. S shall be the clear span plus one-half thickness of stringer.

3.24.2 Edge Distance of Wheel Loads

3.24.2.1 In designing slabs, the center line of the wheel load shall be 1 foot from the face of the curb. If curbs or sidewalks are not used, the wheel load shall be 1 foot from the face of the rail.

3.24.2.2 In designing sidewalks, slabs and supporting members, a wheel load located on the sidewalk shall be 1 foot from the face of the rail. In service load design, the combined dead, live, and impact stresses for this loading shall be not greater than 150 percent of the allowable stresses. In load factor design, 1.0 may be used as the beta factor in place of 1.67 for the design of deck slabs. Wheel loads shall not be applied on sidewalks protected by a traffic barrier.

3.24.3 Bending Moment

The bending moment per foot width of slab shall be calculated according to methods given under Cases A and B, unless more exact methods are used considering tire contact area. The tire contact area needed for exact methods is given in Article 3.30.

In Cases A and B:

S = effective span length, in feet, as defined under "Span Lengths" Articles 3.24.1 and 8.8;
E = width of slab in feet over which a wheel load is distributed;
P = load on one rear wheel of truck (P_{15} or P_{20});
P_{15} = 12,000 pounds for H 15 loading;
P_{20} = 16,000 pounds for H 20 loading.

*The slab distribution set forth herein is based substantially on the "Westergaard" theory. The following references are furnished concerning the subject of slab design.

Public Roads, March 1930, "Computation of Stresses in Bridge Slabs Due to Wheel Loads," by H. M. Westergaard.

University of Illinois, Bulletin No. 303, "Solutions for Certain Rectangular Slabs Continuous over Flexible Supports," by Vernon P. Jensen; Bulletin 304, "A Distribution Procedure for the Analysis of Slabs Continuous over Flexible Beams," by Nathan M. Newmark; Bulletin 315, "Moments in Simple Span Bridge Slabs with Stiffened Edges," by Vernon P. Jensen; and Bulletin 346, "Highway Slab Bridges with Curbs; Laboratory Tests and Proposed Design Method."

3.24.3.1 Case A—Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 Feet Inclusive)

The live load moment for simple spans shall be determined by the following formulas (impact not included):

HS 20 Loading:

$$\left(\frac{S+2}{32}\right)P_{20} = \text{Moment in foot} - \text{pounds per foot} - \text{width of slab} \quad (3\text{-}15)$$

HS 15 Loading: $\left(\frac{S+0.6}{9.6}\right)P_{2}$

$$\left(\frac{S+2}{32}\right)P_{15} = \text{Moment in foot} - \text{pounds per foot} - \text{width of slab} \quad (3\text{-}16)$$

In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moment.

3.24.3.2 Case B—Main Reinforcement Parallel to Traffic

For wheel loads, the distribution width, E, shall be (4 + 0.06S) but shall not exceed 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced slabs shall be designed for the appropriate HS loading.

For simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas:

HS 20 Loading: kg·m 405S

Spans up to and including 50 feet: LLM = 900S foot-pounds

Spans 50 feet to 100 feet: LLM = 1,000 (1.30S-20.0) foot-pounds ×0.45

HS 15 Loading:

Use 3/4 of the values obtained from the formulas for HS 20 loading

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

3.24.4 Shear and Bond

Slabs designed for bending moment in accordance with Article 3.24.3 shall be considered satisfactory in bond and shear.

3.24.5 Cantilever Slabs

3.24.5.1 Truck Loads

Under the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independently of the effects of any edge support along the end of the cantilever. The distribution given includes the effect of wheels on parallel elements.

3.24.5.1.1 Case A—Reinforcement Perpendicular to Traffic

Each wheel on the element perpendicular to traffic shall be distributed over a width according to the following formula:

$\boxed{E = 0.2X + 1.143}$

$$E = 0.8X + 3.75 \quad (3\text{-}17)$$

The moment per foot of slab shall be $\boxed{(P/E)\ X}$ foot-pounds, in which X is the distance in feet from load to point of support.

3.24.5.1.2 Case B—Reinforcement Parallel to Traffic

The distribution width for each wheel load on the element parallel to traffic shall be as follows:

$$E = 0.35X + 3.2, \text{ but shall not exceed 7.0 feet} \quad (3\text{-}18)$$

The moment per foot of slab shall be (P/E) X foot-pounds.

3.24.5.2 Railing Loads

Railing loads shall be applied in accordance with Article 2.7. The effective length of slab resisting post loadings shall be equal to E = 0.8X + 3.75 feet where no parapet is used and equal to E = 0.8X + 5.0 feet where a parapet is used, where X is the distance in feet from the center of the post to the point under investigation. Railing and wheel loads shall not be applied simultaneously.

3.24.6 Slabs Supported on Four Sides

3.24.6.1 For slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be given by the following equations:

$$\text{For uniformly distributed load, } p = \frac{b^4}{a^4 + b^4} \quad (3\text{-}19)$$

$$\text{For concentrated load at center, } p = \frac{b^3}{a^3 + b^3} \quad (3\text{-}20)$$

where,

p = proportion of load carried by short span;
a = length of short span of slab;
b = length of long span of slab.

3.24.6.2 Where the length of the slab exceeds 1½ times its width, the entire load shall be carried by the transverse reinforcement.

3.24.6.3 The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. The moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced by 50 percent. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.

3.24.7 Median Slabs

Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live, and impact stresses shall not be greater than 150 percent of the allowable stresses. Flush median slabs shall be designed without overstress.

3.24.8 Longitudinal Edge Beams

3.24.8.1 Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb.

3.24.8.2 The edge beam of a simple span shall be designed to resist a live load moment of 0.10 PS, where,

P = wheel load in pounds P_{15} or P_{20};
S = span length in feet.

3.24.8.3 For continuous spans, the moment may be reduced by 20 percent unless a greater reduction results from a more exact analysis.

3.24.9 Unsupported Transverse Edges

The design assumptions of this article do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them.

3.24.10 Distribution Reinforcement

3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds 2 feet.

3.24.10.2 The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas:

For main reinforcement parallel to traffic,

$$\text{Percentage} = \frac{100}{\sqrt{S}} \text{ Maximum } 50\% \qquad (3\text{-}21)$$

For main reinforcement perpendicular to traffic,

$$\text{Percentage} = \frac{220}{\sqrt{S}} \text{ Maximum } 67\% \qquad (3\text{-}22)$$

$\frac{120.5}{\sqrt{S}} \leq 67\%$

where, S = the effective span length in feet.

3.24.10.3 For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50 percent of the specified amount shall be used in the outer quarters of the slab span.

3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows.

3.25.1 Transverse Flooring

3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30.

Normal to the direction of flooring span, the wheel load shall be distributed as follows:

Plank floor: the width of plank.

Non-interconnected* nail laminated panel floor: 15 inches, but not to exceed panel width.

Non-interconnected glued laminated panel floor: 15 inches plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated panel floor, with adequate shear transfer between panels**: 15 inches plus thickness of floor, but not to exceed panel width.

Interconnected* glued laminated panel floor, with adequate shear transfer between panels**, not less than 6 inches thick: 15 inches plus twice thickness of floor, but not to exceed panel width.

3.25.1.2 For transverse flooring the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness.

3.25.1.3 One design method for interconnected glued laminated panel floors is as follows: For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers and with panels interconnected using steel dowels, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear.† The maximum shear is for a wheel position assumed to be 15 inches or less from the center line of the support. The maximum moment is for a wheel position assumed to be centered between the supports.

$$M_x = P(.51 \log_{10} s - K) \qquad (3\text{-}23)$$

$$R_x = .034P \qquad (3\text{-}24)$$

Thus,

$$t = \sqrt{\frac{6M_x}{F_b}} \qquad (3\text{-}25)$$

or,

$$t = \frac{3R_x}{2F_v} \text{ whichever is greater} \qquad (3\text{-}26)$$

where,

M_x = primary bending moment in inch-pounds per inch;

R_x = primary shear in pounds per inch;

x = denotes direction perpendicular to longitudinal stringers;

P = design wheel load in pounds;

s = effective deck span in inches;

t = deck thickness, in inches, based on moment or shear, whichever controls;

K = design constant depending on design load as follows:

H 15 K = 0.47

H 20 K = 0.51

F_b = allowable bending stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B);

F_v = allowable shear stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B).

3.25.1.4 The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels shall be based on the following equation:

$$n = \frac{1{,}000}{\sigma_{PL}} \times \left[\frac{\overline{R_y}}{R_D} + \frac{\overline{M_y}}{M_D}\right] \qquad (3\text{-}27)$$

where,

n = number of steel dowels required for the given spans;

σ_{PL} = proportional limit stress perpendicular to grain (for Douglas Fir or Southern pine, use 1,000 psi);

$\overline{R_y}$ = total secondary shear transferred, in pounds, determined by the relationship:

$$\overline{R_y} = 6Ps/1{,}000 \text{ for } s \leq 50 \text{ inches} \qquad (3\text{-}28)$$

or,

$$\overline{R_y} = \frac{P}{2s}(s - 20) \text{ for } s > 50 \text{ inches} \qquad (3\text{-}29)$$

$\overline{M_y}$ = total secondary moment transferred, in inch-pound, determined by the relationship,

$$\overline{M_y} = \frac{Ps}{1{,}600}(s - 10) \text{ for } s \leq 50 \text{ inches} \qquad (3\text{-}30)$$

$$\overline{M_y} = \frac{Ps}{20}\frac{(s - 30)}{(s - 10)} \text{ for } s > 50 \text{ inches} \qquad (3\text{-}31)$$

R_D and M_D = shear and moment capacities, respectively, as given in the following table:

*The terms interconnected and non-interconnected refer to the joints between the individual nail laminated or glued laminated panels.

**This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or other suitable means.

†The equations are developed for deck panel spans equal to or greater than the width of the tire (as specified in Article 3.30), but not greater than 200 inches.

Diameter of Dowel	Shear Capacity R_D	Moment Capacity M_D	Steel Stress Coefficients C_R	C_M	Total Dowel Length Required
in.	lb.	in.-lb.	$l/in.^2$	$l/in.^3$	in.
0.5	600	850	36.9	81.5	8.50
.625	800	1,340	22.3	41.7	10.00
.75	1,020	1,960	14.8	24.1	11.50
.875	1,260	2,720	10.5	15.2	13.00
1.0	1,520	3,630	7.75	10.2	14.50
1.125	1,790	4,680	5.94	7.15	15.50
1.25	2,100	5,950	4.69	5.22	17.00
1.375	2,420	7,360	3.78	3.92	18.00
1.5	2,770	8,990	3.11	3.02	19.50

3.25.1.5 In addition, the dowels shall be checked to ensure that the allowable stress of the steel is not exceeded using the following equation:

$$\sigma = \frac{1}{n}(C_R \overline{R_y} + C_M \overline{M_y}) \qquad (3\text{-}32)$$

where,

σ = minimum yield point of steel pins in pounds per square inch (see Table 10.32.1A);

n, $\overline{R_y}$, $\overline{M_y}$ = as previously defined;

C_R, C_M = steel stress coefficients as given in preceding table.

3.25.2 Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be considered a point loading.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: width of plank;

Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated floor, with adequate shear transfer between panels*, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

*This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or spreader beams located at intervals along the panels or other suitable means.

3.25.3 Longitudinal Glued Laminated Timber Decks

3.25.3.1 Bending Moment

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

$$\text{Load Fraction} = \frac{W_p}{3.75 + \frac{L}{28}} \text{ or } \frac{W_p}{5.00}, \text{ whichever is}$$

greater.

ONE TRAFFIC LANE

$$\text{Load Fraction} = \frac{W_p}{4.25 + \frac{L}{28}} \text{ or } \frac{W_p}{5.50}, \text{ whichever is}$$

greater.

where, W_p = Width of Panel; in feet ($3.5 \leq W_p \leq 4.5$)

L = Length of span for simple span bridges and the length of the shortest span for continuous bridges in feet.

3.25.3.2 Shear

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 Deflections

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 Stiffener Arrangement

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at mid-span with additional stiffeners placed at intervals not to exceed 10 feet. The stiffness factor EI of the stiffener shall not be less than 80,000 kip-in².

3.25.4 Continuous Flooring

If the flooring is continuous over more than two spans, the maximum bending moment shall be assumed as being 80 percent of that obtained for a simple span.

3.26 DISTRIBUTION OF WHEEL LOADS AND DESIGN OF COMPOSITE WOOD-CONCRETE MEMBERS

3.26.1 Distribution of Concentrated Loads for Bending Moment and Shear

3.26.1.1 For freely supported or continuous slab spans of composite wood-concrete construction, as described in Article 20.19.1 Division II, the wheel loads shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear.

3.26.1.2 For composite T-beams of wood and concrete, as described in Article 20.19.2—Division II, the effective flange width shall not exceed that given in Article 10.38.3. Shear connectors shall be capable of resisting both vertical and horizontal movement.

3.26.2 Distribution of Bending Moments in Continuous Spans

3.26.2.1 Both positive and negative moments shall be distributed in accordance with the following table:

Maximum Bending Moments—Percent of Simple Span Moment

	Maximum Uniform Dead Load Moments				Maximum Live Load Moments			
	Wood Subdeck		Composite Slab		Concentrated Load		Uniform Load	
Span	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Interior	50	50	55	45	75	25	75	55
End	70	60	70	60	85	30	85	65
2-Span*	65	70	60	75	85	30	80	75

*Continuous beam of 2 equal spans.

3.26.2.2 Impact should be considered in computing stresses for concrete and steel, but neglected for wood.

3.26.3 Design

The analysis and design of composite wood-concrete members shall be based on assumptions that account for the different mechanical properties of the components. A suitable procedure may be based on the elastic properties of the materials as follows:

$\frac{E_c}{E_w}$ = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section

$\frac{E_c}{E_w}$ = 2 for slab in which the net concrete thickness is at least half the overall depth of the composite section

$\frac{E_s}{E_w}$ = 18.75 (for Douglas fir and Southern pine)

in which,

E_c = modulus of elasticity of concrete;
E_w = modulus of elasticity of wood;
E_s = modulus of elasticity of steel.

3.27 DISTRIBUTION OF WHEEL LOADS ON STEEL GRID FLOORS*

3.27.1 General

3.27.1.1 The grid floor shall be designed as continuous, but simple span moments may be used and reduced as provided in Article 3.24.

*Provisions in this article shall not apply to orthotropic bridge superstructures.

3.27.1.2 The following rules for distribution of loads assume that the grid floor is composed of main elements that span between girders, stringers, or cross beams, and secondary elements that are capable of transferring load between the main elements.

3.27.1.3 Reinforcement for secondary elements shall consist of bars or shapes welded to the main steel.

3.27.2 Floors Filled with Concrete

3.27.2.1 The distribution and bending moment shall be as specified for concrete slabs, Article 3.24. The following items specified in that article shall also apply to concrete filled steel grid floors:

Longitudinal edge beams
Unsupported transverse edges
Span lengths

3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 8.15.2, 8.16.1, and 10.32.

3.27.3 Open Floors

3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1¼ inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H 20, 15 inches for H 15).

3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32.

3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly.

3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS*

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k\frac{S}{L} \qquad (3\text{-}33)$$

where,

N_L = number of design traffic lanes (Article 3.6);
N_B = number of beams ($4 \leq N_B \leq 10$);
S = beam spacing in feet ($6.57 \leq S \leq 11.00$);
L = span length in feet;
k = $0.07\,W - N_L(0.10N_L - 0.26) - 0.20N_B - 0.12$; (3-34)
W = numeric value of the roadway width between curbs expressed in feet ($32 \leq W \leq 66$).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

3.29 MOMENTS, SHEARS, AND REACTIONS

Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress.

3.30 TIRE CONTACT AREA

The tire contact area shall be assumed as a rectangle with an area in square inches of 0.01P, and a Length in Direction of Traffic/Width of Tire ratio of 1/2.5, in which P = wheel load in pounds.

*The provisions of Article 3.12, Reduction in Load Intensity, were not applied in the development of the provisions presented in 3.28.1 and 3.28.2.

Section 8

REINFORCED CONCRETE*

Part A
GENERAL REQUIREMENTS AND MATERIALS

8.1 APPLICATION

8.1.1 General

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

8.1.2 Notations

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

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

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

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

A_b = area of an individual bar, sq. in. (Article 8.25.1)

A_c = area of core of spirally reinforced compression member measured to the outside diameter of the spiral, sq. in. (Article 8.18.2.2.2)

A_{cv} = area of concrete section resisting shear transfer, sq. in. (Article 8.16.6.4.5)

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

A_g = gross area of section, sq. in.

