STANDARD SPECIFICATIONS

for

HIGHWAY BRIDGES

The American Association of State Highway
and Transportation Officials

DIVISION I — DESIGN

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NCHRP Project 20-07 / Task 18

STANDARD SPECIFICATIONS

FOR

HIGHWAY BRIDGES

DIVISION I

DESIGN

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I - DESIGN

SECTION 1 - GENERAL PROVISIONS

1.1 DESIGN ANALYSIS

When the specifications provide for an empirical formula as a design convenience, a rational analysis based on a theory accepted by the Committee on Bridges and Structures of the American Association of State Highway and Transportation Officials, with stresses in accordance with the specifications, will be considered as compliance with the specifications.

1.2 BRIDGE LOCATIONS

The general location of a bridge is governed by the route of the highway it carries. Which, in the case of a new highway, could be one of several routes under consideration. The bridge location should be selected to suit the particular obstacle being crossed. Stream crossings should be located with regard to initial capital cost of bridgeworks and the minimization of total cost including river channel training works and the maintenance measures necessary to reduce erosion. Highway and railroad crossings should provide for possible future works such as road widening.

1.3 WATERWAYS

1.3.1. GENERAL

1.3.1.1 Selecting favorable stream crossings should be considered in the preliminary route determination to minimize construction, maintenance and replacement costs. Natural stream meanders should be studied and, if necessary, channel changes, river training works and other construction which would reduce erosion problems and prevent possible loss of the structure should be considered. The foundations of bridges constructed across channels that have been realigned should be designed for possible deepening and widening of the relocated channel due to natural causes. On wide flood plains, the lowering of approach embankments to provide overflow sections which would pass unusual floods over the highway is a means of preventing loss of structures. Where relief bridges are needed to maintain the natural flow distribution and reduce backwater, caution must be exercised in proportioning the size and in locating such structures to avoid undue scour or changes in the course of the main river channel.

1.3.1.2 Usually, bridge waterways are sized to pass a design flood of a magnitude and frequency consistent with the type or class of highway. In the selection of the waterway opening, consideration should be given to the amount of upstream ponding, the passage of ice and debris and possible scour of the bridge foundations. Where floods exceeding the design flood have occurred, or where superfloods would cause extensive damage to adjoining property or the loss of a costly structure, a larger waterway opening may be warranted. Due consideration should be given to any Federal, State and local requirements.

1.3.1.3 Relief openings, spur-dikes, debris deflectors and channel training works should be used where needed to minimize the effect of adverse flood flow conditions. Where scour is likely to occur, protection against damage from scour should be provided in the design of bridge piers and abutments. Embankment slopes adjacent to structures subject to erosion should be adequately protected by rip-rap, flexible mattresses, retards, spur dikes or other appropriate construction. Clearing of brush and trees along embankments in the vicinity of bridge openings should be avoided to prevent high flow velocities and possible scour. Borrow pits should not be located in areas which would increase velocities and the possibility of scour at bridges.

1.3.2 HYDRAULIC STUDIES

Hydraulic studies of bridge sites are a necessary part of the preliminary design of a bridge and reports of such studies should include applicable parts of the following outline:

1.3.2.1 SITE DATA

(a) Maps, stream cross sections, aerial photographs.

(b) Complete data on existing bridges, including dates of construction and performance during past floods.

(c) Available high water marks with dates of occurrence.

(d) Information on ice, debris and channel stability.

(e) Factors affecting water stages such as high water from other streams, reservoirs, flood control projects and tides.

1.3.2.2 HYDROLOGIC ANALYSIS

(a) Flood data applicable to estimating floods at site, including both historical floods and maximum floods of record.

(b) Flood-frequency curve for site.

(c) Distribution of flow and velocities at site for flood discharges to be considered in design of structure.

(d) Stage-discharge curve for site.

1.3.2.3 HYDRAULIC ANALYSIS

(a) Backwater and mean velocities at bridge opening for various trial bridge lengths and selected discharges.

(b) Estimated scour depth at piers and abutments of proposed structures.

1.4 CULVERT LOCATION, LENGTH, AND WATERWAY OPENINGS

Culvert location, length and waterway openings should be in accordance with the AASHTO Guide on the Hydraulic Design of Culverts.

1.5 ROADWAY DRAINAGE

The transverse drainage of the roadway should be provided by a suitable crown in the roadway surface and longitudinal drainage by camber or gradient. Water flowing downgrade in a gutter section should be intercepted and not permitted to run onto the bridge. Short, continuous span bridges, particularly overpasses, may be built without inlets and the water from the bridge roadway carried downslope by open or closed chutes near the end of the bridge structure. Longitudinal drainage on long bridges should be provided by scuppers or inlets which should be of sufficient size and number to drain the gutters adequately. Downspouts, where required, should be made of rigid corrosion-resistant material not less than 4 inches in least dimension and should be provided with cleanouts. The details of deck drains should be such as to prevent the discharge of drainage water against any portion of the structure or on moving traffic below, and to prevent erosion at the outlet of the downspout. Deck drains may be connected to conduits leading to storm water outfalls at ground level. Overhanging portions of concrete decks should be provided with a drip bead or notch.

1.6 RAILROAD OVERPASSES

1.6.1 CLEARANCES

Structures designed to overpass a railroad shall be in accordance with standards established and used by the affected railroad in its normal practice. These overpass structures shall comply with applicable Federal, State, and local laws.

Regulations, codes, and standards should, as a minimum, meet the specifications and design standards of the American Railway Engineering Association, the Association of American Railroads, and AASHTO.

1.6.2 BLAST PROTECTION

On bridges over railroads with steam locomotives, metal likely to be damaged by locomotive gases, and all concrete surfaces less than 20 feet above the tracks, shall be protected by blast plates. The plates shall be placed to take account of the direction of blast when the locomotive is on level or superelevated tracks by centering them on a line normal to the plane of the two rails at the centerline of the tracks. The plates shall be not less than 4 feet wide and shall be cast-iron, a corrosion and blast resisting alloy or asbestos-board shields, so supported that they may be readily replaced. The thickness of plates and other parts in direct contact with locomotive blast shall be not less than 3/4 inch for cast iron, 3/8 inch for alloy, 1/2 inch for plain asbestos-board, and 7/16 inch for corrugated asbestos-board. Bolts shall be not less than 5/8 inch in diameter. Pockets which may hold locomotive gases shall be avoided as far as practical. All fastenings shall be galvanized or made of corrosion resistant material.

1.7 SUPERELEVATION

The superelevation of the floor surface of a bridge on a horizontal curve shall be provided in accordance with the standard practice of the commission for the highway construction, except that the superelevation shall not exceed 0.10 foot per foot width of roadway.

1.8 FLOOR SURFACES

All bridge floors shall have skid-resistant characteristics.

1.9 UTILITIES

Where required, provisions shall be made for trolley wire supports and poles, lighting pillars, electric conduits, telephone conduits, water pipes, gas pipes, sanitary sewers, and other utility appurtenances.

SECTION 2 - GENERAL FEATURES OF DESIGN

2.1 GENERAL

2.1.1 NOTATIONS

b	=	flange width (Article 2.7.4.3)
C	=	modification factor for concentrated load, P, used in the design of rail members (Article 2.7.1.3.1)
D	=	clear unsupported distance between flange components (Article 2.7.4.3)
d	=	depth of W or I section (Article 2.7.4.3)
F_a	=	allowable axial stress (Article 2.7.4.3)
F_b	=	allowable bending stress (Article 2.7.4.2)
F_v	=	allowable shear stress (Article 2.7.4.2)
F_y	=	minimum yield stress (Article 2.7.4.2)
f_a	=	axial compression stress (Article 2.7.4.3)
h	=	height of top rail above reference surface (Figure 2.7.4.C)
L	=	post spacing (Figure 2.7.4C)
P	=	railing design loading = 10 kips (Article 2.7.1.3 and Figure 2.7.4C)
P'	=	railing design loading equal to P, P/2 or P/3 (Article 2.7.1.3.5)
t	=	web thickness (Article 2.7.4.3)
w	=	pedestrian or bicycle loading (Article 2.7.4.3)

2.1.2 WIDTH OF ROADWAY AND SIDEWALK

The width of roadway shall be the clear width measured at right angles to the longitudinal centerline of the bridge between the bottoms of curbs. If brush curbs or curbs are not used, the clear width shall be the minimum width measured between the nearest faces of the bridge railing.

The width of the sidewalk shall be the clear width, measured at right angles to the longitudinal centerline of the bridge, from the extreme inside portion of the handrail to the bottom of the curb or guard-timber. If there is a truss, girder, or parapet wall adjacent to the roadway curb, the width shall be measured to the extreme walk side of these members.

2.2 STANDARD HIGHWAY CLEARANCES - GENERAL

2.2.1 NAVIGATIONAL

Permits for the construction of crossings over navigable streams, must be obtained from the U.S. Coast Guard, and other appropriate agencies. Requests for such permits from the U.S. Coast Guard should be addressed to the appropriate District Commander.

2.2.2 ROADWAY WIDTH

For recommendations on roadway widths for various volumes of traffic see AASHTO "A Policy on Design Standards-Interstate System," "Geometric Design Standards for Highways Other than Freeways," "A Policy on Geometric Design of Rural Highways," "A Policy on Design of Urban Highways and Arterial Sreets," or "Geometric Design Guide for Local Roads and Streets."

2.2.3 VERTICAL CLEARANCE

Vertical clearance on State trunk highways and interstate systems in rural areas shall be at least 16 feet over the entire roadway width with an allowance for resurfacing. On State trunk highways and interstate routes through urban areas, a 16-foot clearance shall be provided except in highly developed areas. A 16-foot clearance should be provided in both rural and urban areas where such clearance is not unreasonably costly and where needed for defense requirements. Vertical clearance on all other highways shall be at least 14 feet over the entire roadway width with an allowance for resurfacing.

2.2.4 OTHER

The channel openings and clearances shall be acceptable to agencies having jurisdiction over such matters. Channel openings and clearances shall conform in width, height, and location to all Federal, State and local requirements.

2.2.5 CURBS AND SIDEWALKS

The face of the curb is defined as the vertical or sloping surface on the roadway side of the curb. Horizontal measurements of roadway curbs are from the bottom of the face, or, in the case of stepped back curbs, from the bottom of the lower face. Maximum width of brush curbs, if used, shall be 9 inches.

Where curb and gutter sections are used on the roadway approach, at either or both ends of the bridge, the curb height on the bridge may equal or exceed the curb height on the roadway approach. Where no curbs are used on the roadway approaches, the height of the bridge curb above the roadway shall be not less than 8 inches, and preferably not more than 10 inches.

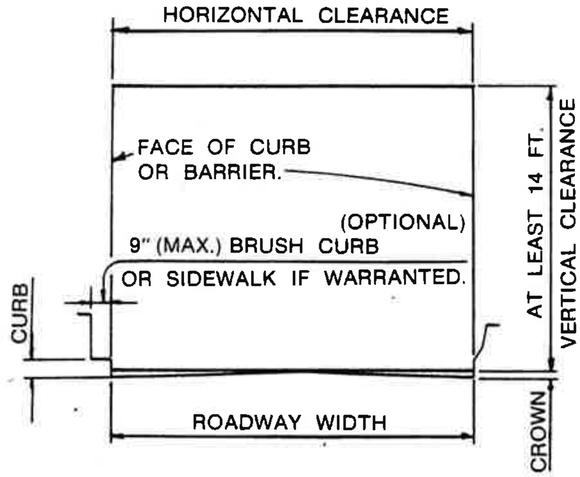
Where sidewalks are used for pedestrian traffic on urban expressways, they shall be separated from the bridge roadway by the use of a combination railing as shown in Figure 2.7.4B.

In those cases where a New Jersey type parapet or a curb is constructed on a bridge, particularly in urban areas that have curbs and gutters leading to a bridge, the same widths between curbs on the approach roadways will be maintained across the bridge structure. A parapet or other railing installed at or near the curb line shall have its ends properly flared, sloped or shielded.

2.3 HIGHWAY CLEARANCES FOR BRIDGES

2.3.1 WIDTH

The horizontal clearance shall be the clear width and the vertical clearance the clear height for the passage of vehicular traffic as shown in Figure 2.3.1.



Clearance Diagram for Bridges
Figure 2.3.1

The roadway width shall generally equal the width of the approach roadway section including shoulders. Where curbed roadway sections approach a structure, the same section shall be carried across the structure.

2.3.2 VERTICAL CLEARANCE

The provisions of Article 2.2.3 shall be used.

2.4 HIGHWAY CLEARANCES FOR UNDERPASSES (See Figure 2.4A)

2.4.1 WIDTH

The pier columns or walls for grade separation structures shall generally be located a minimum of 30 feet from the edges of the through traffic lanes. Where the practical limits of structure costs, type of a structure, volume and design speed of through traffic, span arrangement, skew and terrain make the 30 foot offset impractical, the pier or wall may be placed closer than 30 feet and protected by the use of guard rail or other barrier devices. The guard rail or other device shall be independently supported with the roadway face at least 2'-0" from the face of pier or abutment.

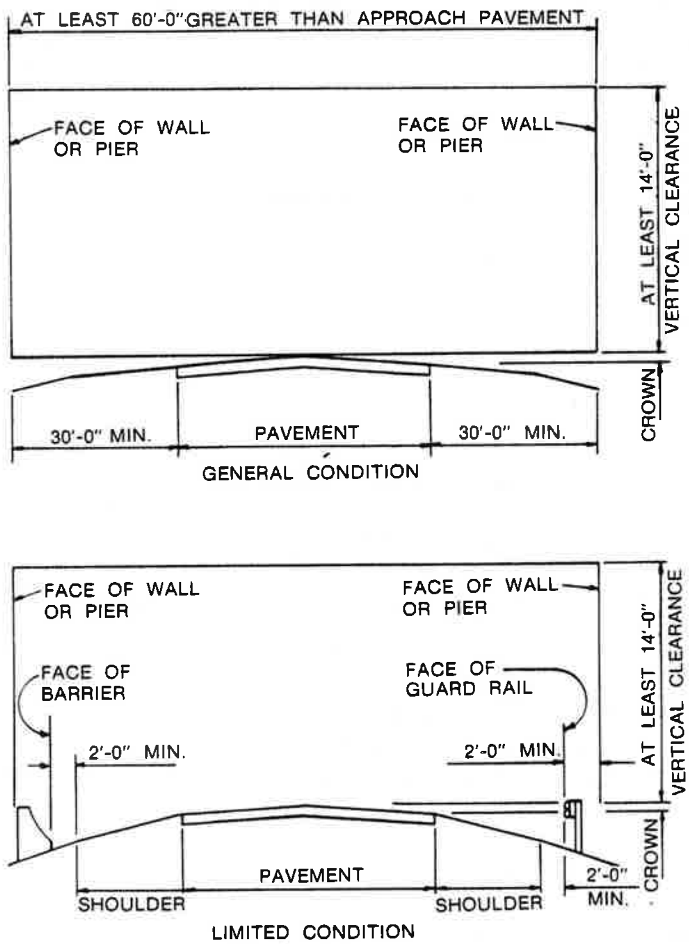
The face of the guard rail or other device shall be at least 2'-0" outside the normal shoulder line.

2.4.2 VERTICAL CLEARANCE

A vertical clearance of not less than 14 feet shall be provided between curbs, or if curbs are not used, over the entire width that is available for traffic.

2.4.3 CURBS

Curbs, if used, shall match those of the approach roadway section.



Clearance Diagrams for Underpasses
Figure 2.4A
See Article 2.4 For General Requirements

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*The barrier to face of wall or pier distance should not be less than the dynamic deflection of the barrier for impact by a full-size automobile at impact conditions of approximately 25 degrees and 60 mph. For information on dynamic deflection of various barriers, see AASHTO "Guide for Selecting, Locating, and Designing Traffic Barriers".

2.5 HIGHWAY CLEARANCES FOR TUNNELS
(See Figure 2.5)

2.5.1 ROADWAY WIDTH

The horizontal clearance shall be the clear width and the vertical clearance the clear height for the passage of vehicular traffic as shown in Figure 2.5.

Unless otherwise provided, the several parts of the structures shall be constructed to secure the following limiting dimensions or clearances for traffic.

The clearances and width of roadway for 2-lane traffic shall be not less than those shown in Figure 2.5. The roadway width shall be increased at least ten feet and preferably twelve feet for each additional traffic lane.

2.5.2 CLEARANCE BETWEEN WALLS

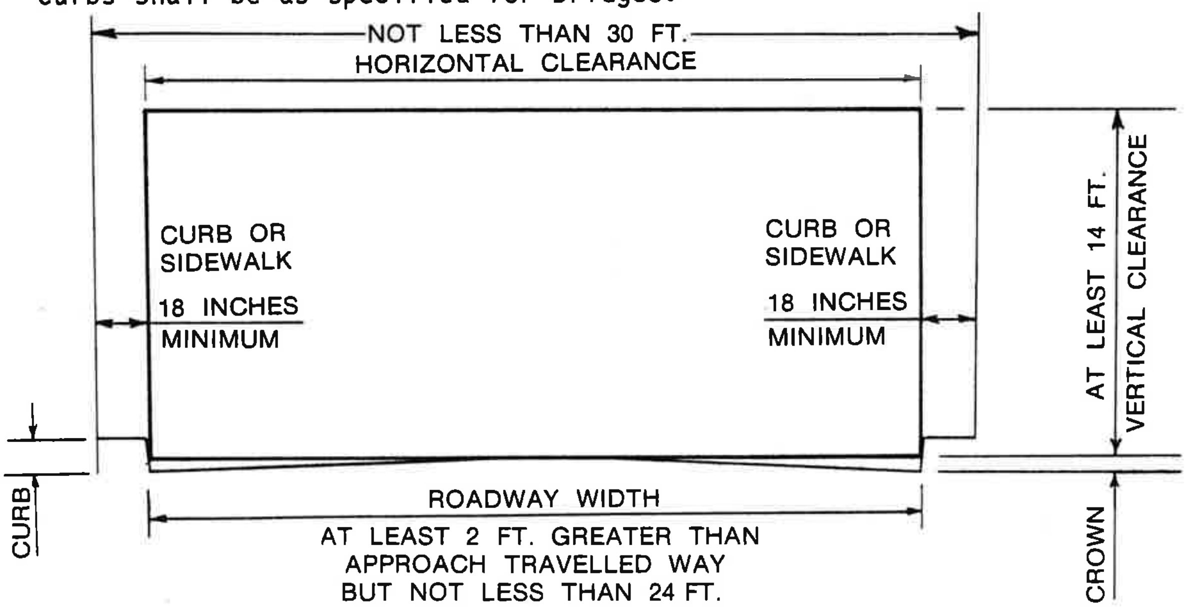
The minimum width between walls of two-lane tunnels shall be 30 feet.

2.5.3 VERTICAL CLEARANCE

The vertical clearance, between curbs, shall be not less than 14 feet.

2.5.4 CURBS

The width of curbs shall be not less than 18 inches. The height of curbs shall be as specified for bridges.



Clearance Diagram for Tunnels
Two Lane Highway Traffic
Figure 2.5

For heavy traffic roads, roadway widths greater than the above minima are recommended.

If traffic lane widths exceed 12 feet the roadway width may be reduced 2'-0" from that calculated from Figure 2.5.

2.6 HIGHWAY CLEARANCES FOR DEPRESSED ROADWAYS

2.6.1 ROADWAY WIDTH

The clear width between curbs shall be not less than that specified for tunnels.

2.6.2 CLEARANCE BETWEEN WALLS

The minimum width between walls for depressed roadways carrying two lanes of traffic shall be 30 feet.

2.6.3 CURBS

The width of curbs shall be not less than 18 inches. The height of curbs shall be as specified for bridges.

2.7 RAILINGS

Railings shall be provided along the edges of structures for protection of traffic and pedestrians.

Except on urban expressways, a pedestrian walkway may be separated from an adjacent roadway by a traffic railing or barrier with a pedestrian railing along the edge of the structure. On urban expressways, the separation shall be made by a combination railing.

2.7.1 VEHICULAR RAILING

2.7.1.1 GENERAL

2.7.1.1.1 While the primary purpose of traffic railing is to contain the average vehicle using the structure, consideration should also be given to (a) protection of the occupants of a vehicle in collision with the railing, (b) protection of other vehicles near the collision, (c) protection of vehicles or pedestrians on roadways underneath the structure, and (d) appearance and freedom of view from passing vehicles.

2.7.1.1.2 Materials for traffic railings shall be concrete, metal, timber, or a combination thereof. Metal materials with less than 10 percent tested elongation shall not be used.

2.7.1.1.3 Traffic railings should provide a smooth, continuous face of rail on the traffic side with the posts set back from the face of rail. Structural continuity in the rail members, in cluding anchorage of ends, is essential. The railing system shall be able to resist the applied loads at all locations.

2.7.1.1.4 Protrusions or depressions at rail joints shall be acceptable provided their thickness or depth is no greater than the wall thickness of the rail member or 3/8", whichever is less.

2.7.1.1.5 Careful attention shall be given to the treatment of railings at the bridge ends. Exposed rail ends, posts, and sharp changes in the geometry of the railing shall be avoided. A smooth transition by means of a continuation of the bridge barrier, guard rail anchored to the bridge end, or other effective means shall be provided to protect the traffic from direct collision with the bridge rail ends.

2.7.1.2 GEOMETRY

2.7.1.2.1 The heights of rails shall be measured relative to the reference surface which shall be the top of the roadway, the top of the future overlay if resurfacing is anticipated, or the top of curb when the curb projection is greater than 9 inches from the traffic face of the railing.

2.7.1.2.2 Traffic railings and traffic portions of combination railings shall not be less than 2'-3" from the top of the reference surface. Parapets designed with sloping traffic faces intended to allow vehicles to ride up them under low angle contacts shall be at east 2'-8" in height.

2.7.1.2.3 The lower element of a traffic or combination railing should consist of either a parapet projecting at least 18" above the reference surface or a rail centered between 15 and 20 inches above the reference surface.

2.7.1.2.4 For traffic railings, the maximum clear opening below the bottom rail shall not exceed 17 inches and the maximum opening between succeeding rails shall not exceed 15 inches. For combination and pedestrian railings, the maximum clear vertical opening between the lowest rail and reference surface shall not exceed 15 inches and the maximum opening between succeeding rails shall not exceed 15 inches.

2.7.1.2.5 The traffic faces of all traffic rails must be within one inch of a vertical plane through the traffic face of the rail closest to traffic.

2.7.1.3 LOADS

2.7.1.3.1 When the height of the top of the top traffic rail exceeds 2'-9", the total transverse load distributed to the traffic rails and posts shall be increased by the factor "C". However, the maximum load applied to any one element need not exceed "P", the transverse design load.

2.7.1.3.2 Rails whose traffic face is more than 1" behind a vertical plane through the face of the traffic rail closest to traffic or centered less than 15 inches above the reference surface shall not be considered to be traffic rails for the purpose of distributing "P" or "CP", but may be considered in determining the maximum clear vertical opening, provided they are designed for a transverse loading equal to that applied to an adjacent traffic rail or "P/2" whichever is less.

2.7.1.3.3 Transverse loads on posts, equal to "P", or "CP", shall be distributed as shown in Figures 2.7.4B and 2.7.4C. A load equal to 1/2 the transverse load on a post shall simultaneously be applied longitudinally, divided among not more than four posts in a continuous rail length. Each traffic post shall also be designed to resist an independently applied inward load equal to 1/4 the outward transverse load.

2.7.1.3.4 The attachment of each rail required in a traffic or combination railing shall be designed to resist a vertical load equal to 1/4 of the transverse design load of the rail. The vertical load shall be applied alternately upward or downward. The attachment shall also be designed to resist an inward transverse load equal to 1/4 the transverse rail design load.

2.7.1.3.5 Rail members shall be designed for a moment, due to concentrated loads, at the center of the panel and at the posts of P'L/6 where L is the post spacing and P' is equal to P, P/2, or P/3, as modified by the factor "C" where required. The handrail members of combination railings shall be designed for a moment at the center of the panel and at the posts of $0.1wL^2$.

2.7.1.3.6 The transverse force on concrete parapet and barrier walls shall be spread over a longitudinal length of 5 feet.

2.7.1.3.7 Railings other than those shown in Figures 2.7.4B and 2.7.4C are permissible provided they meet the requirements of this Article. Railing configurations which have been successfully tested by full scale impact tests are exempt from the provisions of this Article.

2.7.2 BICYCLE RAILING

2.7.2.1 GENERAL

2.7.2.1.1 Bicycle railing shall be used on bridges specifically designed to carry bicycle traffic, and on bridges where specific protection of bicyclists is deemed necessary.

2.7.2.1.2 Railing components shall be designed with consideration to safety; appearance and, when the bridge carries mixed traffic, freedom of view from passing vehicles.

2.7.2.2 GEOMETRY AND LOADS

2.7.2.2.1 The minimum height of a railing used to protect a bicyclist shall be 54 inches, measured from the top of the surface on which the bicycle rides to the top of the top rail.

2.7.2.2.2 Within a band bordered by the riding surface and a line 54 inches above it, horizontal elements of the railing assembly shall have a maximum clear spacing of 15 inches. Vertical elements of the railing assembly shall have a maximum clear spacing of 8 inches. If a railing assembly employs both horizontal and vertical elements, the spacing requirements shall apply to one or the other, but not to both. Chain link fence is exempt from the rail spacing requirements listed above. In general, rails should project beyond the face of posts and/or pickets.

2.7.2.2.3 The minimum design loadings for bicycle railing shall be w = 50 lb. per linear foot transversely and vertically, acting simultaneously on each rail.

2.7.2.2.4 Design loads for rails located more than 54 inches above the riding surface shall be determined by the designer.

2.7.2.2.5 Posts shall be designed for a transverse load of wL (where L is the post spacing) acting at the center of gravity of the upper rail, but at a height not greater than 54 inches.

2.7.2.2.6 Refer to Figures 2.7.4A and 2.7.4C for more information concerning the application of loads.

2.7.3 PEDESTRIAN RAILING

2.7.3.1 GENERAL

2.7.3.1.1 Railing components shall be proportioned commensurate with the type and volume of anticipated pedestrian traffic. Consideration should be given to appearance, safety and freedom of view from passing vehicles.

2.7.3.1.2 Materials for pedestrian railing may be concrete, metal, timber, or a combination thereof.

2.7.3.2 GEOMETRY AND LOADS

2.7.3.2.1 The minimum height of a pedestrian railing shall be 3'-6" measured from the top of the walkway to the top of the upper rail member.

2.7.3.2.2 The minimum design loading for pedestrian railing shall be w = 50 lb. per lin. ft., transversely and vertically, acting simultaneously on each longitudinal member.

Rail members located more than 5'-0" above the walkway are excluded from these requirements.

2.7.3.2.3 Posts shall be designed for a transverse load of wL (where L is the post spacing) acting at the center of gravity of the upper rail or, for high rails, at 5'-0" maximum above the walkway.

2.7.3.2.4 Refer to Figures 2.7.4A and 2.7.4B for more information concerning the application of loads.

2.7.4. STRUCTURAL SPECIFICATIONS AND GUIDELINES

2.7.4.1 Railings shall be designed by the elastic method to the allowable stresses for the appropriate material. For aluminum alloys 6061-T6, 6063-T6, and 6351-T5, the design stresses given in Tables 1.5.2A(1), 1.5.2A(2), 1.5.2(3) of the "AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" shall apply; for cast aluminum alloys A444.0-T4, A356.0-T61, and A356.0-T6, the design stresses given in Table 1.5.2A(5) shall apply. Aluminum railing shall be fabricated and built in accordance with the provisions of Section 6 of April 1976 "Specifications for Aluminum Bridge and other Highway Structures" published by the Aluminum Association for riveted and bolted fabrication, and in accordance with Section 1.5.5 of the "AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," for welded fabrication.

2.7.4.2 The allowable unit stresses for steel shall be as given in Article 10.2, except as modified below.

For steels not generally covered by the "Standard Specifications," but having a guaranteed yield strength, F_y, the allowable unit stress, shall be derived by applying the general formulas as given in the "Standard Specifications" under "Unit Stresses" except as indicated below.

The allowable unit stress for shear shall be $F_v = 0.33F_y$.

Round or oval steel tubes may be proportioned using an allowable bending stress, $F_b = 0.66F_y$ provided the R/t ratio (radius/thickness) is less than or equal to 40.

Square and rectangular steel tubes, and steel W and I sections in bending with tension and compression on extreme fibers of laterally supported compact sections having an axis of symmetry in the plane of loading may be designed for an allowable stress $F_b = 0.60F_y$.

2.7.4.3 The requirements for a compact section are as follows:

(a) The width to thickness ratio of projecting elements of the compression flange of W and I sections shall not exceed

$$\frac{W}{t} \leq \frac{1600}{\sqrt{F_y}} \qquad (2\text{-}1)$$

(b) The width to thickness ratio of the compression flange of square or rectangular tubes shall not exceed

$$\frac{W}{t} \leq \frac{6000}{\sqrt{F_y}} \qquad (2\text{-}2)$$

(c) The D/t ratio of webs shall not exceed

$$\frac{D}{t} \leq \frac{13000}{\sqrt{F_y}} \qquad (2\text{-}3)$$

(d) If subject to combined axial force and bending, the D/t ratio of webs shall not exceed

$$\frac{D}{t} \leq \frac{13{,}300\left[1-1.43\left(\frac{f_a}{F_a}\right)\right]}{\sqrt{F_y}} \qquad (2\text{-}4)$$

but need not be less than

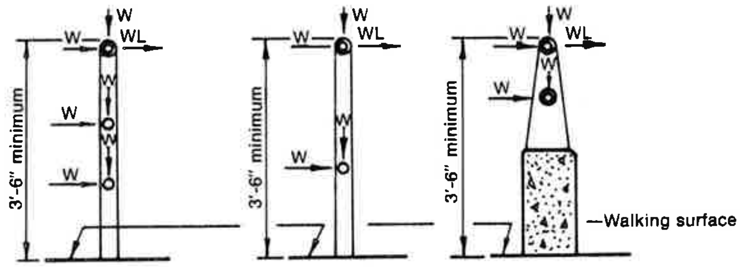
$$\frac{7000}{\sqrt{F_y}} \qquad (2\text{-}5)$$

(e) The distance between lateral supports (ℓ) in inches of W or I sections shall not exceed

$$\ell \leq \frac{2400b}{\sqrt{F_y}} \qquad (2\text{-}6)$$

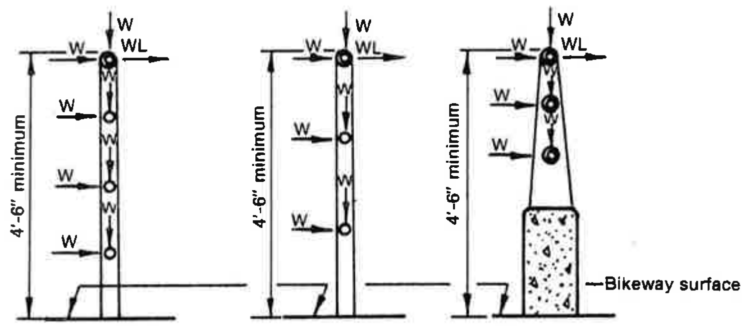
nor

$$\ell \leq \frac{20{,}000{,}000A_f}{d\,F_y} \qquad (2\text{-}7)$$



(To be used adjacent to a sidewalk when highway traffic is separated from pedestrian traffic by a traffic railing.)

PEDESTRIAN RAILING



NOTE:

1. If screening or solid-face is presented then number of rails may be reduced - wind loads must be added if solid face is utilized.

BICYCLE RAILING

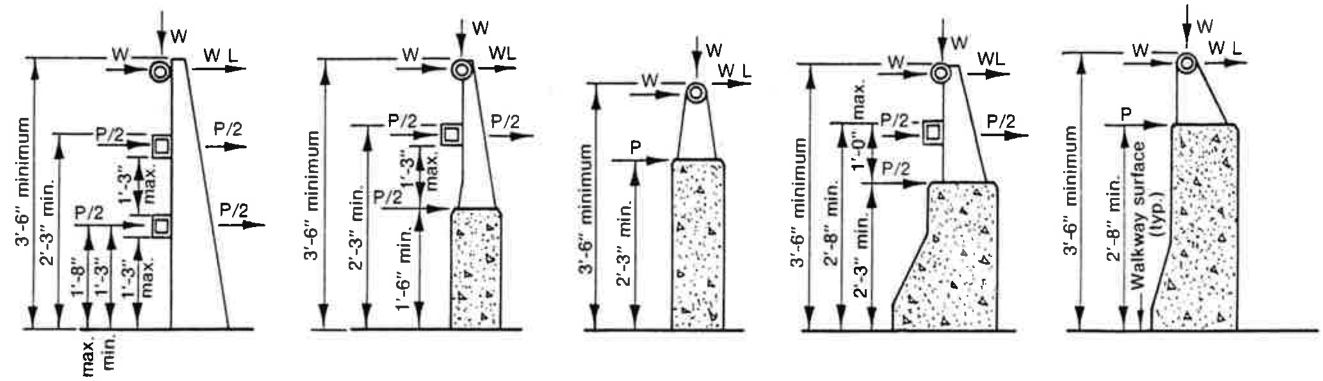
NOTES:

1. Loadings on left are applied to rails.
2. Loads on right are applied to posts.
3. The shapes of rail members are illustrative only. Any material or combination of materials listed in Article 2.7 may be used in any configuration.