A_h = area of shear reinforcement parallel to flexural tension reinforcement, sq. in. (Articles 8.15.5.8 and 8.16.6.8)

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

A_s = area of tension reinforcement, sq. in.

A'_s = area of compression reinforcement, sq. in.

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

A_{sk} = area of skin reinforcement per unit height in one side face, sq. in. per ft. (Article 8.17.2.1.3).

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

A_v = area of shear reinforcement within a distance s

A_{vf} = area of shear-friction reinforcement, sq. in. (Article 8.15.5.4.3)

A_w = area of an individual wire to be developed or spliced, sq. in. (Articles 8.30.1.2 and 8.30.2)

A_1 = loaded area (Articles 8.15.2.1.3 and 8.16.7.2)

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

b = width of compression face of member

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

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

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

b_w = web width, or diameter of circular section (Article 8.15.5.1.1)

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

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

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

d′ = distance from extreme compression fiber to centroid of compression reinforcement, in.

d″ = distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement, in.

d_b = nominal diameter of bar or wire, in.

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

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

EI = flexural stiffness of compression member (Article 8.16.5.2.7)

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

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

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

f'_c = specified compressive strength of concrete, psi

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

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

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

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

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

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

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

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

f_y = specified yield strength of reinforcement, psi

h = overall thickness of member, in.

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

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

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

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

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

k = effective length factor for compression members (Article 8.16.5.2.3)

ℓ_a = additional embedment length at support or at point of inflection, in. (Article 8.24.2.3)

ℓ_d = development length, in. (Articles 8.24 through 8.32)

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

ℓ_{dh} = ℓ_{hb} × applicable modification factor

ℓ_{hb} = basic development length of standard hook in tension, in.

ℓ_u = unsupported length of compression member (Article 8.16.5.2.1)

M = computed moment capacity (Article 8.24.2.3)

M_a = maximum moment in member at stage for which deflection is being computed (Article 8.13.3)

M_b = nominal moment strength of a section at balanced strain conditions (Article 8.16.4.2.3)

M_c = moment to be used for design of compression member (Article 8.16.5.2.7)

M_{cr} = cracking moment (Article 8.13.3)

M_n = nominal moment strength of a section

M_{nx} = nominal moment strength of a section in the direction of the x axis (Article 8.16.4.3)

M_{ny} = nominal moment strength of a section in the direction of the y axis (Article 8.16.4.3)

M_u = factored moment at section

M_{ux} = factored moment component in the direction of the x axis (Article 8.16.4.3)

M_{uy} = factored moment component in the direction of the y axis (Article 8.16.4.3)

M_{1b} = value of smaller end moment on compression member due to gravity loads that result in no appreciable sidesway calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature (Article 8.16.5.2.4)

M_{2b} = value of larger end moment on compression member due to gravity loads that result in no appreciable sidesway calculated by conventional elastic frame analysis, always positive (Article 8.16.5.2.4)

M_{2s} = value of larger end moment on compression member due to lateral loads or gravity loads that result in appreciable sidesway, defined by a deflection Δ, greater than $\ell_u/1500$, calculated by conventional elastic frame analysis, always positive. (Article 8.16.5.2)

n = modular ratio of elasticity = E_s/E_c (Article 8.15.3.4)

N = design axial load normal to cross section occurring simultaneously with V to be taken as positive for compression, negative for tension and to include the effects of tension due to shrinkage and creep (Articles 8.15.5.2.2 and 8.15.5.2.3)

N_c = design tensile force applied at top of bracket of corbel acting simultaneously with V, to be taken as positive for tension (Article 8.15.5.8)

N_u = factored axial load normal to the cross section occurring simultaneously with V_u to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep (Article 8.16.6.2.2)

N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension (Article 8.16.6.8)

P_b = nominal axial load strength of a section at balanced strain conditions (Article 8.16.4.2.3)

P_c = critical load (Article 8.16.5.2.7)

P_o = nominal axial load strength of a section at zero eccentricity (Article 8.16.4.2.1)

P_n = nominal axial load strength at given eccentricity

P_{nx} = nominal axial load strength corresponding to M_{nx}, with bending considered in the direction of the x axis only (Article 8.16.4.3)

P_{ny} = nominal axial load strength corresponding to M_{ny}, with bending considered in the direction of the y axis only (Article 8.16.4.3)

P_{nxy} = nominal axial load strength with biaxial loading (Article 8.16.4.3)

P_u = factored axial load at given eccentricity

r = radius of gyration of cross section of a compression member (Article 8.16.5.2.2)

s = spacing of shear reinforcement in direction parallel to the longitudinal reinforcement, in.

s_w = spacing of wires to be developed or spliced, in.

S = span length, ft

V = design shear force at section (Article 8.15.5.1.1)

v = design shear stress at section (Article 8.15.5.1.1)

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

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

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

v_h = permissible horizontal shear stress (Article 8.15.5.5.3)

V_n = nominal shear strength (Article 8.16.6.1)

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

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

V_u = factored shear force at section (Article 8.16.6.1)

w_c = weight of concrete, lb per cu ft

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

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

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

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

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

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

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

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

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

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

ρ (rho) = tension reinforcement ratio = $A_s/b_w d$, A_s/bd

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

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

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

ρ_w = reinforcement ratio used in Equation (8-4) and Equation (8-48)

δ_b = moment magnification factor for members braced against sidesway to reflect effects of member curvature between ends of compression member

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

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

8.1.3 Definitions

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

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

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

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

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

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

Design strength—Nominal strength multiplied by a strength reduction factor, ϕ.

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

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

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

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

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

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

Service load—Loads without load factors.

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

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

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

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

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

8.2 CONCRETE

The specified compressive strength, f'_c, of the concrete for each part of the structure shall be shown on

*American Society for Testing and Materials.

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

the plans. The requirements for f'_c shall be based on tests of cylinders made and tested in accordance with Section 4—Division II.

8.3 REINFORCEMENT

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

8.3.2 Reinforcement to be welded shall be indicated on the plans and the welding procedure to be used shall be specified.

8.3.3 Designs shall not use a yield strength, f_y, in excess of 60,000 psi.

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

8.3.5 Reinforcement shall conform to the specifications listed in Division II, Section 5, except that, for reinforcing bars, the yield strength and tensile strength shall correspond to that determined by tests on full-sized bars.

Part B
ANALYSIS

8.4 GENERAL

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

8.5 EXPANSION AND CONTRACTION

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

12 m

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

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

$\alpha = 1 \times 10^{-5}$ /c conc
$\alpha = 1.2 \times 10^{-5}$ /c steel

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

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

8.6 STIFFNESS

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

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

8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO

8.7.1 The modulus of elasticity, E_c, for concrete may be taken as $w_c^{1.5}\,33\sqrt{f'_c}$ in psi for values of w_c between 90 and 155 pounds per cubic foot. For normal weight concrete ($w_c = 145$ pcf), E_c may be considered as $57{,}000\sqrt{f'_c}$.

$15100\sqrt{f'_c} \rightarrow$ kg/cm²

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

8.7.3 Poisson's ratio may be assumed as 0.2.

8.8 SPAN LENGTH

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

8.8.2 In analysis of continuous and rigid frame members, distances to the geometric centers of members shall be used in the determination of moments. Moments at faces of support may be used for member design. When fillets making an angle of 45 degrees or more with the axis of a continuous or restrained member are built monolithic with the member and support, the face of support shall be considered at a section where the combined depth of the member and fillet is at least one and one-half times the thickness of the member. No portion of a fillet shall be considered as adding to the effective depth.

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

8.9 CONTROL OF DEFLECTIONS

8.9.1 General

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

recommended

8.9.2 Superstructure Depth Limitations

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

8.9.3 Superstructure Deflection Limitations

When making deflection computations, the following criteria are recommended.

$\Delta_{L+I} < \frac{1}{800} L$, $\frac{1}{1000}$ for pedestrians

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

$\Delta_{L+I} < \frac{1}{300} L$

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

8.10 COMPRESSION FLANGE WIDTH

8.10.1 T-Girder

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

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

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

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

8.10.2 Box Girders

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

8.10.2.2 For integral bent caps, see Article 8.10.1.4.

TABLE 8.9.2 Recommended Minimum Depths for Constant Depth Members

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

[a] When variable depth members are used, values may be adjusted to account for change in relative stiffness of positive and negative moment sections.
S = span length as defined in Article 8.8 in feet.

8.11 SLAB AND WEB THICKNESS

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

8.11.2 The thickness of the bottom slab of a box girder shall be not less than 1/16 of the clear span between girder webs or 5 ½ inches, except that the thickness need not be greater than the top slab unless required by design.

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

8.12 DIAPHRAGMS

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

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

8.12.3 Straight box girder bridges and curved box girder bridges with an inside radius of 800 feet or greater do not require intermediate diaphragms. For curved box girder bridges having an inside radius less than 800 feet, intermediate diaphragms are required unless shown otherwise by tests or structural analysis. For such curved box girders, a maximum diaphragm spacing of 40 feet is recommended to assist in resisting torsion.

8.13 COMPUTATION OF DEFLECTIONS

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

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

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

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

where:

$f_r = 2\sqrt{f'c}$ kg/cm²

$$\boxed{M_{cr} = f_r I_g / y_t} \quad (8\text{-}2)$$

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

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

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

(a) Where the immediate deflection has been based on I_g, the multiplication factor for the long-time deflection shall be taken as 4.
(b) Where the immediate deflection has been based on I_e, the multiplication factor for the long-time deflection shall be taken as $3 - 1.2(A'_s/A_s) \ge 1.6$.

Part C
DESIGN

8.14 GENERAL

8.14.1 Design Methods

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

8.14.1.2 All applicable provisions of this specification shall apply to both methods of design, except Articles

3.5 and 3.17 shall not apply for design by STRENGTH DESIGN.

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

8.14.2 Composite Flexural Members

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

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

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

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

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

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

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

8.14.3 Concrete Arches

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

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

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

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

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

TABLE 8.14.3 Effective Length Factors, k

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

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

8.15 SERVICE LOAD DESIGN METHOD (ALLOWABLE STRESS DESIGN)

8.15.1 General Requirements

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

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

8.15.2 Allowable Stresses

8.15.2.1 Concrete

Stresses in concrete shall not exceed the following:

8.15.2.1.1 Flexure $f_r = 2\sqrt{f'_c}$

Extreme fiber stress in compression, f_c .... $0.40f'_c$

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

Modulus of rupture, f_r, from tests, or, if data are not available: $2\sqrt{f'_c}$

Normal weight concrete .... $7.5\sqrt{f'_c}$

"Sand-lightweight" concrete .... $6.3\sqrt{f'_c}$

"All-lightweight" concrete .... $5.5\sqrt{f'_c}$

8.15.2.1.2 Shear

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

8.15.2.1.3 Bearing Stress

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

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

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

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

8.15.2.2 Reinforcement

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

Grade 40 reinforcement ....20,000 psi

Grade 60 reinforcement ....24,000 psi

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

8.15.3 Flexure

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

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

8.15.3.3 In reinforced concrete members, concrete resists no tension.

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

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

8.15.4 Compression Members

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

8.15.5 Shear

8.15.5.1 Shear Stress

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

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

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

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

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

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

8.15.5.2 Shear Stress Carried by Concrete

8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, the allowable shear stress carried by the concrete, v_c, may be

$0.25\sqrt{f_c} \rightarrow kg/cm^2$

taken as $0.95\sqrt{f'_c}$. A more detailed calculation of the allowable shear stress can be made using:

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

Note:

(a) M is the design moment occurring simultaneously with V at the section being considered.
(b) The quantity Vd/M shall not be taken greater than 1.0.

8.15.5.2.2 Shear in Compression Members

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

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

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

8.15.5.2.3 Shear in Tension Members

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

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

Note:

(a) N is negative for tension.
(b) The quantity N/A_g shall be expressed in pounds per square inch.

8.15.5.2.4 Shear in Lightweight Concrete

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

(a) When f_{ct} is specified, the shear stress, v_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
(b) When f_{ct} is not specified, the shear stress, v_c, shall be multiplied by 0.75 for "all-lightweight" concrete, and

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete"—American Concrete Institute 318 Bulletin may be used.

0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

8.15.5.3 Shear Stress Carried by Shear Reinforcement

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

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

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

8.15.5.3.3 When inclined stirrups are used:

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

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

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

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

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

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

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

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

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

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

8.15.5.4 Shear Friction

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

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

8.15.5.4.3 Shear-friction Design Method

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

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

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

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

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

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

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

concrete placed monolithically 1.4λ

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

concrete placed against hardened concrete not intentionally roughened 0.6λ
concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Art. 8.15.5.4.8 ....0.7λ

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

8.15.5.4.4 Shear stress v shall not exceed $0.09f'_c$ nor 360 psi.

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

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

8.15.5.4.7 For the purpose of Article 8.15.5.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, the interface shall be roughened to a full amplitude of approximately ¼ in.

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

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8.15.5.5 Horizontal Shear Design for Composite Concrete Flexural Members

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

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

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

$$v_{dh} = \frac{V}{b_v d} \qquad (8\text{-}11A)$$

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

(a) When the contact surface is clean, free of laitance, and intentionally roughened, shear stress v_h shall not exceed 36 psi.
(b) When minimum ties are provided in accordance with paragraph 8.15.5.5.5, and the contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 36 psi.
(c) When minimum ties are provided in accordance with paragraph 8.15.5.5.5, and the contact surface is clean, free of laitance, and intentionally roughened to a full magnitude of approximately ¼ in., shear stress v_h shall not exceed 160 psi.
(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by 8.15.5.5.5, permissible v_h may be increased by $72f_y/40{,}000$ psi.

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

8.15.5.5.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing s shall not exceed four times the least web width of support element, nor 24 in.
(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed). All ties shall be adequately anchored into interconnected elements by embedment or hooks.

8.15.5.6 Special Provisions for Slabs and Footings

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

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.1 through 8.15.5.3, except at footings supported on piles, the shear on the critical section shall be determined in accordance with Article 4.4.11.3.
(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but not closer than d/2 to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.6.2 and 8.15.5.6.3.

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

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

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

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

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

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

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

(a) Shear stresses computed by Equation (8-12) shall be investigated at the critical section defined in 8.15.5.6.1(b) and at successive sections more distant from the support.
(b) Shear stress v_c at any section shall not exceed 0.9 $\sqrt{f'_c}$ and v shall not exceed $3\sqrt{f'_c}$.
(c) Where v exceeds 0.9 $\sqrt{f'_c}$, shear reinforcement shall be provided in accordance with Article 8.15.5.3.

8.15.5.7 Special Provisions for Slabs of Box Culverts

For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by:

$$v_c = \sqrt{f'_c} + 2{,}200\rho\left(\frac{Vd}{M}\right) \qquad (8\text{-}14)$$

but v_c shall not exceed 1.8 $\sqrt{f'_c}$. For single cell box culverts only, v_c for slabs monolithic with walls need not be taken less than $1.4\sqrt{f'_c}$, and v_c for slabs simply supported need not be taken less than $1.2\sqrt{f'_c}$. The quantity Vd/M shall not be taken greater than 1.0 where M is the moment occurring simultaneously with V at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.15.5.8 Special Provisions for Brackets and Corbels*

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

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

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

(a) Design of shear-friction reinforcement, A_{vf}, to resist shear, V, shall be in accordance with Article 8.15.5.4. For normal weight concrete, shear stress v shall not exceed $0.09f'_c$ nor 360 psi. For "all lightweight" or "sand-lightweight" concrete, shear stress v shall not exceed $(0.09-0.03a_v/d)f'_c$ nor $(360-126a_v/d)$ psi.
(b) Reinforcement A_f to resist moment $[Va_v + N_c(h - d)]$ shall be computed in accordance with Articles 8.15.2 and 8.15.3.
(c) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c/f_s$. Tensile force N_c shall not be taken less than 0.2V unless special provisions are made to avoid tensile forces.
(d) Area of primary tension reinforcement, A_s, shall be made equal to the greater of (A_f+A_n), or $(2A_{vf}/3+A_n)$.

8.15.5.8.4 Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5(A_s-A_n)$, shall be uni-

*These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83," contains an example design of beam ledges—Part 16, example 16-3.

formly distributed within two-thirds of the effective depth adjacent to A_s.

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

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

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;
(b) bending primary tension bars A_s back to form a horizontal loop; or
(c) some other means of positive anchorage.

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

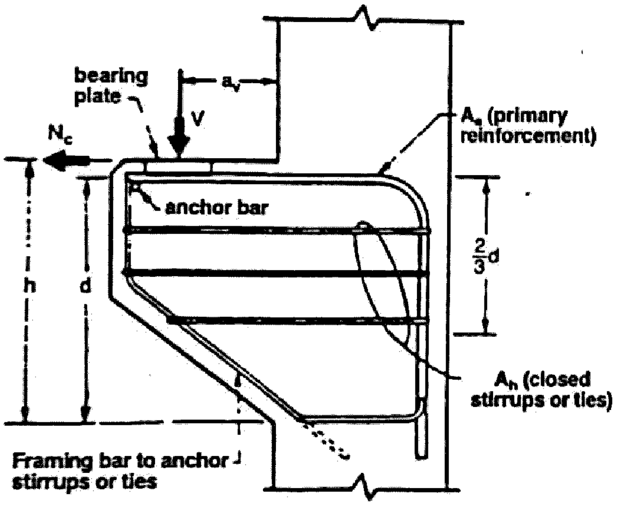

FIGURE 8.15.5.8

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8.16 STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN)

8.16.1 Strength Requirements

8.16.1.1 Required Strength

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

8.16.1.2 Design Strength

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

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

7.6.2(B)6a

(a) Flexure . . . . $\phi = 0.90$
(b) Shear . . . . $\phi = 0.85$
(c) Axial compression with—
Spirals . . . . $\phi = 0.75$
Ties. . . . . $\phi = 0.70$
(d) Bearing on concrete. . . . . $\phi = 0.70$

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

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

8.16.2 Design Assumptions

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

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

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

$\varepsilon_{max} = 0.003$

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

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

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

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

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

8.16.3 Flexure

8.16.3.1 Maximum Reinforcement of Flexural Members

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

8.16.3.1.2 Balanced strain conditions exist at a cross section when the tension reinforcement reaches the strain corresponding to its specified yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

8.16.3.2 Rectangular Sections with Tension Reinforcement Only

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

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

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

where,

$$a = \frac{A_s f_y}{0.85\ f'_c b} \qquad (8\text{-}17)$$

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

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

8.16.3.3 Flanged Sections with Tension Reinforcement Only

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

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

$$\phi M_n = \phi[(A_s - A_{sf})f_y(d-a/2) + A_{sf}f_y\,(d-0.5h_f)] \qquad (8\text{-}19)$$

where,

$$A_{sf} = \frac{0.85 f'_c\,(b-b_w)h_f}{f_y} \qquad (8\text{-}20)$$

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

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

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

where,

$$\rho_f = \frac{A_{sf}}{b_w d} \qquad (8\text{-}23)$$

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

8.16.3.4 Rectangular Sections with Compression Reinforcement

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

If
$$\left(\frac{A_s - A_s'}{bd}\right) \geq 0.85\beta_1\left(\frac{f_c' d'}{f_y d}\right)\left(\frac{87{,}000}{87{,}000 - f_y}\right) \quad (8\text{-}24)$$

then,

$$\phi M_n = \phi[(A_s - A_s')f_y(d - a/2) + A_s' f_y (d - d')] \quad (8\text{-}25)$$

where,

$$a = \frac{(A_s - A_s')f_y}{0.85 f_c' b} \quad (8\text{-}26)$$

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

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

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

where,

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

8.16.3.5 Other Cross Sections

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

8.16.4 Compression Members

8.16.4.1 General Requirements

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

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

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

$$P_{n(max)} = 0.85[0.85 f_c' (A_g - A_{st}) + f_y A_{st}] \quad (8\text{-}29)$$
$$\phi = 0.75$$

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

$$P_{n(max)} = 0.80[0.85 f_c' (A_g - A_{st}) + f_y A_{st}] \quad (8\text{-}30)$$
$$\phi = 0.70$$

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

8.16.4.2 Compression Member Strengths

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

8.16.4.2.1 Pure Compression

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

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

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

8.16.4.2.2 Pure Flexure

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

8.16.4.2.3 Balanced Strain Conditions

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

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

and,

$$\phi M_b = \phi[0.85 f'_c b a_b (d - d'' - a_b/2) + A'_s f'_s (d - d' - d'') + A_s f_y d''] \quad (8\text{-}33)$$

where,

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

and,

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

8.16.4.2.4 Combined Flexure and Axial Load

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

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

8.16.4.3 Biaxial Loading

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

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

when the factored axial load,

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

or,

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

when the factored axial load,

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

8.16.5 Slenderness Effects in Compression Members

8.16.5.1 General Requirements

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

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

8.16.5.2 Approximate Evaluation of Slenderness Effects

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

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

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

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

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

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

8.16.5.2.7 Compression members shall be designed using the factored axial load P_u, derived from a conventional elastic analysis and a magnified factored moment, M_c, defined by:

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

where,

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

$$\delta_s = \frac{1}{1-\dfrac{\Sigma P_u}{\phi \Sigma P_c}} \geq 1.0 \qquad (8\text{-}41A)$$

and,

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad (8\text{-}42)$$

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

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

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1+\beta_d} \qquad (8\text{-}43)$$

or conservatively as,

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

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

$$C_m = 0.6 + 0.4\,(M_{1b}/M_{2b}) \qquad (8\text{-}45)$$

but not less than 0.4.

For all other cases, C_m shall be taken as 1.0.

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

(a) When the computed end eccentricities are less than (0.6 + 0.03h) inches, the computed end moments may be used to evaluate M_{1b}/M_{2b} in Equation (8-45).
(b) If computations show that there is essentially no moment at either end of the member, the ratio M_{1b}/M_{2b} shall be equal to one.

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

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

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

8.16.6 Shear

8.16.6.1 Shear Strength

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

$$V_u \leq \phi V_n \qquad (8\text{-}46)$$

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

$$V_n = V_c + V_s \qquad (8\text{-}47)$$

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

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

8.16.6.2 Shear Strength Provided by Concrete

8.16.6.2.1 Shear in Beams and One-Way Slabs and Footings

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

0.5 1134

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

or, $V_c = 0.53\sqrt{f'_c}\; b_w d$

$$\boxed{V_c = 2\sqrt{f'_c}\,b_w d} \qquad (8\text{-}49)$$

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

Note: $V_c < 0.93\sqrt{f'_c}\; b_w d$

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

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

8.16.6.2.2 Shear in Compression Members

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

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

or,

$$V_c = 2\sqrt{f'_c}\,b_w d \qquad (8\text{-}51)$$

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

8.16.6.2.3 Shear in Tension Members

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

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

Note:
(a) N_u is negative for tension.
(b) The quantity N_u/A_g shall be expressed in pounds per square inch.

8.16.6.2.4 Shear in Lightweight Concrete

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

(a) When f_{ct} is specified, the shear strength, V_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
(b) When f_{ct} is not specified, V_c shall be multiplied by 0.75 for "all lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

8.16.6.3 Shear Strength Provided by Shear Reinforcement

8.16.6.3.1 Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided

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

to satisfy Equations (8-46) and (8-47), but not less than that required by Article 8.19. Shear strength V_s shall be computed in accordance with Articles 8.16.6.3.2 through 8.16.6.3.10.

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

$$V_s = \frac{A_v f_y d}{s} \qquad (8-53)$$

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

8.16.6.3.3 When inclined stirrups are used:

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

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

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

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

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

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

$2\sqrt{f_c} = 1.06\sqrt{f'_c}$

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

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

$4V_c = 2.12\sqrt{f'_c}\, b_w d$

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

8.16.6.4 Shear Friction

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

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

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

8.16.6.4.4 Shear-friction Design Method

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

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

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

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

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

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

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

Concrete placed monolithically 1.4λ
Concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.16.6.4.8 . . . . .1.0λ
Concrete placed against hardened concrete not intentionally roughened 0.6λ
Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.16.6.4.9) 0.7λ

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

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

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

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

8.16.6.4.8 For the purpose of Article 8.16.6.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, the interface shall be roughened to a full amplitude of approximately 1/4 inch.

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

8.16.6.5 Horizontal Shear Strength for Composite Concrete Flexural Members

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

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

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

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

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

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$, in pounds.
(b) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than 80 b_vd, in pounds.
(c) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contract surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately ¼ in., shear strength V_{nh} shall not be taken greater than $350b_vd$, in pounds.
(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by 8.16.6.5.5, shear strength V_{nh} may be increased by $(160f_y/40{,}000)b_vd$, in pounds.

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

8.16.6.5.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_vs/f_y$, and tie spacing, s, shall not exceed four times the least web width of the support element, nor 24 in.
(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks.

8.16.6.6 Special Provisions for Slabs and Footings

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

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.1 through 8.16.6.3 except at footings supported on piles, the shear on the critical section shall be determined in accordance with Article 4.4.11.3.

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

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

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

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

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

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

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

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

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

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

8.16.6.7 Special Provisions for Slabs of Box Culverts

8.16.6.7.1 For slabs of box culverts under 2 feet or more fill, shear strength V_c may be computed by:

$$V_c = \left(2.14\sqrt{f'_c} + 4{,}600\,\rho\frac{V_u d}{M_u}\right) bd \qquad (8\text{-}59)$$

but V_c shall not exceed $4\sqrt{f'_c}$ bd. For single cell box culverts only, V_c for slabs monolithic with walls need not be taken less than $3\sqrt{f'_c}$ bd, and V_c for slabs simply supported need not be taken less than $2.5\sqrt{f'_c}$ bd. The quantity V_ud/M_u shall not be taken greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.16.6.8 Special Provisions for Brackets and Corbels*

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

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

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

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

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

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

(d) Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \le \phi A_nf_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc}

*These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83" contains an example design of beam ledges—Part 16, example 16-3.

shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.
(e) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or:

$$\frac{2A_{vf}}{3} + A_n.$$

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

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

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

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars,
(b) bending primary tension bars A_s back to form a horizontal loop, or
(c) some other means of positive anchorage.

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

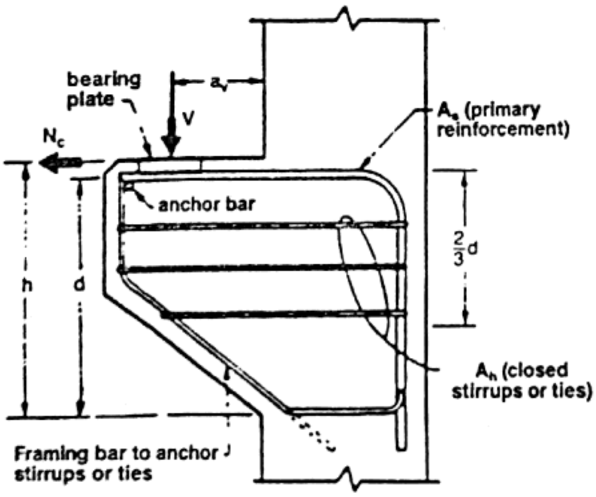

FIGURE 8.16.6.8

8.16.7 Bearing Strength

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

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

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

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

8.16.8 Serviceability Requirements

8.16.8.1 Application

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

8.16.8.2 Service Load Stresses

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

8.16.8.3 Fatigue Stress Limits

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

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

where:

f_f = stress range in kips per square inch;

f_{min} = algebraic minimum stress level, tension positive, compression negative in kips per square inch;

r/h = ratio of base radius to height of rolled-on transverse deformations; when the actual value is not known, use 0.3.

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

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

8.16.8.4 Distribution of Flexural Reinforcement

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

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

where:

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

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

The quantity z in Equation (8-61) shall not exceed 170 kips per inch for members in moderate exposure conditions and 130 kips per inch for members in severe exposure conditions. Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, protection should be provided by increasing the denseness or imperviousness to water or furnishing other protection such as a waterproofing protecting system, in addition to satisfying Equation (8-61).

Part D
REINFORCEMENT

8.17 REINFORCEMENT OF FLEXURAL MEMBERS

8.17.1 Minimum Reinforcement

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

$$\phi M_n \geq 1.2\, M_{cr} \qquad (8\text{-}62)$$

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

8.17.2 Distribution of Reinforcement

8.17.2.1 Flexural Tension Reinforcement in Zones of Maximum Tension

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

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

8.17.2.1.3 If the depth of the side face of a member exceeds 3 feet, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement. The area of skin reinforcement A_{sk} per foot of height on each side face shall be $\leq 0.012\ (d - 30)$. The maximum spacing of skin reinforcement shall not exceed the lesser of d/6 and 12 inches. Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

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

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

8.17.2.3 Bottom Slab Reinforcement for Box Girders

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

8.17.2.3.2 Minimum distributed reinforcement of 0.5 percent of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 18 inches. All transverse reinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook.

8.17.3 Lateral Reinforcement of Flexural Members

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

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

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

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

8.18 REINFORCEMENT OF COMPRESSION MEMBERS

8.18.1 Maximum and Minimum Longitudinal Reinforcement

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

8.18.1.2 The minimum area of longitudinal reinforcement shall not be less than 0.01 times the gross area, A_g, of the section. When the cross section is larger than that required by consideration of loading, a reduced effective area may be used. The reduced effective area shall not be less than that which would require 1 percent of longitudinal reinforcement to carry the loading. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bars shall be No. 5.

8.18.2 Lateral Reinforcement

8.18.2.1 General

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

8.18.2.2 Spirals

Spiral reinforcement for compression members shall conform to the following:

8.18.2.2.1 Spirals shall consist of evenly spaced continuous bar or wire, with a minimum diameter of ⅜ inch.

8.18.2.2.2 The ratio of spiral reinforcement to total volume of core, ρ_s, shall not be less than the value given by:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (8\text{-}63)$$

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

8.18.2.2.3 The clear spacing between spirals shall not exceed 3 inches or be less than 1 inch or 1⅓ times the maximum size of coarse aggregate used.

8.18.2.2.4 Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral bar or wire at each end of a spiral unit.

8.18.2.2.5 Spirals shall extend from top of footing or other support to the level of the lowest horizontal reinforcement in members supported above.

8.18.2.2.6 Splices in spiral reinforcement shall be lap splices of 48 bar or wire diameters but not less than 12 inches, or shall be welded.

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

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

8.18.2.3 Ties

Tie reinforcement for compression members shall conform to the following:

8.18.2.3.1 All bars shall be enclosed by lateral ties which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area may be used instead of bars.

8.18.2.3.2 The spacing of ties shall not exceed the least dimension of the compression member or 12 inches. When two or more bars larger than No. 10 are bundled together, tie spacing shall be one-half that specified above.

8.18.2.3.3 Ties shall be located not more than half a tie spacing from the face of a footing or from the nearest longitudinal reinforcement of a cross-framing member.

8.18.2.3.4 No longitudinal bar shall be more than 2 feet, measured along the tie, from a restrained bar on either side. A restrained bar is one which has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie may be used.

8.18.2.4 Seismic Requirements

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

8.19 LIMITS FOR SHEAR REINFORCEMENT

8.19.1 Minimum Shear Reinforcement

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

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

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

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

$$A_v = \frac{50 b_w s}{f_y} = \frac{3.5 b_w s}{f_y} \qquad (8\text{-}64)$$

حداقل آرماتور برشی

$A_v > A_{v_{min}}$

where b_w and s are in inches.

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

8.19.2 Types of Shear Reinforcement

8.19.2.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to the axis of the member or making an angle of 45 degrees or more with the longitudinal tension reinforcement.
(b) Welded wire fabric with wires located perpendicular to the axis of the member.
(c) Longitudinal reinforcement with a bent portion making an angle of 30 degrees or more with the longitudinal tension reinforcement.
(d) Combinations of stirrups and bent longitudinal reinforcement.
(e) Spirals.

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

8.19.3 Spacing of Shear Reinforcement

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

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8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1 Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least ⅛ square inch per foot in each direction.

حداقل $2.7\ cm^2$

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

$\leq \min\{3h, 45\ cm\}$

8.21 SPACING LIMITS FOR REINFORCEMENT

8.21.1 For cast-in-place concrete the clear distance between parallel bars in a layer shall not be less than 1.5 bar diameters, 1.5 times the maximum size of the coarse aggregate, or 1½ inches.

Max 1.5 1.5

8.21.2 For precast concrete (manufactured under plant control conditions) the clear distance between parallel bars in a layer shall be not less than 1 bar diameter, 1⅓ times the maximum size of the coarse aggregate, or 1 inch.

d_b, 2.5 cm, 1.33

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

2.5 cm

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

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

8.21.6 In walls and slabs the primary flexural reinforcement shall be spaced not farther apart than 1.5 times the wall or slab thickness, or 18 inches.