NOMENCLATURE:

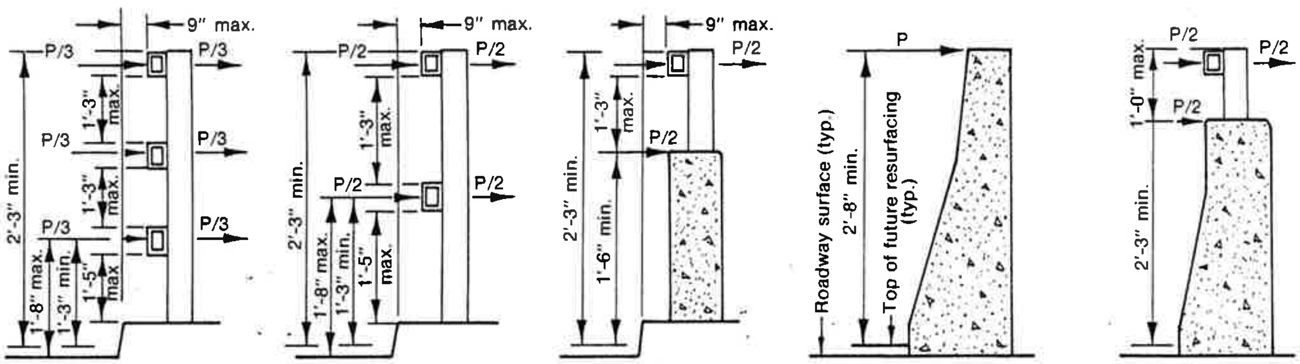
w = Pedestrian or bicycle loading per unit length of rail
L = Post spacing

Figure 2.7.4A



(To be used where there is no curb or curb projects 9" or less from traffic face of railing.)

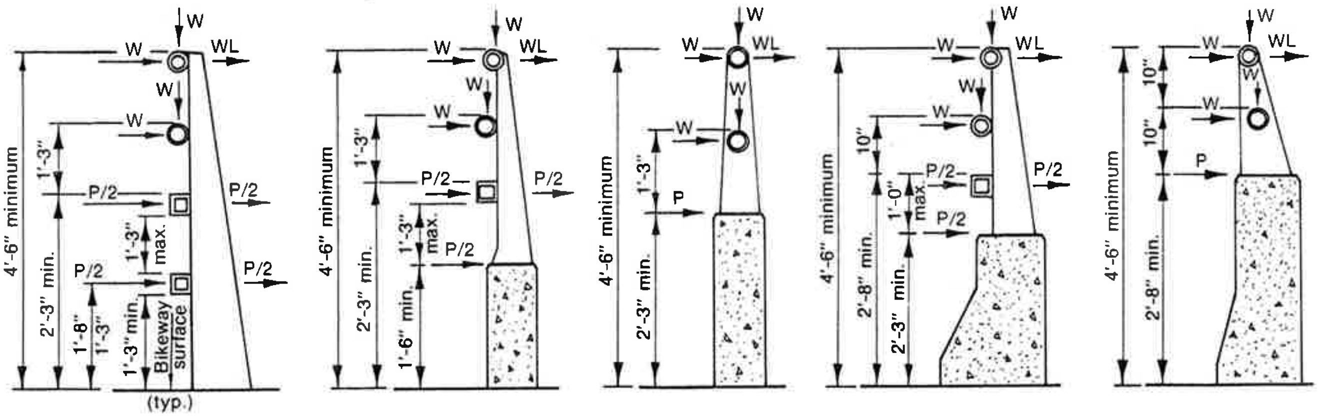
TRAFFIC RAILING



(To be used when curb projects more than 9" from the traffic face of railing).

COMBINATION TRAFFIC AND PEDESTRIAN RAILING

Figure 2.7.4B



COMBINATION TRAFFIC AND BICYCLE RAILING

NOTES:

1. Loadings on left are applied to rails.
2. Loadings on right are applied to posts.
3. The shapes of rail members are illustrative only. Any material or combination of materials listed in Article 2.7 may be used in any configuration.

NOMENCLATURE:

P = Highway design loading = 10 kips
h = Height of top of top rail above reference surface (in.)
L = Post spacing (ft.)
w = Pedestrian loading per unit length of rail

$$C = 1 + \frac{h-33}{18} \geq 1$$

Figure 2.7.4C

SECTION 3 - LOADS

PART A - TYPE OF LOADS

3.1 NOTATIONS

A = maximum expected acceleration of bedrock at the site (Article 3.21.1.2)

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 (Article 3.21.1.2)

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 (Article 3.21.4)

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 (Article 3.21.1.1)

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} (Article 3.21.1.2)

I = impact fraction (Article 3.8.2)

ICE = ice pressure (Article 3.22)

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)

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)

Mx = primary bending moment (Article 3.25.1.3)

$\bar{M}_y$ = total transferred secondary moment (Article 3.25.1.4)

N_B = number of beams (Article 3.28.1)

N_g = number of longitudinal beams (Article 3.23.4)

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 (Article 3.21.1.3)

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)

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 (Article 3.21.1.2)

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)

$\bar{R}_y$ = total secondary shear transferred (Article 3.25.1.4)

S = design speed (Article 3.10.1)

S = soil amplification spectral ratio (Article 3.21.1.2)

S = shrinkage (Article 3.22)

S = average stringer spacing (Article 3.23.2.3.1)

S = spacing of beams (Article 3.23.3)

S = parameter used in determining the load fraction to be applied to precast concrete beams (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 (Article 3.21.1.3)

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 3.7.3A)

W = width of sidewalk (Article 3.14.1.1)

W = wind load on structure (Article 3.22)

W = total dead weight of the structure (Article 3.21.1.1)

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 (Article 3.21.1.2)

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)

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.

3.2.2 Members shall be proportioned using the allowable stresses permitted by the design procedure and the limitations imposed by the material.

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.

3.3 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.

3.3.3 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, 1/4 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:

	#/ft3
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	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, guard rails, and fastenings (per linear foot of track)	200
Stone masonry	170
Asphalt plank, 1 inch thick	9 lb. per square foot

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

3.5.1 For all loadings less than H20, provision shall be made for an infrequent heavy load by applying Loading Combination IA (See Article 3.22), with the live load assumed 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.

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 ten feet.

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.

3.6.3 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 which 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 3.7.2.B. 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.

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 H20. 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.

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*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6.B) 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 were developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing 3/4 as much as the basic truck.

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:

H15 Loading, 1944 Edition shall be designated H15-44
H20 Loading, 1944 Edition shall be designated H20-44
H15-S12 Loading, 1944 Edition shall be designated HS15-44
H20-S16 Loading, 1944 Edition shall be designated HS20-44

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 LIVE LOAD

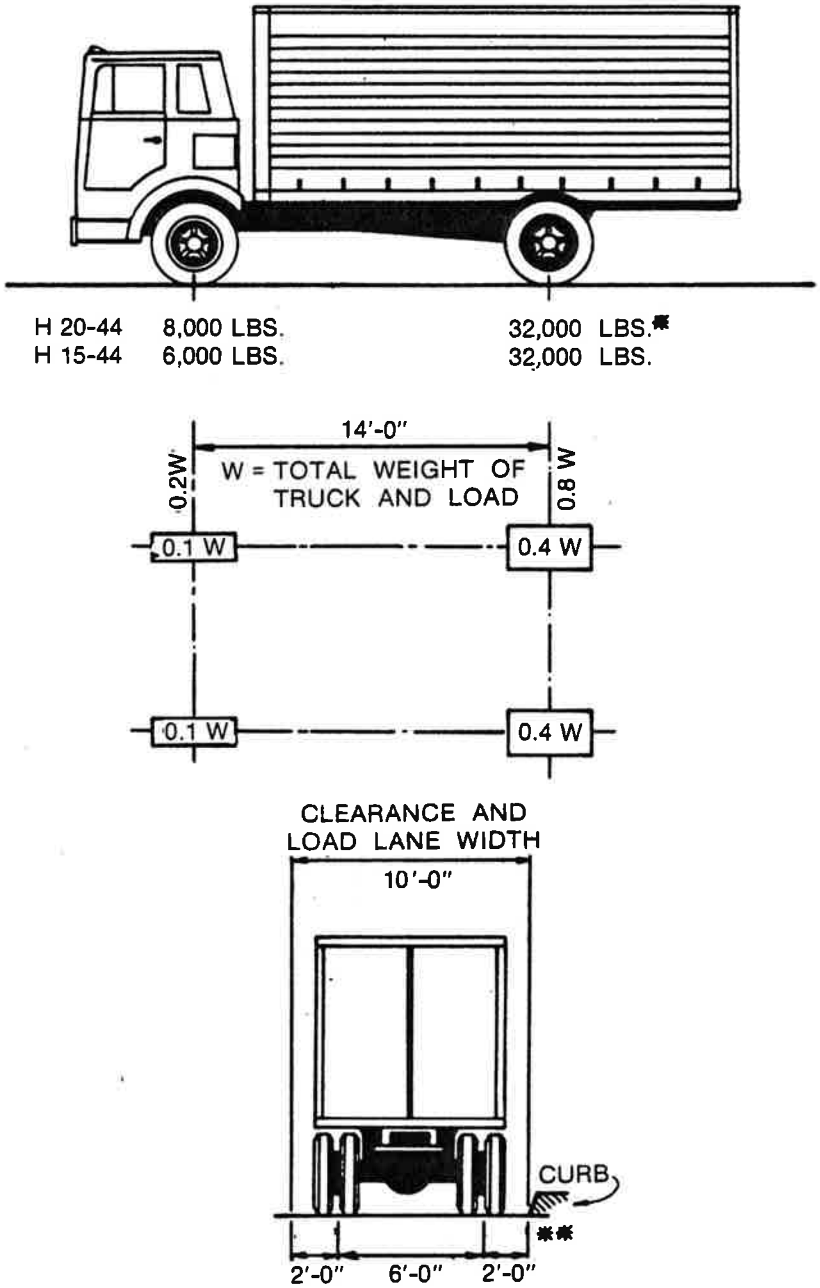
For trunk highways, or for other highways which carry, or which may carry, heavy truck traffic, the minimum live load shall be the HS 15 designated herein.

3.7.5 LOADINGS ON INTERSTATE HIGHWAY BRIDGES

Bridges supporting Interstate highways shall be designed for HS 20-44 loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000, whichever loading produces the greatest stress.

3.7.6 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.

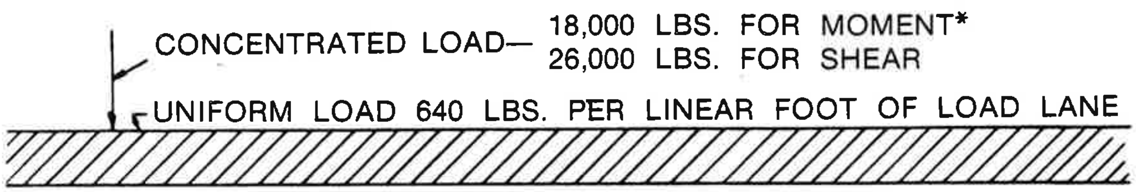


STANDARD H TRUCKS

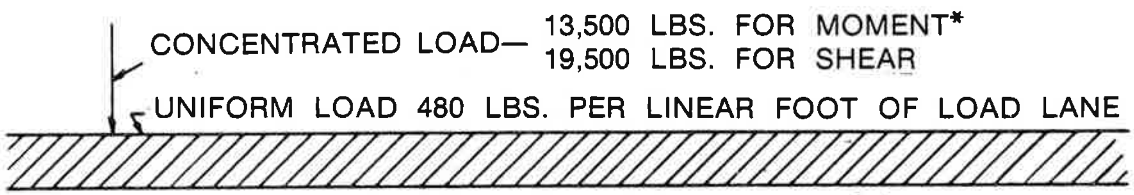
Figure 3.7.6A

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H20 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.)



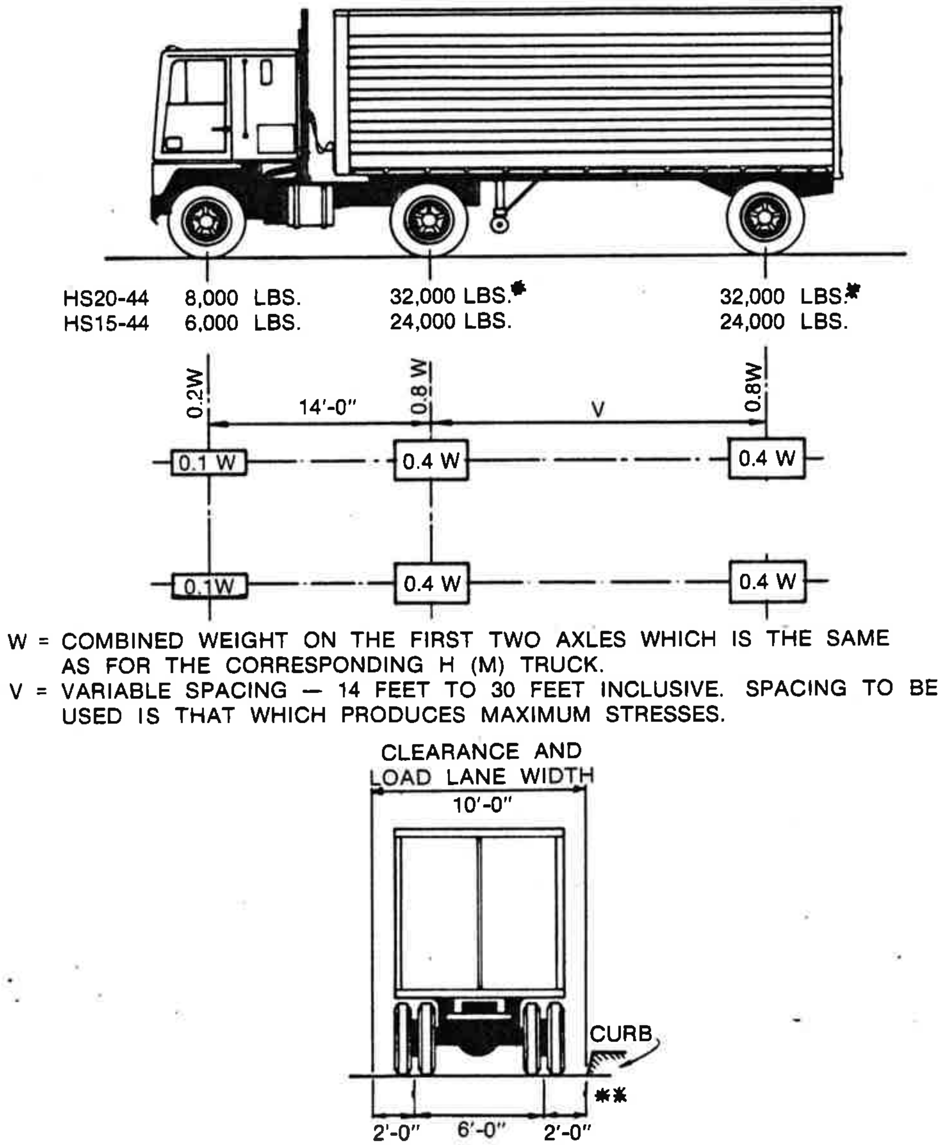
H20-44 LOADING
HS20-44 LOADING



H15-44 LOADING
HS15-44 LOADING

Figure 3.7.6B

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



STANDARD HS TRUCKS

Figure 3.7.7A

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS20 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).

3.7.7 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

Live load stresses produced by H or HS loadings shall be increased for items in Group A to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B.

3.8.1.1 GROUP A

(1) Superstructure, including steel or concrete supporting columns, steel towers, legs of rigid frames and generally those portions of the structure which extend down to the main foundation.

(2) The portions above the ground line of concrete or steel piles which are rigidly connected to the superstructure as in rigid frame or continuous structures:

3.8.1.2 GROUP B

(1) Abutments, retaining walls, piers, piles, except Group A(2).
(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).
L = length in feet of the portion of the span which 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.

(f) For culverts with cover 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

3.9.1 Provision shall be made for the effect of a longitudinal force of five 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.

3.9.2 Provision shall be made for the longitudinal forces due to friction at expansion bearings or shear resistance at elastomeric bearings.

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:

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

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 3.7.2.B 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 type 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 which 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 which 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.

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

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.

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, then 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
For girders and beams .... 50 pounds per square foot

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.

3.15.1.2 GROUP III AND GROUP VI LOADINGS

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 Degrees	Lateral Load PSF	Longitudinal Load PSF	Lateral Load PSF	Longitudinal Load 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 Degrees	Lateral Load lb./ft.	Longitudinal Load 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 to be 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.
Cold climate, from -30 to 120 F.

Concrete Structures	Temperature Rise	Temperature Fall
Moderate climate	30 F	40 F
Cold climate	35 F	45 F

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 FORCE FROM STREAM CURRENT, FLOATING ICE AND DRIFT

All piers and other portions of structures which 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

The effect of flowing water on piers shall be calculated by the formula:

$$P = KV^2 \qquad (3\text{-}4)$$

where:

P = pressure in pounds per square foot.
V = velocity of water in feet per second.
K = a constant, being 1 3/8 for square ends, 1/2 for angle ends where the angle is 30 degrees or less, and 2/3 for circular piers.

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 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:

$$F = C_n ptw \qquad (3\text{-}5)$$

where

F = horizontal ice force on pier, in pounds
C_n = coefficient for nose inclination from table
p = effective ice strength in pounds/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 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 break-up 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 break-up occurs at melting temperatures, but the ice moves in large pieces and is internally sound.

(c) In the order of 300 psi where at break-up 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 break-up 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 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

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

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.

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 the following criteria.

3.21.1 EQUIVALENT STATIC FORCE METHOD

For structures with supporting members of approximately equal stiffness, an equivalent horizontal force (EQ) may be applied to the structure. The distribution of the force shall consider the stiffness of the superstructure and supporting members, abutment restraint, and the deflected position of the structure.

3.21.1.1

$$EQ = C \cdot F \cdot W \qquad (3\text{-}6)$$

where

EQ = The equivalent static horizontal force applied at the center of gravity of the structure.

F = Framing Factor
F = 1.0 for structures where single columns or piers resist the horizontal forces.
F = 0.8 for structures where continuous frames resist horizontal forces applied along the frame.
W = The total dead weight of the structure in pounds.

3.21.1.2

$$C = A \cdot R \cdot S/Z \qquad (3\text{-}7)$$

where

C = Combined Response Coefficient

The calculated coefficient "C" shall not be less than 0.10 for structures with "A" greater than or equal to 0.3 g and 0.06 for structures with "A" less than 0.3g.

Values of coefficients for various depths of alluvium to rock-like material given in Figures 3.21.1A, B, C and D may be used.

A = Maximum expected acceleration of bedrock at the site. Seismic risk map of the United States (shown in Figures 3.21.1 E, F, and G with following assignment of maximum expected rock acceleration may be used. More exact peak rock acceleration values should be used in areas where "Maximum Expected Rock Acceleration" maps are available.

Zone I	A = 0.09 g
Zone II	A = 0.22 g
Zone III	A = 0.50 g

g = 32.2 ft./sec.^{2}
R = Normalized rock response.
S = Soil amplification spectral ratio.
Z = Reduction for ductility and risk assessment.

3.21.1.3

$$T = 0.32\sqrt{\frac{W}{P}} \qquad (3\text{-}8)$$

where

T = The period of vibration of the structure (sec.)
P = Total uniform force, pounds required to cause a one-inch maximum horizontal deflection of the whole structure

The period of vibration may also be computed using dynamic analysis techniques.

3.21.2 RESPONSE SPECTRUM METHOD

3.21.2.1 For complex structures, a response spectrum dynamic approach should be used for seismic analysis.

3.21.2.2 The combined response curves "C" given in figures 3.21.1A, B, C and D, or equivalent curves, modified by the framing factor, "F", may be used as the design response spectrum.

3.21.3 SPECIAL CASES

Structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 33.0 sec. will be considered special cases. These structures will be required to be designed using current seismicity, soil response and dynamic analysis techniques.

3.21.4 DESIGN OF RESTRAINING FEATURES

3.21.4.1 Restraining features to limit the displacement of the superstructure,--i.e., hinge ties, shear blocks, etc.,--shall be designed for the following force:

EQ = 0.25 x contributing DL minus column shears due to EQ. (3-9)

3.21.4.2 "Contributing DL" is determined by examining the entire frame. For example, a simple span fixed at one end and sliding at the other will have the entire superstructure as the "Contributing DL" for longitudinal forces at the fixed abutment, while one half of the superstructure DL will act at each abutment for transverse forces.

3.21.4.3 For a frame, such as a 2-span structure, the full length of the bridge should be used as the contribution length in the longitudinal direction. The resulting force can be reduced by deducting the shear in the column due to earthquake.

3.21.4.4 For hinge restrainers use 0.25 x DL of the smaller of the 2 frames and deduct the column shears due to EQ.

RESPONSE COEFFICIENT "C" FOR VARIOUS VALUES
OF PEAK ROCK ACCELERATION "A"

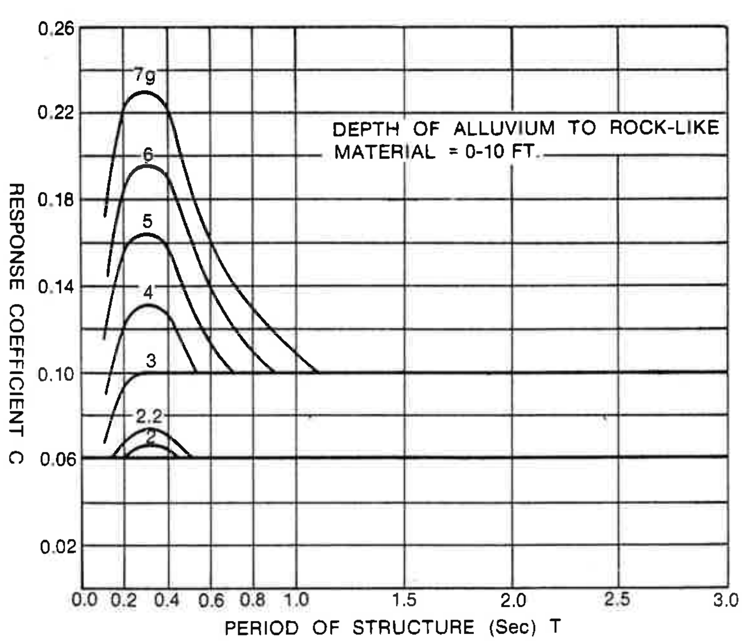


Figure 3.21.1A

RESPONSE COEFFICIENT "C" FOR VARIOUS VALUES OF PEAK ROCK ACCELERATION "A"

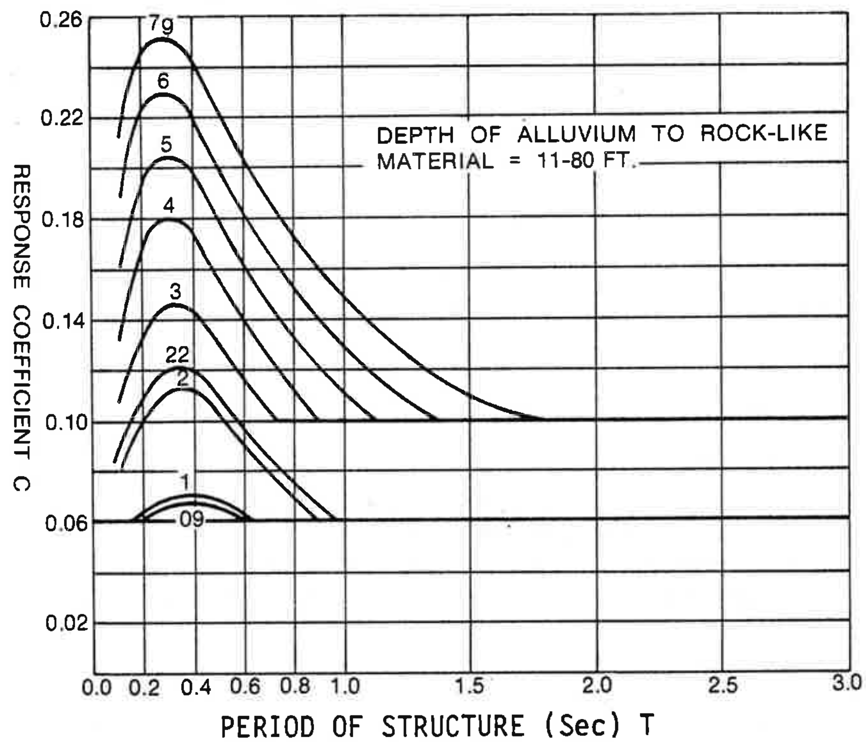


Figure 3.21.1B

RESPONSE COEFFICIENT "C" FOR VARIOUS VALUES OF PEAK ROCK ACCELERATION "A"

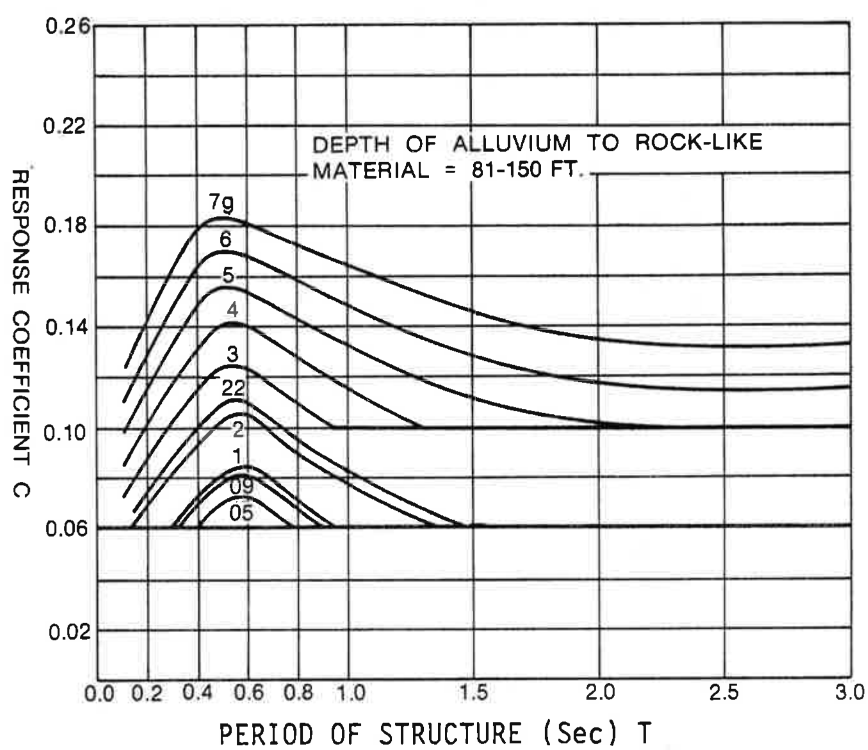


Figure 3.21.1C

RESPONSE COEFFICIENT "C" FOR VARIOUS VALUES
OF PEAK ROCK ACCELERATION "A"

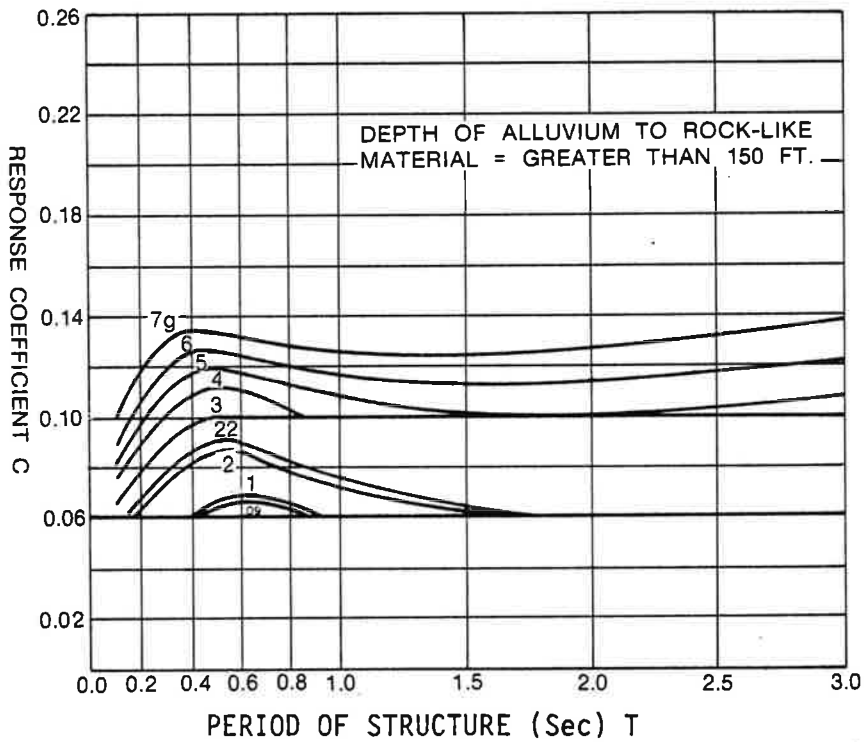


Figure 3.21.1D

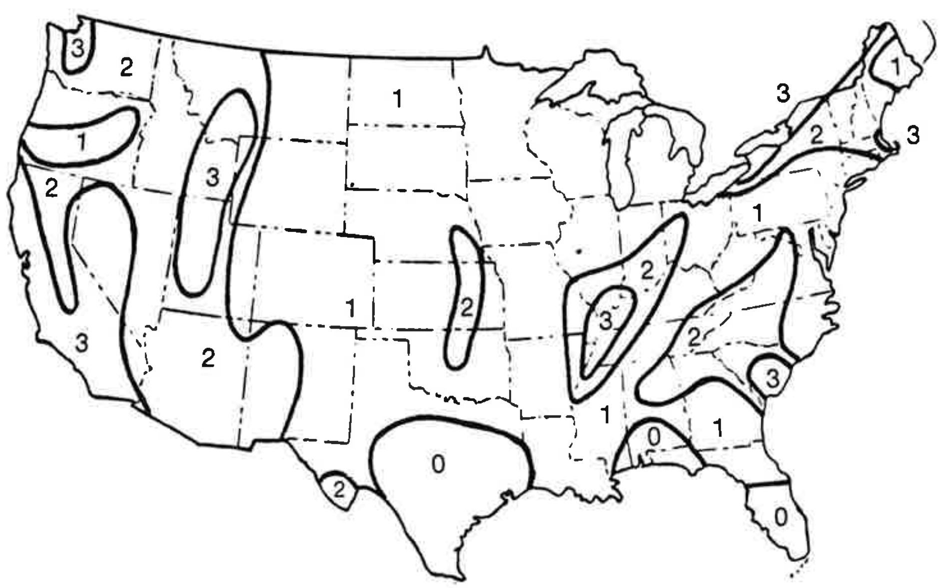


Figure 3.21.1E - Seismic risk map of the United States

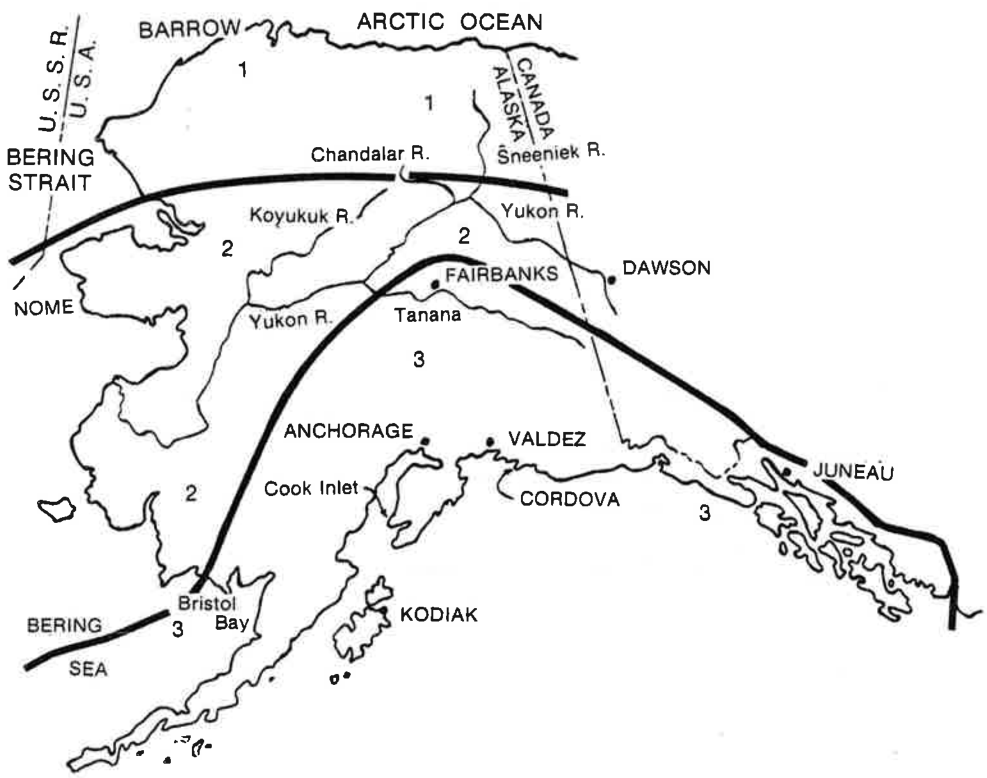


Figure 3.21.1F - Seismic zone map of Alaska

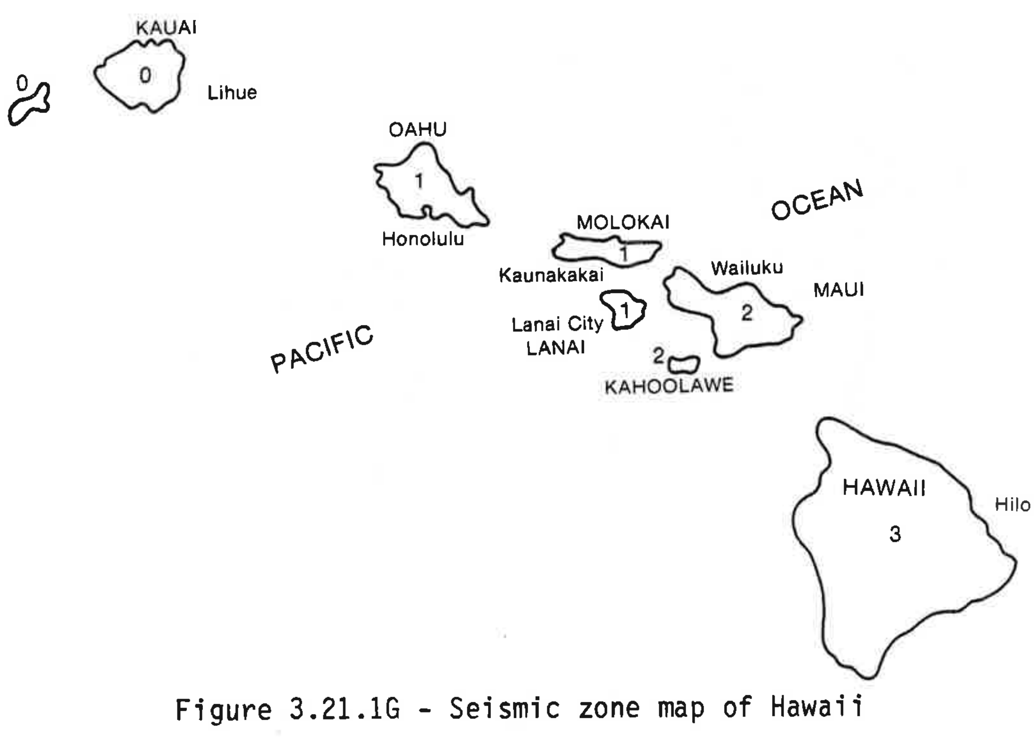


Figure 3.21.1G - Seismic zone map of Hawaii

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:

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

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 shown 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 are only intended for designing structural members by the load factor concept. The actual loads should not be increased by the factors shown in the table when designing foundations (soil pressure, pile loads, etc.). The load factors are also not intended to be used when checking the foundation stability (safety factors against overturning, sliding, etc.) of a structure.