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8.22 PROTECTION AGAINST CORROSION

8.22.1 The following minimum concrete cover shall be provided for reinforcement:

	Minimum Cover (inches)	
Concrete cast against and permanently exposed to earth	3	7.5
Concrete exposed to earth or weather:		
Primary reinforcement	2	5 cm
Stirrups, ties, and spirals	1½	3.8 cm
Concrete deck slabs in mild climates:		
Top reinforcement	2	5 cm
Bottom reinforcement	1	2.5 cm
Concrete deck slabs which have no positive corrosion protection and are frequently exposed to deicing salts:		
Top reinforcement	2½	6.4 cm
Bottom reinforcement	1	2.5 cm
Concrete not exposed to weather or in contact with ground:		
Primary reinforcement	1½	3.8 cm
Stirrups, ties, and spirals	1	2.5 cm
Concrete piles cast against and/or permanently exposed to earth	2	5 cm

8.22.2 For bundled bars, the minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches, except for concrete cast against and permanently exposed to earth in which case the minimum cover shall be 3 inches.

8.22.3 In corrosive or marine environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, by increasing the denseness and imperviousness to water of the protecting concrete or other means. Other means of positive corrosion protection may consist of, but not be limited to, epoxy-coated bars, special concrete overlays, and impervious membranes; or a combination of these means.*

8.22.4 Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

8.23 HOOKS AND BENDS

8.23.1 Standard Hooks

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

(1) 180-deg bend plus $4d_b$ extension, but not less than 2½ in. at free end of bar.

(2) 90-deg bend plus $12d_b$ extension at free end of bar.

cm (3) For stirrup and tie hooks:

(a) No. 5 bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar, or

(b) No. 6, No. 7, and No. 8 bar, 90-deg bend plus $12d_b$ extension at free end of bar, or

(c) No. 8 bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

8.23.2 Minimum Bend Diameters

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

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

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

8.24 DEVELOPMENT OF FLEXURAL REINFORCEMENT

8.24.1 General

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

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

TABLE 8.23.2.1 Minimum Diameters of Bend

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

*For additional information on corrosion protection methods, refer to National Cooperative Highway Research Report 297, "Evaluation of Bridge Deck Protective Strategies."

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

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

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

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

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

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

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

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

8.24.2 Positive Moment Reinforcement

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

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

8.24.2.3 At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by Article 8.25 satisfies Eq. (8-65); except Eq. (8-65) need not be satisfied for reinforcement terminating beyond center line of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$\ell_d \le \frac{M}{V} + \ell_a \qquad (8\text{-}65)$$

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

8.24.3 Negative Moment Reinforcement

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

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

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

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8.25 DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

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

8.25.1 The basic development length shall be:

No. 11 bars and smaller .... $\dfrac{0.04A_bf_y}{\sqrt{f'_c}}$

but not less than .... $0.0004d_bf_y$

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

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

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

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

8.25.2.1 Top reinforcement so placed that more than 12 inches of concrete is cast below the reinforcement 1.4

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

but not less than 1.0

When f_{ct} is not specified

"all lightweight" concrete 1.33

"sand lightweight" concrete 1.18

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

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

less than $6d_b$ 1.5

All other cases 1.15

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

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

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

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

$(A_s \text{ required})/(A_s \text{ provided})$

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

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

8.26 DEVELOPMENT OF DEFORMED BARS IN COMPRESSION

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

8.26.1 The basic development length shall be .... $0.02d_bf_y/\sqrt{f'_c}$

but not less than .... $0.0003d_bf_y$

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

8.26.2.1 Anchorage or development for reinforcement strength is not specifically required, or reinforcement is in excess of that required by analysis .... $(A_s \text{ required})/(A_s \text{ provided})$

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

8.27 DEVELOPMENT OF SHEAR REINFORCEMENT

8.27.1 Shear reinforcement shall extend at least to the centroid of the tension reinforcement, and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its design yield strength. For composite flexural members, all beam shear reinforcement shall be extended into the deck slab or otherwise shall be adequately anchored to assure full beam design shear capacity.

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

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

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

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

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

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

8.27.2.4.2 One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced at least 2 inches from the first wire. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of bend of not less than 8-wire diameters.

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

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

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

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

8.28 DEVELOPMENT OF BUNDLED BARS

The development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

8.29 DEVELOPMENT OF STANDARD HOOKS IN TENSION

8.29.1 Development length ℓ_{dh} in inches, for deformed bars in tension terminating in a standard hook (Article 8.23.1) shall be computed as the product of the basic development length ℓ_{hb} of Article 8.29.2 and the applicable modification factor or factors of Article 8.29.3, but ℓ_{dh} shall not be less than $8d_b$ or 6 inches, whichever is greater.

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

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

8.29.3.1 Bar yield strength:
Bars with f_y other than 60,000 psi .... $f_y/60{,}000$

8.29.3.2 Concrete cover:
For No. 11 bar and smaller, side cover (normal to plane of hook) not less than 2½ in.,

and for 90-deg hook, cover on bar extension beyond hook not less than 2 in. 0.7

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

8.29.3.4 Excess reinforcement:
Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis . . . .(A_s required)/(A_s provided)

8.29.3.5 Lightweight aggregate concrete 1.3

8.29.3.6 Epoxy-coated reinforcement hooked bars with epoxy coating 1.2

8.29.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2½ in., hooked bar shall be enclosed within ties or stirrups spaced along the full development length ℓ_{dh}, not greater than $3d_b$,

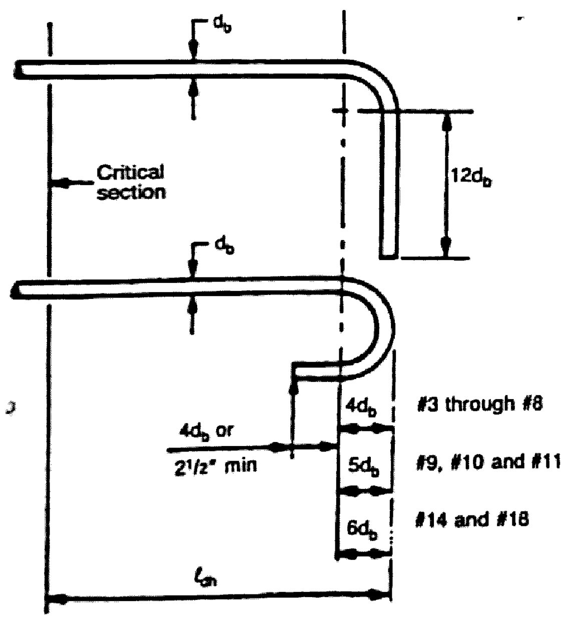


FIGURE 8.29.1 Hooked-Bar Details for Development of Standard Hooks

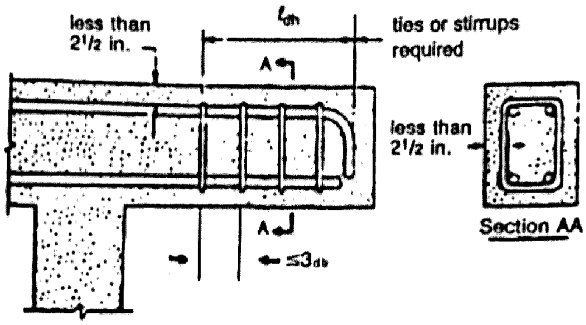


FIGURE 8.29.4 Hooked-Bar Tie Requirements

where d_b is the diameter of the hooked bar. For this case, the factor of Article 8.29.3.3 shall not apply.

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

8.30 DEVELOPMENT OF WELDED WIRE FABRIC IN TENSION

8.30.1 Deformed Wire Fabric

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

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

$$0.03d_b\,(f_y - 20{,}000)/\sqrt{f'_c}\,* \qquad (8\text{-}66)$$

but not less than,

$$0.20\,\frac{A_w}{s_w}\cdot\frac{f_y}{\sqrt{f'_c}} \qquad (8\text{-}67)$$

8.30.1.3 The basic development length of welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 8.25.

*The 20,000 has units of psi.

8.30.2 Smooth Wire Fabric

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

$$0.27\frac{A_w}{s_w}\cdot\frac{f_y}{\sqrt{f'_c}} \qquad (8\text{-}68)$$

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

8.31 MECHANICAL ANCHORAGE

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

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

8.32 SPLICES OF REINFORCEMENT

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

8.32.1 Lap Splices

8.32.1.1 Lap splices shall not be used for bars larger than No. 11, except as provided in Articles 8.32.4.1 and 4.4.11.4.1.

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

8.32.1.3 Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than ⅕ the required length of lap or 6 inches.

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

8.32.2 Welded Splices and Mechanical Connections

8.32.2.1 Welded splices or other mechanical connections may be used. Except as provided herein, all welding shall conform to the latest edition of the American Welding Society publication, "Structural Welding Code Reinforcing Steel."

8.32.2.2 A full-welded splice shall have bars butted and welded to develop in tension at least 125 percent of the specified yield strength of the bar.

8.32.2.3 A full-mechanical connection shall develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

8.32.2.4 Welded splices and mechanical connections not meeting requirements of Articles 8.32.2.2 and 8.32.2.3 may be used in accordance with Article 8.32.3.4.

8.32.3 Splices of Deformed Bars and Deformed Wire in Tension

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

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

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

8.32.3.3 Welded splices or mechanical connections used where the area of reinforcement provided is less than twice that required by analysis shall meet the requirements of Article 8.32.2.2 or 8.32.2.3.

TABLE 8.32.3.2 Tension Lap Splices

	Maximum Percent of A_s Spliced within Required Lap Length		
(A_s provided)/(A_s required)*	50	75	100
Equal to or Greater than 2	Class A	Class A	Class B
Less than 2	Class B	Class C	Class C

*Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

8.32.3.4 Welded splices or mechanical connections used where the area of reinforcement provided is at least twice that required by analysis shall meet the following:

8.32.3.4.1 Splices shall be staggered at least 24 inches and in such manner as to develop at every section at least twice the calculated tensile force at that section but not less than 20,000 psi for the total area of reinforcement provided.

8.32.3.4.2 In computing tensile force developed at each section, spliced reinforcement may be rated at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to ℓ_d required to develop the specified yield strength f_y.

8.32.3.5 Splices in tension tie members shall be made with a full-welded splice or a full-mechanical connection in accordance with Article 8.32.2.2 or 8.32.2.3. Splices in adjacent bars shall be staggered at least 30 inches.

8.32.4 Splices of Bars in Compression

8.32.4.1 Lap Splices in Compression

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

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

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

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

8.32.4.2 End-Bearing Splices

In bars required for compression only, the compressive stress may be transmitted by bearing of square cut ends held in concentric contact by a suitable device. Bar ends shall terminate in flat surfaces within 1½ degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly. End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

8.32.4.3 Welded Splices or Mechanical Connections

Welded splices or mechanical connections used in compression shall meet the requirements of Article 8.32.2.2 or 8.32.2.3.

8.32.5 Splices of Welded Deformed Wire Fabric in Tension

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

8.32.5.2 Lap splices of welded deformed wire fabric, with no cross wires within the lap splice length, shall be determined as for deformed wire in accordance with Article 8.32.3.1.

8.32.6 Splices of Welded Smooth Wire Fabric in Tension

The minimum length of lap for lap splices of welded smooth wire fabric shall be in accordance wi'h the following:

8.32.6.1 When the area of reinforcement provided is less than twice that required by analysis at the splice location, the length of overlap measured between the outermost cross wires of each fabric sheet shall not be less than one spacing of cross wires plus 2 inches or less than 1.5 ℓ_d, or 6 inches.

8.32.6.2 When the area of reinforcement provided is at least twice that required by analysis at the splice location, the length of overlap measured between the outermost cross wires of each fabric sheet shall not be less than 1.5 ℓ_d or 2 inches.

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_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, or 10.53.1.2, and 10.56.3)
A_{fc} = area of compression flange (Article 10.48.4.1)
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_w = area of web of beam (Article 10.53.1.2)
a = distance from center of bolt under consideration to edge of plate in inches (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 Article 10.34.2.1.3)
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, 10.37.3.2, 10.39.4.5.1, 10.48.1, 10.48.2, 10.48.5.3, 10.50, 10.51.5.5, and 10.55.3)
C = web buckling coefficient (Articles 10.34.4, 10.48.5.3, 10.48.8, and 10.50(e))
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, Article 10.48.4.1)
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 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48.1, 10.48.2, 10.48.5, 10.48.6, 10.48.8, 10.49.2, 10.49.3.2, 10.50 (d), 10.50.1.1.2, 10.50.2.1, and 10.55.2)

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 10.48.2.1(b), 10.48.4.1, 10.49.2, 10.49.3 and, 10.50(d))

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 10.50.1.1.2 and 10.50.2.1)

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. (Article 10.13, Table 10.32.1A, Articles 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)

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)

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 10.37.1.2 and 10.55.1)

F_{cr} = buckling stress of the compression flange plate or column (Articles 10.51.1, 10.51.5, 10.54.1.1, and 10.54.2.1)

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_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_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_y^r = specified minimum yield point of the reinforcing steel (Articles 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 10.32.1A and 10.32.3B, Article 10.18.4)

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

F_v = allowable shear stress (Tables 10.32.1A, 10.32.3B and Articles 10.32.2, 10.32.3, 10.34.4, 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_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, 10.38.5, 10.39.4, 10.40.2.2, 10.41.4.6, 10.46, 10.48, 10.49, 10.50, 10.51.5, and 10.54)

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

F_{yw} = specified minimum yield strength of the web (Article 10.53.1)

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'_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_{dl1} = top flange compressive stress due to non-composite dead load (Article 10.34.2.1, 10.34.2.2, and 10.50(c))

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

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

f_v = unit shear stress (Articles 10.32.3.2.3 and 10.34.4.4)

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

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

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 10.34.4, 10.39.4.3, 10.48.8, and 10.51.5.4)

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)

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

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 = moments at the ends of a member

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

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 10.48.4.1 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 10.48, 10.50.1, 10.50.2, 10.51.1, 10.53.1, and 10.54.2.1)

M_y = moment capacity at first yield (Article 10.50.1.1.2)

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_s = allowable slip resistance (Article 10.32.3.2.1)
- P_u = maximum axial compression capacity (Article 10.54.1.1)
- 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 10.40.2.1.1, 10.53.1.2, and 10.53.1.3)
- R_b = bending capacity reduction factor (Articles 10.48.4.1 and 10.53.1.3)
- 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 (Article 10.36)
- r_y = radius of gyration with respect to the Y-Y axis (Article 10.48.1.1)
- r' = radius of gyration in inches of the compression flange about the axis in the plane of the web (Table 10.32.1A, 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, Article 10.48.4.1)
- 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 10.34.2.1, 10.39.4.2, 10.48.1.1, 10.48.2.1, 10.50, and 10.51.5.1)
- 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 (Article 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, 10.50.1.1.2)
- t_w = web thickness, in (Articles 10.15.2.1, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48, 10.49.2, 10.49.3, 10.55.2, and 10.56.3)
- 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_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 10.34.4, 10.48.5.3, 10.48.8, and 10.53.1.4)
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)
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_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 = ratio of web plate yield strength to stiffener plate yield strength (Articles 10.34.4 and 10.48.5.3)
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)
β = area of the web divided by the area of the tension flange (Articles 10.40.2 and 10.53.1.2)
ρ = Fyw/Fyf (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 may conform to one of the designations listed below and in Table 10.2B, in addition to the designations listed in Table 10.2A.

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).

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.*

TABLE 10.2A

Type	Minimum Material Properties Structural Steel: Structural Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High-Yield Strength, Quenched and Tempered Alloy Steel	
AASHTO Designation[a,c]	M 270 Grade 36	M 270 Grade 50	M 270 Grade 50W	M 270 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 70W	A 709 Grades 100/100W[b]	
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 2 1/2 in. to 4 in. incl.
Shapes[d]	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]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 A709 Grades 100/100W.

[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] 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.

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

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[b] Class G
Minimum Yield Point, psi F_y	36,000[a]	33,000	37,500	50,000	50,000

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

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

10.2.6 Cast Steel, Ductile Iron Castings, Malleable Castings, Cast Iron, and Bronze or Copper Alloy

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.

10.2.6.4 Bronze or Copper-Alloy

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

Part B
DESIGN DETAILS

10.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

10.3.1 Allowable Fatigue Stress

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 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.

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

تنش های مجاز خستگی برای ترکیبات بارهایی دارای بار زنده و باد اعمال

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.

برای بار باد تعداد سیکل تنش ۱۰۰۰۰۰ استفاده میشود

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.

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

***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*

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

×70.3 = kg/cm²

Nonredundant Load Path Structures

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

*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.

[a]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.

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

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

[d]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.

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.

10.3.3.4 M 270 Grades 100/100W steel shall be supplied to Zone 2 requirements as a minimum.

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.

10.5 DEPTH RATIOS

$\frac{D}{L} \geq \frac{1}{25}$

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. $\frac{D}{L} \geq \frac{1}{25}$

10.5.3 For trusses the ratio of depth to length of span preferably should not be less than 1/10. $\frac{D}{L} \geq \frac{1}{10}$

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.*

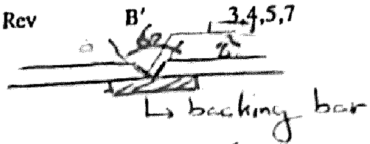
10.6 DEFLECTION

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.

*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 f)	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. 20 mm	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[b]	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



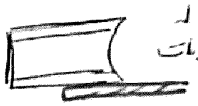


TABLE 10.3.1B (Continued)

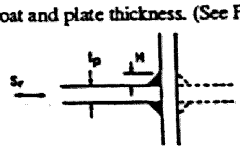
General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness 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[b,c]	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. 1.25 cm	T or Rev	C	14
	(b) Detail thickness > 0.5 in.	T or Rev	See Note[d]	
	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[b,c,e]	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[b,e]	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]"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.
[c]Transversely loaded partial penetration groove welds are prohibited.
[d]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.)

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



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.
[e]Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.
[f]See Wattar, Albrecht and Sahli, Journal of Structural Engineering, ASCE, Vol. III, No. 6, June 1985, pp. 1235–1249.

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.

$\Delta_{L+I} < \frac{L}{800} \rightarrow m$ $\Delta_{L+I} < \frac{L}{1000} \rightarrow m$ for pedestrians

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.

$\Delta_{L+I} < \frac{L}{300} \rightarrow$

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.

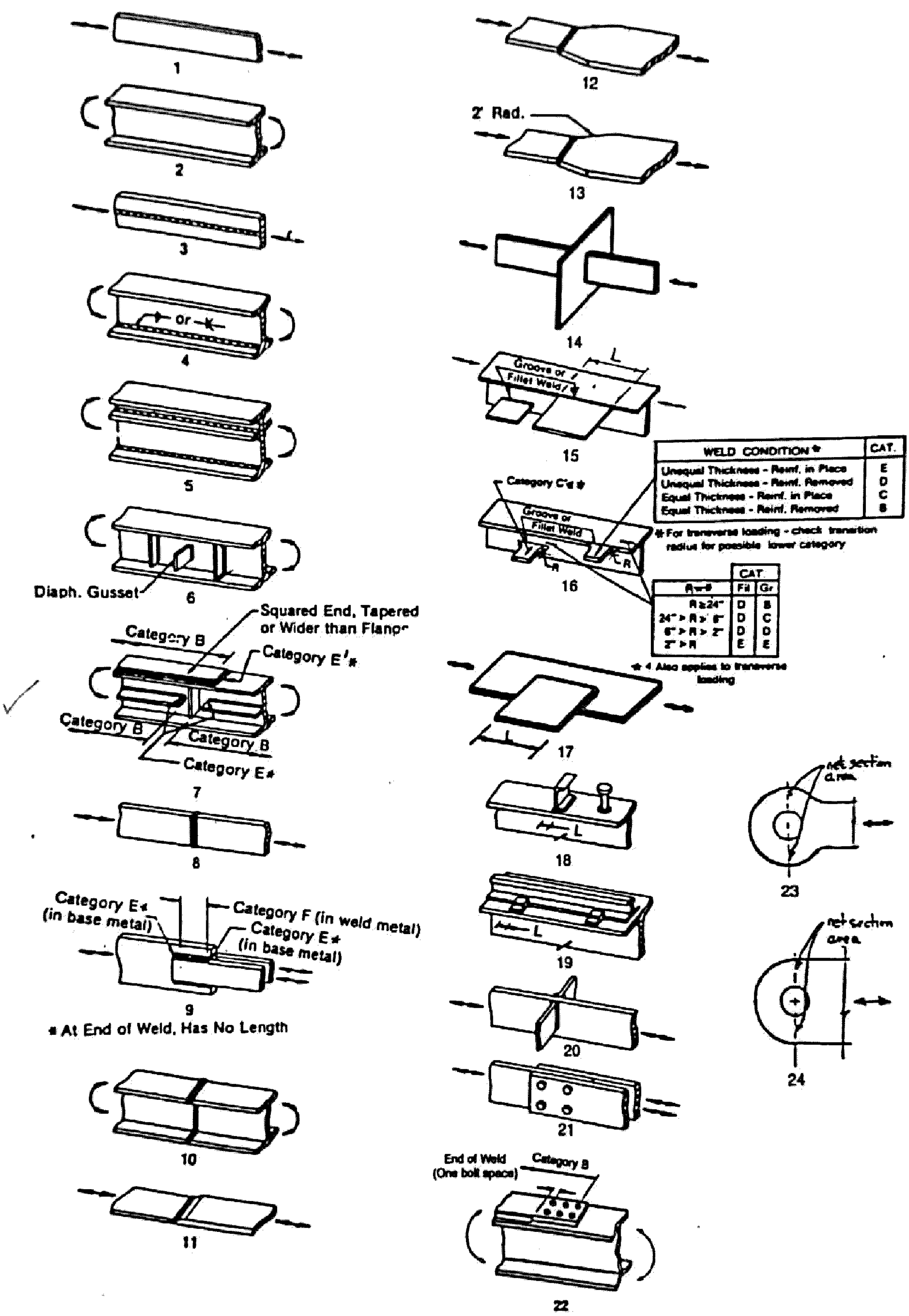


FIGURE 10.3.1C Illustrative Examples

TABLE 10.3.2A Stress Cycles

Main (Longitudinal) Load Carrying Members

Type of Road	Case	ADTT[a]	Truck Loading	Lane Loading[b]
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.

10.6.5 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 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.*

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

kL < 12.

*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 10.3.3A 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

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.

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.

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

8 mm

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.

5.8 mm

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 angel 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 percent 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.

90 m

30 m

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.

روابط ورق‌های پوششی

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.

$L_{cp} \geq (2d + 0.9)$ m

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.

> 2 cm

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.

single: $t_{cp} \leq 2\, t_{flange}$

$\Sigma t_{cp} \leq 2.5\, t_{flange}$

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 theoret-

- تعداد قطع کاور پلیت‌ها حداقل ۲ برابر عرض c.p. است در صورتیکه با استفاده از طول درز جوش داده شود
- " " تنها ... 1.5 برابر عرض c.p است ورق ورق ... پیوسته جوش داده شود
- در انتهای قطع شده جوش c.p باید بصورت پیوسته اجرا شود - جوش انتهایی c.p به تیر باید پیوسته باشد و ... تا ... گردد
- عرض c.p. معمولاً کمتر از عرض بال تیر است ... باید ... از ... باشد
- در انتهای c.p ... شود

ical 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 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. برای بارهای

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 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:

$$R = \frac{14bD}{\sqrt{F_y}\,\psi\, t_w} \qquad (10\text{-}1)$$

$$R = \frac{7{,}500b}{F_y \psi} \qquad (10\text{-}2)$$

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:

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

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

$\Delta_R = 0$ 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 non-symmetrical 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 percent) 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 percent 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 degrees for double lacing and 60 degrees 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 percent.*

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.

*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.

10.16.14.5 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 The strength of members connected by high-strength bolts and rivets shall be determined by the gross section for compression members. For members primarily in bending, the gross section shall also be used, except that if more than 15 percent of each flange area is removed, that amount removed in excess of 15 percent shall be deducted from the gross area. In no case shall the design tensile stress on the net section exceed 0.50 F_u, when using service load design method or 1.0 F_u, when using strength design method, where F_u equals the minimum tensile strength of the steel, except that for M 270 Grades 100/100W steels the design tensile stress on the net section shall not exceed 0.46 F_u when using the service load design method. Splices may be made with rivets, by high-strength bolts, or by the use of welding. Splices, whether in tension, compression, bending, or shear, 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 at the point of splice and the allowable stress of the member at the same point but, in any event, not less than 75 percent of the allowable stress in the member. Splices in the case of strength design method shall be designed for not less than the average of the required strength at the point of splice and the strength of the member at the same point but, in any event, not less than 75 percent of the strength of the member. Where a section changes at a splice, the small section is to be used for the above splice requirements.

10.18.1.2 If splice plates are not in direct contact with the parts which they connect, the number of fasteners on each side of the joint shall be in excess of the number required for a direct contact splice to the extent of at least two extra transverse lines of fasteners for each intervening plate, except as provided in Article 10.18.1.3 and 10.18.6.

10.18.1.3 Fillers in high strength bolted slip-critical connections need not be extended and developed, but eccentricity of forces at short, thick fillers must be considered.

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

10.18.2 Beams and Girders

10.18.2.1 Web splice plates and their connections shall be designed for the portion of the design moment resisted by the web and for the moment due to eccentricity of the shear introduced by the splice connection. Web plates shall be spliced symmetrically by plates on each side. The splice plates for shear shall extend the full depth of the girder between flanges. In the splice there shall be not less than two rows of rivets or bolts on each side of the joint.

10.18.2.2 Flange splice plates need be designed only for the portion of the design moment not resisted by the web.

10.18.2.3 As an alternate, splices of rolled flexural members may be proportioned for a shear equal to the actual maximum shear multiplied by the ratio of the splice design moment and the actual moment at the splice.

10.18.2.4 For riveted and bolted flexural members, splices in flange parts shall not be used between field splices except by special permission of the Engineer. In any one flange not more than one part shall be spliced at the same cross section. If practicable, splices shall be located at points where there is an excess of section.

10.18.2.5 In continuous spans, splices preferably shall be made at or near points of contraflexure.

10.18.3 Columns

10.18.3.1. 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 percent of the lower allowable design stress of the sections spliced.

10.18.3.2 Splices in truss chords and columns shall be located as near to the panel points as practicable and usually on that 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.

10.18.4 Tension Members

10.18.4.1. For tension members and splice material, the gross section shall be used unless the net section area is less than 85 percent of the corresponding gross area, in which case that amount removed in excess of 15 percent shall be deducted from the gross area.

10.18.4.2 In no case shall the design tensile stress on the net section exceed 0.50 F_u when using service load design or 1.0 F_u when using strength design method, where F_u equals the minimum tensile strength of the steel.

10.18.4.3 For M 270 Grades 100/100W steels, the design tensile stress on net section shall not exceed 0.46 F_u when using service load design method.

10.18.4.4 For calculating the net section, the provisions of Article 10.16.14 shall apply.

10.18.5 Welding

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 In welded splices any filler ¼ inch or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted, with sufficient weld to transmit the splice plate load applied at the surface of the filler as an eccentric load.

10.18.5.4 The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than ¼ inch thick shall have its edges made flush with the edges of the splice plate. The weld size necessary to carry the splice plate load shall be increased by the thickness of the filler plate.

10.18.5.5 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.

10.18.6 Fillers

When fasteners carrying loads pass through fillers thicker than ¼ inch, except in high-strength bolted connections designed as slip-critical connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough additional fasteners to distribute the total stress in the member uniformly over the combined section of the member and the filler. As an alternate, an equivalent number of additional fasteners may be passed through the gusset or splice material without extending the filler. Fillers ¼ inch or more in thickness shall consist of not more than two plates, unless special permission is given by the Engineer.

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 percent 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 percent 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.

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

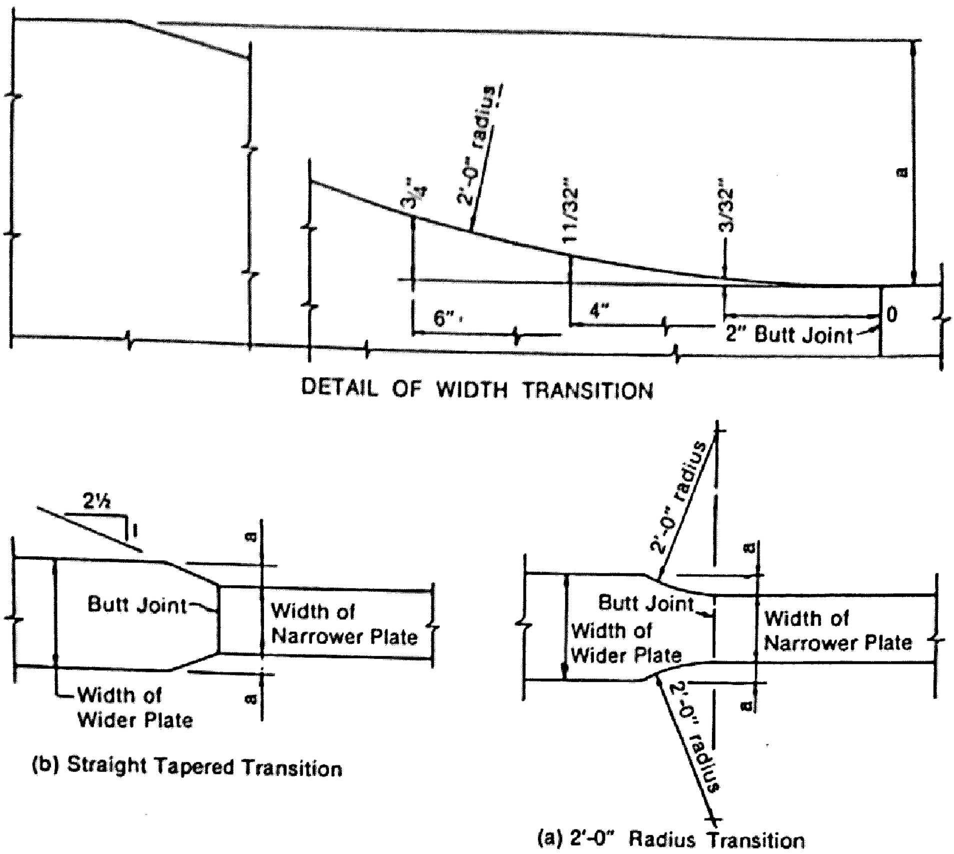


FIGURE 10.18.5A Splice Details

of floor beams and stringers shall be not less than 3/8 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. 7.5 m Diaphragms for rolled beams shall be at least 1/3 and preferably 1/2 the beam depth and for plate girders shall be at least 1/3 and preferably 3/4 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 twenty degrees (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:

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

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

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

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

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 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 ¹⁄₁₆ 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 Joined (T)		Minimum Size of Fillet Weld*		
in.	mm	in.	mm	
$T \leq 3/4$	$T \leq 19.0$	1/4	6	Single-pass welds must be used
$3/4 < T$	$19.0 < T$	5/16	8	

* 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.

** 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

طول موثر جوش $L_{eff} \geq 4\,C_w$, 3.8 cm

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

4 برابر اندازه جوش

3.8 cm

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 degrees) 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.

TABLE 10.24.2 Nominal Hole Dimension

	Hole Dimensions			
Bolt Dia.	Standard (Dia.)	Oversize (Dia.)	Short Slot (Width × Length)	Long Slot (Width × Length)
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2 1/2
≥1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

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 degrees) 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 degrees) 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.

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 $\frac{5}{16}$ 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 $\frac{5}{16}$ 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 $\frac{3}{4}$ inch or $\frac{7}{8}$ inch in diameter. Fasteners $\frac{5}{8}$ inch in diameter shall not be used in members carrying calculated stress except in $2\frac{1}{2}$-inch legs of angles and in flanges of sections requiring $\frac{5}{8}$-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, $\frac{5}{8}$-inch fasteners may be used in 2-inch legs, $\frac{3}{4}$-inch fasteners in $2\frac{1}{2}$-inch legs, $\frac{7}{8}$-inch fasteners in 3-inch legs, and 1-inch fasteners in $3\frac{1}{2}$-inch legs.

10.24.4.4 Structural shapes which do not admit the use of $\frac{5}{8}$-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, 3½ inches
For ⅞-inch fasteners, 3 inches
For ¾-inch fasteners, 2½ inches
For ⅝-inch fasteners, 2¼ 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 1½ 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 percent 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 percent, and the net section back of the pin hole not less than 100 percent 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 percent.

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 percent. The net section back of the pin hole shall not be less than 75 percent 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 percent 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 percent 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 percent.

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 a minimum of, to support 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 one-half inch (½"). For form work spans (L) of 10 feet (10') or less, or a deflection of L/240 or three-quarters inch (¾"), form work for spans L over 10 feet (10').

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.

فلز جوش

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: تنش های جوش < تنش های فلز مادر

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: جوش گوشه

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

where,

F_v = allowable basic shear stress;

F_u = tensile strength of the electrode classification but not greater than the tensile strength of the connected part. اگر از مصالح فولادی ST-37 استفاده شود $F_u = 38$

در رابطه (10-12) F_u مربوط به الکترود است E60

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.