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 than 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 lead shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H20, Group IA loading combination shall be used (See Article 3.5).

Table 3.22.1A
Table of Coefficients γ and β

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
			β FACTORS														
GROUP		γ	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		2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3		0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3		0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3		1	0	1		1	1	0.3	1	1	0	0	0		
	IV	1.3		1	0	1		1	1	0	0	0	1	0	0		
	V	1.25		0	0	0		1	1	1	0	0	1	0	0		
	VI	1.25		1	0	1		1	1	0.3	1	1	1	0	0		
	VII	1.3		0	0	0		1	1	0	0	0	0	1	0		
	VIII	1.3		1	0	1		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

$(L+I)_n$ - 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.

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

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 = 0.70 for vertical loads on Reinforced Concrete Boxes.
β_E = 1.00 for lateral loads on Reinforced Concrete Boxes.
β_E = 1.00 for vertical and lateral loads on all other culverts.

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 = 1.0 for vertical earth pressure
β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity ....
β_D = 1.0 when checking member for maximum axial load and minimum moment ....
For Column Design

β_D = 1.0 for flexural and tension members
β_E = 1.0 for Rigid Culverts including Reinforced Concrete Boxes
β_E = 1.5 for Flexible Culverts

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

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 end at which the stress is being determined.

3.23.1.2 Lateral distribution of the wheel load shall be that produced by assuming the flooring to act as a simple span between stringers or beams. For loads 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.

---

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

**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.

Table 3.23.1

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or More Traffic Lanes
Timber:[1]		
Plank[2]	S/4.0	S/3.75
Nail laminated[3] 4" thick or multiple layer[4] floors over 5" thick ...	S/4.5	S/4.0
Nail laminated[3] 6" or more thick	S/5.0 If S exceeds 5' use footnote 6.	S/4.25 If S exceeds 6.5' use footnote 6.
Glued Laminated[5] 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 6.	S/5.0 If S exceeds 7.5' use footnote 6.
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 6.	S/4.5 If S exceeds 7' use footnote 6.
Concrete:		
On Steel I-Beam Stringers[7] and Prestressed Concrete Girders ..	S/7.0 If S exceeds 10' use footnote 6.	S/5.5 If S exceeds 14' use footnote 6.
On Concrete T-Beams	S/6.5 If S exceeds 6' use footnote 6.	S/6.0 If S exceeds 10' use footnote 6.
On Timber Stringers	S/6.0 If S exceeds 6' use footnote 6.	S/5.0 If S exceeds 10' use footnote 6.
Concrete Box Girders[8]	S/8.0 If S exceeds 12' use footnote 6.	S/7.0 If S exceeds 16' use footnote 6.
On Steel Box Girders	See Article 10.39.2.	
On Prestressed Concrete Spread Box Beams	See Article 3.28.	

Table 3.23.1
(continued)

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or More Traffic Lanes
Steel Grid:		
(Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6' use footnote 6.	S/5.0 If S exceeds 10.5' use footnote 6.
Steel Bridge Corrugated Plank[9] (2" min. depth)	S/5.5	S/4.5

S = average stringer spacing in feet.

[1]Timber dimensions shown are for nominal thickness.

[2]Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (See Article 20.17 - Division II).

[3]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 preceeding piece (See Article 20.18 - Division II).

[4]Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (See Article 20.17 - Division II).

[5]Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (See Article 20.1.1 - Division II).

[6]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.

[7]"Design of I-Beam Bridges" by N. M. Newmark - Proceedings, ASCE, March 1948.

[8]The sidewalk live load (see Article 3.15) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

[9]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" x 2" steel corrugated plank. The factors should yield safe results for other corrugation configurations provided primary bending stiffness is the same as or greater than the 6" x 2" corrugated plank used in the studies.

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.

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:

$$\frac{S}{5.5} \quad \text{where } S = 6' \text{ or less}$$

$$\frac{S}{4.0 + 0.25S} \quad \text{where } S \text{ is more than } 6' \text{ and less than } 14'$$

When S is 14' or more, use footnote 6, Article 3.23.2.2.

S = distance in feet between outside and adjacent interior stringers.

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, 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 longitudidnal 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

Kind of Floor	Fraction of Wheel Load to Each Floor Beam
Plank[1,2]	$\frac{S}{4}$
Nail laminated[3] or glued laminated[5], 4-inches in thickness, or multiple layer[4] floors more than 5 inches thick	$\frac{S}{4.5}$
Nail laminated[3] or glued laminated[5], 6 inches or more in thickness	$\frac{S^*}{5}$
Concrete	$\frac{S^*}{6}$
Steel grid (less than 4 inches thick)	$\frac{S}{4.5}$
Steel grid (4 inches or more)	$\frac{S^*}{6}$
Steel bridge corrugated plank (2 inches minimum depth)	$\frac{S}{5.5}$

S = spacing of floor beams in feet.

*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.

See Table 3.23.1 for footnotes 1 through 5.

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 which are placed side by side on the supports. The interaction between the beams is developed by continuous longitudinal shear keys and lateral bolts which may, or may not, be prestressed.

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 relations:

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

where

$$S = \frac{12N_L + 9}{N_g} \qquad (3\text{-}12)$$

$$D = 5 + \frac{N_L}{10} + \left(3 - \frac{2N_L}{7}\right)\left(1 - \frac{C}{3}\right)^2 \quad \text{when} \quad C \leq 3 \qquad (3\text{-}13)$$

$$D = 5 + \frac{N_L}{10} \quad \text{when} \quad C > 3 \qquad (3\text{-}14)$$

N_L = Total number of traffic lanes from Article 3.6.

N_g = Number of longitudinal beams

C = $K(W/L)$, a stiffness parameter

W = Overall width of bridge

L = Span length, feet

VALUES OF K TO BE USED IN C = K(W/L)

Bridge Type	Beam Type and Deck Material	K
Multi-beam	Non voided rectangular beams	0.7
	Rectangular beams with circular voids	0.8
	Box section beams	1.0
	Channel 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.

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 walls (without haunches). S shall be the clear span.

(b) Slabs supported on steel stringers. S shall be the distance between edges of flanges plus 1/2 of the stringer flange width.

(c) Slabs supported on timber stringers. S shall be the clear span plus 1/2 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 one foot from the face of the curb. If curbs or sidewalks are not used, the wheel load shall be one 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 one 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.

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*The slab distribution set forth herein is based, substantially, upon 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 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 H15 loading
- P_{20} = 16,000 pounds for H20 loading

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:

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

HS15 Loading:

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

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.

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:

HS20 Loading:

Spans up to and including 50 feet: LLM = 900S foot-pounds
Spans 50 feet to 100 feet: LLM = 1000 (1.30S-20.0) foot-pounds

HS15 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.

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:

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

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

Case B - Reinforcement Parallel to Traffic

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

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

The moment per foot of slab shall be $\frac{P}{E} X$ 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:

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

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

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 1 1/2 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 all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds two 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:

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

For main reinforcement perpendicular to traffic:

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

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 direction of span:

Over width of tire as given in Article 3.30.

Normal to direction of span:

Plank floor: the width of plank

Non-interconnected[1] 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[2]: 15 inches plus thickness of floor, but not to exceed panel width.

Interconnected[1] glued laminated panel floor, with adequate shear transfer between panels[2], 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

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

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

and shear.* The maximum shear is for a wheel position assumed to be 15 inches or less from the centerline of the support. The maximum moment is for a wheel position assumed to be centered between the supports.

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

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

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.-lb./in.)
- R_x = primary shear (lb/in)
- x = denotes direction perpendicular to longitudinal stringers
- P = design wheel load (lb)
- s = effective deck span (in)
- t = deck thickness (in) based on moment or shear, whichever controls
- K = design constant depending on design load as follows:
  - H15 $K = 0.47$
  - H20 $K = 0.51$
- F_b = allowable bending stress, psi, based on load applied parallel to the wide face of the laminations. See Tables 13.2.2A and B.
- F_v = allowable shear stress, psi, based on load applied parallel to the wide face of the laminations. See Tables 13.2.2A and B.

*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.

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:

$$n = \frac{1,000}{\sigma_{PL}} \times \left[\frac{\bar{R}_y}{\bar{R}_D} + \frac{\bar{M}_y}{\bar{M}_D}\right] \qquad (3\text{-}27)$$

where

n = number of steel dowels required for the given span, s

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 (lb) determined by the relationship

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

or

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

$\overline{M_y}$ = total secondary moment transferred (in-lb) determined by the relationship

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

$$\overline{M_y} = \left[\frac{Ps}{20}\right]\left[\frac{(s-30)}{(s-10)}\right] \text{ for } s > 50 \text{ in.} \qquad (3\text{-}31)$$

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

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.	1/in.^{2}	1/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:

$$\sigma = \frac{1}{\bar{n}} (C_R \bar{R}_y + C_M \bar{M}_y) \tag{3-32}$$

Where

σ = minimum yield point of steel pins (psi) (See Table 10.32.1A)

$n, \bar{R}_y, \bar{M}_y$ = as previously defined.

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

3.25.2 LONGITUDINAL FLOORING

3.25.2.1 In direction of span:

Point loading

3.25.2.2 Normal to direction of span:

Plank floor: width of plank
Non-interconnected nail laminated or glued laminated panel floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated or glued laminated panel

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.

3.25.3 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[a]	65	70	60	75	85	30	80	75

[a]Continuous beam of 2 equal spans.

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*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.

3.26.2.2 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 which 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.

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

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

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 1 1/4 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 contact length of the rear tire along the element.

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:

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

*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.

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 H15, H20, HS15, and HS20 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.

SECTION 4 - FOUNDATIONS

4.1 NOTATIONS

d = average diameter of pile (Article 4.3.7)

d_p = diameter of a round pile or depth of H pile (Article 4.5.7)

E = efficiency or decimal fraction of the single pile value to be used for each pile in the group (Article 4.3.7)

f'_c = specified compressive strength of concrete (Article 4.3.3)

f_{ce} = concrete stress in pile due to prestress after all losses (Article 4.3.3)

m = number of rows in each group (Article 4.3.7)

n = number of piles in each row (Article 4.3.7)

s = center to center spacing of piles (Article 4.3.7)

β = ratio of long side to short side of footing (Article 4.5.6)

ϕ = arctan (d/s) (Article 4.3.7)

4.2 BEARING CAPACITY OF FOUNDATION SOILS

4.2.1 THEORETICAL ESTIMATION

The bearing capacity of the foundation soil may be estimated using accepted theories.* Such theories are based on the measurement of soil parameters such as cohesion and angle of friction or on the results of field tests such as the standard penetration test or the shear vane test.

4.2.2 LOAD TESTS

The bearing capacity may also be determined by load tests in excavated foundation pits. Load tests have a limited depth influence, however, and may not disclose long-term consolidation and so should not be used without drilling or probing to determine the soil profile below the foundation.

4.2.3 APPROXIMATE VALUES

Where testing is not carried out, the bearing capacity and angle of friction of broad basic groups of materials given in Tables 4.2.3A and 4.2.3B may be used. These values should be used conservatively; for

*Reference may be made to "Soil Mechanics in Engineering Practice," by Terzaghi and Peck,.

example, in determining lateral pressures, the minimum angle of friction shall be taken.

Table 4.2.3A

BEARING CAPACITY

	Safe Bearing Capacity Tons per Square Foot	
Material	Minimum	Maximum
Alluvial soils	1/2	1
Clays	1	4
Sand, confined	1	4
Gravel	2	4
Cemented sand and gravel	5	10
Rock	5	--

Table 4.2.3B

ANGLE OF FRICTION	
Earth, Loam .... 30^{o} to 45^{o}	Gravel ...... 30^{o} to 40^{o}
Dry Sand .... 25 to 35	Cinders ..... 25 to 40
Moist Sand .... 30 to 45	Coke .... 30 to 45
Wet Sand .... 15 to 30	Coal .... 25 to 35
Compact Earth ..... 35 to 40	

4.3 PILES

4.3.1 GENERAL

4.3.1.1 In general, piling shall be considered when footings cannot, at a reasonable expense, be founded on rock or other solid foundation material. At locations where soil conditions would normally permit the use of spread footings but conditions are such that erosion may occur, piles may be used as a protection against scour.

4.3.1.2 In general, the penetration for any pile shall be not less than 10 feet into hard cohesive or dense granular material nor less than 20 feet into soft cohesive or loose granular material. Piles for trestle or pile bents shall meet these requirements and, additionally, unless refusal is encountered, shall penetrate not less than 1/3 the length of the pile.

4.3.1.3 For foundation work, no piling shall be used to penetrate a soft or loose upper stratum overlying a hard or firm stratum unless the piles penetrate the hard or firm stratum by a sufficient distance to fix the ends against lateral movement of the bottom end of the pile.

4.3.2 LIMITATIONS ON THE USE OF UNTREATED TIMBER PILES

4.3.2.1 Untreated timber piles may be used for temporary construction, revetments, fenders and similar work, and in permanent construction under the following conditions:

(a) For foundation piling when the cutoff is below permanent ground water level.

(b) For trestle construction when it is economical to do so, though treated piles are preferable.

(c) They shall not be used where they will, or may be, exposed to marine borers.

4.3.2.2 The limitations of use of treated timber piles are given in Section 21 - Division II.

4.3.3 DESIGN LOADS

4.3.3.1 The design loads for piles shall be according to Article 4.3.4. Piles shall be designed to carry the entire superimposed load, no allowance being made for the supporting value of the material between the piles.

4.3.3.2 The supporting power of piles shall be determined by the application of test loads or by the use of formulas as specified in Article 3.6 - Division II.

4.3.4 LOAD CAPACITY OF PILES

4.3.4.1 GENERAL

4.3.4.1.1 The design load on a pile shall not be greater than its load capacity as determined from the minimum of the following cases:

Case A: the capacity of the pile as a structural member.
Case B: the capacity of the pile to transfer its load to the ground.
Case C: the capacity of the ground to support the load from the pile or piles.

4.3.4.1.2 The values of each of these cases shall be determined by making subsurface investigations or tests and by referring to all other available information. Consideration shall also be given to:

(1) The difference between the supporting capacity of a single pile and that of a group of piles.

(2) The capacity of the underlying strata to support the load of the pile group.

(3) The effects on adjacent structures of driving the additional piles.

(4) The possibility of scour and its effect.

(5) The transmission of forces from consolidating soils.

4.3.4.2 CASE A - CAPACITY AS A STRUCTURAL MEMBER

4.3.4.2.1 For portions of piles in air or water, or in soil not capable of providing adequate lateral support throughout the pile length to prevent buckling, the structural design provisions for compression members of Section 8, Section 9, Section 10, or Section 13 shall apply except: timber piles shall be designed in accordance with Article 13.3 using the allowable unit stresses given in Article 13.2 for lumber and in Table 4.3.4.2A.

4.3.4.2.2 For concrete-filled pipe piles, where corrosion may be expected, 1/16 inch shall be deducted from the shell thickness to allow for reduction in section due to corrosion. Area of shell shall be included in determining percentage of reinforcement, ρ.

4.3.4.3 CASE B - CAPACITY OF THE PILE TO TRANSFER LOAD TO THE GROUND

4.3.4.3.1 POINT-BEARING PILES

A pile shall be considered to be a point-bearing pile when placed or driven on or into a material which is capable of developing the pile load by direct bearing at the point with a reasonable factor of safety.

Table 4.3.4.2A

ALLOWABLE WORKING STRESS FOR ROUND TIMBER PILES

Species	Allowable Unit Working Stress Compression Parallel to Grain for Normal Duration of Loading psi
Ash, white	1,200
Beech	1,300
Birch	1,300
Chestnut	900
Cypress, Southern	1,200
Cypress, Tidewater red	1,200
Douglas fir, coast type	1,200
Douglas fir, inland	1,100
Elm, rock	1,300
Elm, soft	850
Gum, black and red	850
Hemlock, Eastern	800
Hemlock, West Coast	1,000
Hickory	1,650
Larch	1,200
Maple, hard	1,300
Oak, red and white	1,100
Pecan	1,650
Pine, Lodgepole	800
Pine, Norway	850
Pine, Southern	1,200
Pine, Southern, dense	1,400
Poplar, yellow	800
Redwood	1,100
Spruce, Eastern	850
Tupelo	850

The allowable load at the tip of the pile shall not exceed the following:

(a) For round timber piles, the values in Table 4.3.4.2A times the pile tip area.

For sawn timber piles, the values applicable to "wet condition" for allowable compression parallel to grain, in accordance with Article 13.2.

(b) For concrete piles, $0.33f'_c$ times the gross cross sectional area of the concrete.

(c) For concrete-filled piles, $0.40\ f'_c$ times the total actual area of the concrete and steel or the point bearing capacity as determined by loading test piles.

(d) For steel H-piles and unfilled tubular steel piles, 9,000 psi over the cross sectional area of the pile tip, not including the area of any pile tip reinforcement, or the point bearing capacity as determined by loading test piles.

(e) For prestressed concrete piles fully embedded in soils providing lateral support, a stress of $0.33f'_c - 0.27f_{ce}$ times the gross cross sectional area of the concrete where f_{ce} is the concrete stress in the pile due to prestressing, after all losses.

4.3.4.3.2 FRICTION PILES

4.3.4.3.2.1 A pile shall be considered to be a friction pile if its point does not rest on or in a material which is capable of developing the pile load by direct bearing at the point.

4.3.4.3.2.2 The load-carrying capacity of friction piles shall be determined by one or more of the following methods:

(a) Driving and loading test piles. The safe allowable load shall be as defined by Article 3.6.1 - Division II.

(b) Pile-driving experience in the vicinity. When piles are designed on the basis of experience in the vicinity, due consideration will be given to the variation in pile types and lengths, and to the variation of the soil strata. Where possible, the complete driving records of all piles in the vicinity shall be examined and compared with the driving records of the project piles.

(c) Adequate tests of the soil strata through which the pile is to be driven. These tests should be projected and compared, if possible with tests of similar material through which piles of known capacity have been driven.

4.3.4.4 CASE C - CAPACITY OF THE GROUND TO SUPPORT THE LOAD DELIVERED BY THE PILE

4.3.4.4.1 For the evaluation of the capacity of the ground, preference shall be given to test loading or satisfactory subgrade investigation.

4.3.4.4.2 In the absence of pile tests, the capacity of the ground to support the load delivered by the pile shall be determined from the results of the applicable subsurface investigations.

4.3.4.4.2.1 POINT-BEARING PILES

Sufficient borings shall be made to determine the thickness and quality of the stratum in which the point bearing is to be developed. If that stratum is of sufficient thickness and is underlain by a firm material, no reduction need be made for group action of piles. In general, piles should not rest on a thin stratum of hard material which is underlain by a thick stratum of soft or yielding material, but where this condition cannot be avoided, group action should be considered and the design loads reduced accordingly.

4.3.4.4.2.2 FRICTION PILES

4.3.4.4.2.2.1 Borings shall be carried well below the tips of the piles in order to determine the characteristics of the underlying material. In most cases a study of those borings will suffice to determine whether or not the underlying soil will support the loads to be delivered to it, but in doubtful or special cases, especially large foundation areas and important footings, the material should be investigated more thoroughly by soil mechanics methods.

4.3.4.4.2.2.2 A single row of piles shall not be considered as a group provided that they are not spaced closer center to center than 2-1/2 times the nominal diameter or dimension. In those cases where piles are driven in groups into plastic material, the design load shall be determined by the loading of a group of piles or an allowance shall be made according to Article 4.3.4.7 for the difference between the supporting capacity of a single pile and that of a group of piles.

4.3.4.5 MAXIMUM DESIGN LOADS FOR PILES

4.3.4.5.1 In those cases where it is not feasible to make the required subsurface investigations or test loads, the maximum assumed design load for piles shall be as given in the Table 4.3.4.5. These values may be increased for certain combinations of loads as specified in Article 3.22.

4.3.4.5.2 The assumed pile loads shall be substantiated by using a pile driving formula to determine the allowable load when the piles are driven, as provided in Article 3.6.2 - Division II.

Table 4.3.4.5

ASSUMED PILE LOADS

Types of Piles				
Size or Diameter at Butt* inches	Timber tons	Concrete tons	Steel (Friction)-tons	Steel Point-Bearing
8	--	--	16	9,000 pounds per sq. in. of point area, not including the area of any pile tip reinforcement.
10	20	20	20	
12	24	24	24	
14	28	28	28	
16	32	32	--	
20	--	40	--	
24	--	50	--	

*Timber piles, diameter to be measured 3 feet from butt.

4.3.4.6 UPLIFT

4.3.4.6.1 Friction piles may be considered to resist an intermittent but not sustained uplift equivalent to 40 percent of the above loads providing proper provision is made for the anchorage at the top and sufficient skin friction is developed. In no case shall uplift exceed the weight of material, after allowing for buoyancy, surrounding the embedded portion of the pile.

4.3.4.6.2 In seals, the bond between timber, steel or concrete piles and surrounding concrete may be assumed to be ten pounds per square inch. The total bond force used shall be no greater than the resistance of the pile to uplift.

4.3.4.7 GROUP PILE LOADING

Where the capacity of a group of friction piles driven into plastic material is not determined by test loading, the following Converse-Labarre* formula is suggested to determine the reduction of a single pile load for a group pile load:

$$E = 1 - \emptyset \frac{(n-1)m+(m-1)n}{90mn} \qquad (4\text{-}1)$$

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*Refer to "Pile Foundations, Theory - Design - Practice," by Robert D.. Chillis.

Where

E	= the efficiency or the decimal fraction of the single pile value to be used for each pile in the group.
n	= the number of piles in each row.
m	= the number of rows in each group.
d	= the average diameter of the pile.
s	= center to center spacing of piles.
Tan ϕ	= d/s

ϕ is numerically equal to the angle expressed in degrees.

4.3.5 REQUIRED SUBSURFACE INVESTIGATIONS

4.3.5.1 POINT-BEARING PILES

Sufficient borings shall be made to determine the presence, position, and thickness of the material which is capable of developing point bearing, and the log of borings shall show the nature of the overlying strata in order that the extent of lateral support may be determined. If the point-bearing stratum is of doubtful thickness and quality, the borings shall be made to such sufficient depth below this stratum that the capacity of a friction pile may be determined.

4.3.5.2 FRICTION PILES

Borings shall be made to an elevation well below the expected elevation of the pile tips and accurate logs of these borings shall be made. In those cases where the piles are to be designed on the basis of soil tests, undisturbed samples shall be taken on all strata which will have appreciable influence on the capacity of the pile.

4.3.5.3 COMBINATION POINT-BEARING AND FRICTION PILES

Piles shall be classified as either (1) point-bearing or (2) friction. Those cases where adequate strength is developed by both point-bearing and friction may be designed under either of these classifications.

4.3.5.4 SCOUR

Subsurface investigations shall be made which will determine the probable depth of scour or flotation of material and the condition of lateral support of the pile.

4.3.6 SPACING, CLEARANCES AND EMBEDMENT

4.3.6.1 FOOTINGS

4.3.6.1.1 Footings shall be so proportioned that pile spacing shall be not less than 2 feet 6 inches center to center. The distance from the side of any pile to the nearest edge of the footing shall be not less than 9 inches.

4.3.6.1.2 The tops of piles shall project not less than 12 inches into the concrete after all damaged pile material has been removed, but in special cases it may be reduced to 6 inches.

4.3.6.2 BENT CAPS

Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover at the sides of the piles shall be a minimum of six inches. The piles shall project at least six inches and preferably nine inches into the cap, although concrete piles may project a lesser distance into the cap if the projection of the pile reinforcement is sufficient to provide for adequate bond.

4.3.7 BATTER PILES

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation or when increased rigidity of the entire structure is required, batter piles shall be used in the foundation.

4.3.8 BUOYANCY

The effect of hydrostatic pressure shall be considered in the design as provided in Article 3.19.

4.3.9 PRECAST CONCRETE PILES

4.3.9.1 Precast concrete piles shall be of approved size and shape but may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for that portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns.

4.3.9.2 In general, concrete piles shall have a cross sectional area, measured above the taper, of not less than 140 square inches and when they are to be used in salt water they shall have a cross sectional area of not less than 220 square inches. If a square section is employed, the corners shall be chamfered at least one inch.

4.3.9.3 The diameter of tapered piles measured 2 feet from the point shall be not less than 8 inches where, for all pile cross sections, the diameter shall be considered as the least dimension through the center.

4.3.9.4 Piles, preferably, shall be cast with a driving point and for hard driving, preferably shall be shod with a metal shoe of approved pattern.

4.3.9.5 Where steel points are not used, points shall be not less than 6 inches in diameter and the pile shall be beveled, tapered or sloped uniformly from the point to 2 feet from the point.

4.3.9.6 Vertical reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile, except that if more than four bars are used, the number may be reduced to four in the bottom 4 feet of the pile. The amount of reinforcement shall be at least 1-1/2 percent of the total cross section measured above the taper.

4.3.9.7 The full length of vertical steel shall be enclosed with spiral reinforcement or equivalent hoops.

4.3.9.8 The spiral reinforcement at the ends of the pile shall have a pitch of 3 inches and gage of not less than No. 5 (U.S. Steel Wire Gage). In addition the top 6 inches of the pile shall have five turns of spiral winding at one-inch pitch. For the remainder of the pile, the lateral reinforcement shall be a No. 5 gage spiral with not more than 6-inch pitch, or 1/4-inch round hoops spaced on not more than 6-inch centers.

4.3.9.9 The reinforcement shall be placed at a clear distance from the face of the pile of not less than 2 inches and, when the piles are for use in salt water or alkali soils, this clear distance shall be not less than 3 inches.

4.3.9.10 Piles may be spliced provided that the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing that provides equal results may be considered for approval.

4.3.9.11 In computing stresses due to handling, the static loads shall be increased by 50 percent as an allowance for impact and shock.

4.3.10 CAST-IN-PLACE CONCRETE PILES

4.3.10.1 Cast-in-place concrete piles shall be, in general, cast in metal shells which shall remain permanently in place. However, other types of cast-in-place concrete piles, plain or reinforced, cased or uncased, may be used if the soil conditions permit their use and if their design and the method of placing are satisfactory.

4.3.10.2 Cast-in-place concrete piles may have a uniform section or may be tapered over any portion.

4.3.10.3 The minimum area at the butt of the pile shall be 100 square inches and the minimum diameter at the tip of pile shall be 8 inches. Above the butt or taper, the minimum size, shall be as specified for precast piles.

4.3.10.4 Cast-in-place piles, carrying axial loads only where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced when the soil provides adequate lateral support. Those portions of cast-in-place piles which are not supported laterally shall be designed as reinforced concrete columns in accordance with Articles 8.15.4 and 8.16.4 and the reinforcing steel shall extend ten feet below the plane where the soil provides adequate lateral restraint. Where the shell is more than 0.12 inch in thickness, it may be considered as reinforcement.

4.3.10.5 Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection. The embedment of the reinforcement into the pile cap shall be as specified for precast piles.

4.3.10.6 The shell shall be of sufficient thickness and strength so that it will hold its original form and show no harmful distortion after it and adjacent shells have been driven and the driving core, if any, has been withdrawn. The plans shall stipulate that alternative designs of the shell must be approved by the Engineer before any driving is done.

4.3.10.7 Piles may be spliced provided that the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing providing equal results may be considered for approval.

4.3.11 STEEL H-PILES

4.3.11.1 THICKNESS OF METAL

Steel piles shall have a minimum thickness of web of .400 inch. Splice plates shall be not less than 3/8 inch thick.

4.3.11.2 SPLICES

Piles shall be spliced to develop the net section of pile. The flanges and web shall be either spliced by butt welding or with plates that are welded, riveted or bolted. Bolted splices shall only be used on projects where a small number of piles are required and where facilities for riveting or welding are not available. Splices shall be detailed on the contract plans.

4.3.11.3 CAPS

In general, caps are not required for steel piles embedded in concrete. Reference is made to Research Report No. 1, "Investigation of the Strength of the Connection between a Concrete Cap and the Embedded end of the Steel H-Pile" - Department of Highways, State of Ohio, for a discussion of this subject and for the results of tests pertinent to it.

4.3.11.4 SCOUR

If heavy scour is expected, consideration shall be given to designing the portion of the pile which would be exposed as a column.

4.3.11.5 LUGS, SCABS, AND CORE-STOPPERS

These devices may be used to increase the bearing capacity of the pile where necessary. They may consist of structural shapes, welded, riveted or bolted, of plates welded between the flanges, or of timber or concrete blocks securely fastened.

4.3.12 UNFILLED TUBULAR STEEL PILES

4.3.12.1 THICKNESS OF METAL

Piles shall have a minimum wall thickness not less than indicated in the following table:

Outside Diameter	Less than 14 inches	14 inches and over
Wall Thickness	.25 inch	.375 inch

4.3.12.2 SPLICES

Piles shall be spliced to develop the full section of the pile. The piles shall be spliced either by butt welding or by the use of welded sleeves. Splices shall be detailed on the contract plans.

4.3.12.3 DRIVING

Tubular steel piles may be driven either closed or open ended. Closure plates should not extend beyond the perimeter of the pile.

4.3.12.4 COLUMN ACTION

Where the piles are to be used as part of a bent structure or where heavy scour is anticipated that would expose a portion of the pile, the pile shall be investigated for column action. The provisions of Article 4.3.13 shall apply to unfilled tubular steel piles.

4.3.13 PROTECTION AGAINST CORROSION

Where conditions of exposure warrant, concrete encasement shall be used on steel piles and steel shells or 1/16 inch shall be deducted from the thickness of all exposed surfaces in computing the area of steel in the piles or shells.

4.3.14 PRESTRESSED CONCRETE PILES

4.3.14.1 Prestressed concrete piles, which are generally octagonal, square, or circular, shall be of approved size and shape. Air entrained concrete shall be used in piles which are subject to freezing and thawing or wetting and drying. Concrete in prestressed piles shall have a minimum compressive strength (f'_c) of 5000 psi at 28 days. Prestressed concrete piles may be solid or hollow. For hollow piles precautionary measures should be taken to prevent breakage due to internal water pressure during driving, ice pressure in trestle piles and gas pressure due to decomposition of material used to form the void.

4.3.14.2 Main reinforcement shall be spaced and stressed so as to provide a compressive stress on the pile after losses, f_{ce}, generally not less than 700 psi to prevent cracking during handling and installation. Piles shall be designed to resist stresses developed during handling as well as under service load conditions. Bending stresses shall be investigated for all conditions of handling, taking into account the weight of the pile plus 50 percent allowance for impact, with tensile stresses limited to $5\sqrt{f'_c}$.

4.3.14.3 The full length of vertical reinforcement shall be enclosed with spiral reinforcement. For piles up to 24 inches in diameter, spiral wire shall be No. 5 (U.S. Steel Wire Gage). Spiral reinforcement at the ends of these piles shall have a pitch of 3 inches for approximately sixteen turns. In addition, the top 6 inches of pile shall have five turns of spiral winding at 1-inch pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 6-inch pitch. For piles having a diameter greater than 24 inches, the wire shall be No. 4 (U.S. Steel Wire Gage). Spiral reinforcement at the end of these piles shall have a pitch of 2 inches for approximately sixteen turns. In addition, the top 6 inches shall have 4 turns of spiral winding at 1-1/2 inches. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 4-inch pitch. The reinforcement shall be placed at a clear distance from the face of the prestressed pile of not less than 2 inches.