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 ⅜ inch thick, countersunk fasteners shall not be assumed to carry stress. In metal ⅜ 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 [illegible])

10.32.3.1.5 All bolts except high-strength bolts, tensioned to the requirements of Division II, Table 11.5A

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[f]	
AASHTO Designation[e,h]		M270 Grade 36	M 270 Grade 50	M 270 Grade 50W	M 270 Grade 70W	M 270 Grades 100/100W	
Equivalent ASTM Designation[h]		A 709 Grade 36	A 709 Grade 50	A 709 Grade 50W	A 709 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 for high-strength bolts or rivets. Use net section when member has any open holes larger than 1¼ inch diameter such as perforations.	$0.55F_y$	20,000	27,000	27,000	38,000	Not Applicable	
	$0.46F_u$	Not Applicable				51,000	46,000
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending. Satisfy both Gross and Net Section criterion.	Gross[i] Section $0.55F_y$	20,000	27,000	27,000	38,000	Not Applicable	
	Net Section $0.50F_u$	29,000	32,500	35,000	45,000	Not Applicable	
	Net Section $0.46F_u$	Not Applicable				51,000	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	$\boxed{0.55F_y}$	20,000	27,000	27,000	38,000	55,000	49,000

(B) Partially supported or is unsupported[a,b]

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

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.

	Structural Carbon Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High-Yield Strength Quenched and Tempered Alloy Steel	
Compression in concentrically loaded columns[c] with $C_c = (2\pi^2E/F_y)^{1/2}$ = $\pi\sqrt{\frac{2E}{F_y}}$	126.1	107.0	107.0	90.4	75.7	79.8
when $KL/r \leq C_c$ ST37 برای $C_c = 129.5 \simeq 130$						
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2F_y}{4\pi^2E}\right]$ =	$16{,}980 - 0.53(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$	$23{,}580 - 1.03(KL/r)^2$	$33{,}020 - 2.02(KL/r)^2$	$47{,}170 - 4.12(KL/r)^2$	$42{,}450 - 3.33(KL/r)^2$

$\left[1 - \left(\frac{KL/r}{C_c}\right)^2\right]$

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[f]	
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[d]	$0.80F_y$	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[g]	$0.80F_y$	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] For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures*, 3d 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.

[b] 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]).

[c] 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.

[d] See also Article 10.32.4.

[e] 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.

[f] 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.

[g] 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.

[h] 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

[i] When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1¼ inch-diameter, such as perforations, shall be deducted.

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

Type of Fastener	Tension[b]	Bearing[c]	Shear Bearing-Type Connection[b]
(A) Low-Carbon Steel Bolts[a] Turned Bolts (ASTM A 307) Ribbed Bolts	18,000	20,000	11,000
(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] ASTM A 307 bolts shall not be used in connections subject to fatigue.
[b] Applies to fastener cross-sectional area based upon nominal body diameter.
[c] Applies to nominal diameter of fastener multiplied by the thickness of the metal.

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 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 percent, 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:

$$P_s = F_s A_b N_b N_s$$

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.

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

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

[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] 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] 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.
[e] If material thickness or joint details preclude threads in the shear plane, multiply tabulated values by 1.25.
[f] F_u = specified minimum tensile strength of connected material.
[g] 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.
[h] 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.
[i] The allowable bearing force for the connection is equal to the sum of the allowable bearing forces for the individual bolts in the connection.
[j] 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).

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 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

$$Q=\left[\frac{3b}{8a}-\frac{t^3}{20}\right]T \qquad (10\text{-}13)$$

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.

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.

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:

$$f_v = F_s(1 - 1.88f_t/F_u) \qquad (10\text{-}14)$$

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 $f_v/F_v \le 0.33$

$$F_t' = F_t \qquad (10\text{-}15)$$

for $f_v/F_v > 0.33$

$$F_t' = F_t\sqrt{1-(f_v/F_v)^2} \qquad (10\text{-}16)$$

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.

$$s^2 + (kt)^2 = S^2 \qquad (10\text{-}17)$$

s = the computed rivet or bolt unit stress in shear;
t = the computed rivet or bolt unit stress in tension, including any stress due to prying action;
S = the allowable rivet or bolt unit stress in shear;
k = a constant: 0.75 for rivets; 0.6 for high-strength bolts with thread excluded from shear plane.

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

10.32.4.2 Bearing per linear inch on expansion rockers and rollers shall not exceed the values obtained by the following formulas:

Diameters up to 25 inches:

$$p = \frac{F_y - 13{,}000}{20{,}000}600d \qquad (10\text{-}18)$$

Diameters 25 to 125 inches:

$$p = \frac{F_y - 13{,}000}{20{,}000}3{,}000\sqrt{d} \qquad (10\text{-}19)$$

where:

p = allowable bearing in pounds per linear inch;
d = diameter of rocker or roller in inches;
F_y = minimum yield point in tension of steel in the roller or bearing plate, whichever is the smaller.

Expansion rollers shall be not less than 4 inches in diameter.

10.32.4.3 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.

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[b] Class G
Minimum Yield Point, psi	F_y	36,000[a]	33,000	37,500	50,000	50,000
Stress in Extreme Fiber, psi	$0.80F_y$	29,000[a]	26,000	30,000	40,000	40,000
Shear, psi	$0.40F_y$	14,000[a]	13,000	15,000	20,000	20,000
Bearing on Pins not Subject to Rotation, psi[c]	$0.80F_y$	29,000[a]	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[a]	13,000	15,000	20,000	20,000

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

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

[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.

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.

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

TABLE 10.32.5.1A Allowable Stresses—Cast Steel and Ductile Iron

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

inches (average) beyond the edge of shoe or plate. Otherwise, the unit stresses permitted will be 75 percent 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 percent of the allowable shear for girder webs. See the related provisions of Article 10.34.6. $F_v < 0.75 F_v$ not use stiff

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 percent of that flange. That area in excess of 15 percent 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.

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:

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$$\frac{b}{t} = \frac{3{,}250}{\sqrt{f_b}} \quad \text{but in no case shall } b/t \text{ exceed } 24 \qquad (10-20)$$

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

$F_y = 24..$ $\frac{b}{t} < 24$

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:

$$\frac{b}{t} = \frac{3{,}250}{\sqrt{f_{dl}}} \quad \text{but in no case shall b/t exceed 24} \qquad (10\text{-}21)$$

DL

where f_{dl} 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 ⅝ 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:

$$\frac{b'}{t} = \frac{1{,}625}{\sqrt{f_b}} \quad \text{but in no case shall b'/t exceed 12} \qquad (10\text{-}22)$$

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:

$$\frac{b'}{t} = \frac{1{,}625}{\sqrt{f_{dl}}} \quad \text{but in no case shall b'/t exceed 12} \qquad (10\text{-}23)$$

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_{dl} is the top flange compressive stress due to non-composite dead load.

F_y

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:

STIFF STIFF

$$t_w = \frac{D\sqrt{f_b}}{23{,}000} \quad \text{(See Figure 10.34.3.1A.)} \qquad (10\text{-}24)$$

$F_y = 2400$

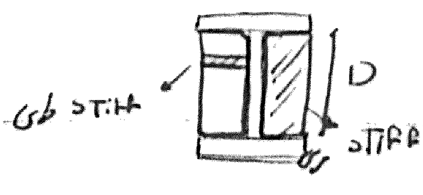
but in no case shall the thickness be less than D/170. $\frac{D}{t_w} \geq 178$

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):

$F_y = 2400$ kg/cm² ST-37

if $\frac{D}{t_w} < 170$ No L-ST needed

if $\frac{D}{t_w} < 390$ L-ST needed



ST-37 → $\frac{D}{165}$

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

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:

with L.ST

$$t_w = \frac{D\sqrt{f_b}}{46{,}000} \quad \text{(See Figure 10.34.3.1A)} \quad (10\text{-}25)$$

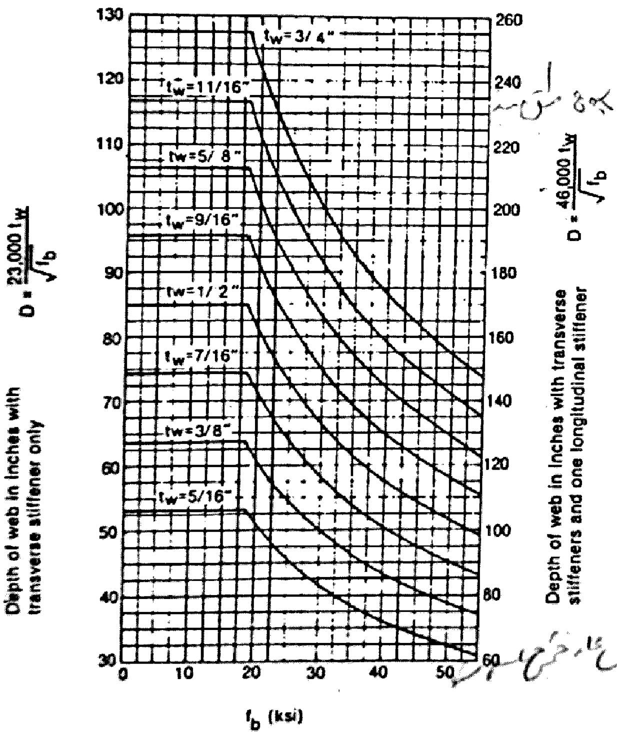
but in no case shall the thickness be less than D/340.

10.34.3.2.2 Where the calculated bending stress in the flange equals the allowable bending stress, the thickness of the web plate stiffened with transverse stiffeners in combination with one longitudinal stiffener shall not be less than (where the Y.P. is for the flange material):

ST-37 → D/340

36,000 psi, Y.P. Min. D/330
50,000 psi, Y.P. Min. D/280
70,000 psi, Y.P. Min. D/230
90,000 psi, Y.P. Min. D/210
100,000 psi, Y.P. Min. D/200

In the above, D (depth of web) is the clear unsupported distance in inches between flange components, t_w is the web thickness, and f_b is the calculated flange bending stress.



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

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:

$f_v = \frac{V}{D t_w}$

$$F_v = \frac{7.33 \times 10^7}{(D/t_w)^2} \le \frac{F_y}{3} \qquad (10\text{-}26)$$

$\frac{D}{t_w} \le 150$

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:

$$F_v = \frac{F_y}{3}\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \qquad (10\text{-}27)$$

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

$$\text{for} \quad \frac{D}{t_w} < \frac{6{,}000\sqrt{k}}{\sqrt{F_y}}$$

C=1.0

for:

$$\frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \le (D/t_w) \le \frac{7{,}500\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}28)$$

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

$C \ge 0.8$

for:

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

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

$C \le 0.8$

where:

$$k = 5 + \frac{5}{(d_o/D)^2} \qquad (10\text{-}28B)$$

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

($F_y/3$) in Equation (10-27) 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. For panels without longitudinal stiffeners, the spacing of these stiffeners shall not exceed $D[260/(D/t_w)]^2$ to ensure efficient handling, fabrication, and erection of the girders.

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):

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

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 stress, F_s, shall be limited to:

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

only when (10-27) is used

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 (10-27).

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 mid-plane of the web 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,

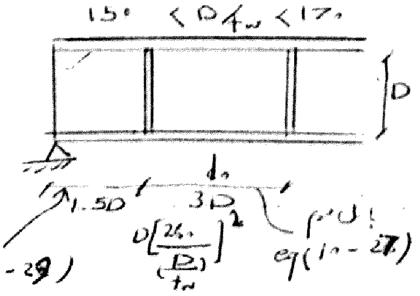
I = minimum permissible moment of inertia of any type of transverse intermediate stiffener in inches[4];
J = required ratio of rigidity of one transverse stiffener to that of the web plate;
d_o = actual distance between stiffeners in inches;
D = unsupported depth of web plate between flange components in inches for transversely stiffened girders, or maximum subpanel depth in inches for longitudinally stiffened girders;
t_w = thickness of the web plate in inches.

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

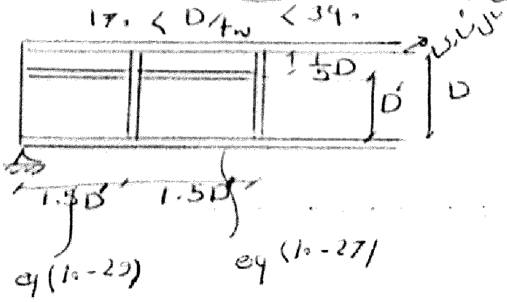
$$A = [0.15BDt_w(1 - C)(f_v/F_v) - 18\,t_w^2]Y \qquad (10\text{-}32a)$$

where Y is the ratio of web plate yield strength to stiffener plate yield strength: B = 1.0 for stiffener pairs, 1.8 for sin-

$150 < D/t_w < 170$



$170 < D/t_w < 340$



gle 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 center line of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener shall be D/5 from the inner surface or leg of the compression flange component. 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:

$$\frac{b'\sqrt{f_b}}{2{,}250} \qquad (10\text{-}34)$$

where:

- b' = width of stiffeners;
- f_b = calculated compressive bending stress in the flange.

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 maximum subpanel 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 maximum subpanel 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 except that the maximum subpanel depth shall be used instead of the total panel depth, D.

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,

cr stiff

$$t \geq \frac{b'}{12}\sqrt{\frac{F_y}{33{,}000}} \rightarrow kg/m^2 \qquad (10\text{-}35)$$

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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:

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

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:

$$\frac{b}{t} = \frac{1{,}625}{\sqrt{f_a}} = \frac{431}{\sqrt{f_a\ (kg/cm^2)}} \qquad (10\text{-}38)$$

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

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

and

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

where

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

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.

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

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

where

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

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

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:

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

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

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

where KL is as defined above.

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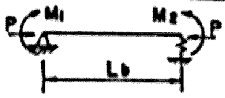
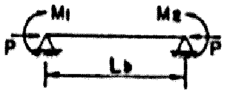
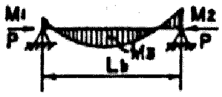
TABLE 10.36A Bending-Compression Interaction Coefficients

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		$\left[(0.4)\frac{M_1}{M_2} + 0.6\right] \geq 0.4$
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.

10.37.1.4 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

$$I_s = 0.75\, D t_w^3 \qquad (10\text{-}51)$$

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

$$I_s = 2.2\, D t_w^3 \qquad (10\text{-}53)$$

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

$$0.2 \leq \frac{f_b}{f_a + f_b} \leq 0.7 \qquad (10\text{-}55)$$

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	n
2,000–2,300	= 11
2,400–2,800	= 10
2,900–3,500	= 9 → 2.4 - 246 kg/cm²
3,600–4,500	= 8 → 250 - 316
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 and for determining stiffness factors used in calculating moments and shears.

10.38.1.7 The steel beams, especially if not supported by intermediate falsework, shall be investigated for stability and strength in accordance with Articles 10.50(c) through (g) during the time the concrete is in place and before it has hardened.

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 4.8 mm ← 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. 5 cm

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 one-twelfth 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 percent 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 In the negative moment regions of continuous spans, the minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1 percent of the cross-sectional area of the concrete slab. Two-thirds of this required reinforcement is to be placed in the top layer of slab within the effective width. Placement of distribution steel as specified in Article 3.24.10 is waived within the effective width.

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);

*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.

Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches;

I = moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

(In the formula, the compressive 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:

$$Z_r = Bw \qquad (10\text{-}59)$$

$\to B \times 0.1786$

Welded studs (for H/d ≥ 4):

$$Z_r = \alpha d^2 \qquad (10\text{-}60)$$

$\to \times 0.70$

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 → 914
10,600 for 500,000 cycles → 745
7,850 for 2,000,000 cycles → 552
5,500 for over 2,000,000 cycles; 387

B = 4,000 for 100,000 cycles → 714
3,000 for 500,000 cycles → 536
2,400 for 2,000,000 cycles → 429
2,100 for over 2,000,000 cycles; → 375

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. 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.

$P = \dfrac{\Sigma Z_r}{S_r}$

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 cover-plates;

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:

$$P_3 = A_s^r F_y^{r*} \qquad (10\text{-}65)$$

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P_3 is equal to zero.

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: 144.52

$$S_u = 550\left(h + \frac{t}{2}\right)W\sqrt{f_c'} \qquad (10\text{-}66)$$

Welded studs (for H/d > 4):

$$S_u = 0.4d^2\sqrt{f_c'E_c} \qquad (10\text{-}67)$$

where:

E_c = modulus of elasticity of the concrete in pounds per square inch;

$$E_c = w^{3/2}33\sqrt{f_c'} \qquad (10\text{-}68)$$

$1510\sqrt{f_c}$

S_u = ultimate strength of individual shear connector in pounds;

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.

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 percent 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 percent of the average distance center-to-center

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P_3 is equal to zero.

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 percent 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:

1625

$$\frac{b}{t} = \frac{6{,}140}{\sqrt{F_y}} \qquad (10\text{-}73)$$

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 60, the stress in an unstiffened bottom flange shall not exceed the value determined by the use of the formula:

$$f_b = 0.55F_y - 0.224F_y \times \left[1 - \sin\left(\frac{\pi}{2} \times \frac{13{,}300 - \frac{b\sqrt{F_y}}{t}}{7{,}160}\right)\right] \qquad (10\text{-}74)$$

x 377

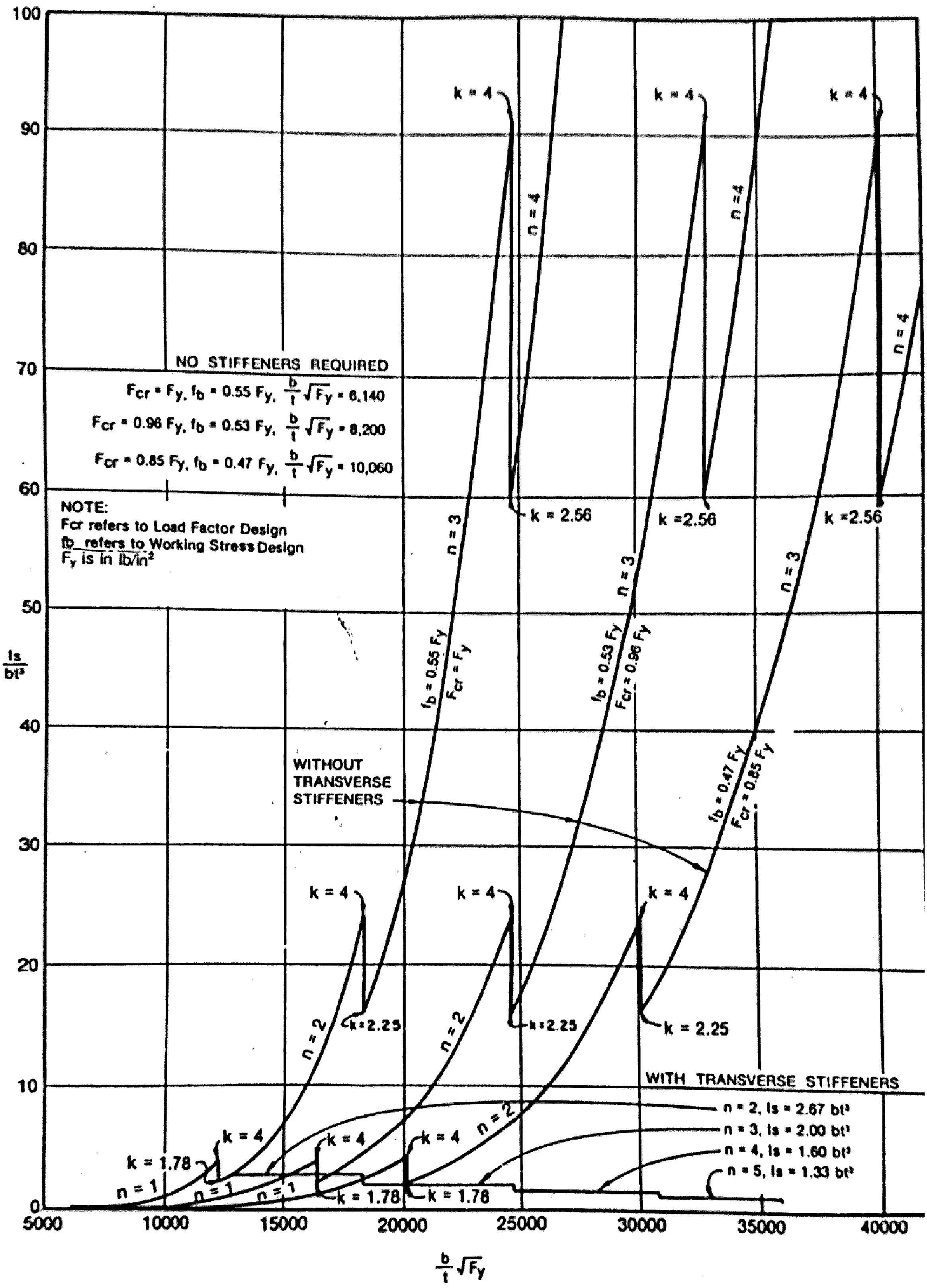
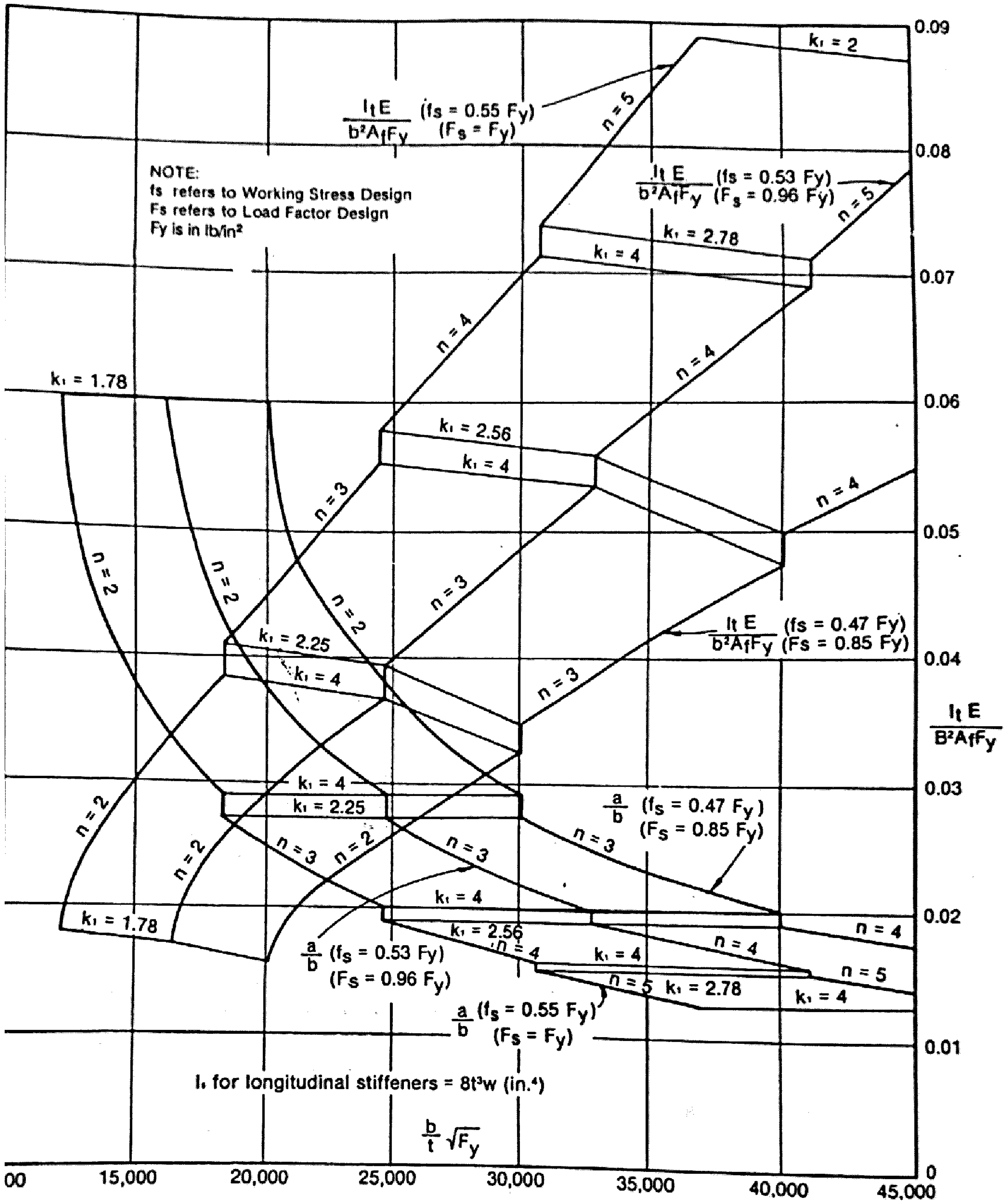


FIGURE 10.39.4.3A. Longitudinal Stiffeners—Box Girder Compression Flange



RE 10.39.4.3B. Spacing and Size of Transverse Stiffeners (for Flange Stiffened Longitudinally and Transversely)

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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:

90 kg/cm²

$$f_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^6 \qquad (10\text{-}75)$$

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.

10.39.4.3 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:

$$I_s = \phi\, t_f^3 w \qquad (10\text{-}76)$$

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.

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:

81.

$$\frac{w}{t} = \frac{3{,}070\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}77)$$

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:

$$f_b = 0.55F_y - 0.224F_y \times$$

×3.77

$$\left[1 - \sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k} - \dfrac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k}}\right)\right] \qquad (10\text{-}78)$$

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:

1.0 kg/cm²

$$f_b = 14.4\,k(t/w)^2 \times 10^6 \qquad (10\text{-}79)$$

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.

10.39.4.4 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:

$$I_s = 8\, t_f^3 w \qquad (10\text{-}80)$$

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:

$$I_t = 0.10(n+1)^3 w^3 \frac{f_s}{E}\frac{A_f}{a} \qquad (10\text{-}81)$$

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.

*In solving these equations a value of k between 2 and 4 generally should be assumed.

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:

$$\frac{w}{t} = \frac{3{,}070\sqrt{k_1}}{\sqrt{F_y}} \qquad (10\text{-}82)$$

where:

$$k_1 = \frac{[1+(a/b)^2]^2 + 87.3}{(n+1)^2(a/b)^2[1+0.1(n+1)]} \qquad (10\text{-}83)$$

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:

$$f_b = 0.55F_y - 0.224F_y \times \left[1 - \sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k_1} - \frac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k_1}}\right)\right] \qquad (10\text{–}84)$$

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:

$$f_b = 14.4k_1\left(\frac{t}{w}\right)^2 \times 10^6 \qquad (10\text{-}85)$$

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:

$$R_w = \frac{F_y S_s}{2b} \qquad (10\text{-}86)$$

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:

$$R_s = \frac{F_y S_s}{nb} \qquad (10\text{-}87)$$

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:

$$\frac{b'}{t'} = \frac{2{,}600}{\sqrt{F_y}} \qquad (10\text{-}88)$$

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 percent 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 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 percent 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.)

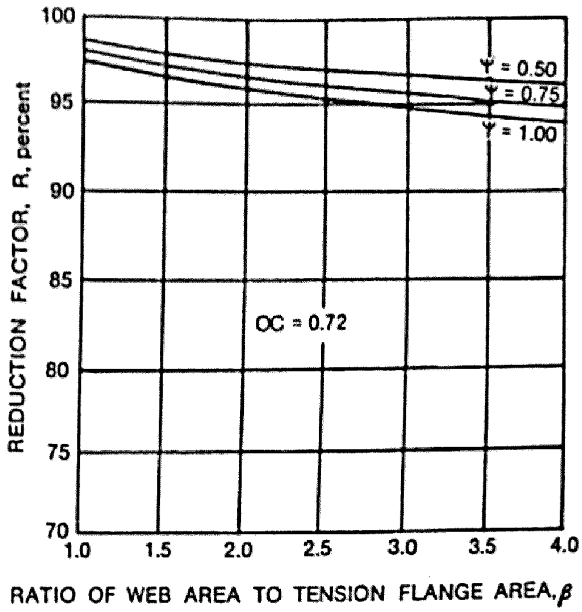


FIGURE 10.40.2.1A

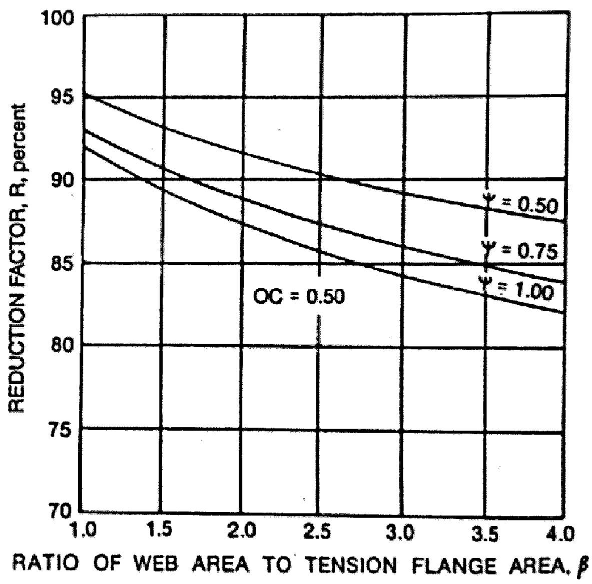


FIGURE 10.40.2.1B

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;*

*Bottom flange of orthotropic deck bridges.

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.

10.40.2.2 Shear

The design of the web for a hybrid girder shall be in compliance with specification Article 10.34.3 except that Equation (10-27) 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 equation for the allowable average shear stress in the web of transversely stiffened hybrid girders:

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

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 and the minimum thickness of the web plate according to Article 10.34.3, f_b shall be taken as the calculated bending stress in the compression flange divided by the reduction factor, R.

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.

*Bottom flange of orthotropic dock bridges.

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.*

*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.

10.41.3.2 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.

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:

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

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-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 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 $0.85_c{}'$.

10.45.4 Tensile strength of concrete shall be neglected in flexural calculations.

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 SYMMETRICAL BEAMS AND GIRDERS

10.48.1 Compact Sections

Symmetrical I-shaped beams and girders with high resistance to local buckling and proper bracing to resist lateral torsional buckling qualify as compact sections. Compact sections are able to form plastic hinges with an inelastic rotation capacity of three times the elastic rotation corresponding to the plastic moment.

Rolled or fabricated I-shaped beams and fabricated girders meeting the requirements of Article 10.48.1.1 below shall be considered compact sections and the maximum strength shall be as computed:

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

Where F_y is the specified yield point of the steel being used, Z is the plastic section modulus.*

10.48.1.1 Beams and girders designed as compact sections shall meet the following requirements: (For certain frequently used steels these requirements are listed in Table 10.48.1.2A.)

(a) Projecting compression flange element:

$$\frac{b'}{t} \le \frac{2{,}055}{\sqrt{F_y}} \qquad (10\text{-}93)$$

where b′ is the width of the projecting flange element, t is the flange thickness.

(b) Web thickness:

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

where D is the clear distance between the flanges, 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:

$$\frac{D}{t_w} + 9.35\left(\frac{b'}{t}\right) \le \frac{33{,}650}{\sqrt{F_{yf}}} \qquad (10\text{-}95)$$

where F_{yf} is the yield strength of the compression flange.

(c) Lateral bracing:

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

where L_b is the distance between points of bracing of the compression flange, r_y is the radius of gyration of

*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
b'/t	10.8	9.2
D/t_w	101	86
L_b/r_y (M_l/M_u = 0*)	100	72
L_b/r_y (M_l/M_u = 1*)	39	28

* For values of M_l/M_u other than 0 and 1, use Equation (10-95).

the steel section with respect to the Y-Y axis, M_l 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_l/M_u) is positive when moments cause single curvature between brace points. (M_l/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:

$$P \leq 0.15 F_yA \qquad (10\text{-}97)$$

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 stress-strain diagrams that exhibit a yield plateau followed by a strain hardening range. Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A709 Grades 36, 50, and 50W) 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 of compact section complying with the provision of Articles 10.48.1.1, negative moments over 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.

10.48.2 Braced Noncompact Sections

For rolled or fabricated I-shaped beams and fabricated girders not meeting the requirements of Article 10.48.1.1 but meeting the requirements of paragraph 10.48.2.1 below, the maximum strength shall be computed as:

$$M_u = F_yS \qquad (10\text{-}98)$$

where S is the section modulus.

10.48.2.1 The above equation is applicable to beams and girders meeting the following requirements:

(a) Projecting compression flange element:

$$\frac{b'}{t} \leq \frac{2{,}200}{\sqrt{F_y}} \qquad (10\text{-}99)$$

where $M < M_u$, b'/t may be increased by the ratio $\sqrt{M_u/M}$.

(b) Web thickness:

$$\frac{D_c}{t_w} \leq \frac{15{,}400}{\sqrt{F_y}} \qquad (10\text{-}100)$$

where D_c is the depth of the web in compression equal to $\frac{D}{2}$ for symmetrical girders.

(c) Spacing of lateral bracing for compression flange:

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

where d is the depth of beam or girder, and A_f is the flange area.

(d) Maximum axial compression:

$$P \leq 0.15\ F_yA. \qquad (10\text{-}102)$$

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.