4.3.14.4 Large diameter hollow cylinder piles shall be of approved size and shape. The wall thickness for cylinder piles shall not be less than 5 inches. The grouting of post-tensioning tendons shall be in accordance with Article 4.33.9 - Division II.

4.3.14.5 When prestressed concrete piles are spliced, the splice shall be capable of resisting the forces applied after the pile is spliced.

4.4 FOOTINGS

4.4.1 GENERAL

4.4.1.1 Provisions of this Article shall apply for design of isolated footings and, where applicable, to combined footings and mats (footings supporting more than one column, pier or wall).

4.4.1.2 Circular or regular polygon-shaped concrete columns or piers may be treated as square members with the same area for location of critical sections for moment, shear, and development of reinforcement in footings.

4.4.2 DEPTH

4.4.2.1 The depth of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, footings of all structures, other than culverts, which are exposed to the erosive action of stream currents shall be founded at a depth preferably not less than 4 feet below the permanent stream bed. Stream piers and arch abutments shall be founded at a depth preferably not less than 6 feet below stream bed. The above preferred minimum depths shall be increased as conditions may require.

4.4.2.2 Footings not exposed to the action of stream currents shall be founded on a firm foundation and below frost.

4.4.2.3 Excavations which would increase the hydraulic gradient and hence would cause foundation soils to be loosened by the upward flow of water should not be used.

4.4.2.4 Intrusion failures should be prevented by requiring a base course between rip-rap and fine soils and by requiring proper gradation of drainage backfill behind abutments.

4.4.3 ANCHORAGE

Footings on inclined smooth solid rock surfaces which are not restrained by an overburden of resistant material, shall be effectively anchored by means of anchor bolts, dowels, keys or other suitable means.

4.4.4 DISTRIBUTION OF PRESSURE

4.4.4.1 Footings shall be designed to keep the maximum soil pressures within safe bearing values. In order to prevent unequal settlement, footings shall be designed to keep the pressure as nearly uniform as practicable. In footings supported on piles, the spacing of the piles shall be such as to secure as nearly equal loads on each pile as may be practicable.

4.4.4.2 When footings support more than one column, pier, or wall, distribution of soil pressure shall be consistent with properties of the soil and the structure, and with established principles of soil mechanics.

4.4.5 LOADS AND REACTIONS

4.4.5.1 Footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry the computed proportion of the total footing load.

4.4.5.2 When a single isolated footing supports a column, pier or wall, the footing shall be assumed to act as a cantilever. When footings support more than one column, pier or wall, the footing slab shall be designed for the actual conditions of continuity and restraint.

4.4.6 MOMENT IN FOOTINGS

4.4.6.1 External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over the entire area of footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, pier or wall. In the case of columns that are not square or rectangular, the critical section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, the critical section shall be taken as halfway between the middle and edge of the wall. For footings under metallic column bases, the critical section shall be taken as halfway between the column face and the edge of the metallic base.

4.4.6.2 In one-way footings, and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

4.4.6.3 In two-way rectangular footings, reinforcement shall be distributed as follows:

4.4.6.3.1 Reinforcement in the long direction shall be distributed uniformly across entire width of footing.

4.4.6.3.2 For reinforcement in the short direction, a portion of the total reinforcement given by the following equation shall be distributed uniformly over a band width (centered on centerline of column or pier) equal to the length of the short side of footing. Remainder of reinforcement required in the short direction shall be distributed uniformly outside center band width of footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta+1)} \qquad (4\text{-}2)$$

Where β is the ratio of long side to short side of footing.

4.4.7 SHEAR IN FOOTINGS

4.4.7.1 Computation of shear in footings, and location of critical section, shall be in accordance with Article 8.15.5.6 or 8.16.6.6. Location of critical section shall be measured from face of column, pier or wall, for footings supporting a column, pier or wall. For footings supporting a column or pier with metallic base plates, the critical section shall be measured from the location defined in Article 4.4.6.

4.4.7.2 For footings supported on piles, shear on the critical section shall be in accordance with the following, where d_p is the diameter of a round pile or depth of H pile at footing base:

(a) Entire reaction from any pile whose center is located $d_p/2$ or more outside the critical section shall be considered as producing shear on that section.

(b) Reaction from any pile whose center is located $d_p/2$ or more inside the critical section shall be considered as producing no shear on that section.

(c) For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the critical section shall be based on linear interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

4.4.8 DEVELOMENT OF REINFORCEMENT

4.4.8.1 Computation of development of reinforcement in footings shall be in accordance with Articles 8.24 through 8.32.

4.4.8.2 Critical sections for development of reinforcement shall be assumed at the same locations as defined in Article 4.4.6, and at all other vertical planes where changes of section or reinforcement occur. See also Article 8.24.1.5.

4.4.9 TRANSFER OF FORCE AT BASE OF COLUMN

4.4.9.1 All forces and moments applied at base of column or pier shall be transferred to top of footing by bearing on concrete and by reinforcement.

4.4.9.2 Lateral forces shall be transferred to supporting footing in accordance with shear-transfer provisions in Article 8.15.5.4 or 8.16.6.4.

4.4.9.3 Bearing on concrete at contact surface between supporting and supported member shall not exceed concrete bearing strength for either surface as given in Article 8.15.2 or 8.16.7.

4.4.9.4 Reinforcement shall be provided across interface between supporting and supported member either by extending main longitudinal reinforcement into footings, or by dowels. Reinforcement across interface shall be sufficient to satisfy all of the following.

(a) Reinforcement shall be provided to transfer all force that exceeds concrete bearing strength in supporting or supported member.

(b) If required loading conditions include uplift, total tensile force shall be resisted by reinforcement.

(c) Area of reinforcement shall not be less than 0.005 times gross area of supported member, with a minimum of 4 bars.

4.4.9.5 Diameter of dowels, if used, shall not exceed diameter of longitudinal reinforcement by more than 0.15 inch.

4.4.9.6 For transfer of force by reinforcement, development of reinforcement in supporting and supported member shall be in accordance with Articles 8.24 through 8.32.

4.4.9.7 At footings, #14 and #18 main longitudinal reinforcement, in compression only, may be lap sliced with footing dowels to provide the required area, but not less than that required by Article 4.4.9.4. Dowels shall not be larger than #11 and shall extend into the column a distance not less than the development length of the #14 or #18 bars or the splice length of the dowels, whichever is greater; and into the footing a distance not less than the development length of the dowels.

4.4.10 PLAIN CONCRETE FOOTINGS

4.4.10.1 Design stresses in plain concrete footings or pedestals shall be computed assuming a linear stress distribution. For footings and pedestals cast against soil, effective thickness used in computing stresses shall be taken as the overall thickness minus 3 inches. Extreme fiber stress in tension shall not exceed that specified in Article 8.15.2.1.1. Bending need not be considered unless projection of footing from face of supported member exceeds footing thickness.

4.4.10.2 The ratio of unsupported height to average least lateral dimension of plain concrete pedestals shall not exceed 3.

SECTION 5 - RETAINING WALLS

5.1 GENERAL

Retaining walls shall be designed to withstand earth pressure, including any live load surcharge, and the weight of the wall, in accordance with the general principles specified for abutments.

Stone masonry and plain concrete walls shall be of the gravity type. Reinforced concrete walls may be a cantilever, counterforted, buttressed, or cellular type.

5.2 BASE OR FOOTING SLABS

The rear projection or heel of base slabs shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used.

The base slabs of cantilever walls shall be designed as cantilevers supported by the wall.

The base slabs of counterforted and buttressed walls shall be designed as fixed or continuous beams of spans equal to the distance between counterforts or buttresses.

5.3 VERTICAL WALLS

The vertical stems of cantilever walls shall be designed as cantilevers supported at the base.

The vertical or face walls of counterforted and buttressed walls shall be designed as fixed or continuous beams. The face walls shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcement.

5.4 COUNTERFORTS AND BUTTRESSES

Counterforts shall be designed as T-beams. Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts there shall be a system of horizontal and vertical bars or stirrups to anchor the face walls and base slab to the counterfort. These stirrups shall be anchored as near to the outside faces of the face walls, and as near to the bottom of the base slab as practicable.

5.5 REINFORCEMENT FOR TEMPERATURE

Except in gravity walls, not less than 1/8 square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

5.6 EXPANSION AND CONTRACTION JOINTS

Contraction joints shall be provided at intervals not exceeding 30 feet and expansion joints at intervals not exceeding 90 feet for gravity or reinforced concrete walls.

5.7 DRAINAGE

The filling material behind all retaining walls shall be drained by weep holes with French drains, placed at suitable intervals. In counterforted walls there shall be at least one drain for each pocket formed by the counterforts.

SECTION 6 - CULVERTS

6.1 CULVERT LOCATION, LENGTH, AND WATERWAY OPENINGS

Recommendations on culvert location, length, and waterway openings are given in the AASHTO Guide on Hydraulic Design of Culverts.

6.2 DEAD LOADS

Vertical and horizontal earth pressures on culverts may be computed by recognized or appropriately documented analytical techniques based on the principles of soil mechanics and soil structure interaction, or design pressures shall be calculated as being the result of an equivalent fluid weight as follows:

6.2.1 Culvert in trench, or culvert untrenched on yielding foundation.

A. Rigid culverts except reinforced concrete boxes:

(1) For vertical earth pressure - 120 pcf
For lateral earth pressure - 30 pcf
(2) For vertical earth pressure - 120 pcf
For lateral earth pressure - 120 pcf

B. Reinforced concrete boxes:

For vertical earth pressure - 120 pcf
For lateral earth pressure - 30 pcf

C. Flexible Culverts:

For vertical earth pressure - 120 pcf
For lateral earth pressure - 120 pcf

6.2.2 Culvert untrenched on unyielding foundation.

A special analysis is required.

6.3 FOOTINGS

Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In any location subject to erosion, aprons or cut-off walls shall be used at both ends of the culvert and, where necessary, the entire floor area between the wing walls shall be paved. Baffle walls or struts across the unpaved bottom of a culvert barrel shall not be used where the stream bed is subject to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

6.4 DISTRIBUTION OF WHEEL LOADS THROUGH EARTH FILLS

6.4.1 When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over a square with sides equal to 1-3/4 times the depth of fill.

6.4.2 When such areas from several concentrations overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab. For single spans, the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments. When the depth of fill is less than 2 feet the wheel load shall be distributed as in slabs with concentrated loads. When the calculated live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills exceeds the live load and impact moment calculated according to Article 3.24, then the latter moment shall be used.

6.5 DISTRIBUTION REINFORCEMENT

Where the depth of fill exceeds two feet reinforcement to provide for the lateral distribution of concentrated loads is not required.

6.6 DESIGN

For culvert design guidelines see Section 17.

SECTION 7 - SUBSTRUCTURES

7.1 PIER SPACING, ORIENTATION AND TYPE

Piers shall be located to meet navigational clearance requirements and to give a minimum interference to flood flow. In general, piers should be placed parallel with the direction of the stream current at flood stage. Adequate provision should be made for drift and ice by increasing span lengths and vertical clearances, by selecting proper pier types and by using debris deflectors. Special precautions against scour are required when large cofferdams are placed in unstable stream beds.

7.2 PIERS

7.2.1 GENERAL

7.2.1.1 Piers shall be designed to withstand dead load, live loads on the roadway, wind loads acting on the pier and superstructure, forces due to stream current, floating ice and drift, and longitudinal forces at the fixed ends of spans.

7.2.1.2 Where necessary, piers shall be protected against abrasion within the limits of damage by floating ice or debris by facing them with granite, vitrified brick, timber or other suitable material.

7.2.2 PIER NOSE

In streams carrying ice or drift, the pier nose shall be designed as an ice breaker. When a steel angle or other metal nosing is used, it shall be effectively secured to the masonry by means of suitable anchors.

7.3 TUBULAR STEEL PIERS

7.3.1 USE

Wherever possible, tubular steel piers should not be used and shall never be used in locations where they will be subjected to lateral earth pressure. In the special cases where their use may be permitted, the following requirements shall apply.

7.3.2 DEPTH

The general requirements governing the depths of foundations given in Section 4 shall apply in the case of tubular steel piers except that steel tubes resting upon gravel foundation without piling shall be carried to a depth greater than 8 feet below the permanent bed of the stream and to such additional depth as may be necessary to eliminate all danger of undermining.

7.3.3 PILES

Piles supporting tubular piers shall thoroughly brace the tubes by extending into the concrete filling a sufficient distance, which in general, shall not be less than 6 to 8 feet.

7.3.4 DIMENSIONS OF SHELL

The minimum thickness of the metal in the shells of tubular piers shall be 5/16 inch; this thickness shall be increased where necessary to secure strength and rigidity for placing the shell. The base of the pier shall be designed for safe pile or soil bearing capacities as specified herein, but when the diameter required by these values is greater than that required for the superstructure bearing, the diameter may be reduced at any splice point. The minimum diameter of steel cylinders used for piers shall be 42 inches.

7.3.5 SPLICES AND JOINTS

All horizontal joints shall be butt joints. Vertical joints may be lapped if the corners of the plates are properly scarfed. When field splicing is necessary the lower section of the tube shall extend at least 2 feet above the water line when in position.

7.3.6 BRACING

The tubes of cylinder piers shall be provided with adequate bracing which, in general, shall consist of a steel or concrete girder diaphragm effectively secured to the tubes. The depth of this diaphragm shall be as great as conditions will permit.

7.4 ABUTMENTS

7.4.1 GENERAL

7.4.1.1 Abutments shall be designed to withstand earth pressure as specified in Article 3.20, the weight of the abutment and bridge superstructure, live load on the superstructure or approach fill, wind forces and longitudinal forces when the bearings are fixed, and longitudinal forces due to friction or shear resistance of bearings. The design shall be investigated for any combination of these forces which may produce the most severe condition of loading.

7.4.1.2 Abutments shall be designed to be safe against overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles at the point of maximum pressure.

7.4.1.3 In computing stresses in abutments, the weight of filling material directly over an inclined or stepped rear face, or over the base of a reinforced concrete spread footing may be considered as part of the effective weight of the abutment. In the case of a spread footing, the rear projection shall be designed as a cantilever supported at the abutment stem and loaded with the full weight of the superimposed material, unless a more exact method is used.

7.4.1.4 The cross section of stone masonry or plain concrete abutments shall be proportioned to avoid the introduction of tensile stress in the material.

7.4.2 REINFORCEMENT FOR TEMPERATURE

Except in gravity abutments, not less than 1/8 square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced to resist the formation of temperature and shrinkage cracks.

7.4.3 WINGWALLS

7.4.3.1 Wingwalls shall be of sufficient length to retain the roadway embankment to the required extent and to furnish protection against erosion. The wingwall lengths shall be computed using the required roadway slopes.

7.4.3.2 Reinforcing bars or suitable rolled sections shall be spaced across the junction between wingwalls and abutments to tie them together. Such bars shall extend into the masonry on each side of the joint far enough to develop the strength of the bar as specified for bar reinforcement, and shall vary in length so as to avoid planes of weakness in the concrete at their ends. If bars are not used, an expansion joint shall be provided and the wing wall shall be keyed into the body of the abutment.

7.4.4 DRAINAGE

The filling material behind abutments shall be drained by weep holes with French drains, placed at suitable intervals.

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 = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, sq. in.. When the flexural reinforcement consists of several bar sizes or wires 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 (Article 8.16.8.4)

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 8.16.6.4.4)

A_g = gross area of section, sq. in.

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_{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 8.15.5.5.2)

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

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

EI = flexural stiffness of compression member (Article 8.16.5.2.7)

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

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

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

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, $\sqrt{\text{lb.}}$/in.

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_h = tensile stress developed by a standard hook, psi (Article 8.29.1)

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 (See 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 8.33)

ℓ_e = equivalent embedment length, in. (Articles 8.29 and 8.32)

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_1 = value of smaller end moment on compression member calculated from a conventional elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature (Article 8.16.5.2.4)

M_2 = value of larger end moment on compression member calculated from a conventional elastic analysis, always positive (Article 8.16.5.2.4)

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_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)

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 8.15.5.5.2)

v_h = permissible horizontal shear stress (Article 8.15.5.5.4)

V_n = nominal shear strength (Article 8.16.6.1)

V_{nh} = nominal horizontal shear strength (Article 8.16.6.5.2)

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

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

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

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

β_d = ratio of maximum dead load moment to maximum total load moment, always positive (Article 8.16.5.2.7)

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)

δ (delta) = moment magnification factor (Article 8.16.5.2.9)

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

ξ (xi) = constant for standard hook (Article 8.29.1)

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

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

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

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

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

= A_s/b_wd

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.

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 C567), 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.

Effective area of reinforcement - The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

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

Embedment length, equivalent (ℓ_e) - Length of embedded reinforcement that can develop the same stress as that which can be developed by a hook or mechanical anchorage.

End anchorage - Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof beyond the point of zero stress in the reinforcement.

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 C330).

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

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

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

8.2 CONCRETE

The specified compressive strength, f'_c, of the concrete for each part of the structure shall be shown on the plans. 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 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, provision for temperature changes shall be made in simple spans when the span length exceeds 40 feet.

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

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

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

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

8.6 STIFFNESS

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

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

8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO

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

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 which may adversely affect the strength or serviceability of the structure at service load.

8.9.2 SUPERSTRUCTURE DEPTH LIMITATIONS

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

Table 8.9.2

RECOMMENDED MINIMUM DEPTHS FOR CONSTANT DEPTH MEMBERS[a]

Superstructure Type	Minimum Depth[b] in feet
Bridge slabs with main reinforcement parallel or perpendicular to traffic	$\frac{S+10}{30} \geq 0.542$
T-Girders	$\frac{S+9}{18}$
Box-Girders	$\frac{S+10}{20}$

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

[b]Recommended values for continuous spans; simple spans should have about 10 percent greater depth.

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

8.10 COMPRESSION FLANGE WIDTH

8.10.1 T-GIRDER

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

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

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

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

8.10.2 BOX GIRDERS

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

8.10.2.2 For integral bent caps, see Article 8.10.1.4.

8.11 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 1/2 in., 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 the effective moment of inertia as follows:

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

where

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

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

For continuous spans, the effective 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.

8.13.4 Unless values are obtained by a more comprehensive analysis, the additional long-term 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 the factor $[2 - 1.2 (A_s' / A_s)] \geq 0.6$.

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.

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

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

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

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

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

8.15 SERVICE LOAD DESIGN METHOD (Allowable Stress Design)

8.15.1 GENERAL REQUIREMENTS

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

8.15.1.2 Development and splices of reinforcement shall be as required in Articles 8.24 through 8.33.

8.15.2 ALLOWABLE STRESSES

8.15.2.1 CONCRETE

Stresses in concrete shall not exceed the following:

8.15.2.1.1 FLEXURE

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

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:

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

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

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

8.15.2.1.2 SHEAR

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

8.15.2.1.3 BEARING STRESS

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

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

When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest 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 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.

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*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete" - ACI 318-77 may be used.

8.15.5.2 SHEAR STRESS CARRIED BY CONCRETE

8.15.5.2.1 SHEAR IN BEAMS AND ONE-WAY SLABS AND FOOTINGS

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

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

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 psi.

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 psi.

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 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 The provisions of this Article may 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 with relative displacement along the assumed crack resisted by friction maintained by shear-friction reinforcement across the crack. Shear friction reinforcement shall be placed approximately perpendicular to the assumed crack.

8.15.5.4.3 The required area of shear-friction reinforcement, A_{vf}, shall be computed by:

$$A_{vf} = V/(f_s \mu) \qquad (8\text{-}10)$$

The coefficient of friction, μ, shall be 1.4 for concrete cast monolithically, 1.0 for concrete cast against hardened concrete, and 0.7 for concrete cast against as-rolled structural steel.

8.15.5.4.4 Shear strength, V, shall not exceed $0.09\, f'_c\, A_{cv}$ nor $360\, A_{cv}$ in pounds where A_{cv} is the area of the concrete section resisting shear transfer.

8.15.5.4.5 Additional reinforcement shall be provided for direct tension across the assumed crack.

8.15.5.4.6 Shear-friction reinforcement shall be well distributed across the assumed crack and shall be adequately anchored on both sides by embedment, hooks, or welding to special devices.

8.15.5.4.7 When concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 inch. When shear is transferred between as-rolled steel and concrete, the steel shall be free of loose rust, loose mill scale, grease, paint, or other foreign matter.

8.15.5.5 HORIZONTAL SHEAR DESIGN FOR COMPOSITE CONCRETE FLEXURAL MEMBERS

8.15.5.5.1 Provision shall be made for full transfer of horizontal shear forces at contact surfaces of interconnected elements.

8.15.5.5.2 Unless calculated in accordance with Article 8.15.5.5.3, the design horizontal shear stress at any cross section shall be computed by

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

where d shall be for the entire composite section.

8.15.5.5.3 Horizontal shear may be investigated by computing, in any segment not exceeding 1/10 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.

8.15.5.5.4 Horizontal shear may be transferred at contact surfaces using the permissible horizontal shear stress v_h stated below.

(a) When contact surfaces are 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 Article 8.15.5.5.6 and contact surfaces are 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 Article 8.15.5.5.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 inch, shear stress v_h shall not exceed 160 psi.

(d) When design horizontal shear stress, v_{dh}, exceeds 160 psi, design for horizontal shear shall be in accordance with Article 8.15.5.4.

8.15.5.5.5 When direct tension exists across any contact surface between interconnected elements, horizontal shear transfer by contact may be assumed only when minimum ties are provided in accordance with Article 8.15.5.5.6.

8.15.5.5.6 Ties for horizontal shear

(a) When ties are provided to transfer horizontal shear, the tie area shall not be less than that required by Article 8.19.1 and the tie spacing shall not exceed four times the least web width of support element nor 24 inches.

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

(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

8.15.5.7.1 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 $1.8\sqrt{f'_c}$. v_c need not be taken less than $1.4\sqrt{f'_c}$ for slabs monolithic with walls, or $1.2\sqrt{f'_c}$ for simply supported slabs. 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.

8.16 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:

(a) Flexure .... ϕ=0.90
(b) Shear .... ϕ=0.85
(c) Axial compression with
spirals .... ϕ=0.75
ties .... ϕ=0.70
(d) Bearing on concrete .... ϕ=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 0.10 $f'_c A_g$ or ϕP_b, whichever is smaller, to zero.

8.16.1.2.3 The development and splice lengths of reinforcement specified in Articles 8.24 through 8.33 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.

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*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.

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

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

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

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:

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

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

where

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

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

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

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:

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

where

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

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

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

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

where

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

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

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

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

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

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

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 8.18.2.1.

$$P_{n(max)}=0.85[0.85\, f'_c(A_g-A_{st})+f_yA_{st}] \qquad (8\text{-}29)$$

and $\phi = 0.75$

(b) For members with tie reinforcement conforming to Article 8.18.2.2.

$$P_{n(max)}=0.80[0.85\, f'_c(A_g-A_{st})+f_yA_{st}] \qquad (8\text{-}30)$$

and $\phi = 0.70$.

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

8.16.4.2 COMPRESSION MEMBER STRENGTHS

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

8.16.4.2.1 PURE COMPRESSION

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

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

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:

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

and

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

where

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

and

$$f_s' = 87{,}000\left[1-\left(\frac{d'}{d}\right)\left(\frac{87{,}000+f_y}{87{,}000}\right)\right] \leq f_y \qquad (8\text{-}35)$$

8.16.4.2.4 COMBINED FLEXURE AND AXIAL LOAD

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

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

8.16.4.3 BIAXIAL LOADING

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

$$\frac{1}{P_{nxy}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o} \tag{8-36}$$

when the factored axial load,

$$P_u \geq 0.1f'_c A_g \tag{8-37}$$

or

$$\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \leq 1 \tag{8-38}$$

when the factored axial load,

$$P_u < 0.1f'_c A_g \tag{8-39}$$

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 $34-(12M_1/M_2)$.

M_1 is the value of the smaller end moment on the compression member calculated from a conventional elastic analysis. M_1 is positive if the member is bent in single curvature, negative if bent in double curvature. M_2 is the value of the larger end moment on the compression member calculated from conventional elastic analysis and is always positive.

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 with $k\ell_u/r$ is greater than 100, an analysis as defined in Article 8.16.5.1.1 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:

$$M_c = \delta M_2 \tag{8-40}$$

where

$$\delta = \frac{C_m}{1-\dfrac{P_u}{\phi P_c}} \geq 1.0 \tag{8-41}$$

and

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

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

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

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

$$C_m = 0.6 + 0.4\ (M_1/M_2) \qquad (8\text{-}45)$$

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 or that computed end eccentricities are less than (0.6+0.03h) inches, M_2 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_1/M_2 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_1/M_2 in equation (8-45).

(b) If computations show that there is essentially no moment at either end of the member, the ratio M_1/M_2 shall be equal to one.

8.16.5.2.9 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.

8.16.5.2.10 In structures which 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.

8.16.5.2.11 When a group of compression members on one level comprise a frame, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of δ shall be computed for the entire group. P_u and P_c shall be taken as the summation ΣP_u and ΣP_c for all compression members in the group. In designing each member in the group, δ shall be taken as the larger of (a) the value computed for the group as a whole, or (b) the value computed for the individual compression member assuming its ends to be braced against sidesway.

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

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

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

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

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

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete" - ACI 318-77 may be used.

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

or

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

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:

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

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

8.16.6.2.2 SHEAR IN COMPRESSION MEMBERS

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

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

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 psi.

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

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

Note:

(a) N_u is negative for tension.

(b) The quantity N_u/A_g shall be expressed in psi.

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

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

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

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

8.16.6.3.3 When inclined stirrups are used:

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

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

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

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

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

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

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

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

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

8.16.6.4 SHEAR FRICTION

8.16.6.4.1 These provisions may 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 A crack shall be assumed to occur along the shear plane with relative displacement along the assumed crack resisted by friction maintained by shear-friction reinforcement across the crack. Shear-friction reinforcement shall be placed approximately perpendicular to the assumed crack.

8.16.6.4.3 Design of cross sections subject to shear transfer shall be based on Equation (8-46), where shear strength V_n shall be computed by

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

where A_{vf} is the area of shear friction reinforcement. The coefficient of friction, μ, shall be 1.4 for concrete cast monolithically, 1.0 for concrete cast against hardened concrete, and 0.7 for concrete cast against as-rolled structural steel.

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

8.16.6.4.5 Additional reinforcement shall be provided for direct tension across the assumed crack.

8.16.6.4.6 Shear-friction reinforcement shall be well distributed across the assumed crack and shall be adequately anchored on both sides by embedment, hooks, or welding to special devices.

8.16.6.4.7 When concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 inch. When shear is transferred between as-rolled steel and concrete, the steel shall be free of loose rust, loose mill scale, grease, paint, or other foreign matter.

8.16.6.5 HORIZONTAL SHEAR DESIGN FOR COMPOSITE CONCRETE FLEXURAL MEMBERS

8.16.6.5.1 Provision shall be made for full transfer of horizontal shear forces at contact surfaces of interconnected elements.

8.16.6.5.2 Unless calculated in accordance with Article 8.16.6.5.3, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \tag{8-57}$$

where V_u is the factored shear force at the section considered and V_{nh} is the nominal horizontal shear strength.

8.16.6.5.3 Horizontal shear may be investigated by computing in any segment not exceeding 1/10 of the span, the actual change in compressive or tensile force to be transferred, and provisions made to transfer that factored force as horizontal shear V_u between interconnected elements.

8.16.6.5.4 Factored horizontal shear may be transferred at contact surfaces using the permissible shear strength V_{nh} stated below.

(a) When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not exceed 80 $b_v d$ in pounds.

(b) When minimum ties are provided in accordance with Article 8.16.6.5.6 and contact surfaces are clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not exceed 80 $b_v d$ in pounds.

(c) When minimum ties are provided in accordance with Article 8.16.6.5.6 and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 inch, shear strength V_{nh}, shall not exceed 350 $b_v d$ in pounds.

(d) When the factored shear force V_u at the section considered exceeds $\phi(350\ b_v d)$, design for horizontal shear shall be in accordance with Article 8.16.6.4.

8.16.6.5.5 When direct tension exists across any contact surface between interconnected elements, horizontal shear transfer by contact may be assumed only when minimum ties are provided in accordance with Article 8.16.6.5.6.

8.16.6.5.6 When ties are provided to transfer horizontal shear, the tie areas shall not be less than that required by Article 8.19.1 and tie spacing shall not exceed four times the least dimension of the supported element or 24 inches. The ties may consist of single bars or wire, multiple leg stirrups, or the vertical legs of welded wire fabric. All ties shall be adequately anchored into the 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 Article 8.16.6.1 through 8.16.6.3.

(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.

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

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

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

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

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

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

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

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

8.16.6.7 SPECIAL PROVISIONS FOR 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 $4\sqrt{f_c'}\,bd$. V_c need not be taken less than $3\sqrt{f_c'}\,bd$ for slabs monolithic with walls designed as rigid frames or $2.5\sqrt{f_c'}\,bd$ for simply supported slabs. The quantity $V_u d/M_u$ 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.

8.16.7 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 frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

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

8.16.8 SERVICEABILITY REQUIREMENTS

8.16.8.1 APPLICATION

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

8.16.8.2 SERVICE LOAD STRESSES

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

8.16.8.3 FATIGUE STRESS LIMITS

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

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

where:

f_f = stress range, ksi

f_{min} = algebraic minimum stress level, tension positive, compression negative, ksi

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:

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

where,

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

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar or wire, inches.

The quantity z in the 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.

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

8.17.2 DISTRIBUTION OF REINFORCEMENT

8.17.2.1 FLEXURAL TENSION REINFORCEMENT IN ZONES OF MAXIMUM TENSION

8.17.2.1.1 Where flanges of T-girders and box-girders are in tension, tension reinforcement shall be distributed over an effective tension flange width equal to 1/10 the girder span length or a width as defined in Article 8.10, 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 1/4 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 2 feet, longitudinal reinforcement having a total area at least equal to 10 percent of the area of the flexural tension reinforcement shall be placed near the side faces of the member and distributed in the zone of flexural tension with a spacing not more than the web width or 12 inches. Such reinforcement may be included in computing the flexural capacity only if a stress and strain compatibility analysis is made to determine stresses in the individual bars or wires.

8.17.2.2 TRANSVERSE DECK SLAB REINFORCEMENT IN T-GIRDERS AND BOX GIRDERS

At least 1/3 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 #3 in size for longitudinal bars that are #10 or smaller, and at least #4 in size for #11, #14, #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 which 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 one percent of longitudinal reinforcement to carry the loading. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bars shall be #5.

8.18.2 LATERAL REINFORCEMENT

8.18.2.1 GENERAL

In a compression member which 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 3/8 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

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

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 1 1/3 times the maximum size of coarse aggregate used.

8.18.2.2.4 Anchorage of spiral reinforcement shall be provided by 1 1/2 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 #3 in size for longitudinal bars that are #10 or smaller, and at least #4 in size for #11, #14, #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 #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

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

where b_w and s are in inches.

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

8.19.2 TYPES OF SHEAR REINFORCEMENT

8.19.2.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to the axis of the member or making an angle of 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.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1 Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least 1/8 square inch per foot in each direction.

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

8.21 SPACING LIMITS FOR REINFORCEMENT

8.21.1 For cast-in-place concrete the clear distance between parallel bars in a layer shall not be less than 1.5 bar diameters, 1.5 times the maximum size of the coarse aggregate, or 1 1/2 inches.

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, 1 1/3 times the maximum size of the coarse aggregate, or 1 inch.

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

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

8.21.5 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle. Bars larger than #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.

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
Concrete exposed to earth or weather	
Primary reinforcement	2
Stirrups, ties, and spirals	1 1/2
Concrete deck slabs	
Top reinforcement	2
Bottom reinforcement	1
Concrete not exposed to weather or in contact with ground	
Primary reinforcement	1 1/2
Stirrups, ties, and spirals	1

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.

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 hooks" as used herein shall mean either:

8.23.1.1 A 180-degree bend plus an extension of at least 4 bar diameters but not less than 2 1/2 inch at the free end of the bar, or

8.23.1.2 A 90-degree bend plus an extension of at least 12 bar diameters at the free end of the bar, or

8.23.1.3 For stirrup and tie hooks only, either a 90-degree or a 135-degree bend plus an extension of at least 6 bar diameters but not less than 2 1/2 inches at the free end of the bar.

8.23.2 MINIMUM BEND DIAMETERS

8.23.2.1 Except as provided in Articles 8.23.2.2 and 8.23.2.3, the diameter of bend measured on the inside of Grade 40 or Grade 60 bars shall be not less than the values given in Table 8.23.2.1.

Table 8.23.2.1

MINIMUM DIAMETERS OF BEND

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

8.23.2.2 For Grade 40 bars of size #3 to #11 inclusive with bends not exceeding 180 degrees, the minimum diameter of bend shall not be less than 5 bar diameters.

8.23.2.3 The inside diameter of bend for stirrups and ties shall not be less than 4 bar diameters for sizes #5 and smaller, and 5 bar diameters for sizes #6 to #8 inclusive.