10.48.2.3 The maximum bending strength of members not meeting the web requirements of Article 10.48.2.1(b) or the lateral bracing requirements of Article 10.48.2.1(c) shall be computed from the provisions of Article 10.48.4.1.

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	11.6	9.8	8.3	7.3	7.0
$\dfrac{L_b d}{A_f}$	556	400	286	222	200
D/t_w	162	138	116	103	97

10.48.3 Transitions

The maximum strength of members 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 Unbraced Sections

10.48.4.1 For members not meeting the lateral bracing requirements of Article 10.48.2.1(c), or the web thickness requirements of Article 10.48.2.1(b), and with 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, within the limits of $0.1 \le I_{yc}/I_y \le 0.9$, the maximum strength shall be computed as:

$$M_u = M_r R_b \qquad (10\text{-}103a)$$

R_b = 1 for longitudinally stiffened girders meeting the requirements of Articles 10.48.6 and 10.49.3. For all other members:

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

D_c = depth of web in compression (in.) = $\dfrac{D}{2}$ for symmetrical girders;

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 members with a compression flange area equal to or greater than the tension flange area;

= 12,500 for members with a compression flange area less than the tension flange area.

The moment capacity of the member, M_r, cannot exceed the yield moment, M_y. In addition M_r cannot exceed the lateral torsional buckling moment given below:

For members with $\dfrac{D_c}{t_w} \le \dfrac{\lambda}{\sqrt{F_y}}$ or with longitudinally stiffened webs:

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

For members with $\dfrac{\lambda}{\sqrt{F_y}} < \left(\dfrac{D_c}{t_w}\right) \le \dfrac{18{,}250^{a}}{\sqrt{F_y}}$:

for $L_b \le L_p$

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

for $L_r \ge L_b > L_p$

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

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

for $L_b \ge L_r$

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

L_b = unbraced length of the compression flange, in.

L_p = $9{,}500 r'/\sqrt{F_y}$, inches.

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.

$J = \dfrac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, respectively (in.^{4}).

$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3(M_1/M_2)^2 \le 2.3$ 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.[a]

If the web slenderness D_c/t_w for the maximum design loads exceeds the upper limit of $18{,}250/\sqrt{F_y}$, either the section shall be modified to comply with the limit, or longitudinal stiffeners shall be provided.[b]

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:

$$\frac{D}{t_w} \le \frac{36{,}500}{\sqrt{F_y}} \qquad (10\text{-}104)$$

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

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, or 10.48.4.1, 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:

$$\frac{b'}{t} \le \frac{2{,}600}{\sqrt{F_y}} \qquad (10\text{-}105)$$

where b′ is the projecting width of the stiffener, and F_y is the yield strength of the transverse stiffener.

The gross cross-sectional area of intermediate transverse stiffeners shall not be less than:

$$A = [0.15BDt_w(1 - C)(V/V_u) - 18t_w^2]Y \qquad (10\text{-}106)$$

where Y is the ratio of web plate yield strength to stiffener plate yield strength; 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 (10-106) 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 midplane of the web shall be not less than:

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

where:

$$J = 2.5(D/d_o)^2 - 2, \text{ but not less than } 0.5 \qquad (10\text{-}108)$$

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 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:

[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. 135.

[b] The upper limit on D_c/t_w of $18{,}250/\sqrt{F_y}$ may be waived for composite girders without longitudinal stiffeners in accordance with Article 10.50(d) when checking formulas (10-102d) through (10-102g) for factored noncomposite dead load only.

$$\frac{D}{t_w} \le \frac{73{,}000}{\sqrt{F_y}} \qquad (10\text{-}109)$$

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

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 Article 10.48.2 or Article 10.48.4.1 as applicable, subject to the requirement of Article 10.48.8.1.

10.48.6.3 The shear capacity of girders with one longitudinal stiffener shall be computed by Article 10.48.8.1.

The dimensions of the longitudinal stiffener shall be such that:

(a) the width-to-thickness ratio is not greater than that given by Article 10.48.5.3.
(b) the rigidity of the stiffener is not less than:

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

(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 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 except that the maximum subpanel depth shall be used instead of the total panel depth, D. 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 at D/5.

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 rolled or fabricated I-shaped beams and fabricated girders shall be computed as follows:

For beams and girders with 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 girders with stiffened webs and (d_o/D) less than or equal to 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:

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

$$C = 1.0$$

$$\text{for } \frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \le \frac{D}{t_w} \le \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

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

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

$$C = \frac{4.5\times10^7 k}{\left(\frac{D}{t_w}\right)^2 F_y} \qquad (10\text{-}117)$$

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 subjected to simultaneous action of shear and bending moment with the magnitude of the moment higher than $0.75M_u$, the shear shall be limited to not more than:

$$V/V_u = 2.2 - (1.6M/M_u) \qquad (10\text{-}118)$$

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. For panels without longitudinal stiffeners, the spacing of these stiffeners shall not exceed $D[260/(D/t_w)]^2$ to ensure efficient handling, fabrication, and erection of the girders.

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 maximum subpanel 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:

$$V = CV_p \qquad (10\text{-}119)$$

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 limited to 1.5 times the maximum subpanel depth.

10.49 UNSYMMETRICAL BEAMS AND GIRDERS

10.49.1 General

For beams and girders symmetrical about the vertical axis of the cross section but unsymmetrical with respect to the horizontal centroidal axis, the provisions of Articles 10.48.1 through 10.48.4 shall be applicable.

10.49.2 Unsymmetrical 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:

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

10.49.3 Longitudinally Stiffened Unsymmetrical Sections

10.49.3.1 Longitudinal stiffeners shall be required on unsymmetrical sections when the web thickness is less than that 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 provided that:

(a) When D_c exceeds D/2, the longitudinal stiffener is placed $2D_c/5$ from the inner surface or the leg of the compression flange element.
(b) When D_c exceeds D/2, the web thickness, t_w, shall meet the requirement:

$$\frac{D_c}{t_w} \leq \frac{36{,}500}{\sqrt{F_y}} \qquad (10\text{-}121)$$

10.49.4 Unsymmetrical Braced Noncompact Sections

Unsymmetrical braced, noncompact rolled or fabricated I-shaped beams and fabricated girders shall be designed and evaluated by the provisions of Article 10.48.2.1.

10.49.5 Unbraced Unsymmetrical Sections

Unsymmetrical sections which do not satisfy the lateral bracing requirements of Article 10.48.2.1(c) shall be designed and evaluated by the provisions of Article 10.48.4.1.

10.50 COMPOSITE BEAMS AND GIRDERS

Composite beams and girders shall be so proportioned that the criteria on the next page 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. In such application the value of D_c shall be taken as the clear distance between the neutral axis of the composite section for live loads and the compression flange.
(c) The ratio of the projecting top compression flange plate width to thickness shall not exceed the value determined by the formula:

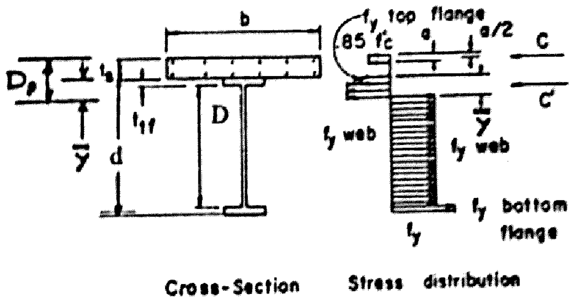
$$\frac{b'}{t} = \frac{2{,}200}{\sqrt{1.3f_{dl1}}} \qquad (10\text{-}122)$$

where f_{dl1} is the top-flange compressive stress due to noncomposite dead load.
(d) The maximum moment capacity of noncompact sections when considering noncomposite dead loads with a load factor of $\gamma = 1.3$ shall be computed by Article 10.48.4.1, except that D_c/t_w of the steel section may exceed $\frac{18{,}250}{\sqrt{F_y}}$ but D/t_w shall not exceed $\frac{36{,}500}{\sqrt{F_y}}$ for girders without longitudinal stiffeners. D_c/t_w of the steel section shall not exceed $\frac{36{,}500}{\sqrt{F_y}}$ for girders with longitudinal stiffeners.
(e) The maximum shear due to noncomposite dead load with a load factor of $\gamma = 1.3$ shall not exceed the shear buckling capacity of the web in Article 10.48.8.1 (Equation 10-112).
(f) The moment capacity at first yield shall be computed considering the application of the dead and live loads to the steel and composite sections.
(g) The casting or placing sequence for the composite concrete deck should be considered in meeting the requirements of Articles 10.50(c), 10.50(d), and 10.50(e).

10.50.1 Positive Moment Sections of Composite Beams and Girders

10.50.1.1 Compact Sections

The maximum strength, M_u, of compact composite beams and girders in the positive-moment regions shall be computed in accordance with Article 10.50.1.1.2. The stress-strain diagram of the steel shall exhibit a yield plateau followed by a strain-hardening range.



PLASTIC STRESS DISTRIBUTION

FIGURE 10.50A

Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A 709 Grades 36, 50, and 50W) 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) $$C = 0.85f'_c\, bt_s + (AF_y)_c \qquad (10\text{-}123)$$

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.

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

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 (10-123), the top portion of the steel section will be subjected to the compressive force on the next page:

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

(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 beams and girders in positive-moment regions 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 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 beams and girders 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 $D_p \leq D'$

$$M_u = M_p \qquad (10\text{-}129b)$$

for $D' < D_p \leq 5D'$

$$M_u = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \qquad (10\text{-}129c)$$

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(f), 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(f);

$(M_u - M_s)_{Pier}$ = 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), M_u 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 maximum bending strength, M_u, of the section shall be taken as the moment at first yield determined as specified in Article 10.50(f).

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 percent 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 of Composite Beams and Girders

The maximum bending strength, M_u, of composite beams and girders in the negative moment regions shall be computed in accordance with Articles 10.48 and 10.49 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 beams and girders in negative bending qualify as compact when their steel section meets the requirements of Article 10.48.1.1, and the stress-strain diagram of the steel exhibits a yield plateau followed by a strain hardening range. Steels such as AASHTO M 270 Grades 36, 50, and 50W (ASTM A 709, Grades 36, 50, and 50W) 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 (10-93) and (10-94) 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 the requirements of Article 10.48.2.1, the maximum strength, M_u, of the section shall be taken as the moment at first yielding determined as specified in Article 10.50(f). If the requirements of Article 10.48.2.1(b) or Article 10.48.2.1(c) are not satisfied, M_u shall be calculated according to the provisions specified in Article 10.48.4.1. In this case, the web slenderness shall not exceed the requirement given by Equation (10-103) or Equation (10-108), as applicable, subject to the corresponding requirements of Article 10.49.2 or 10.49.3.

10.50.2.3

In the negative moment regions of continuous spans, the minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1 percent of the cross-sectional area of the concrete slab. Two-thirds of this required reinforcement is to be placed in the top layer of slab within the effective width. Placement of distribution steel as specified in Article 3.24.10 is waived within the effective width.

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 cen-

*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.

ter-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 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 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:

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

where F_{cr} is the buckling stress of the bottom flange plate as given in Article 10.51.5.

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,

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

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,

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

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.

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.

10.51.6 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.

10.52 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), 10.48.2.1(b), 10.48.4.1, 10.48.5.1, 10.48.6.1, 10.49.2, 10.49.3.2(b), and 10.50(d) use the F_y of the compression flange.
(2) In Articles 10.48.6.3(a) and 10.48.6.3(c) use the F_y of the adjacent flange. 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.

10.53.1 Noncomposite Hybrid Girders

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} 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 F_{yf}.

10.53.1.2 Braced Noncompact Sections

The equation of Article 10.48.2 for the maximum strength of noncompact sections shall be replaced by the expression:

$$M_u = F_{yf}SR \qquad (10\text{-}146)$$

For symmetrical sections:

$$R = \frac{12+\beta(3\rho-\rho^3)}{12+2\beta} \qquad (10\text{-}147)$$

where:

$\rho = F_{yw}/F_{yf}$

$\beta = A_w/A_f$

For unsymmetrical sections,

$$R = 1-\left[\frac{\beta\psi(1-\rho)^2(3-\psi+\rho\psi)}{6+\beta\psi(3-\psi)}\right] \qquad (10\text{-}148)$$

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.

10.53.1.3 Unbraced Noncompact Sections

The strength of unbraced noncompact hybrid sections shall be calculated in accordance with Article 10.48.4.1 with Equation (10-103a) replaced by the expression:

$$M_u = M_rR_bR \qquad (10\text{-}148a)$$

and the yield moment calculated as:

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

where the appropriate R is determined from Article 10.53.1.2 above, and R_b is determined by Equation (10-103b).

10.53.1.4 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:

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

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.

10.53.2 Composite Hybrid Girders

The maximum strength of the composite section shall be the moment at first yielding of the flanges 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.

10.54 COMPRESSION MEMBERS

10.54.1 Axial Loading

10.54.1.1 Maximum Capacity

The maximum strength of concentrically loaded columns shall be computed as:

$$P_u = 0.85A_sF_{cr} \quad (10\text{-}150)$$

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:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2E}\left(\frac{KL_c}{r}\right)^2\right] \quad (10\text{-}151)$$

$$\text{for } \frac{KL_c}{r} \leq \sqrt{\frac{2\pi^2E}{F_y}} \quad (10\text{-}152)$$

$$F_{cr} = \frac{\pi^2E}{\left(\frac{KL_c}{r}\right)^2} \quad (10\text{-}153)$$

$$\text{for } \frac{KL_c}{r} > \sqrt{\frac{2\pi^2E}{F_y}} \quad (10\text{-}154)$$

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.

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.*

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

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:

$$\frac{P}{0.85A_sF_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_sF_e}\right)} \leq 1.0 \quad (10\text{-}155)$$

$$\frac{P}{0.85A_sF_y} + \frac{M}{M_p} \leq 1.0 \quad (10\text{-}156)$$

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;

$$F_e = \frac{E\pi^2}{\left(\frac{KL_c}{r}\right)^2} = \text{the Euler Buckling stress in the plane of bending;} \quad (10\text{-}157)$$

- 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.

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:

$$C = 0.6 + 0.4a\text{, but not less than } 0.4 \quad (10\text{-}158)$$

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:

$$\frac{b'}{t_s} = \frac{2{,}200}{\sqrt{f_a + \dfrac{f_b}{3}}} \text{ maximum } \frac{b'}{t_s} = 12 \qquad (10\text{-}164)$$

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:

$$\phi R = \phi F A_b \qquad (10\text{-}166a)$$

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 $f_v/F_v \leq 0.33$

$$F_t' = F_t \qquad (10\text{-}167)$$

for $f_v/F_v > 0.33$

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

where:

f_v = computed rivet or bolt stress in shear, ksi;
F_v = design shear strength of rivet or bolt from Table 10.56A or Table 10.57A, 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 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 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:

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

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:

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

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:

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

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	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[c]	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 but not greater than the tensile strength of the connected parts.
[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 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.
[e] If material thickness or joint details preclude threads in the shear plane, multiply values by 1.25.

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;
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

10.57.1 Noncomposite Beams and Girders

For noncomposite beams and girders, the maximum flange stress caused by D + 5(L + I)/3 shall not exceed $0.8RF_{yf}$ where R is the hybrid girder reduction factor specified in Article 10.53.1.2, equal to 1.0 for nonhybrid sections, and F_{yf} is the specified minimum yield stress of the flange. For such beams and girders designed for Group 1A loading, the maximum flange stress caused by D + 2.2(L + I) shall not exceed $0.8RF_{yf}$. In the case of moment redistribution under the provisions of Article 10.48.1.3 the above limitation shall apply to the modified moments but not to the original moments.

10.57.2 Composite Beams and Girders

For composite beams and girders, the maximum flange stress caused by D + 5(L + I)/3 shall not exceed $0.95RF_{yf}$ where R is the hybrid girder reduction factor specified in Article 10.53.1.2, equal to 1.0 for nonhybrid sections, and F_{yf} is the specified minimum yield stress of the flange. For such beams and girders designed for Group 1A loading, the maximum flange stress caused by D + 2.2(L + I) shall not exceed $0.95RF_{yf}$. In computing dead load stresses the presence or absence of temporary supports during the construction shall be considered.

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.

10.57.3 Slip-Critical Joints

10.57.3.1 In addition to the requirements of 10.56.1.3.1 and 10.56.1.3.2 for fasteners, the force caused by D + 5(L + I)/3, for H or HS truck load only, on a slip-critical joint shall not exceed the design slip force (ϕR_s) given by:

$$\phi R_s = \phi F_s A_b N_b N_s \qquad (10\text{-}172)$$

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:

$$\phi R_s' = \phi R_s (1 - 1.88 f_t/F_u) \qquad (10\text{-}173)$$

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 (10-172), 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.

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.