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

8.24 DEVELOPMENT OF FLEXURAL REINFORCEMENT

8.24.1 GENERAL

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

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

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

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

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

8.24.1.4 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 #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 the following:

$$\ell_d \leq \frac{M}{V} + \ell_a \qquad (8\text{-}64)$$

where M is the computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed. V is the maximum shear force at the section. At a support, ℓ_a shall be the sum of the embedment length beyond the center of the support and the equivalent embedment length of any hook or mechanical anchorage provided. 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.

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:

#11 bar[1] and smaller .... $\frac{0.04A_b f_y}{\sqrt{f'_c}}$

but not less than[2] .... $0.0004d_b f_y$

#14 bars[3] .... $\frac{0.085\ f_y}{\sqrt{f'_c}}$

#18 bars[3] .... $\frac{0.11f_y}{\sqrt{f'_c}}$

deformed wire[4] .... $\frac{0.03d_b f_y}{\sqrt{f'_c}}$

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

8.25.2.1 Top reinforcement 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[5] .... $\frac{6.7\sqrt{f'_c}}{f_{ct}}$

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.

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[1]The constant has the unit of $1/\sqrt{\text{lb.}}$

[2]The constant has the unit of $\text{in}^2/\text{lb.}$

[3]The constant has the unit of $\text{in}^2/\sqrt{\text{lb.}}$

[4]The constant has the unit of $\text{in}/\sqrt{\text{lb.}}$

[5]The constant has the unit of $\sqrt{\text{lb.}}/\text{in.}$

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

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

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

$(A_s \text{ required}) / (A_s \text{ provided})$

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 8.33.3 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[1] .... $0.02 d_b f_y / \sqrt{f'_c}$

but not less than[2] ..... $0.0003\, d_b f_y$

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

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[1] The constant has the unit of $\text{in}/\sqrt{\text{lb}}$.

[2] The constant has the unit of in^2/lb.

8.27 DEVELOPMENT OF SHEAR REINFORCEMENT

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

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

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

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

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

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

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

8.27.2.4.2 One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced at least 2 inches from the first wire. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of bend of not less than 8-wire diameters.

8.27.3 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 HOOKS

8.29.1 Standard hooks shall be considered to develop a tensile stress in bar reinforcement $f_h = \xi\sqrt{f'_c}$ where ξ is not greater than the values in Table 8.29.1. The value of ξ may be increased by 30 percent where enclosure, consisting of external concrete or internal closed ties, spirals or stirrups, is provided perpendicular to the plane of the hook.

8.29.2 An equivalent embedment length, ℓ_e, of a standard hook may be computed using the provisions of Article 8.25 by substituting f_h for f_y and ℓ_e for ℓ_d.

8.29.3 Hooks shall not be considered effective in developing reinforcement in compression.

Table 8.29.1

ξ VALUES[a]

	f_y=60 ksi		f_y=40 ksi
Bar Size	Top Bars	Other Bars	All Bars
#3 to #5	540	540	360
#6	450	540	360
#7 to #9	360	540	360
#10	360	480	360
#11	360	420	360
#14	330	330	330
#18	220	220	220

[a]The constant has the units of $\sqrt{\text{lb}}$/in.

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 Articles 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 8.33.5 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:

$$0.03d_b\ (f_y - 20{,}000)/\sqrt{f'_c}\ ^{1} \qquad (8\text{-}65)$$

but not less than

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

8.30.1.3 The basic development length of welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 8.25.

8.30.2 SMOOTH WIRE FABRIC

8.30.2.1 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:

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

modified by (A_s required)/(A_s provided) for reinforcement in excess of that required by analysis and by factor of Article 8.25.2 for lightweight aggregate concrete, but ℓ_d shall not be less than 6 inches except in computation of lap splices by Article 8.33.6.

8.31 MECHANICAL ANCHORAGES

Any mechanical device capable of developing the strength of the reinforcement without damage to the concrete may be used as an anchorage.

8.32 COMBINATION OF DEVELOPMENT LENGTH

The development length ℓ_d, in tension, may consist of a combination of the equivalent embedment length, ℓ_e, of a hook or mechanical anchorage plus any additional embedment length of the reinforcement measured from the point of tangency of the hook.

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[1]The 20,000 has units of psi.

8.33 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.

8.33.1 LAP SPLICES

8.33.1.1 Lap splices shall not be used for bars larger than #11, except as provided in Article 4.4.9.7.

8.33.1.2 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 8.33.3 or 8.33.4 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.

8.33.1.3 Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than 1/5 the required length of lap or 6 inches.

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

8.33.2 WELDED SPLICES AND MECHANICAL CONNECTIONS

8.33.2.1 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."

8.33.2.2 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.

8.33.2.3 A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

8.33.2.4 Welded splices and mechanical connections not meeting requirements of Articles 8.33.2.2 and 8.33.2.3 may be used in accordance with Article 8.33.3.4.

8.33.3 SPLICES OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

8.33.3.1 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 .... $1.0\ell_d$
Class B splice .... $1.3\ell_d$
Class C splice .... $1.7\ell_d$

8.33.3.2 Lap splices of deformed bars and deformed wire in tension shall conform to Table 8.33.3.2.

Table 8.33.3.2

TENSION LAP SPLICES

(A_s provided)/(A_s required)[a]	Maximum Percent of A_s Spliced within Required Lap Length		
	50	75	100
Equal to or greater than 2	Class A	Class A	Class B
Less than 2	Class B	Class C	Class C

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

8.33.3.3 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 Articles 8.33.2.2 or 8.33.2.3.

8.33.3.4 Welded splices or mechanical connections used where the area of reinforcement provided is at least twice that required by analysis shall meet the following:

8.33.3.4.1 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.

8.33.3.4.2 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.

8.33.3.5 Splices in tension tie members shall be made with a full welded splice or a full mechanical connection in accordance with Article 8.33.2.2 or 8.33.2.3. Splices in adjacent bars shall be staggered at least ℓ_d.

8.33.4 SPLICES OF BARS IN COMPRESSION

8.33.4.1 LAP SPLICES IN COMPRESSION

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

In compression members where ties along the splice have an effective area not less than 0.0015hs, the lap splice length may be multipled 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.

8.33.4.2 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 1 1/2 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.

8.33.4.3 WELDED SPLICES OR MECHANICAL CONNECTIONS

Welded splices or mechanical connections used in compression shall meet the requirements of Article 8.33.2.2 or 8.33.2.3.

8.33.5 SPLICES OF WELDED DEFORMED WIRE FABRIC IN TENSION

8.33.5.1 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.

8.33.5.2 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 8.33.3.1.

8.33.6 SPLICES OF WELDED SMOOTH WIRE FABRIC IN TENSION

8.33.6.1 The minimum length of lap for lap splices of welded smooth wire fabric shall be in accordance with the following:

8.33.6.1.1 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.

8.33.6.1.2 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 9 - PRESTRESSED CONCRETE

PART A - GENERAL REQUIREMENTS AND MATERIALS

9.1 APPLICATION

9.1.1 GENERAL

The specifications of this section are intended for design of prestressed concrete bridge members. Members designed as reinforced concrete, except for a percentage of tensile steel stressed to improve service behavior, shall conform to the applicable specifications of Section 8.

Exceptionally long span or unusual structures require detailed consideration of effects which under this Section may have been assigned arbitrary values.

9.1.2 NOTATIONS

A_s = area of non-prestressed tension reinforcement (Articles 9.7 and 9.19)

A'_s = area of compression reinforcement (Article 9.19)

A^*_s = area of prestressing steel (Article 9.17)

A_{sf} = steel area required to develop the compressive strength of the overhanging portions of the flange (Article 9.17)

A_{sr} = steel area required to develop the compressive strength of the web of a flanged section (Articles 9.17 - 9.19)

A_v = area of web reinforcement (Article 9.20)

b = width of flange or flanged member or width of rectangular member

b' = width of a web of a flanged member

CR_c = loss of prestress due to creep of concrete (Article 9.16)

CR_s = loss of prestress due to relaxation of prestressing steel (Article 9.16)

D = nominal diameter of prestressing steel (Articles 9.17 and 9.27)

d = distance from extreme compressive fiber to centroid of the prestressing force, or to centroid of negative moment reinforcing for precast girder bridges made continuous

ES = loss of prestress due to elastic shortening (Article 9.16)

e = base of Naperian logarithms (Article 9.16)

f_{cds} = average concrete compressive stress at the c.g. of the prestressing steel under full dead load (Article 9.16)

f_{cir} = average concrete stress at the c.g. of the prestressing steel at time of release (Article 9.16)

f'_c = compressive strength of concrete at 28 days

f'_{ci} = compressive strength of concrete at time of initial prestress (Article 9.15)

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20)

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone) (Article 9.20)

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20)

Δf_s = total prestress loss, excluding friction (Article 9.16)

f_{se} = effective steel prestress after losses

f^*_{su} = average stress in prestressing steel at ultimate load

f'_s = ultimate strength of prestressing steel (Articles 9.15 and 9.17)

f_{sy} = yield strength of non-prestressed conventional reinforcement in tension (Articles 9.19 and 9.20)

f'_y = yield strength of non-prestressed conventional reinforcement in compression (Article 9.19)

f^*_y = yield point stress of prestressing steel (Article 9.15)

h = overall depth of member (Article 9.20)

I = moment of inertia about the centroid of the cross section (Article 9.20)

K = friction wobble coefficient per foot of prestressing steel (Article 9.16)

L = length of prestressing steel element from jack end to point x (Article 9.16)

M_{cr} = moment causing flexural cracking at section due to externally applied loads (Article 9.20)

M_{max} = maximum factored moment at section due to externally applied loads (Article 9.20)

M_u = design moment strength of a section (Articles 9.17 and 9.18)

p = A_s/bd, ratio of non-prestressed tension reinforcement (Articles 9.5 and 9.19)

p^* = A^*_s/bd, ratio of prestressing steel (Articles 9.17 - 9.19)

p' = A'_s/bd, ratio of compression reinforcement (Article 9.19)

Q = statical moment of cross sectional area, above or below the level being investigated for shear, about the centroid (Article 9.20)

SH = loss of prestress due to concrete shrinkage (Article 9.16)

s = longitudinal spacing of the web reinforcement (Article 9.20)

t = average thickness of the flange of a flanged member (Articles 9.17 and 9.18)

T_o = steel stress at jacking end (Article 9.16)

T_x = steel stress at any point x (Article 9.16)

v = permissible horizontal shear stress (Article 9.20)

V_c = nominal shear strength provided by concrete (Article 9.20)

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment (Article 9.20)

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web (Article 9.20)

V_d = shear force at section due to unfactored dead load (Article 9.20)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (Article 9.20)

V_p = vertical component of effective prestress force at section (Article 9.20)

V_s = nominal shear strength provided by shear reinforcement (Article 9.20)

V_u = factored shear force at section (Article 9.20)

Y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension (Article 9.20)

μ = friction curvature coefficient (Article 9.16)

α = total angular change of prestressing steel profile in radians from jacking end to point x (Article 9.16)

9.1.3 DEFINITIONS

The following terms are defined for general use. Specialized definitions appear in individual articles.

Anchorage Seating - Deformation of anchorage or seating of tendons in anchorage device when prestressing force is transferred from jack to anchorage device.

Bonded Tendon - Prestressing tendon that is bonded to concrete either directly or through grouting.

Coating - Material used to protect prestressing tendons against corrosion, to reduce friction between tendon and duct, or to debond prestressing tendons.

Couplers (Couplings) - Means by which prestressing force is transmitted from one partial-length prestressing tendon to another.

Creep of Concrete - Time-dependent deformation of concrete under sustained load.

Curvature Friction - Friction resulting from bends or curves in the specified prestressing tendon profile.

Debonding (blanketing) - Wrapping, sheathing, or coating prestressing strand to prevent bond between strand and surrounding concrete.

Duct - Hole or void formed in prestressed member to accommodate tendon for post-tensioning.

Effective Prestress - Stress remaining in concrete due to prestressing after all calculated losses have been deducted, excluding effects of superimposed loads and weight of member; stress remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load.

Elastic Shortening of Concrete - Shortening of member caused by application of forces induced by prestressing.

End Anchorage - Length of reinforcement, or mechanical anchor, or hook, or combination thereof, beyond point of zero stress in reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member.

End Block - Enlarged end section of member designed to reduce anchorage stresses.

Friction (post tensioning) - Surface resistance between tendon and duct in contact during stressing.

Grout Opening or Vent - Inlet, outlet, vent, or drain in post-tensioning duct for grout, water, or air.

Jacking Force - Temporary force exerted by device that introduces tension into prestressing tendons.

Loss of Prestress - Reduction in prestressing force resulting from combined effects of strains in concrete and steel, including effects of elastic shortening, creep and shrinkage of concrete, relaxation of steel stress, and for post-tensioned members, friction and anchorage seating.

Post-Tensioning - Method of prestressing in which tendons are tensioned after concrete has hardened.

Precompressed Zone - Portion of flexural member cross-section compressed by prestressing force.

Prestressed Concrete - Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

Pretensioning - Method of prestressing in which tendons are tensioned before concrete is placed.

Relaxation of Tendon Stress - Time-dependent reduction of stress in prestressing tendon at constant strain.

Shear Lag - Non-uniform distribution of bending stress over the cross section.

Shrinkage of Concrete - Time-dependent deformation of concrete caused by drying and chemical changes (hydration process).

Tendon - Wire, strand, or bar, or bundle of such elements, used to impart prestress to concrete.

Transfer - Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

Transfer Length - Length over which prestressing force is transferred to concrete by bond in pretensioned members.

Wobble Friction - Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment.

Wrapping or Sheathing - Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.

9.2 CONCRETE

The specified compressive strength, f'_c, of the concrete for each part of the structure shall be shown on the plans. The requirements for f'c shall be based on tests of cylinders made and tested in accordance with Division II.

9.3 REINFORCEMENT

9.3.1 PRESTRESSING STEEL

Wire, strands, or bars shall conform to one of the following specifications:

"Uncoated Stress-Relieved Wire for Prestressed Concrete" AASHTO M204.

"Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete" AASHTO M203.

"Uncoated High-Strength Steel Bar for Prestressing Concrete" ASTM A722.

Wire, strands, and bars not specifically listed in AASHTO M204, AASHTO M203, or ASTM A722 may be used provided they conform to the minimum requirements of these specifications.

9.3.2 NON-PRESTRESSED REINFORCEMENT

Non-prestressed reinforcement shall conform to the requirements in Article 8.3

PART B - ANALYSIS

9.4 GENERAL

Members shall be proportioned for adequate strength using these specifications as minimum guidelines. Continuous beams and other statically indeterminate structures shall be designed for adequate strength and satisfactory behavior. Behavior shall be determined by elastic analysis, taking into account the reactions, moments, shear, and axial forces produced by prestressing, the effects of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlement.

9.5 EXPANSION AND CONTRACTION

9.5.1 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.

9.5.2 Movements not otherwise provided for, including shortening during stressing, shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads or other devices.

9.6 SPAN LENGTH

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam. The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth.

9.7 FRAMES AND CONTINUOUS CONSTRUCTION

9.7.1 CAST-IN-PLACE POST-TENSIONED BRIDGES

The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

9.7.2 BRIDGES COMPOSED OF SIMPLE-SPAN PRECAST PRESTRESSED GIRDERS MADE CONTINUOUS

9.7.2.1 GENERAL

When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans.

9.7.2.2 POSITIVE MOMENT CONNECTION AT PIERS

9.7.2.2.1 Provision shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

9.7.2.2.2 Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi.

9.7.2.3 NEGATIVE MOMENTS

9.7.2.3.1 Negative moment reinforcement shall be proportioned by strength design with load factors in accordance with Article 9.14.

9.7.2.3.2 The effect of initial precompression due to prestress in the girders may be neglected in the negative moment calculation of ultimate strength if the maximum precompression stress is less than $0.4f'_c$ and the continuity reinforcement, p, in the deck slab is less than 0.015; where $p = A_s/bd$.

9.7.2.3.3 The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete.

9.7.2.4 COMPRESSIVE STRESS IN GIRDERS AT PIERS AT SERVICE LOADS

The compressive stress in ends of girders at piers resulting from addition of the effects of prestressing and negative live load bending shall not exceed $0.60f'_c$.

9.7.3 PRECAST SEGMENTAL BOX GIRDERS

9.7.3.1 GENERAL

9.7.3.1.1 Elastic analysis and beam theory may be used in the design of precast segmental box girder structures.

9.7.3.1.2 In the analysis of precast segmental box girder bridges, no tension shall be permitted across any joint between segments during any stage of erection or service loading.

9.7.3.1.3 In addition to the usual substructure design considerations, unbalanced cantilever moments due to segment weights and erection loads shall be accommodated in pier design or with auxiliary struts. Erection equipment which can eliminate these unbalanced moments may be used.

9.7.3.2 FLEXURE

The transverse design of precast segments for flexure shall consider the segment as a rigid box frame. Top slabs shall be analyzed as variable depth sections considering the fillets between the top slab and webs. Wheel loads shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location (see Article 3.11). Transverse post-tensioning of top slabs is generally recommended.

9.7.3.3 TORSION

In the design of the cross section, consideration shall be given to the increase in web shear resulting from eccentric loading or geometry of structure.

9.8 EFFECTIVE FLANGE WIDTH

9.8.1 T-BEAMS

9.8.1.1 For composite prestressed construction where slabs or flanges are assumed to act integrally with the beam, the effective flange width shall conform to the provisions for T-girder flanges in Article 8.10.1.

9.8.1.2 For monolithic prestressed construction, with normal slab span and girder spacing, the effective flange width shall be the distance center-to-center of beams. For very short spans, or where girder spacing is excessive, analytical investigations shall be made to determine the anticipated width of flange acting with the beam.

9.8.1.3 For monolithic prestressed design of isolated beams, the flange width shall not exceed 15 times the web width and shall be adequate for all design loads.

9.8.2 BOX GIRDERS

9.8.2.1 For cast-in-place box girders with normal slab span and girder spacing, where the slabs are considered an integral part of the girder, the entire slab width shall be assumed to be effective in compression.

9.8.2.2 For box girders of unusual proportions, including segmental box girders, methods of analysis which consider shear lag shall be used to determine stresses in the cross section due to longitudinal bending.

9.8.2.3 Adequate fillets shall be provided at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required.

9.9 FLANGE AND WEB THICKNESS - BOX GIRDERS

9.9.1 TOP FLANGE

The minimum top flange thickness for non-segmental box girders shall be 1/16 of the clear distance between girders or 6 inches, whichever is greater, except the minimum thickness may be reduced for factory produced precast elements to 5 1/2 inches. The top flange thickness for segmental box girders shall be determined in accordance with Article 9.7.3.2.

9.9.2 BOTTOM FLANGE

The minimum thickness of the bottom flange shall be determined by maximum allowable unit stresses as specified in Article 9.15 but in no case shall be less than 1/16 of the clear span between girders or 5 1/2 inches, whichever is the greater, except the minimum thickness may be reduced for factory produced precast elements to 5 inches.

9.9.3 WEB

Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

9.10 DIAPHRAGMS

9.10.1 GENERAL

Diaphragms shall be provided in accordance with Articles 9.10.2 and 9.10.3 except that diaphragms may be omitted where tests or structural analysis show adequate strength.

9.10.2 T-BEAMS

Diaphragms or other means shall be used at span ends to strengthen the free edge of the slab and to transmit lateral forces to the substructure. Intermediate diaphragms shall be placed between the beams at the points of maximum moment for spans over 40 feet.

9.10.3 BOX GIRDERS

9.10.3.1 For spread box beams, diaphragms shall be placed within the box and between boxes at span ends and at the points of maximum moment for spans over 80 feet.

9.10.3.2 For precast box multi-beam bridges, diaphragms are required only if necessary for slab end support or to contain or resist transverse tension ties.

9.10.3.3 For cast-in-place box girders, diaphragms or other means shall be used at span ends to resist lateral forces and maintain section geometry. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater.

9.10.3.4 For precast segmental box girders, diaphragms shall be placed within the box at span ends. Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater.

9.10.3.5 For all types of prestressed boxes in bridges with inside radius of curvature less than 800 feet, intermediate diaphragms may be required and the spacing and strength of diaphragms shall be given special consideration in the design of the structure.

9.11 DEFLECTIONS

9.11.1 GENERAL

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

9.11.2 PRECAST SEGMENTAL BOX GIRDERS

Deflections shall be calculated prior to manufacture of segments, based on the anticipated production and erection schedules. Calculated deflections shall be used as a guide against which erected deflection measurements are checked.

9.12 DECK PANELS

9.12.1 GENERAL

9.12.1.1 Precast prestressed deck panels, that are used as permanent forms spanning between stringers, may be designed compositely with the cast-in-place portion of the slabs to support additional dead loads and live loads.

9.12.1.2 The panels shall be analyzed assuming they support their self weight, any construction loads and the weight of the cast-in-place concrete, and shall be analyzed assuming they act compositely with the cast-in-place concrete to support moments due to additional dead loads and live loads.

9.12.2 BENDING MOMENT

9.12.2.1 Live load moments shall be computed in accordance with Article 3.24.3.

9.12.2.2 In calculating stresses in the deck panel due to negative moment near the stringer, no compression due to prestressing shall be assumed to exist.

PART C - DESIGN

9.13 GENERAL

9.13.1 DESIGN THEORY AND GENERAL CONSIDERATIONS

9.13.1.1 Members shall meet the strength requirements specified herein.

9.13.1.2 Design shall be based on strength (Load Factor Design) and on behavior at service conditions (Allowable Stress Design) at all load stages that may be critical during the life of the structure from the time prestressing is first applied.

9.13.1.3 Stress concentrations due to the prestressing shall be considered in the design.

9.13.1.4 The effects of temperature and shrinkage shall be considered.

9.13.2 BASIC ASSUMPTIONS

The following assumptions are made for design purposes for monolithic members.

9.13.2.1 Strains vary linearly over the depth of the member throughout the entire load range.

9.13.2.2 Before cracking, stress is linearly proportional to strain.

9.13.2.3 After cracking, tension in the concrete is neglected.

9.13.3 COMPOSITE FLEXURAL MEMBERS

Composite flexural members consisting 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 shall conform to the provisions of Articles 8.14.2.1 through 8.14.2.4, 8.14.2.6, and the following:

9.13.3.1 Where an entire member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Articles 9.20.1 through 9.20.3.

9.13.3.2 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 9.20.4.

9.13.3.3 In structures with a cast-in-place slab on precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the beams. Because the tensile shrinkage develops over an extended time period, the effect on the beams is reduced by creep. Differential shrinkage may influence the cracking load and the beam deflection profile. When these factors are particularly significant, the effect of differential shrinkage should be added to the effect of loads.

9.14 LOAD FACTORS

The computed strength capacity shall not be less than the largest value from load factor design in Article 3.22.

The following strength capacity reduction factors shall be used:

For factory produced precast prestressed concrete members $\phi = 1.0$
For post-tensioned cast-in-place concrete members $\phi = 0.95$
For shear $\phi = 0.90$

9.15 ALLOWABLE STRESSES

The design of precast prestressed members ordinarily shall be based on $f'_c = 5{,}000$ psi. An increase to 6,000 psi is permissible where, in the Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the Engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths. The provisions of this Section are equally applicable to prestressed concrete structures and components designed with lower concrete strengths.

9.15.1 PRESTRESSING STEEL

Temporary stress before loss due to creep and shrinkage .... $0.70f'_s$
Stress at service load[1] after losses .... $0.80\ f^*_y$

(Overstressing to $0.80\ f'_s$ for short periods of time may be permitted provided the stress, after transfer to concrete in pretensioning or seating of anchorage in post-tensioning, does not exceed $0.70\ f'_s$).

9.15.2 CONCRETE

9.15.2.1 TEMPORARY STRESSES BEFORE LOSSES DUE TO CREEP AND SHRINKAGE:

Compression

Pretensioned members .... $0.60\ f'_{ci}$
Post-tensioned members .... $0.55\ f'_{ci}$

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[1]Service load consists of all loads contained in Article 3.2 but does not include overload provisions.

Tension

Precompressed tensile zone .... No temporary allowable stresses are specified. See Article 9.15.2.2 for allowable stresses after losses.

Other Areas

In tension areas with no bonded reinforcement .... 200 psi or $3\sqrt{f'_{ci}}$

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed .... $7.5\sqrt{f'_{ci}}$

9.15.2.2 STRESS AT SERVICE LOAD AFTER LOSSES HAVE OCCURRED:

Compression .... $0.40\ f'_c$

Tension in the precompressed tensile zone

(a) For members with bonded reinforcement .... $6\sqrt{f'_c}$

For severe corrosive exposure conditions, such as coastal areas .... $3\sqrt{f'_c}$

(b) For members without bonded reinforcement 0

Tension in other areas is limited by the allowable temporary stresses specified in Article 9.15.2.1.

9.15.2.3 CRACKING STRESS*

Modulus of rupture from tests or if not available.

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

For sand-lightweight concrete .... $6.3\sqrt{f'_c}$

For all other lightweight concrete .... $5.5\sqrt{f'_c}$

9.15.2.4 ANCHORAGE BEARING STRESS

Post-tensioned anchorage at service load .... 3,000 psi (but not to exceed $0.9\ f'_{ci}$)

*Refer to Article 9.18.

9.16 LOSS OF PRESTRESS

9.16.1 FRICTION LOSSES

Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations shall be shown on the plans. These friction losses shall be calculated as follows:

$$T_o = T_x e^{(KL+\mu\alpha)} \qquad (9\text{-}1)$$

When $(KL + \mu\alpha)$ is not greater than 0.3, the following equation may be used:

$$T_o = T_x (1+KL+\mu\alpha) \qquad (9\text{-}2)$$

The following values for K and μ may be used when experimental data for the materials used are not available:

Type of Steel	Type of Duct	K/ft.	μ
Wire or ungalvanized strand	Bright Metal Sheathing	0.0020	0.30
	Galvanized Metal Sheathing	0.0015	0.25
	Greased or Asphalt-Coated and Wrapped	0.0020	0.30
	Galvanized Rigid	0.0002	0.25
High-strength bars	Bright Metal Sheathing	0.0003	0.20
	Galvanized Metal Sheathing	0.0002	0.15

Friction losses occur prior to anchoring but should be estimated for design and checked during stressing operations. Rigid ducts shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete. Rigid ducts may be fabricated with either welded or interlocked seams. Galvanizing of the welded seam will not be required.

9.16.2 PRESTRESS LOSSES

9.16.2.1 GENERAL

Loss of prestress due to all causes, excluding friction, may be determined by the following method.* The method is based

*Should more exact prestress losses be desired, data representing the materials to be used, the methods of curing, the ambient service condition and any pertinent structural details should be determined for use in accordance with a method of calculating prestress losses that is supported by appropriate research data.

on normal weight concrete and one of the following types of prestressing steel: 250 or 270 ksi, seven-wire, stress-relieved strand; 240 ksi stress-relieved wires; or 145 to 160 ksi smooth or deformed bars. For data regarding the properties and effects of lightweight aggregate concrete and low-relaxation tendons, refer to documented tests or see authorized suppliers.

TOTAL LOSS

$$\Delta f_s = SH + ES + CR_c + CR_s \qquad (9\text{-}3)$$

where

Δf_s = total loss excluding friction in psi
SH = loss due to concrete shrinkage in psi
ES = loss due to elastic shortening in psi
CR_c = loss due to creep of concrete in psi
CR_s = loss due to relaxation of prestressing steel in psi

9.16.2.1.1 SHRINKAGE

Pretensioned Members

$$SH = 17{,}000 - 150\ RH \qquad (9\text{-}4)$$

Post-tensioned Members

$$SH = 0.80\ (17{,}000 - 150\ RH) \qquad (9\text{-}5)$$

where RH = mean annual ambient relative humidity in percent (see Figure 9.16.2.1.1).

9.16.2.1.2 ELASTIC SHORTENING

Pretensioned Members

$$ES = \frac{E_s}{E_{ci}} f_{cir} \qquad (9\text{-}6)$$

Post-tensioned Members*

$$ES = 0.5 \frac{E_s}{E_{ci}} f_{cir} \qquad (9\text{-}7)$$

*Certain tensioning procedures may alter the elastic shortening losses.

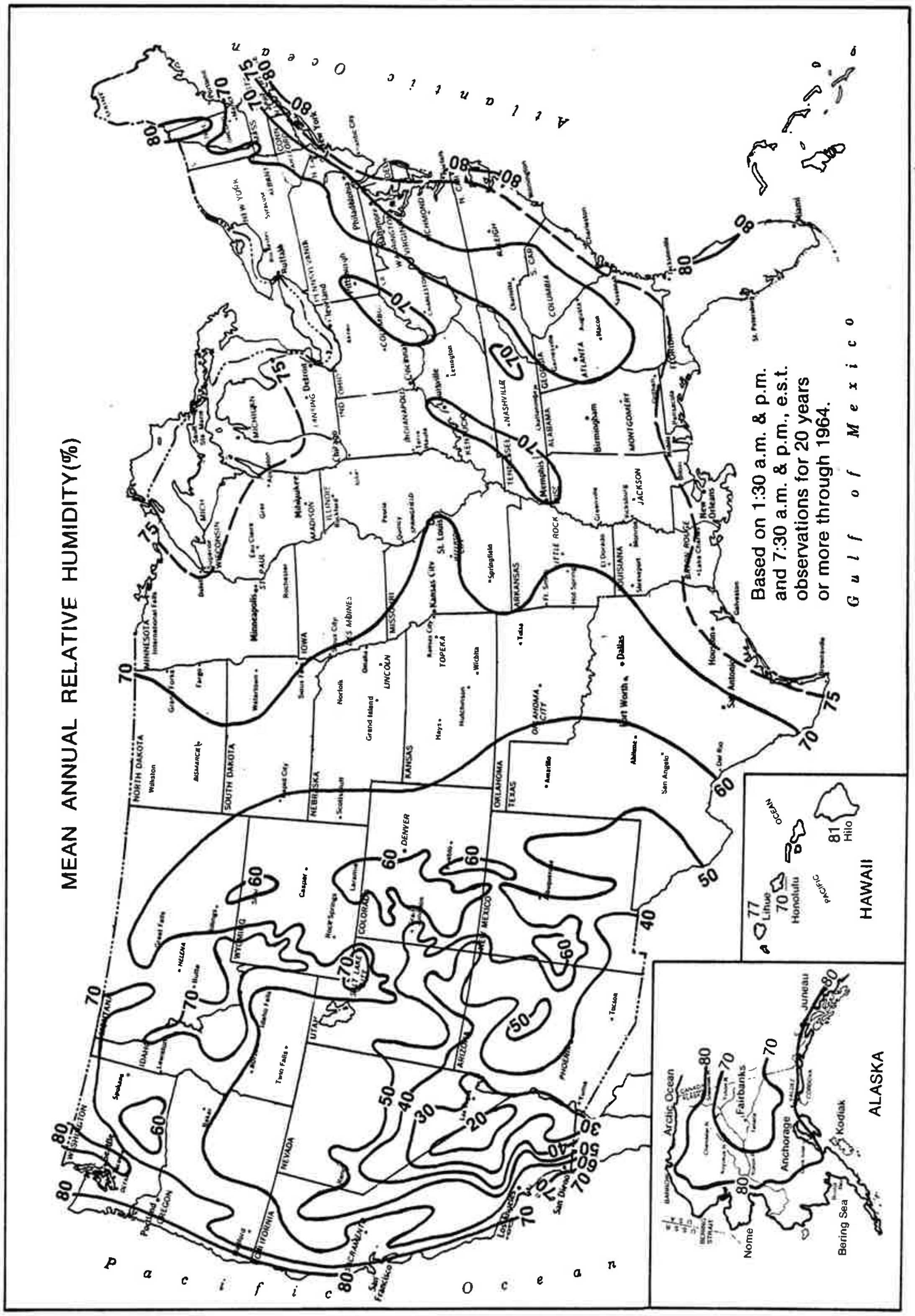


FIGURE 9.16.2.1.1

where

E_s = modulus of elasticity of prestressing steel strand which can be assumed to be 28 x 10^6 psi.

E_{ci} = modulus of elasticity of concrete in psi at transfer of stress which can be calculated from:

$$E_{ci} = 33w^{3/2}\sqrt{f'_{ci}} \qquad (9\text{-}8)$$

where w is the concrete unit weight in lb/ft^3 and f'_{ci} is in psi

f_{cir} = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer, f_{cir} shall be computed at the section or sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing the concrete for pretensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 f'_s for typical pretensioned members.)

9.16.2.1.3 CREEP OF CONCRETE

Pretensioned and post-tensioned members.

$$CR_c = 12\ f_{cir} - 7\ f_{cds} \qquad (9\text{-}9)$$

where

f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied.

9.16.2.1.4 RELAXATION OF PRESTRESSING STEEL*

Pretensioned members

250 to 270 ksi Strand

$$CR_s = 20{,}000 - 0.4\ ES - 0.2\ (SH + CR_c) \qquad (9\text{-}10)$$

*The relaxation losses are based on an initial stress of 0.70f'_s.

Post-tensioned members

250 to 270 ksi Strand

$$CR_s = 20{,}000 - 0.3\ FR - 0.4\ ES - 0.2\ (SH + CR_c) \qquad (9\text{-}11)$$

240 ksi Wire

$$CR_s = 18{,}000 - 0.3\ FR - 0.4\ ES - 0.2\ (SH + CR_c) \qquad (9\text{-}12)$$

145 to 160 ksi Bars

$$CR_s = 3{,}000$$

where

FR = friction loss stress reduction in psi below the level of 0.70 f'_s at the point under consideration, computed according to Article 9.16.1.

ES, SH, and CR_c = appropriate values as determined for either pre-tensioned or post-tensioned members.

9.16.2.2 ESTIMATED LOSSES

In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of usual design. These loss values are based on use of normal weight concrete, normal prestress levels, and average exposure conditions. For exceptionally long spans, or for unusual designs, the method in Article 9.16.2.1 or a more exact method shall be used.

Table 9.16.2.2

ESTIMATE OF PRESTRESS LOSSES

Type of Prestressing Steel	Total Loss f'_c = 4,000 psi	Total Loss f'_c = 5,000 psi
Pretensioning		
Strand	------	45,000 psi
Post-Tensioning[a]		
Wire or Strand	32,000 psi	33,000 psi
Bars	22,000 psi	23,000 psi

[a]Losses due to friction are excluded. Friction losses should be computed according to Article 9.16.1.

9.17 FLEXURAL STRENGTH

9.17.1 GENERAL

Prestressed concrete members may be assumed to act as uncracked members subjected to combined axial and bending stresses within specified service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included in pretensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted.

9.17.2 RECTANGULAR SECTIONS

For rectangular or flanged sections in which the neutral axis lies within the flange, the ultimate flexural strength shall be assumed as

$$M_u = A^*_s f^*_{su} d \left(1 - 0.6 \frac{p^* f^*_{su}}{f'_c}\right) \quad (9\text{-}13)$$

9.17.3 FLANGED SECTIONS

If the neutral axis falls outside the flange (usually if the flange thickness is less than 1.4 $dp^*f^*_{su}/f'_c$), the ultimate flexural strength shall be assumed as

$$M_u = A_{sr} f^*_{su} d \left(1 - 0.6 \frac{A_{sr} f^*_{su}}{b'df'_c}\right) + 0.85\, f'_c\, (b-b')t(d-0.5t) \quad (9\text{-}14)$$

where $A_{sr} = A^*_s - A_{sf}$ (9-15)

A_{sr} = the steel area required to develop the ultimate compressive strength of the web of a flanged section.

$A_{sf} = 0.85\, f'_c\, (b-b')t/f^*_{su}$ (9-16)

A_{sf} = the steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

9.17.4 STEEL STRESS

9.17.4.1 Unless the value of f^*_{su} can be more accurately known from detailed analysis, the following values may be used:

Bonded members .... $f^*_{su} = f'_s \left(1 - 0.5 \frac{p^* f'_s}{f'_c}\right)$ (9-17)

Unbonded members ...... $f^*_{su} = f_{se} + 15{,}000$ (9-18)

provided that:

(1) The stress-strain properties of the prestressing steel approximate those specified in Article 4.33.10 - Division II.

(2) The effective prestress after losses is not less than 0.5 f'_s.

9.17.4.2 At ultimate load, the stress in the prestressing steel of precast deck panels shall be limited to:

$$f^*_{su} = \frac{\ell_x}{D} + \frac{2}{3} f_{se} \qquad (9\text{-}19)$$

but shall not be greater than f^*_{su} as given by the equations in Article 9.17.4.1. In the above equation:

D = nominal diameter of strand, in.

f_{se} = effective stress in prestressing strand after losses, ksi.

ℓ_x = distance from end of prestressing strand to center of panel, in.

9.18 MAXIMUM AND MINIMUM STEEL PERCENTAGE

9.18.1 MAXIMUM STEEL

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general the reinforcement index shall be such that:

$$p^* \frac{f^*_{su}}{f'_c} \quad \text{for rectangular sections} \qquad (9\text{-}20)$$

and

$$A_{sr} \frac{f^*_{su}}{b'df'_c} \quad \text{for flanged sections} \qquad (9\text{-}21)$$

does not exceed 0.30. For steel with reinforcement indices greater than this, the ultimate flexural strength shall be assumed not greater than:

$$M_u = 0.25\ f'_c\ bd^2 \text{ for rectangular sections, or} \qquad (9\text{-}22)$$

$$M_u = 0.25\ b'd^2f'_c + 0.85\ f'_c(b-b')t(d-0.5t) \text{ for flanged sections} \qquad (9\text{-}23)$$

9.18.2 MINIMUM STEEL

9.18.2.1 The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate load in flexure at the critical section at least 1.2 times the cracking load calculated on the basis of the modulus of rupture (refer to Article 9.15.2.3.)

9.18.2.2 The minimum amount of non-prestressed longitudinal reinforcement provided in the cast-in-place portion of slabs utilizing precast prestressed deck panels shall be 0.25 square inch per foot of slab width.

9.19 NON-PRESTRESSED REINFORCEMENT

Non-prestressed reinforcement may be considered as contributing to the tensile strength of the beam at ultimate strength in an amount equal to its area times its yield point, provided that

$$\frac{pf_{sy}}{f'_c} + \frac{p^*f^*_{su}}{f'_c} - \frac{p'f'_y}{f'_c} \leq 0.3 \quad \text{for rectangular sections, or} \tag{9-24}$$

$$\frac{A_sf_{sy}}{b'df'_c} + \frac{A_{sr}f^*_{su}}{b'df'_c} - \frac{A'_sf'_y}{b'df'_c} \leq 0.3 \quad \text{for flanged sections} \tag{9-25}$$

9.20 SHEAR

9.20.1 GENERAL

9.20.1.1 Prestressed concrete flexural members, except solid slabs and footings, shall be reinforced for shear and diagonal tension stresses. Voided slabs shall be investigated for shear, but shear reinforcement may be omitted if the factored shear force, V_u, is less than half the shear strength provided by the concrete ϕV_c.

9.20.1.2 Web reinforcement shall consist of stirrups perpendicular to the axis of the member or welded wire fabric with wires located perpendicular to the axis of the member. Web reinforcement shall extend to a distance d from the extreme compression fiber 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. Web reinforcement shall be anchored at both ends for its design yield strength in accordance with the provisions of Article 8.27.

9.20.1.3 Members subject to shear shall be designed so that

$$V_u \leq \phi (V_c+V_s) \qquad (9\text{-}26)$$

where V_u is the factored shear force at the section considered, V_c is the nominal shear strength provided by concrete and V_s is the nominal shear strength provided by web reinforcement.

9.20.1.4 When the reaction to the applied loads introduces compression into the end regions of the member, sections located at a distance less than h/2 from the face of the support may be designed for the same shear V_u as that computed at a distance h/2.

9.20.1.5 Reinforced keys shall be provided in the webs of precast segmental box girders to transfer erection shear. Possible reverse shearing stresses in the shear keys shall be investigated, particularly in segments near a pier. At time of erection, the shear stress carried by the shear key shall not exceed $2\sqrt{f_c'}$.

9.20.2 SHEAR STRENGTH PROVIDED BY CONCRETE

9.20.2.1 The shear strength provided by concrete, V_c, shall be taken as the lesser of the values V_{ci} or V_{cw}.

9.20.2.2 The shear strength, V_{ci}, shall be computed by

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (9\text{-}27)$$

but need not be less than $1.7\sqrt{f'_c}\,b'd$ and d need not be taken less than 0.8h.

The moment causing flexural cracking at the section due to externally applied loads, M_{cr}, shall be computed by

$$M_{cr} = \frac{I}{Y_t} (6 \sqrt{f'_c} + f_{pe} - f_d) \qquad (9\text{-}28)$$

The maximum factored moment and factored shear at the section due to externally applied loads, M_{max} and V_i, shall be computed from the load combination causing maximum moment at the section.

9.20.2.3 The shear strength, V_{cw}, shall be computed by

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3\, f_{pc})\, b'd + V_p \qquad (9\text{-}29)$$

but d need not be taken less than 0.8h.

9.20.2.4 For a pretensioned member in which the section at a distance h/2 from the face of support is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

9.20.3 SHEAR STRENGTH PROVIDED BY WEB REINFORCEMENT

9.20.3.1 The shear strength provided by web reinforcement shall be taken as

$$V_s = \frac{A_v f_{sy} d}{s} \qquad (9\text{-}30)$$

where A_v is the area of web reinforcement within a distance s. V_s shall not be taken greater than $8 \sqrt{f'_c}\, b'd$.

9.20.3.2 The spacing of web reinforcing shall not exceed 0.75h or 24 inches. When V_s exceeds $4 \sqrt{f'_c}\, b'd$, this maximum spacing shall be reduced by one-half.

9.20.3.3 The minimum area of web reinforcement shall be

$$A_v = \frac{50\, b's}{f_{sy}} \qquad (9\text{-}31)$$

where b' and s are in inches and f_{sy} is in psi.

9.20.3.4 The design yield strength of web reinforcement, f_{sy}, shall not exceed 60,000 psi.

9.20.4 HORIZONTAL SHEAR DESIGN-COMPOSITE FLEXURAL MEMBERS

9.20.4.1 Composite members shall be interconnected in accordance with Articles 9.20.4.2 through 9.20.4.4 to transfer shear along contact surfaces and to prevent separation of elements.

9.20.4.2 Full transfer of the ultimate horizontal shear forces may be assumed when contact surfaces are clean and intentionally roughened, minimum vertical ties are provided in accordance with Article 9.20.4.4, all stirrups are fully anchored into all intersecting components, and the web members are designed to resist the entire vertical shear. Otherwise, ultimate horizontal shear stress shall be calculated and limited according to Articles 9.20.4.3 and 9.20.4.4.

9.20.4.3 In lieu of the requirements of Article 9.20.4.2, ultimate horizontal shear stress may be computed by the formula $v = V_u Q/Ib$. To resist the computed shear stress, the following values of shear capacity shall be assumed at the contact surface:

When the minimum steel tie requirements of Article 9.20.4.4 are met .... 75 psi

When the minimum steel tie requirements of Article 9.20.4.4 are met and the contact surface of the precast element is clean and intentionally roughened .... 300 psi

In addition to the above values, for each percent of the contact surface provided by stirrup and vertical tie reinforcement crossing the joint in excess of the percentage provided by the minimum requirements of Article 9.20.4.4 .... 150 psi

9.20.4.4 All web reinforcement shall extend into cast-in-place decks. The minimum total area of vertical ties per linear foot of span shall be not less than the area of two #3 bars spaced at 12 inches. Web reinforcement may be used to satisfy the vertical tie requirement. The spacing of vertical ties shall not be greater than four times the average thickness of the composite flange and in no case greater than 24 inches.

9.20.5 HORIZONTAL SHEAR - BOX GIRDERS

The horizontal shearing unit stress at the junction of the flange and the monolithic fillet joining it to the girder web shall not exceed $0.15f'_c$.

9.21 ANCHORAGE ZONES

9.21.1 For beams with post-tensioning tendons, end blocks shall be used to distribute the concentrated prestressing forces at the anchorage. Where all tendons are pretensioned wires or 7-wire strand, the use of end blocks will not be required. End blocks shall have sufficient area to allow the spacing of the prestressing steel as specified in Article 9.25. Preferably, they shall be as wide as the narrower flange of the beam. They shall have a length at least equal to three-fourths of the depth of the beam and in any case 24 inches. In post-tensioned members, a closely spaced grid of both vertical and horizontal bars shall be placed near the face of the end block to resist bursting stresses. Amounts of steel in the end grid should follow recommendations of the supplier of the anchorage. Where such recommendations are not available the amount of steel in the grid shall be designed and shall consist of at least #3 bars on 3-inch centers in each direction placed not more than 1 1/2 inches from the inside face of the anchor bearing plate.

9.21.2 Closely spaced reinforcement shall be placed both vertically and horizontally throughout the length of the end block in accordance with accepted methods of end block stress analysis.

9.21.3 In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total prestressing force shall be placed within the distance of d/4 of the end of the beam, the end stirrups to be as close to the end of the beam as practicable. For at least the distance d from the end of the beam, nominal reinforcement shall be placed to enclose the prestressing steel in the bottom flange. For box girders, transverse reinforcement shall be provided and anchored by extending the leg into the web of the girder.

9.22 CONCRETE STRENGTH AT STRESS TRANSFER

Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the members, is at least 4,000 psi for pretensioned members and 3,500 psi for post-tensioned members.

9.23 DECK PANELS

9.23.1 Deck panels shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the panels are placed on the supporting stringers. The top surface of the panels shall be roughened in such a manner as to insure composite action between the precast and cast-in-place concrete.

9.23.2 Reinforcing bars, or equivalent mesh, shall be placed in the panel transverse to the strands to provide at least 0.11 square inches per foot of panel.

9.24 FLANGE REINFORCEMENT

Bar reinforcement for cast-in-place T-beam and box girder flanges shall conform to the provisions in Articles 8.17.2.2 and 8.17.2.3 except that the minimum reinforcement in bottom flanges shall be 0.3 percent of the flange section.

9.25 COVER AND SPACING OF STEEL

9.25.1 MINIMUM COVER

The following minimum concrete cover shall be provided for prestressing and conventional steel:

9.25.1.1 Prestressing Steel and Main Reinforcement .... 1 1/2 in.

9.25.1.2 Slab Reinforcement

9.25.1.2.1 Top of Slab .... 1 1/2 in.

When deicers are used .... 2 in.

9.25.1.2.2 Bottom of Slab .... 1 in.

9.25.1.3 Stirrups and Ties .... 1 in.

9.25.1.4 When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.

9.25.2 MINIMUM SPACING

9.25.2.1 The minimum clear spacing of prestressing steel at the ends of beams shall be as follows:

Pretensioning steel: three times the diameter of the steel or 1 1/3 times the maximum size of the concrete aggregate, whichever is greater.

Post-tensioning ducts: 1 1/2 inches or 1 1/2 times the maximum size of the concrete aggregate, whichever is the greater.

9.25.2.2 Prestressing strands in deck panels shall be spaced symmetrically and uniformly across the width of the panel. They shall not be spaced farther apart than 1 1/2 times the total composite slab thickness or more than 18 inches.

9.25.3 BUNDLING

9.25.3.1 When post-tensioning steel is draped or deflected, post-tensioning ducts may be bundled in groups of three maximum, provided that the spacing specified in Article 9.25.2 is maintained in the end 3 feet of the member.

9.25.3.2 Where pretensioning steel is bundled, all bundling shall be done in the middle third of the beam length and the deflection points shall be investigated for secondary stresses.

9.25.4 SIZE OF DUCTS

9.25.4.1 For tendons made up of a number of wires, bars, or strands, duct area shall be at least twice the net area of the prestressing steel.

9.25.4.2 For tendons made up of a single wire, bar, or strand, the duct diameter shall be at least 1/4 inch larger than the nominal diameter of the wire, bar, or strand.

9.26 POST-TENSIONING ANCHORAGES AND COUPLERS

9.26.1 Anchorages, couplers, and splices for bonded post-tensioned reinforcement shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. Couplers and splices shall be placed in areas approved by the Engineer and enclosed in a housing long enough to permit the necessary movements. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

9.26.2 The anchorages of unbonded tendons shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The total elongation under ultimate load of the tendon shall not be less than 2 percent measured in a minimum gauge length of 10 feet.

9.26.3 For unbonded tendons, a dynamic test shall be performed on a representative specimen and the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full size tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10 percent of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

9.26.4 Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

9.26.5 Anchorages, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion.

9.27 EMBEDMENT OF PRESTRESSED STRAND

9.27.1 Three or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than

$$\left(f^*_{su} - \frac{2}{3} f_{se}\right) D \qquad (9\text{-}32)$$

where

D is the nominal diameter in inches, f^*_{su} and f_{se} are kips per square inch and the parenthetical expression is considered to be without units.

9.27.2 Investigations may be limited to those cross sections nearest each end of the member which are required to develop their full ultimate capacity.

9.27.3 Where strand is debonded at the end of a member and tension at service load is allowed in the pre-compressed tensile zone, the development length required above shall be doubled.

9.28 BEARINGS

Bearing devices for prestressed concrete structures shall be designed in accordance with Article 10.29 and Section 14.

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, 10.48.1.1, 10.48.2.1, 10.48.4.3, 10.48.5.5 and 10.55.1)

A_F = amplification factor (Articles 10.37.1.1 and 10.55.1)

$(AF_y)_{bf}$ = product of area and yield point for bottom flange of steel section (Article 10.50.1.1.1)

$(AF_y)_c$ = product of area and yield point of that part of reinforcing which lies in the compression zone of the slab (Article 10.50.1.1.1)

$(AF_y)_{tf}$ = product of area and yield point for top flange of steel section (Article 10.50.1.1.1)

$(AF_y)_w$ = product of area and yield point for web of steel section (Article 10.50.1.1.1)

A_f = area of flange (Articles 10.39.4.4.2, 10.48.2.1, 10.53.1.2 and 10.56.3)

A_s^r = total area of longitudinal reinforcing steel of 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 10.32.3.2 and 10.56.2)

a = spacing of transverse stiffeners (Article 10.39.4.4.2)

a = depth of stress block (Article 10.50.1.1.1)

a = ratio of numerically smaller to the larger end moment (Article 10.54.2.2)

B = a constant based on the number of stress cycles (Article 10.38.5.1.1.)

B = constant for stiffeners (Article 10.48.5.5.)

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 inches (Articles 10.32.3.2.2 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 = distance from center of bolt to toe of fillet of connected part (Article 10.56.2)

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, 10.48.4.1, 10.48.5.5, 10.50, 10.51.5.5, and 10.55.3)

C = web buckling coefficient (Articles 10.34.4 and 10.48.5.3)

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_c = column slenderness ratio dividing elastic and inelastic buckling (Table 10.32.1A)

C_{mx} = a coefficient about x axis (Article 10.36)

C_{my} = a coefficient about the Y axis (Article 10.36)

c = thickness of concrete slab (Article 10.38.5.1.2)

c = buckling stress coefficient (Article 10.51.5)

D = clear distance between flanges in inches (Article 10.15.2)

D = depth of beam in feet (Article 10.13)

D = clear unsupported distance between flange components (Articles 10.48.2.1, 10.48.5, 10.48.6, 10.49.2, 10.49.3.2 and 10.55.2)

D = depth of web (Articles 10.34.3, 10.34.4, and 10.37.2)

D = unsupported depth of web plate between flanges in inches (Articles 10.34.4, 10.34.5 and 10.40.2.2)

D_c = the clear distance between the neutral axis and the compression flange (Articles 10.49.2 and 10.49.3)

D_c = moments caused by dead load acting on composite girder (Article 10.50.1.2.2)

D_s = moments caused by dead load acting on steel girder (Article 10.50.1.2.2)

d = bolt diameter (Table 10.32.3B and Article 10.24.6.2)

d = diameter of stud in inches (Article 10.38.5.1)

d = depth of beam or girder (Articles 10.48.1 and 10.48.2)

d = diameter of rocker or roller in inches (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, 10.40.2.2, 10.48.5.3, 10.48.5.5 and 10.48.6.3)

d_w = depth of web (Article 10.50.1.1.1)

E = modulus of elasticity of steel, psi (Table 10.32.1A and Articles 10.15.3, 10.36, 10.37, 10.39.4.4.2, 10.48.4.1, 10.53.1.3, 10.54.1, and 10.55.1)

E_c = modulus of elasticity of concrete, psi (Article 10.38.5.1.2)

F = maximum compressive stress in psi (Article 10.41.4.6)

F_a = allowable axial unit stress (Table 10.32.1A and Articles 10.36, 10.37.1.1 and 10.55.1)

F_b = allowable bending unit stress (Table 10.32.1A and Articles 10.37.1.1 and 10.55.1)

F_e = Euler buckling stress (Articles 10.37.1.1, 10.54.2.1, and 10.55.1)

F_{cr} = the 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_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_s = limiting bending stress (Article 10.34.4)

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

F_y^r = specified minimum yield point of the reinforcing steel (Article 10.38.5.1.2)

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

F_u = specified minimum tensile strength (Tables 10.32.1A and 10.32.3B, Articles 10.18.4, 10.24.6.2)

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, 10.40.2.2.1, and 10.57.3)

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, 10.51.1 and 10.54)

F_{yf} = specified minimum yield strength of the flange (Articles 10.53.1)

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 = the 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)

$f_{d\ell}$ = the top flange compressive stress due to non-composite dead load (Article 10.34.2.4 and 10.50.1)

f_p = computed bearing stress due to design load (Table 10.32.3B and Article 10.24.6.2)

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 10.32.3.2.3, 10.56.1.3.2 and 10.57.3.2)

f_v = unit shear stress (Articles 10.32.3.2.3, 10.34.4.4 and 10.57.3.2)

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 inches (Articles 10.16.14 and 10.24.5)

H = height of stud in inches (Article 10.38.5.1.1)

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

I = moment of inertia, in^4 (Articles 10.34.4, 10.34.5 10.38.5.1.1, 10.48.5.5 and 10.48.6.3)

I_s = the 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)

J = the required ratio of rigidity of one transverse stiffener to that of the web plate (Articles 10.34.4 and 10.48.5.5)

K = effective length factor in plane of buckling (Table 10.32.1A and Articles 10.37, 10.54.1, 10.54.2 and 10.55.1)

k = a constant: 0.75 for rivets; 0.6 for high-strength bolts with thread excluded from shear plane (Article 10.32.3.2.4)

k = buckling coefficient (Articles 10.39.4.3 and 10.51.5.4)

k = a 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 inches (Article 10.54.1.1)

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

M = maximum bending moment (Article 10.48.5.4 and 10.54.2.1)

M_1&M_2 = moments at the ends of a member (Table 10.36A)

M_1&M_2 = moments at two adjacent braced points (Article 10.48.1.)

M_c = column moment (Article 10.56.3.2)

M_p = the full plastic moment of the section (Article 10.54.2.1.)

M_u = maximum bending strength (Articles 10.48, 10.51.1, 10.53.1, and 10.54.2.1)

N_1&N_2 = the 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_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, P_1, P_2, & P_3 = force in the slab (Article 10.38.5.1.2)

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

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 10.32.3.2 and 10.56.2)

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

Q_u = ultimate strength of a shear connector (Article 10.50.1.1.1)

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))

Rev = signifies 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, inches (Table 10.32.1A and 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)

S = the allowable rivet or bolt unit stress in shear (Article 10.32.3.2.4)

S = section modulus (Articles 10.48.2, 10.48.4.1, 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 (Articles 10.39.4.4 and 10.48.6.3)

S_t = section modulus of longitudinal or transverse stiffener (Article 10.48.6.3)

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

s = the computed rivet or bolt unit stress in shear (Article 10.32.3.2.4)

T = range in tensile stress (Table 10.3.1B)

T = direct tension per bolt due to external load (Table 10.32.3B and 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 (Articles 10.24.5 and 10.35.2)

t = thickness of members in compression (Article 10.35.2)

t = thickness of thinnest part connected, in inches (Articles 10.32.3.2.2 and 10.56.2)

t = thickness of longitudinal stiffener (Article 10.39.4.3)

t = the computed rivet or bolt unit stress in tension including any stress due to prying action (Article 10.32.3.2.4)

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

t = flange thickness in inches (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 inches (Article 10.38.5.1.2)

t = thickness of stiffener (Article 10.48.5.5)

t_b = thickness of flange delivering concentrated force (Article 10.56.3)

t_c = thickness of flange of member to be stiffened (Article 10.56.3)

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

t_s = thickness of stiffener (Article 10.37.2 and 10.55.2)

t_s = slab thickness (Article 10.50.1.1.1)

t_w = web thickness in inches (Articles 10.15.2.1, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.40.2.2, 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, 10.48.1.1, 10.48.5.5 and 10.51.3)

V_p = equals the shear yielding strength of the web (Articles 10.48.5.3, 10.48.5.5 and 10.53.1.4)

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

V_u = maximum shear force (Articles 10.48.2.1, 10.48.5.3, 10.48.5.4, 10.48.5.5 and 10.53.1.4)

V_v = vertical shear (Article 10.39.3.1)

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

W = length of a channel shear connector, in inches (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 = a fraction of a wheel load (Article 10.39.2)

w = the 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, in pounds per cubic feet (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 (Article 10.48.5.5)

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

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

Z = the 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 = a 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 inches (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 inches (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)

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 M169 (ASTM A108) Steel Forgings, Carbon and Alloy, for General Industrial Use, AASHTO M 102 (ASTM A 668).

Table 10.2A

MINIMUM MATERIAL PROPERTIES STRUCTURAL STEEL					
Type	Structural Steel	High Strength Low-Alloy Steel		High Yield Strength, Quenched and Tempered Alloy Steel	
AASHTO Designation[a]	M 183	M 223	M 222	M 244	
Equivalent ASTM Designation	A 36	A 572 Grade 50	A 588	A 514[b]	A 517[b,c]
Thickness of Plates	Up to 8" incl.[e]	Up to 2" incl.	Up to 4" incl.	Up to 2 1/2"incl.	Over 2 1/2" to 4" incl.
Shapes[d]	All Groups[e]	Shapes thru 426 lb/ft.	All Groups	Not Applicable	Not Applicable
Minimum Tensile Strength F_u	58,000	65,000	70,000	110,000	100,000
Minimum Yield Point or Minimum Yield Strength F_y	36,000	50,000	50,000	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 ASTM A 514/A 517, 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 ASTM A 514/A 517 steel.

[c]Materials ordered to ASTM A 517 specifications shall comply with toughness requirements of AASHTO M 244.

[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 x 230 to 300 incl.
W33 x 200 to 240 incl.
W14 x 142 to 211 incl.
W12 x 120 to 190 incl.

Group 4 includes the following W shapes: W14 x 219 to 550 incl.
Group 5 includes the following W shapes: W14 x 605 to 730 incl.
For breakdown of Groups 1 and 2 see ASTM A 6.

[e]Limited to 4" thickness for structural members other than bearing assembly components.

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 dia. or less	M 102 To 20" in dia.	M 102 To 20" in dia.	M 102 To 10" in dia.	M 102 To 20" in dia.
ASTM Designation Grade or Class	A 108 Grades 1016 to 1030 inc.	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.

10.2.4 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 AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges.

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 296) and for Ductile Iron Castings, ASTM A 536.

10.2.6.2 MALLEABLE CASTINGS

Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47. Physical properties are shown in Table 10.2.6.2A.

Table 10.2.6.2A

AASHTO Designation	M 103	M 192	M 192		M 163	None
ASTM Designation	A 27	A 486	A 486		A 296	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

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, 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 Articles 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 shown in Table 10.3.1B and illustrated in Figure 10.3.1C.

Main load carrying components subjected to tensile stresses which may be considered non-redundant 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 Non-redundant Load Path Structures. Examples of non-redundant 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.

Table 10.3.1A

REDUNDANT LOAD PATH STRUCTURES[1]

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	60	36	24	24
B	45	27.5	18	16
C	32	19	13	10 12[b]
D	27	16	10	7
E	21	12.5	8	5
E'	16	9.4	5.8	2.6
F	15	12	9	8

NON-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	36	24	24	24
B	27.5	18	16	16
C	19	13	10 12[b]	9 11[b]
D	16	10	7	5
E[c]	12.5	8	5	2.5
F	12	9	8	7

[1]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 non-redundant load path structures.

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 Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members without attachments, built-up of plates, or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	T or Rev.	B	3,4,5,7
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges	T or Rev.	C	6
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends			
	(a) Flange thickness ≤ 0.8 in.	T or Rev.	E	7
	(b) Flange thickness > 0.8 in.	T or Rev.	E'	7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by nondestructive inspection.	T or Rev.	B	8,10,14
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2 1/2, with grinding in the direction of applied stress, and weld soundness established by nondestructive inspection.	T or Rev.	B	11,12

[a]"T" signifies range in tensile stress only: "Rev." signifies a range of stress involving both tension and compression during a stress cycle.

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 and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2 1/2 when reinforcement is not removed and weld soundness is established by nondestructive inspection	T or Rev.	C	8,10,11, 12,14
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev.	D	13
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, is greater than 12 times the plate thickness or greater than 4 inches long.	T or Rev.	E	13
	Base metal at details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length when weld soundness transverse to the direction of stress is established by nondestructive inspection.			
	(a) When provided with transition radius equal to or greater than 24 in. and weld end ground smooth	T or Rev.	B	14
	(b) When provided with transition radius less than 24 in. but not less than 6 in. and weld end ground smooth	T or Rev.	C	14

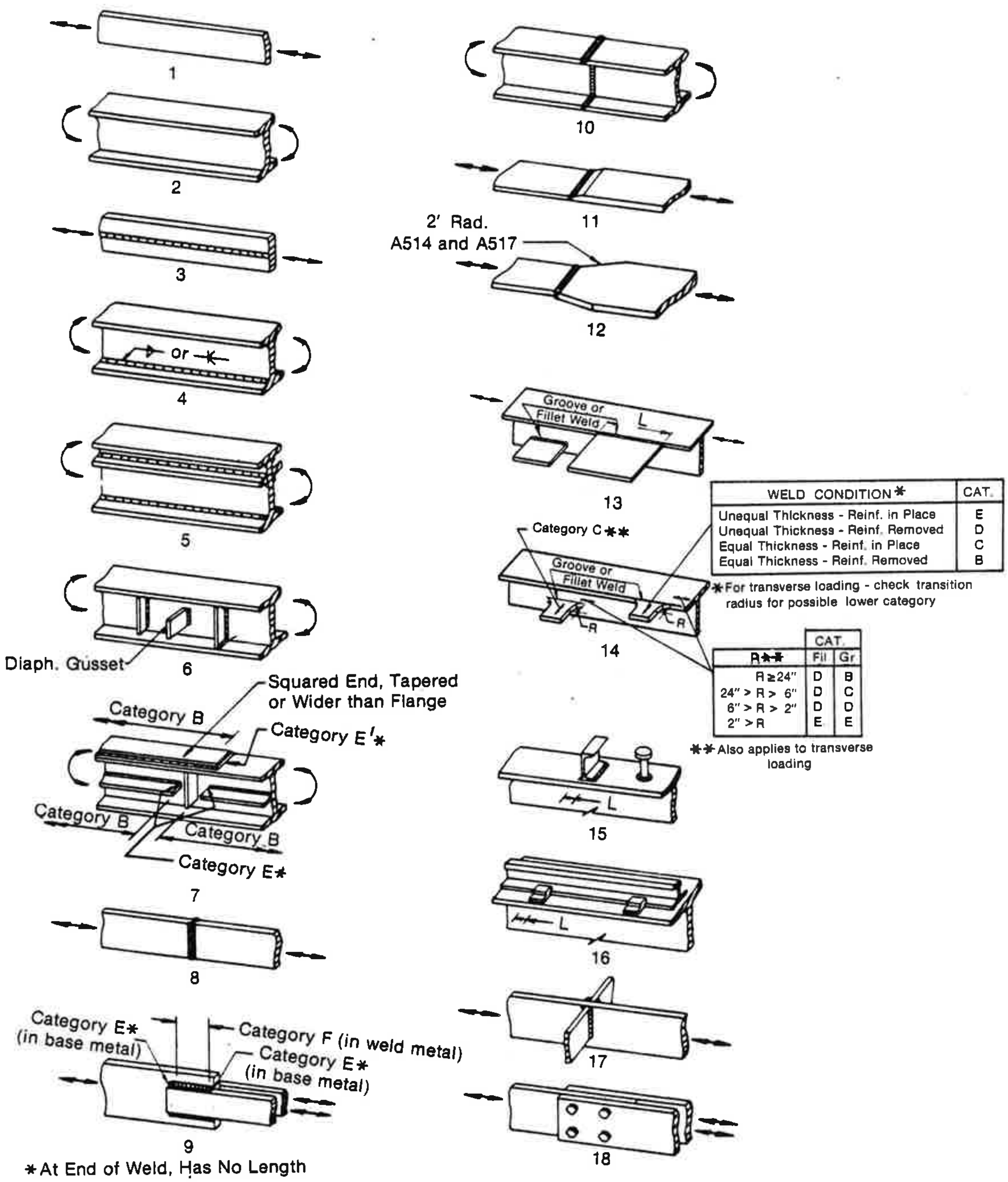
Table 10.3.1B
(continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	(c) When provided with transition radius less than 6 in. but not less than 2 in. and weld end ground smooth	T or Rev.	D	14
	(d) When provided with transition radius between 0 in. and 2 in.	T or Rev	E	14
Fillet[b] Welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	---
	Base metal adjacent to fillet welded attachments with length L, in direction of stress less than 2 in. and stud-type shear connectors	T or Rev.	C	13,15,16,17
	Base metal at details attached by fillet welds with detail length, L, in direction of stress between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev.	D	13,15,16
	Base metal at attachment-details with detail length, L, in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in.	T or Rev.	E	7,9,13,16
	Base metal at details attached by fillet welds regardless of length in direction of stress (shear stress on the throat of fillet welds governed by stress category F)			
	(a) When provided with transition radius equal to or greater than 2 in. and weld end ground smooth	T or Rev.	D	14

[b]Gusset plates attached to girder flanges with only transverse fillet welds, not recommended.

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)
	(b) When provided with transition radius between 0 in. and 2 in.	T or Rev.	E	14
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 connected material.	T or Rev.	B	18
	Base metal at net section of high-strength bolted bearing-type connections	T or Rev.	B	18
	Base metal at net section of riveted connections	T or Rev.	D	18
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	9



WELD CONDITION*	CAT.
Unequal Thickness - Reinf. in Place	E
Unequal Thickness - Reinf. Removed	D
Equal Thickness - Reinf. in Place	C
Equal Thickness - Reinf. Removed	B

R**	CAT. Fil	CAT. Gr
R ≥ 24"	D	B
24" > R > 6"	D	C
6" > R > 2"	D	D
2" > R	E	E

Figure 10.3.1C
Illustrative Examples

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.

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.

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.

10.3.3 CHARPY V-NOTCH IMPACT REQUIREMENTS

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

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

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

10.3.3.4 A 514 steel shall be supplied to Zone 2 requirements as a minimum.

Table 10.3.3A

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

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

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

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

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

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*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.

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

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

10.6 DEFLECTION

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

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

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

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

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

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

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

10.7.2 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 betweeen 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 in railings, shall be not less than 5/16" in thickness. The web thickness of rolled beams or channels shall not be less than 0.23". The thickness of closed ribs in orthotropic decks shall not be less than 3/16".

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" minimum.

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

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

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

10.9 EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION

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

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

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

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 and 10.50.1.1.

10.13 COVER PLATES

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

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

10.13.3 The maximum thickness of a single cover plate on a flange shall not be greater than 2 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 2 1/2 times the flange thickness.

10.13.4 Any partial length welded cover plate shall extend beyond the theoretical end by the terminal distance, or 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 is the section at which the stress in the flange without that cover plate equals the allowable service load stress exclusive of fatigue considerations. The terminal distance is 2 times the nominal cover plate width for cover plates not welded across their ends, and 1 1/2 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.

10.14 CAMBER

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

10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

10.15.1 SCOPE

This section pertains to rolled beams and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified minimum yield point greater than 50,000 psi 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 centerline 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:

$$R = \frac{14bD}{\sqrt{F_y}\,\psi t_w} \qquad (10\text{-}1)$$

$$R = \frac{7500b}{F_y \psi} \qquad (10\text{-}2)$$

In these equations, F_y is the specified minimum yield point in ksi 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:

$$\Delta = (\Delta_{DL}/\Delta_m)\,[\Delta_m + (0.02L^2F_y/EY_o)] \qquad (10\text{-}3)$$

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), and L is the span length for simple spans or the distance between a simple end support and the dead load contraflexure point; or the distance between points of dead load contraflexure for continuous spans. L is measured in inches.

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 or side segments of 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 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 1 1/2 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 1/4 times the distance between points of support and the length of intermediate stay plates not less than 3/4 of that distance. In lateral struts and other secondary members, the over-all length of end and intermediate stay plates shall be not less than 3/4 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 3/4 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 2/3 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
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 1 1/2 times the maximum width of the member. Beyond this point, the pitch shall be increased gradually for a length equal to 1 1/2 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.

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

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 which gives the least net width.

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

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

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

10.16.14.6 The diameter of the hole shall be taken as 1/8 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 $0.50F_u$, when using service load design method or $1.0F_u$, when using strength design method, where F_u equals the minimum tensile strength of the steel, except that for A514 and A517 steels the design tensile stress on the net section shall not exceed $0.46F_u$ 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 10.18.6.

10.18.1.3 Fillers in high-strength bolted friction type 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 A514 and A517 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 1/4 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 stress 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 stress and shall be long enough to avoid overstressing the filler along the toe of the weld. Any filler less than 1/4 inch thick shall have its edges made flush with the edges of the splice plate. The weld size necessary to carry the splice plate stress 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, except that for A 514 and A 517 steels only, the transition illustrated in

Figure 10.18.5A(a) is permitted. At butt weld splices joining material of different thicknesses there shall be a uniform slope between the offset surfaces of not more than 1 in 2 1/2 with respect to the surface of either part.

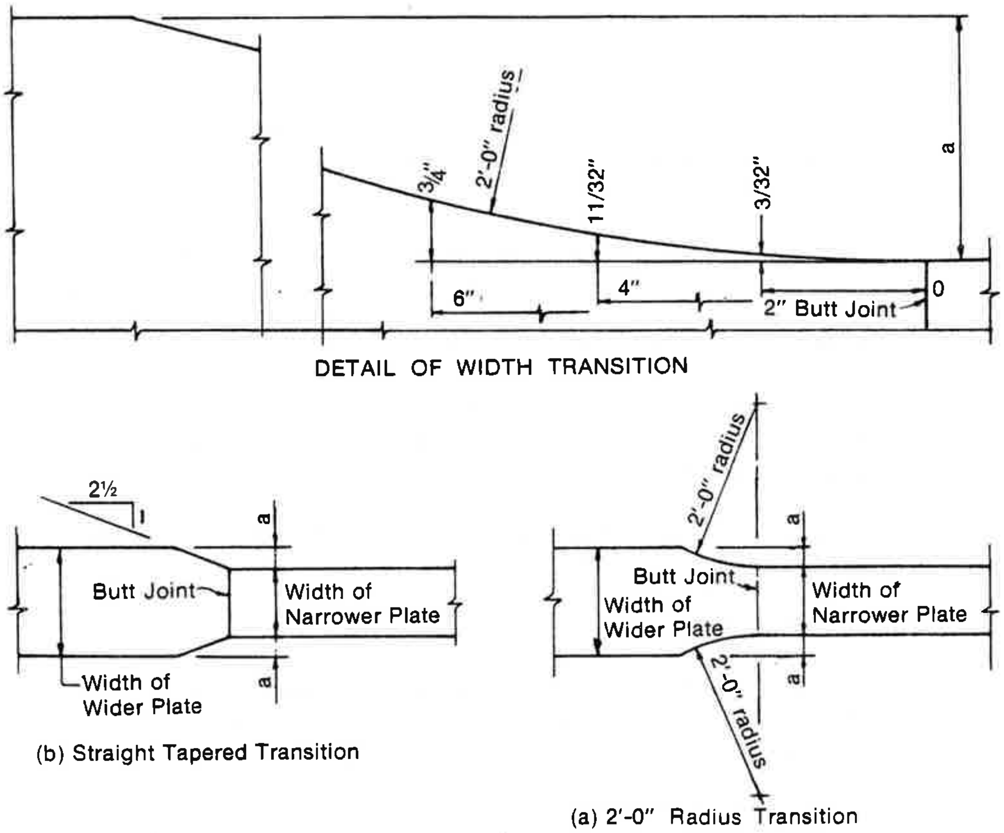


Figure 10.18.5A - Splice Details

10.18.6 FILLERS

10.18.6.1 If fasteners carrying stress pass through fillers, the fillers preferably shall be extended beyond the gusset or splice material, and the extension secured by enough additional fasteners to carry the stress in the filler, which stress is to be calculated as the total stress in the member divided by the combined area of the member plus the fillers. As an alternate, the additional fasteners may be passed through the gusset or splice material without extending the filler.

10.18.6.2 If the filler is less than 1/4 inch thick it shall not be extended beyond the splicing material and additional fasteners are not required. Fillers 1/4 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 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 in case of load factor design method 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.

10.19.2 END CONNECTIONS OF FLOOR BEAMS AND STRINGERS

10.19.2.1 The end connection shall be designed for the loads specified. The end connection angles 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.

10.20 DIAPHRAGMS AND CROSS FRAMES

10.20.1 GENERAL

Rolled beam and plate girder spans shall be provided with cross frames or diaphragms at each support and with intermediate cross frames or diaphragms placed in all bays and spaced at intervals not to exceed 25 feet. Diaphragms for rolled beams shall be at least 1/3 and preferably 1/2 the beam depth and for plate girders shall be at least 1/2 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: $R = [0.2272L - 11]\ S_d^{-2/3}$ when no bottom lateral bracing is provided (10-6)

$R = [0.059L - 0.64]\ S_d^{-1/2}$ when bottom lateral bracing is provided (10-7)

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

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

W = Wind loading along the exterior flange (lbs./foot)

S_d = Diaphragm Spacing (feet)

L = Span Length (feet)

t_f = Thickness of Flange (inches)

b_f = Width of Flange (inches)

10.20.2.2 Diaphragms & Cross Frames. The maximum horizontal force (F_D) in the transverse diaphragms and cross frames is obtained from the following:

With or Without Bracing

$$F_D = 1.14WS_d \tag{10-10}$$

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 2 1/2 inches. There shall be not less than 2 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 which 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, 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 metal to be welded, weld metal, and welding design details shall conform to the requirements of the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges, 1981, and subsequent AASHTO Interim Specifications.

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 10.24 - Division 2.

10.23.2 MINIMUM SIZE OF FILLET WELDS

10.23.2.1 The minimum fillet weld size shall be as shown in the following table. Weld size is determined by the thicker of the two parts joined unless a larger size is requiured by calculated stress. The weld size need not exceed the thickness of the thinner part joined.

Material Thickness of Thicker Part Joined (inches)	Minimum Size of Fillet Weld (inches)
To 1/2 inclusive	3/16
Over 1/2 to 3/4	1/4
Over 3/4 to 1 1/2	5/16
Over 1 1/2 to 2 1/4	3/8
Over 2 1/4 to 6	1/2
Over 6	5/8

10.23.2.2 The minimum size seal weld shall be 3/16 inch fillet weld.

10.23.3 MAXIMUM EFFECTIVE 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 1/4 inch thick the maximum size may be equal to the thickness of the material.

(2) Along edges of material 1/4 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.

10.23.4 MINIMUM EFFECTIVE LENGTH OF FILLET WELDS

The minimum effective length of a fillet weld shall be four times its size and in no case less than 1 1/2 inches.

10.23.5 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.

10.23.6 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, the nominal diameter shall be used, except as otherwise noted.

10.24.1.2 High-strength bolts may be substituted for Grade 1 rivets, AASHTO M 228 (ASTM A-502).

10.24.1.3 All bolts except high-strength bolts, 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 friction-type or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be friction-type.

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

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 multipled by the thickness of the metal on which it bears. In metal less than 3/8 inch thick, countersunk rivets, turned bolts or ribbed bolts shall not be assumed to carry stress. In metal 3/8 inch thick and over, one-half the depth of countersink shall be omitted in calculating the bearing area.

10.24.1.8 Bolts in bearing-type connections shall have the threads excluded from the shear planes of the contact surfaces between the connected parts. 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 lengths greater than the specified thread length as an allowance for thread run out.

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.

10.24.2 HOLES

Where shown in the design drawings and at other locations approved by the designer, oversize, short-slotted, and long-slotted holes may be used with high-strength bolts 5/8 inch and larger in diameter proportioned to meet the allowable unit stresses given in Articles 10.32.3 and 10.56.1.1 except as hereinafter restricted:

10.24.2.1 Oversize holes are 3/16 inch larger than bolts 7/8 inch and less in diameter, 1/4 inch larger than bolts 1 inch in diameter and 5/16 inch larger than bolts 1 1/8 inch and greater in diameter. They may be used in any or all plies of friction-type connections. Hardened washers shall be installed over exposed oversize holes.

10.24.2.2 Short-slotted holes are 1/16 inch wider than the bolt diameter and have a length which does not exceed the oversize diameter provisions of Article 10.24.2.1 by more than 1/16 inch. They may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type connections but shall be normal to the direction of the load in bearing-type connections. Hardened washers shall be installed over exposed short-slotted holes.

10.24.2.3 Long-slotted holes are 1/16 inch wider than the bolt diameter and have a length more than allowed in Article 10.24.2.2 but not more than 2 1/2 times the bolt diameter. In friction-type connections, they may be used without regard to direction of loading. In bearing-type connections, the long diameter of the slot shall be normal to the direction of loading. Long-slotted holes may be used in only one of the connected parts of either a friction-type or bearing-type connection at an individual faying surface. Structural plate washers or a continuous bar not less than 5/16 inch in thickness are required to cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to completely cover the slot after installation. If hardened washers are required they shall be placed over the plate or bar.

10.24.2.4 When enlarged or slotted holes are used, the distances between edges of adjacent holes or edges of holes and edges of members shall not be less than permitted with conventional size holes under Articles 10.24.4 and 10.24.6.

10.24.3 SIZE OF FASTENERS (Rivets or High-Strength Bolts)

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

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

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

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

10.24.4 SPACING OF FASTENERS

10.24.4.1 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. The pitch of fasteners shall be governed by the requirements for sealing or stitch whichever is the minimum.

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

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

10.24.5 MAXIMUM PITCH OF SEALING AND STITCH FASTENERS

10.24.5.1 SEALING FASTENERS

For sealing, the pitch on a single line adjacent to a free edge of an outside plate or shape shall not exceed 4 inches + 4t or 7 inches. If there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge, at a gage "g" less than 1 1/2 inches + 4t therefrom, the staggered pitch 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, "g"=gage between fasteners in inches.

10.24.5.2 STITCH FASTENERS

In built-up members where two or more plates or shapes are in contact, stitch fasteners shall be used to insure uniform action 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.

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

10.24.6 EDGE DISTANCE OF FASTENERS

10.24.6.1 GENERAL

The minimum distance from the center of any fastener to a sheared or flame cut edge shall be:

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

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

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

In the flanges of beams and channels the distance shall be:

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

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

10.24.6.2 SPECIAL CONDITIONS

In bearing-type connections having no more than two bolts in a line parallel to the direction of stress, the distance between the center of the nearest bolt and that end of a connected member towards which the pressure from the bolt is directed shall be not less than $2.2df_p/F_u$ or 1.5d, whichever is the larger value.

where d = bolt diameter
f_p = computed bearing stress due to design load
F_u = specified minimum tensile strength of the connected part

10.24.7 LONG RIVETS

Rivets subjected to calculated stress and having a grip in excess of 4 1/2 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 held against lateral movement on the pins.

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 1/8 of the width, nor less than 1/2 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 centerline 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 1/2 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 cleaned easily.

10.29.4 SOLE PLATES AND MASONRY PLATES

10.29.4.1 Sole plates and masonry plates shall have a minimum thickness of 3/4 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, 1 1/4 inches in diameter, set 12 inches in the masonry.

Spans 101 to 150 feet, 2 bolts, 1 1/2 inches in diameter, set 15 inches in the masonry.

Spans greater than 150 feet; 4 bolts, 1 1/2 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 5/8 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.

PART C - SERVICE LOAD DESIGN METHOD

ALLOWABLE STRESS DESIGN

10.31 SCOPE

Allowable stress design is the standard design method for all structure types. It is a method for proportioning structural members using design loads and forces, allowable stresses, and design limitations for the appropriate material under service conditions.

10.32 ALLOWABLE STRESSES

10.32.1 STEEL

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

10.32.2 WELD METAL

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

Butt Welds

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

Fillet Welds

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

where

F_v = allowable basic shear stress
F_u = tensile strength of the electrode classification but not greater than the tensile strength of the connected part.

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

Plug Welds

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

Table 10.32.1A

ALLOWABLE STRESSES
STRUCTURAL STEEL
(in pounds per square inch)

Type		Structural Carbon Steel	High-Strength Low-Alloy Steel		High Yield Strength Quenched and Tempered Alloy Steel	
AASHTO Designation[e]		M 183	M 223	M 222	M 244	
Equivalent ASTM Designation		A 36	A 572 Grade 50	A 588	A 514[f]	A 517[f,h]
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1 1/4" diam. such as perforations.	$0.55F_y$	20,000	27,000	27,000	Not Applicable	
	$0.46F_u$	Not Applicable			51,00	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[j] Section $0.55F_y$	20,000	27,000	27,000	Not Applicable	
	Net Section $0.50F_u$	29,000	32,500	35,000	Not Applicable	
	Net Section $0.46F_u$	Not Applicable			51,000	46,000

Table 10.32.1A

ALLOWABLE STRESSES
STRUCTURAL STEEL
(in pounds per square inch)
(continued)

Type	Structural Carbon Steel	High-Strength Low-Alloy Steel		High Yield Strength Quenched and Tempered Alloy Steel	
Axial tension in memberswithout holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section $0.55F_y$	20,000	27,000	27,000	55,000	49,000
Compression in extreme fibers of rolled shapes girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete $0.55F_y$	20,000	27,000	27,000	55,000	49,000

Table 10.32.1A

ALLOWABLE STRESSES
STRUCTURAL STEEL
(in pounds per square inch)
(continued)

Type	Structural Carbon Steel	High-Strength Low-Alloy Steel		High Yield Strength Quenched and Tempered Alloy Steel	
(B) Partially supported or is unsupported with $\frac{\ell}{b}$ not greater than[a,b]	36	30	30	22	23
$F_b = 0.55F_y\left[1-\frac{\left(\frac{\ell}{r'}\right)^2 F_y}{4\pi^2 E}\right] =$	$20{,}000-7.5\frac{\ell^2}{b}$	$27{,}000-14.4\frac{\ell^2}{b}$	$27{,}000-14.4\frac{\ell^2}{b}$	$55{,}000-58\frac{\ell^2}{b}$	$49{,}000-47\frac{\ell^2}{b}$
with $(r')^2 = \frac{b^2}{12}$					

Table 10.32.1A

ALLOWABLE STRESSES
STRUCTURAL STEEL
(in pounds per square inch)
(continued)

Type	Structural Carbon Steel	High-Strength Low-Alloy Steel		High Yield Strength Quenched and Tempered Alloy Steel	
Compression in concentrically loaded columns[c]					
with $C_c=(2\pi^2E/F_y)^{1/2}$ =	126.1	107.0	107.0	75.7	79.8
when $KL/r \leq C_c$					
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2F_y}{4\pi^2E}\right]$ =	$16{,}980-.53(KL/r)^2$	$23{,}580-1.03(KL/r)^2$	$23{,}580-1.03(KL/r)^2$	$47{,}170-4.12(KL/r)^2$	$42{,}450-3.33(KL/r)^2$
when $KL/r > C_c$					
$F_a = \frac{\pi^2E}{F.S.(KL/r)^2}$ =	$\frac{135{,}000{,}740}{(KL/r)^2}$				
with F.S.=2.12					
Shear in girder webs, gross section $F_v=0.33F_y$	12,000	17,000	17,000	33,000	30,000

Table 10.32.1A

ALLOWABLE STRESSES
STRUCTURAL STEEL
(in pounds per square inch)
(continued)

Type		Structural Carbon Steel	High-Strength Low-Alloy Steel		High Yield Strength Quenched and Tempered Alloy Steel	
Bearing on milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80F_y$	29,000	40,000	40,000	80,000	72,000
Stress in extreme fiber or pins[d]	$0.80F_y$	29,000	40,000	40,000	80,000	72,000
Shear in pins	$F_v=0.40F_y$	14,000	20,000	20,000	40,000	36,000
Bearing on pins not subject to rotation[g]	$0.80F_y$	29,000	40,000	40,000	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	40,000	36,000
Bearing on power-driven rivets and high-strength bolts (or as limited by the allowable bearing on the fasteners)	$1.35F_u$	78,500	88,000	94,500	148,000	135,000

FOOTNOTES FOR TABLE 10.32.1A

[a]Continuous or cantilever beams or girders may be proportioned for negative moment at interior supports for an allowable unit stress 20 percent higher than permitted by this formula but in no case exceeding allowable unit stress for compression flange supported its full length. 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. For continuous beams and girders, ℓ may be taken as the distance from interior support to point of dead load contraflexure if this distance is less than designated above.

For cantilever beam and girders, ℓ may be taken as twice the distance from the support to the end of the cantilever, if this distance is less than designated above.

r' = radius of gyration, in inches, of the compression flange about the axis in the plane of the web.

b = flange width, in inches.

[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 ASTM A 514/A 517, 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 ASTM A 514/A 517 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.

FOOTNOTES FOR TABLE 10.32.1A

[h]Materials ordered to ASTM A 517 specifications shall comply with toughness requirements of AASHTO M 244.

[j]When the area of holes deducted for high strength bolts or rivets is more than 15% of the gross area, that area in excess of 15% shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1 1/4" diam. such as perforations shall be deducted.

10.32.3 FASTENERS (Rivets and Bolts)

Allowable stresses for fasteners shall be as listed in Tables 10.32.3A, 10.32.3.B, and 10.32.3C.

Table 10.32.3A

ALLOWABLE STRESSES FOR LOW CARBON STEEL BOLTS
AND POWER DRIVEN RIVETS in psi

Type of Fastener	Tension	Bearing	Shear Bearing-Type Connection
(A) Low Carbon Steel bolts[a] Turned Bolts (ASTM A 307) and Ribbed bolts	13,500[b]	20,000	11,000
(B) Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven). Structural Steel Rivet AASHTO M 228, Grade 1 (ASTM A 502 Grade 1)	------	40,000	13,500
Structural Steel Rivet (High-Strength) AASHTO M 228, Grade 2 (ASTM A 502 Grade 2)	------	40,000	20,000

[a]ASTM A307 bolts shall not be used in connections subject to fatigue.
[b]Based on area at the root of thread.

10.32.3.1 GENERAL

10.32.3.1.1 In proportioning fasteners the nominal diameter shall be used except as otherwise noted.

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

Table 10.32.3B

ALLOWABLE STRESSES FOR HIGH-STRENGTH BOLTS[a]
(in ksi)

Load Condition	Hole Type	AASHTO M 164 (ASTM A 325)[f] Bolts	AASHTO M 253 (ASTM A 490) Bolts
Applied Tension[b] (T)	Standard, oversize, or slotted	39.5	48.5
Shear (F_v):[c] Friction-Type Connection:[c]	Standard Oversize Short Slotted Long Slotted	16.0 13.5 13.5 11.5	20.0 17.0 17.0 14.5
Shear (F_v):[d] Bearing-Type Connection:			
Threads in Any Shear Plane	Standard or Slotted	19.0	25.0
No Threads in Shear Plane	Standard or Slotted	27.0	36.0
Bearing[e] (f_p)	Standard or slotted	$\frac{LF_u}{2.2d}$ or $1.35F_u$ (whichever is smaller)	

[a]The tabulated stresses, except for bearing stress, apply to the nominal area of bolts used in any grade of steel.

[b]For allowable working stresses when bolts are subjected to fatigue loading in tension, see Article 10.32.3.3.

[c]Applicable for contact surfaces with clean mill scale. When the designer has specified special treatment of the contact surfaces in a friction-type connections, values in Table 10.32.3C may be substituted.

[d]In bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of an axial force exceeds 50 inches tabulated values shall be reduced by 20 percent.

[e]L is the distance in inches measured in the line of force from the center line of a bolt to the nearest edge of an adjacent bolt or to the end of the connected part toward which the force is directed; d is the diameter of the bolts; and F_u is the lowest specified minimum tensile strength of the connected parts.

[f]AASHTO M 164 (ASTM A 325) 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 222 (ASTM A 588) steel.

Table 10.32.3C

ALLOWABLE SHEAR STRESSES[a] FOR
HIGH STRENGTH BOLTS USED FOR FRICTION-TYPE CONNECTORS,
BASED UPON SURFACE CONDITION OF BOLTED PARTS, in ksi

Class of Surface[b]	Surface Condition of Bolted Parts	Standard Holes		Oversize Holes and Short-Slotted Holes		Long-Slotted Holes	
		M 164 (A 325)	M 253 (A 490)	M 164 (A 325)	M 253 (A 490)	M 164 (A 325)	M 253 (A 490)
A	Clean mill scale	16.0	20.0	13.5	17.0	11.5	14.5
B	Blast-cleaned carbon and low alloy steel	25.0	31.0	21.0	26.5	17.5	21.5
C	Blast-cleaned quenched and tempered steel	17.0	21.0	14.5	18.0	12.0	15.0
D	Hot-dip galvanized and roughened	19.5	24.5	16.5	20.5	13.5	17.0
E	Blast-cleaned, organic zinc rich paint	19.0	23.5	16.0	20.0	13.0	16.0
F	Blast-cleaned, inorganic zinc rich paint	26.5	33.5	22.5	28.5	18.5	23.5
G	Blast-cleaned, metallized with zinc	26.5	33.5	22.5	28.5	18.5	23.5
H	Blast-cleaned, metallized with aluminum	27.0	34.0	23.0	29.0	19.0	24.0
I	Vinyl Wash	15.0	18.5	12.5	16.0	10.5	13.0

[a]Values from this table are applicable only when they do not exceed the lowest appropriate allowable working stresses for bearing-type connections, taking into account the position of threads relative to shear planes and, if required, the 20 percent reduction due to joint length. (See Table 10.32.3B, Footnote d).

[b]See Article 10.17.3 - Division II for further definition of Class of Surface.

10.32.3.1.3 Bolts in bearing-type connections shall have the threads excluded from the shear planes of the contact surfaces between the connected parts. 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 lengths greater than the specified thread length as an allowance for thread run out.

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 Article 10.24.6.

10.32.3.1.5 All bolts except high-strength bolts, 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 friction-type or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be friction-type.

10.32.3.1.7 The percentage of unit stress increase shown in Article 3.2, Loading Combinations, shall not apply to allowable stresses in bolted friction-type connections using high-strength bolts.

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

10.32.3.2 APPLIED TENSION, COMBINED TENSION AND SHEAR

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

10.32.3.2.2 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.

10.32.3.2.3 For combined shear and tension in friction-type joints using high-strength bolts where applied forces reduce the total clamping force on the friction plane, the allowable unit shearing stress, f_v, shall not exceed the values obtained from the following equations:

For AASHTO M 164 (ASTM A 325) bolts:

$$f_v = F_v\ (1-1.59 \times 10^{-5} f_t) \tag{10-14}$$

For AASHTO M 253 (ASTM A 490) bolts:

$$f_v = F_v\ (1-1.27 \times 10^{-5} f_t) \tag{10-15}$$

where

f_t = tensile stresses due to applied loads including any stress due to prying action, in pounds per square inch.

F_v = allowable shear stress from Table 10.32.3B or Table 10.32.3C.

10.32.3.2.4 Where rivets or high-strength bolts in bearing type connections are subject to both shear and tension, the combined stress shall not exceed values obtained from the following equation:

$$s^2 + (kt)^2 = S^2 \tag{10-16}$$

where

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

10.32.3.3 FATIGUE

10.32.3.3.1 When subjected to tensile fatigue loading, high-strength bolts may be designed for the combined external and prying forces using the allowable tensile stress given in Table 10.32.3B within the following limits:

Connections subject to not more than 20,000 cycles of such combined tensile stress.

Connections subject to more than 20,000 cycles but not more than 500,000 cycles of direct tension, where the prying load on the bolts does not exceed 10 percent of the externally applied load.

Connections subject to more than 500,000 cycles of direct tension, where the prying load of the bolts does not exceed 5 percent of the externally applied load.

10.32.3.3.2 If the prying load exceeds the limits specified above, the bolts shall be designed, on the basis of the external force alone, using a reduced allowable stress as prescribed below:

For connections subject to more than 20,000 cycles but not more than 500,000 cycles, the reduced stress shall be 60 percent of the value given in Table 10.32.3B for applied tension.

For connections subject to more than 500,000 cycles, the reduced stress shall be 50 percent of the value given in Table 10.32.3B.

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

$$p = \frac{F_y - 13{,}000}{20{,}000}\ 600d \qquad (10\text{-}17)$$

Diameters from 25 to 125 inches

$$p = \frac{F_y - 13{,}000}{20{,}000}\ 3{,}000\sqrt{d} \qquad (10\text{-}18)$$

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) Steel Forgings, Carbon and Alloy, for General Industrial Use, AASHTO M 102 (ASTM A 668) are shown in Table 10.32.4.3A.

Table 10.32.4.3A

AASHTO Designation with Size Limitations	------	M 169 4" in dia. or less	M 102 To 20" in dia.	M 102 To 20" in dia.	M 102 To 10" in dia.	M 102 To 20" 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 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.

[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 296) 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. The allowable stresses in pounds per square inch shall be in accordance with Table 10.32.5.1A.

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

10.32.5.3 CAST IRON

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

Bending in Extreme Fiber	3,000
Shear	3,000
Direct Compression, short columns	12,000

10.32.5.4 BRONZE OR COPPER-ALLOY

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

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

Table 10.32.5.1A

AASHTO Designation	M 103	M 192	M 192		M 163	None
ASTM Designation	A 27	A 486	A 486		A 296	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

10.32.5.5 BEARING ON MASONRY

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

Granite	800
Sandstone and Limestone	400

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

10.32.5.5.3 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.

10.34 PLATE GIRDERS

10.34.1 GENERAL

10.34.1.1 Girders shall be proportioned by the moment of inertia method. In calculating the net moment of inertia of riveted plate girders, the gravity axis of the gross section shall be used and the moment of inertia of all holes on each side of the axis shall be deducted. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section.

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 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:

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

10.34.2.1.4 Where the calculated compressive bending stress equals .55 F_y the $\left(\frac{b}{t}\right)$ 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
90,000 psi, Y.P. Min. b/t=15
100,000 psi, Y.P. Min. b/t=14

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

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

$$\frac{b}{t} = \frac{3{,}860}{\sqrt{f_{d\ell 1}}} \qquad (10\text{-}20)$$

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

10.34.2.2 RIVETED OR BOLTED GIRDERS

10.34.2.2.1 Flange angles shall form as large a part of the area of the flange as practicable. Side plates shall not be used except where flange angles exceeding 7/8 inch in thickness otherwise would be required.

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

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

10.34.2.2.3 Where the calculated compressive bending stress equals .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
90,000 psi, Y.P. Min. b'/t= 7.5
100,000 psi, Y.P. Min. b'/t= 7

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

$$\frac{b'}{t} = \frac{1{,}930}{\sqrt{f_{d\ell 1}}} \qquad (10\text{-}22)$$

In the above b' is the width of a flange angle, t is the thickness, f_b is the calculated maximum compressive stress, and $f_{d\ell 1}$ is the top flange compressive stress due to noncomposite dead load.

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

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

10.34.2.2.7 At least one cover plate of the top flange shall extend the full length of the girder except when the flange is covered with concrete. Any cover plate which is not full length shall extend beyond the theoretical cut off 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 cut off point of the cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations.

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

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

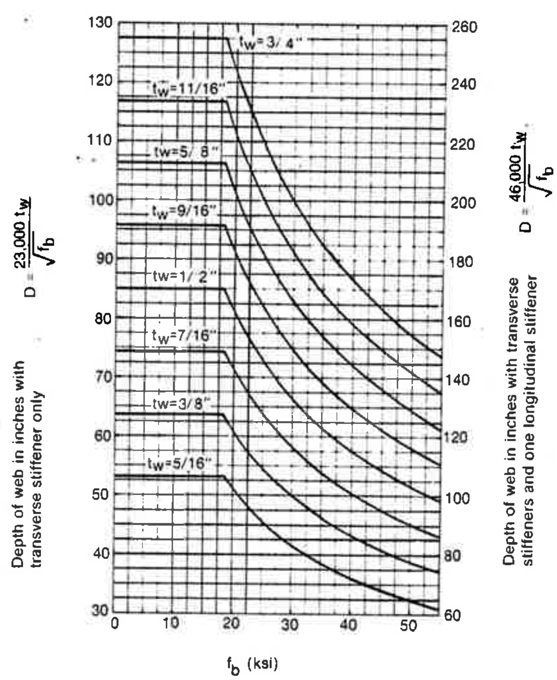
10.34.3 THICKNESS OF WEB PLATES

10.34.3.1 GIRDERS NOT STIFFENED LONGITUDINALLY

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

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

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



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

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 upon the requirements for transverse stiffeners), shall not be less than (where the Y.P. is for the flange material):

36,000 psi, Y.P. Min. D/165
50,000 psi, Y.P. Min. D/140
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:

$$t_w = \frac{D\sqrt{f_b}}{46,000} \quad \text{(See Figure 10.34.3.1A)} \qquad (10\text{-}24)$$

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):

36,000 psi, Y.P. Min. D/330
50,000 psi, Y.P. Min. D/280
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.

10.34.4 TRANSVERSE INTERMEDIATE STIFFENERS

10.34.4.1 Transverse intermediate stiffeners may be omitted if the web thickness is not less than D/150 and 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.

$$F_v = \frac{5.625 \times 10^7}{(D/t_w)^2} \leq \frac{F_y}{3} \qquad (10\text{-}25)$$

where

D = The unsupported depth of web plate between flanges in inches.
t_w = The 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 limited to 1.5D.

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

(See Figure 10.34.4A and 10.34.4B.)

where

$$C = \frac{2.2 \times 10^8 \left[1 + (D/d_o)^2\right]}{F_y(D/t_w)^2} \leq 1 \qquad (10\text{-}27)$$

d_o = spacing of intermediate stiffener

$\left(\frac{F_y}{3}\right)$ in equation (10-26) can be replaced by the allowable shearing stress listed in Table 10.32.1A.

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 D/2.

$$F_v = \frac{7 \times 10^7 \left[1 + (D/d_o)^2\right]}{(D/t_w)^2} \leq F_y/3 \qquad (10\text{-}28)$$

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

$$F_s = (.754 - .34 f_v/F_v)F_y \qquad (10\text{-}29)$$

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 the previous equation.

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/68
50,000 psi, Y.P. Min. D/58
90,000 psi, Y.P. Min. D/43
100,000 psi, Y.P. Min. D/41

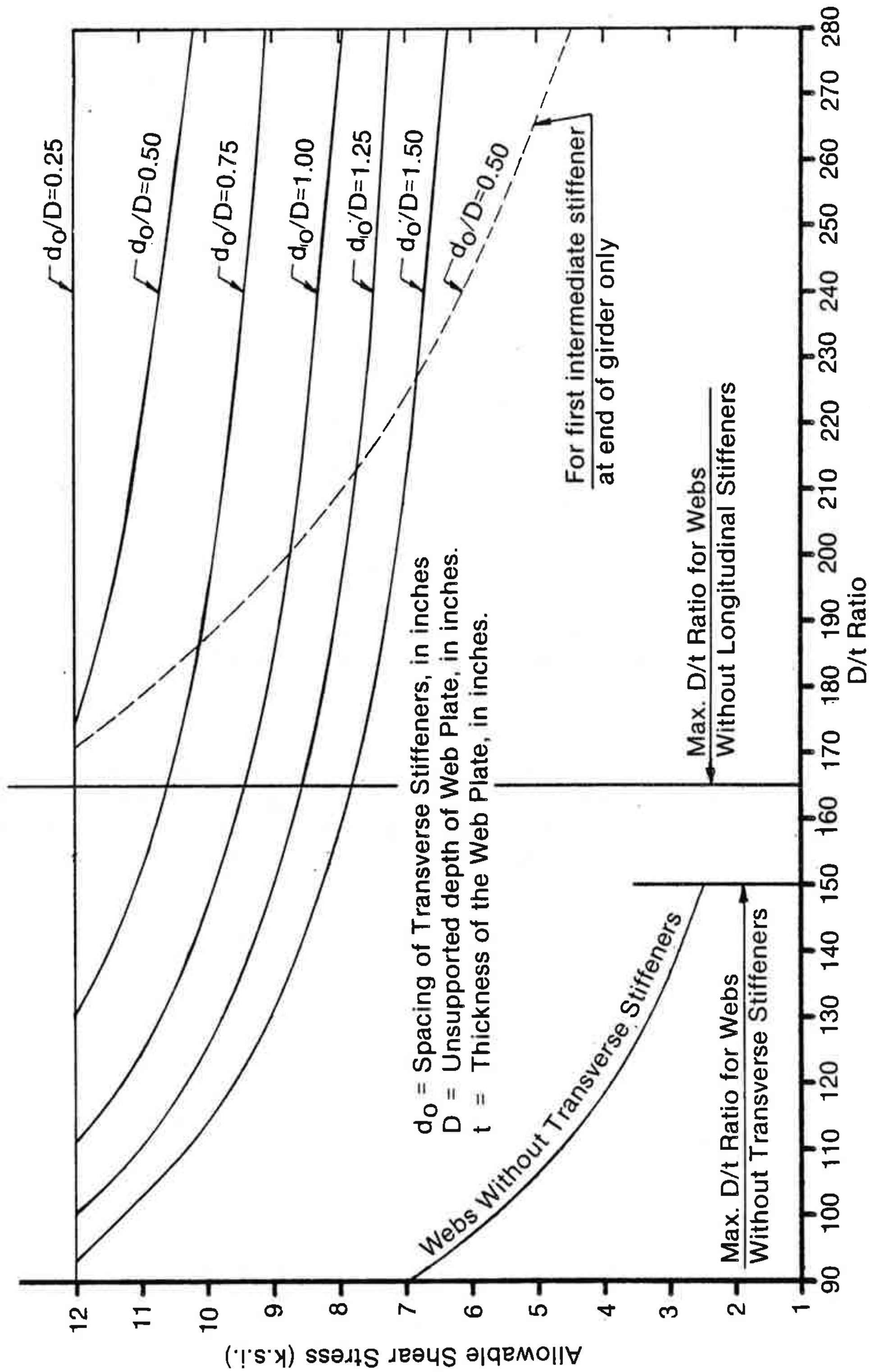


Figure 10.34.4A

Allowable Shear Stress vs. D/t Ratio

(M183 Steel, F_y = 36 ksi)

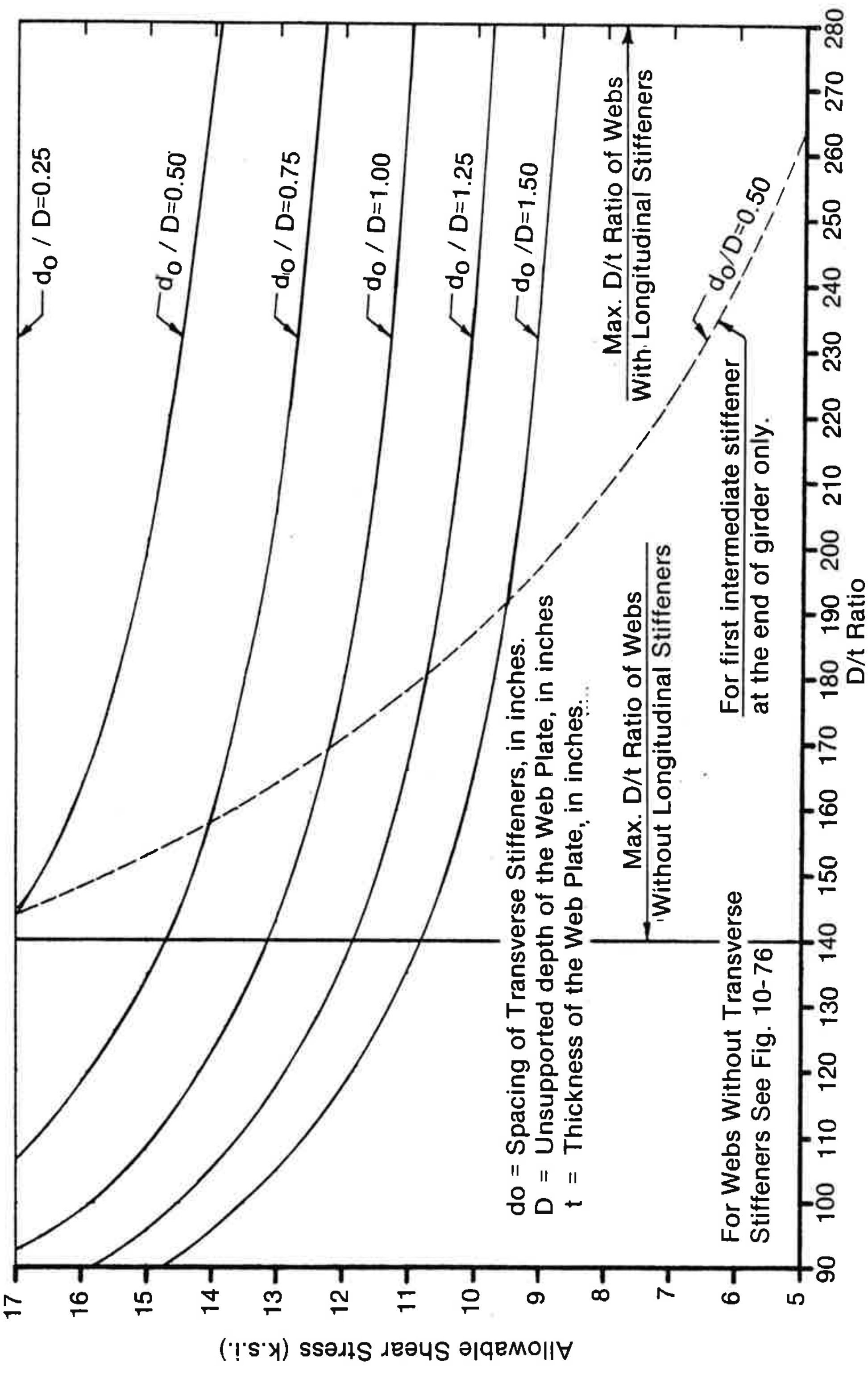


Figure 10.34.4A

Allowable Shear Stress vs. D/t Ratio

(M183 Steel, F_y = 36 ksi)

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, consideration shall be given to the need for this attachment if the location of the stiffener or its use as a connector plate for a diaphragm or cross frame will produce out-of-plane movements in a welded web to flange connection

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:

$$I = d_o t_w^3 J \qquad (10\text{-}30)$$

where

$$J = 2.5\ (D/d_o)^2 - 2, \text{ but not less than } 0.5 \qquad (10\text{-}31)$$

In these expressions,

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

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 $4t_w$ nor 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.

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 A36 steel.

10.34.5 LONGITUDINAL STIFFENERS

10.34.5.1 The centerline 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:

$$I = Dt_w^3 \left(2.4 \frac{d_o^2}{D^2} - 0.13\right) \qquad (10\text{-}32)$$

where:

I = the minimum moment of inertia of the longitudinal stiffener about its edge in contact with the web plate, in inches^4.
D = the unsupported distance between flange components, in inches
t_w = the thickness of the web plate, in inches
d_o = the actual distance between transverse stiffeners, in inches

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

$$t_s \geq \frac{b'\sqrt{f_b}}{2,250} \qquad (10\text{-}33)$$

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 The shear stress on the girder web when stiffened by the use of a longitudinal stiffener shall be determined in accordance with Article 10.34.4.

10.34.6 BEARING STIFFENERS

10.34.6.1 WELDED GIRDERS

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

$$\frac{b'}{12}\sqrt{\frac{F_y}{33{,}000}} \qquad (10\text{-}34)$$

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

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

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:

$$V = \frac{P}{100}\left[\frac{100}{\left(\frac{\ell}{r}\right)+10} + \frac{\frac{\ell}{r}F_y}{3,300,000}\right] \qquad (10\text{-}36)$$

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, the outstanding legs of angles, and perforated plates at the perforations, the b/t ratio of the plates or angle segments, when used in compression, shall not be greater than the value obtained by use of the formula:

$$\frac{b}{t} = \frac{1,625}{\sqrt{f_a}} \qquad (10\text{-}37)$$

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
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:

$$\frac{b}{t} = \frac{4{,}000}{\sqrt{f_a}} \qquad (10\text{-}38)$$

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
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:

$$\frac{b}{t} = \frac{5{,}000}{\sqrt{f_a}} \qquad (10\text{-}39)$$

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
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:

$$\frac{b}{t} = \frac{6{,}000}{\sqrt{f_a}} \qquad (10\text{-}40)$$

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
90,000 psi, Y.P. Min. b/t = 30
100,000 psi, Y.P. Min. b/t = 29

In the above expressions--

f_a = the computed compressive stress
b = the width (defined as indicated for each expression)
t = the 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:

$$\frac{f_a}{F_a} + \frac{C_{mx}\, f_{bx}}{\left(1-\dfrac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_{my}\, f_{by}}{\left(1-\dfrac{f_a}{F'_{ey}}\right)F_{by}} \leq 1.0 \qquad (10\text{-}41)$$

and

$$\frac{f_a}{0.472F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \text{ (at points of support)} \qquad (10\text{-}42)$$

where

$$F'_e = \frac{\pi^2 E}{F.S.\ (K_b L_b / r_b)^2} \qquad (10\text{-}43)$$

And

fa = 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.

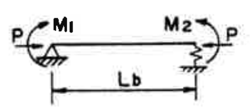
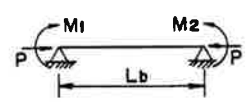
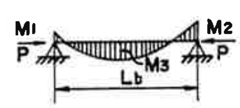
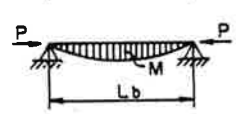
L_b = Actual unbraced length in the plane of bending.

r_b = Radius of gyration in the plane of bending.

C_{mx}, C_{my} = A coefficient about the X axis and Y axis respectively, whose value is taken from Table 10.36A.

F.S. = Factor of Safety = 2.12.

Table 10.36A

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 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:

$$A_F = \frac{1}{1-\frac{1.70T}{AF_e}} \qquad (10\text{-}44)$$

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

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad \text{(Euler buckling stress)} \qquad (10\text{-}45)$$

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:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad (10\text{-}46)$$

where

f_a is the computed axial stress

f_b is the calculated bending stress, including moment amplification, at the extreme fiber.

F_a is the allowable axial unit stress

F_b is the allowable bending unit stress

10.37.1.3 For buckling in the vertical plane:

$$F_a = \frac{F_y}{2.12}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right] \text{ with KL as defined above} \quad (10\text{-}47)$$

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:

$$\frac{D}{t_w} = \frac{5{,}000}{\sqrt{f_a}}, \text{ maximum } D/t_w = 60 \quad (10\text{-}48)$$

10.37.2.2 If one longitudinal stiffener is used at mid-depth of the web, maximum D/t_w shall be as follows:

$$\frac{D}{t_w} = \frac{7{,}500}{\sqrt{f_a}}, \text{ maximum } D/t_w = 90 \quad (10\text{-}49)$$

and the moment of inertia of the stiffener about an axis parallel to the web and at the base of the stiffener shall be equal to:

$$I_s = 0.75\ Dt_s^3 \quad (10\text{-}50)$$

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:

$$\frac{D}{t_w} = \frac{10{,}000}{\sqrt{f_a}}, \text{ maximum } D/t_w = 120 \quad (10\text{-}51)$$

and the moment of inertia of each stiffener shall be:

$$I_s = 2.2\ Dt_s^3 \quad (10\text{-}52)$$

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:

$$\frac{b'}{t_s} = \frac{1{,}625}{\sqrt{f_a + \frac{f_b}{3}}}, \text{ maximum } \frac{b'}{t_s} = 12 \qquad (10\text{-}53)$$

10.37.2.5 Web plate equations apply between limits:

$$0.2 \leq \frac{f_b}{f_a + f_b} \leq 0.7 \qquad (10\text{-}54)$$

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:

$$\frac{b}{t_f} = \frac{4{,}250}{\sqrt{f_a + f_b}}, \text{ maximum } b/t_f = 47 \qquad (10\text{-}55)$$

10.37.3.2 The b'/t_f ratio for the overhang width of flange plates shall be not greater than:

$$\frac{b'}{t_f} = \frac{1{,}625}{\sqrt{f_a + f_b}} \quad \text{maximum } b'/t_f = 12 \qquad (10\text{-}56)$$

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=145pcf) 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, psi.

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
= 2000 - 2300		= 11
2400 - 2800		= 10
2900 - 3500		= 9
3600 - 4500		= 8
4600 - 5900		= 7
6000 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 should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. If concrete is on the tension side of the neutral axis, it shall not be considered in computing moments of inertia or resisting moments except for deflection calculations. Mechanical anchorages shall be provided to tie the sections together and to develop stresses on the plane joining the concrete and the steel.

10.38.1.7 The steel beams, especially if not supported by intermediate falsework shall be investigated for stability 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 which are 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 which permit a thorough compaction of the concrete in order to insure that their entire surfaces are in contact with the concrete. They shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

10.38.2.2 The capacity of stud and channel shear connectors welded to the girders is given in Article 10.38.5. Channel shear connectors shall have at least 3/16-inch fillet welds placed along the heel and toe of the channel.

10.38.2.3 The clear depth of concrete cover over the tops of the shear connectors shall be not less than 2 inches. Shear connectors shall penetrate at least 2 inches above bottom of slab.

10.38.2.4 The clear distance between the edge of a girder flange and the edge of the shear connectors shall be not less than one inch.

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, nor six times the thickness of the slab, nor 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 which 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 which 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 In continuous spans, the positive moment portion 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 contraflexure 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.

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:

$$S_r = \frac{V_r Q}{I} \qquad (10\text{-}57)$$

where

S_r = the 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 = the range of shear due to live loads and impact in kips. At any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads).

Q = the 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 in^3.

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*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.

I = the 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 in^4.

(In the above, 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

$$Z_r = Bw \qquad (10\text{-}58)$$

Welded studs (for ratios of H/d equal to or greater than 4)

$$Z_r = \alpha d^2 \qquad (10\text{-}59)$$

where

w = the length of a channel shear connector in inches measured in a transverse direction on the flange of a girder.

d = diameter of stud, in inches

α = 13,000 for 100,000 cycles
10,600 for 500,000 cycles
7,850 for 2,000,000 cycles
5,500 for over 2,000,000 cycles

B = 4,000 for 100,000 cycles
3,000 for 500,000 cycles
2,400 for 2,000,000 cycles
2,100 for over 2,000,000 cycles

H = height of stud in inches

The required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one transverse girder cross-section (ΣZ_r) by the horizontal range of shear S_r. Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged.

10.38.5.1.2 ULTIMATE STRENGTH

The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength.

The number of shear connectors required equal or exceed the number given by the formula:

$$N_1 = \frac{P}{\phi S_u} \tag{10-60}$$

where

N_1 = the number of connectors between points of maximum positive moment and adjacent end supports.

S_u = the ultimate strength of the shear connector as given below.

ϕ = a 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:

$$P_1 = A_sF_y \tag{10-61}$$

or

$$P_2 = 0.85f'_cbc \tag{10-62}$$

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.
c = 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:

$$N_2 = \frac{P+P_3}{\phi S_u} \tag{10-63}$$

At points of maximum negative moment the force in the slab is taken as:

$$P_3 = A_s^r F_y^r * \qquad (10\text{-}64)$$

where

A_s^r = total area of longitudinal reinforcing steel at the interior support within the effective flange width.

F_y^r = specified minimum yield point of the reinforcing steel.

The ultimate strength of the shear connector is given as follows:

Channels:

$$S_u = 550 \left(h+\frac{t}{2}\right) W \sqrt{f'_c} \qquad (10\text{-}65)$$

Welded studs: $(H/d \geq 4)$:

$$S_u = 0.4d^2 \sqrt{f'_c E_c} \qquad (10\text{-}66)$$

where

E_c = Modulus of Elasticity of the concrete, psi

$$E_c = w^{3/2} 33 \sqrt{f'_c} \qquad (10\text{-}67)$$

S_u = ultimate strength of individual shear connector, in pounds.

h = the average flange thickness of the channel flange, in inches.

t = the 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, psi.

d = diameter of stud, in inches.

w = unit weight of concrete, in lbs. per cu. ft.

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

*When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments P_3 is equal to zero.

concrete is not used in computing section properties for negative moments, shall be computed by the formula:

$$N_c = A_r^s f_r / Z_r \qquad (10\text{-}68)$$

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 = the 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 1/3 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 non-parallel 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 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:

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} \qquad (10\text{-}69)$$

where

$$R = \frac{N_w}{\text{Number of Box Girders}} \qquad (10\text{-}70)$$

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 nor 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:

$$V_w = V_v/\cos \qquad (10\text{-}71)$$

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 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, the transverse bending stresses resulting from distortion of the span, 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 1/5 the span length. If the flange plate width exceeds 1/5 of the span, an amount equal to 1/5 of the span only shall be considered effective.

10.39.4.1.2 For continuous spans, the criteria above shall be applied to the lengths between points of contraflexure.

10.39.4.2 COMPRESSION FLANGES UNSTIFFENED

10.39.4.2.1 Unstiffened compression flanges designed for the basic allowable stress of 0.55 F_y shall have a width to thickness ratio equal to or less than the value obtained by the use of the formula:

$$\frac{b}{t} = \frac{6,140}{\sqrt{F_y}} \qquad (10\text{-}72)$$

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:

$$f_b = 0.55F_y - 0.224F_y\left[1 - \sin\left(\frac{\pi}{2} \times \frac{13{,}300 - \dfrac{b\sqrt{F_y}}{t}}{7{,}160}\right)\right] \qquad (10\text{-}73)$$

10.39.4.2.3 For values of b/t exceeding $\dfrac{13{,}300}{\sqrt{F_y}}$ the stress in the flange shall not exceed the value given by the formula:

$$f_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^6 \qquad (10\text{-}74)$$

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 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:

$$I_s = \phi\, t^3 w \qquad (10\text{-}75)$$

where

$\phi = 0.07\, k^3 n^4$ for values of n greater than 1

$\phi = 0.125\, k^3$ for a value of 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

*In solving these equations a value of k between 2 and 4 generally should be assumed.

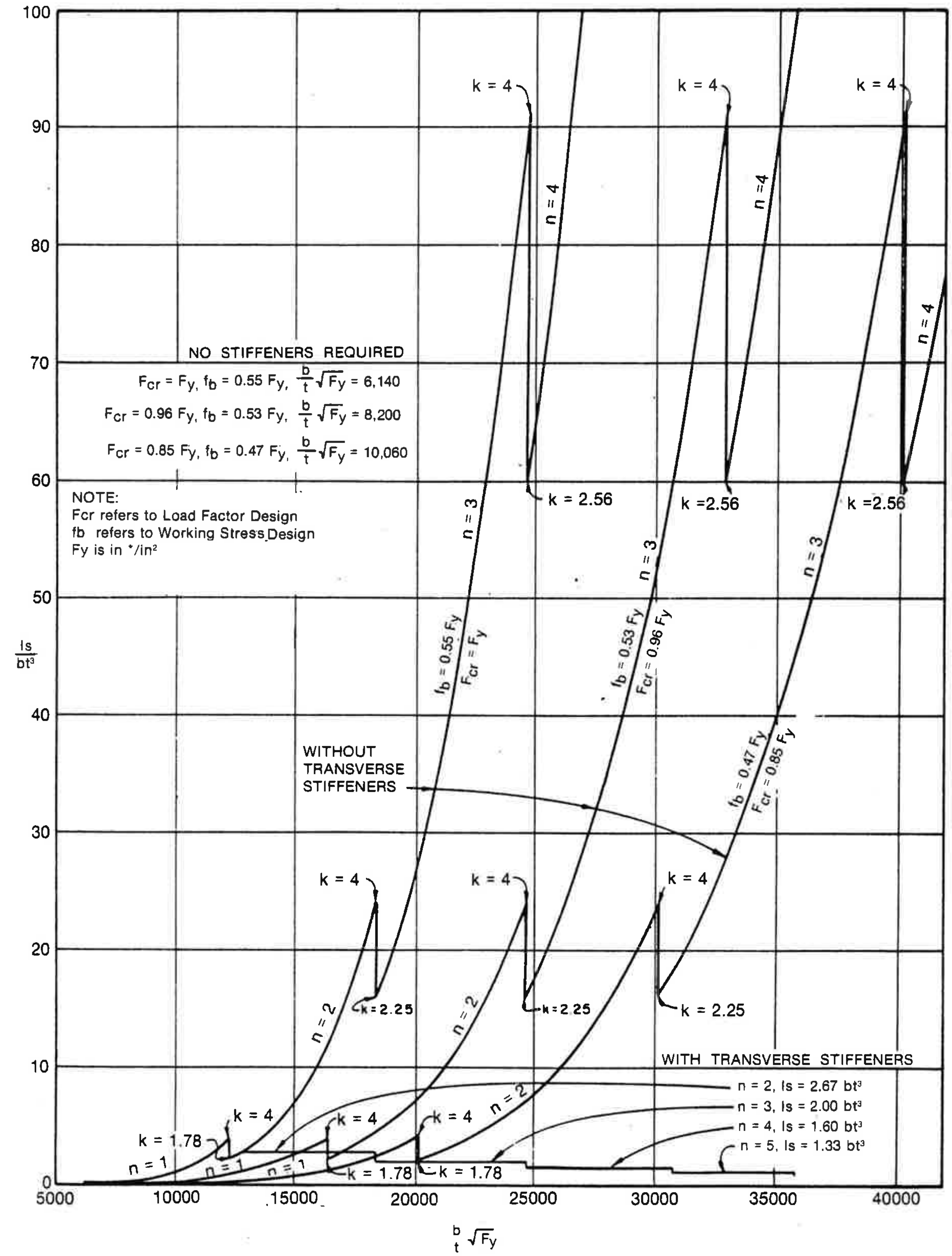

Figure 10.39.4.3A

Longitudinal Stiffeners - Box Girder Compression Flange

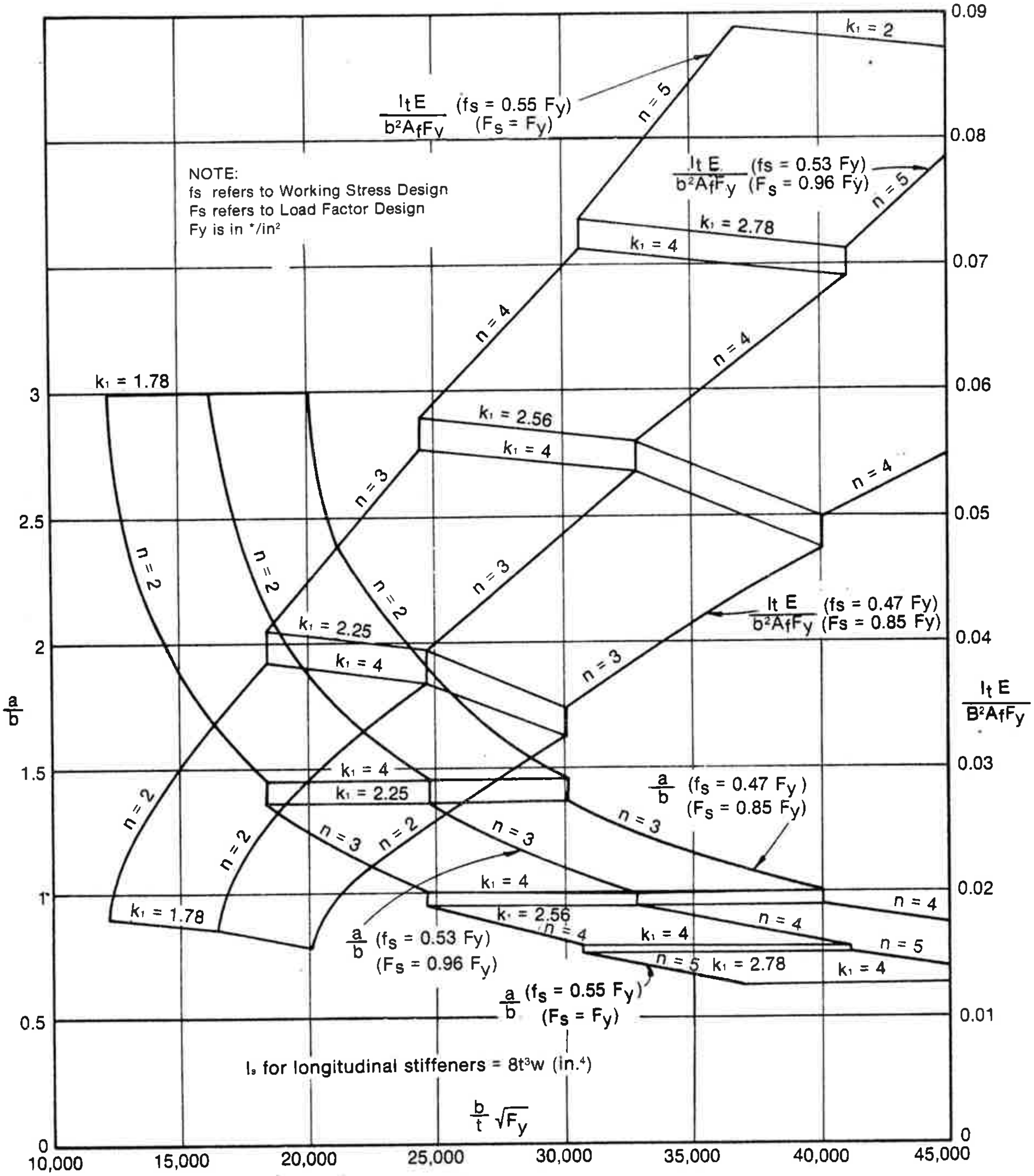

Figure 10.39.4.3B

Spacing and Size of Transverse Stiffeners

(For Flange Stiffened Longitudinally and Transversely)

10.39.4.3.2 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 F_y, the ratio w/t shall not exceed the value given by the formula:

$$\frac{w}{t} = \frac{3{,}070\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}76)$$

10.39.4.3.3 For greater values of w/t but not exceeding 60 or $\frac{6{,}650\sqrt{k}}{\sqrt{Fy}}$

whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula:

$$f_b = 0.55F_y - 0.224F_y\left[1 - \sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k} - \frac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k}}\right)\right] \qquad (10\text{-}77)$$

10.39.4.3.4 For values of w/t exceeding $\frac{6{,}650\sqrt{k}}{\sqrt{Fy}}$ but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14.4\ k(t/w)^2 \times 10^6 \qquad (10\text{-}78)$$

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:

$$I_s = 8\ t^3 w \qquad (10\text{-}79)$$

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:

$$I_t = 0.10(n+1)^3 w^3 \frac{f_s}{E} \frac{A_f}{a} \qquad (10\text{-}80)$$

where

A_f = area of bottom flange including longitudinal stiffeners

a = spacing of transverse stiffeners

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

E = modulus of elasticity of steel

10.39.4.4.3 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 F_y, the ratio w/t for the longitudinal stiffeners shall not exceed the value given by the formula:

$$\frac{w}{t} = \frac{3{,}070\sqrt{k_1}}{\sqrt{F_y}} \qquad (10\text{-}81)$$

where

$$k_1 = \frac{[1+(a/b)^2]^2+87.3}{(n+1)^2(a/b)^2[1+0.1(n+1)]} \qquad (10\text{-}82)$$

10.39.4.4.4 For greater values of w/t, but not exceeding 60 or $\dfrac{6{,}650\sqrt{k_1}}{\sqrt{F_y}}$, whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula:

$$f_b = 0.55F_y - 0.224F_y\left[1-\sin\left(\frac{\pi}{2} \times \frac{6{,}650\sqrt{k_1} - \dfrac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k_1}}\right)\right] \qquad (10\text{-}83)$$

10.39.4.4.5 For values of w/t exceeding $\frac{6,650\sqrt{k_1}}{\sqrt{F_y}}$

but not exeeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula:

$$f_b = 14.4k_1\left(\frac{t}{w}\right)^2 \times 10^6 \qquad (10\text{-}84)$$

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:

$$R_w = \frac{F_y S_s}{2b} \qquad (10\text{-}85)$$

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:

$$R_s = \frac{F_y S_s}{nb} \qquad (10\text{-}86)$$

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:

$$\frac{b'}{t'} = \frac{2,600}{\sqrt{F_y}} \qquad (10\text{-}87)$$

where

b' = width of any outstanding stiffener element
t' = thickness 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. If fillet welds are used, they shall be 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, composite box girders, and orthotropic-deck 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.

$$R = 1 - \frac{\beta\psi(1-\alpha)^2(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)} \qquad (10\text{-}88)$$

(See Figures 10.40.2.1A and 10.40.2.1B)

where

α = the minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange.*

β = the area of the web divided by the area of the tension flange.*

ψ = the 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 average calculated shear stress in the web shall not exceed the allowable shear stress listed in Article 10.34.4; except the allowable shear stress of transversely stiffened girder is limited to:

$$F_v = \frac{7\times10^7\,[1+(D/d_o)^2]}{(D/t_w)^2} \leq F_y/3 \text{ (See Figure 10.40.2.2A)} \qquad (10\text{-}89)$$

*Bottom flange of orthotropic deck bridges.

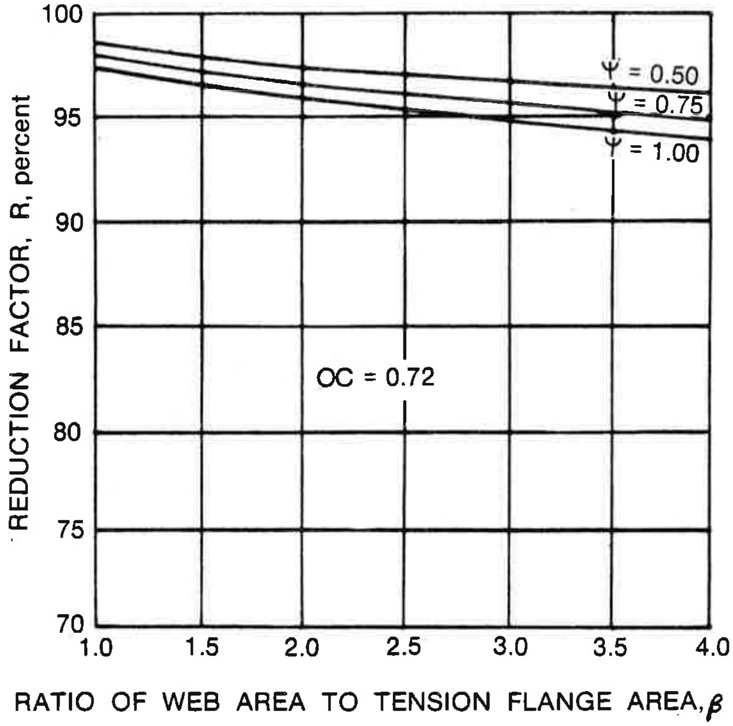

FIGURE 10.40.2.1A

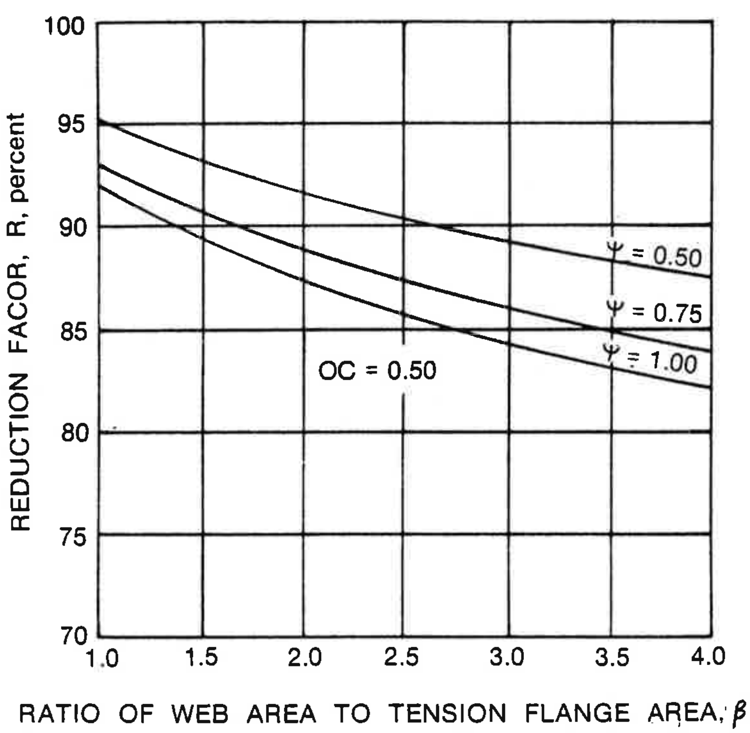

FIGURE 10.40.2.1B

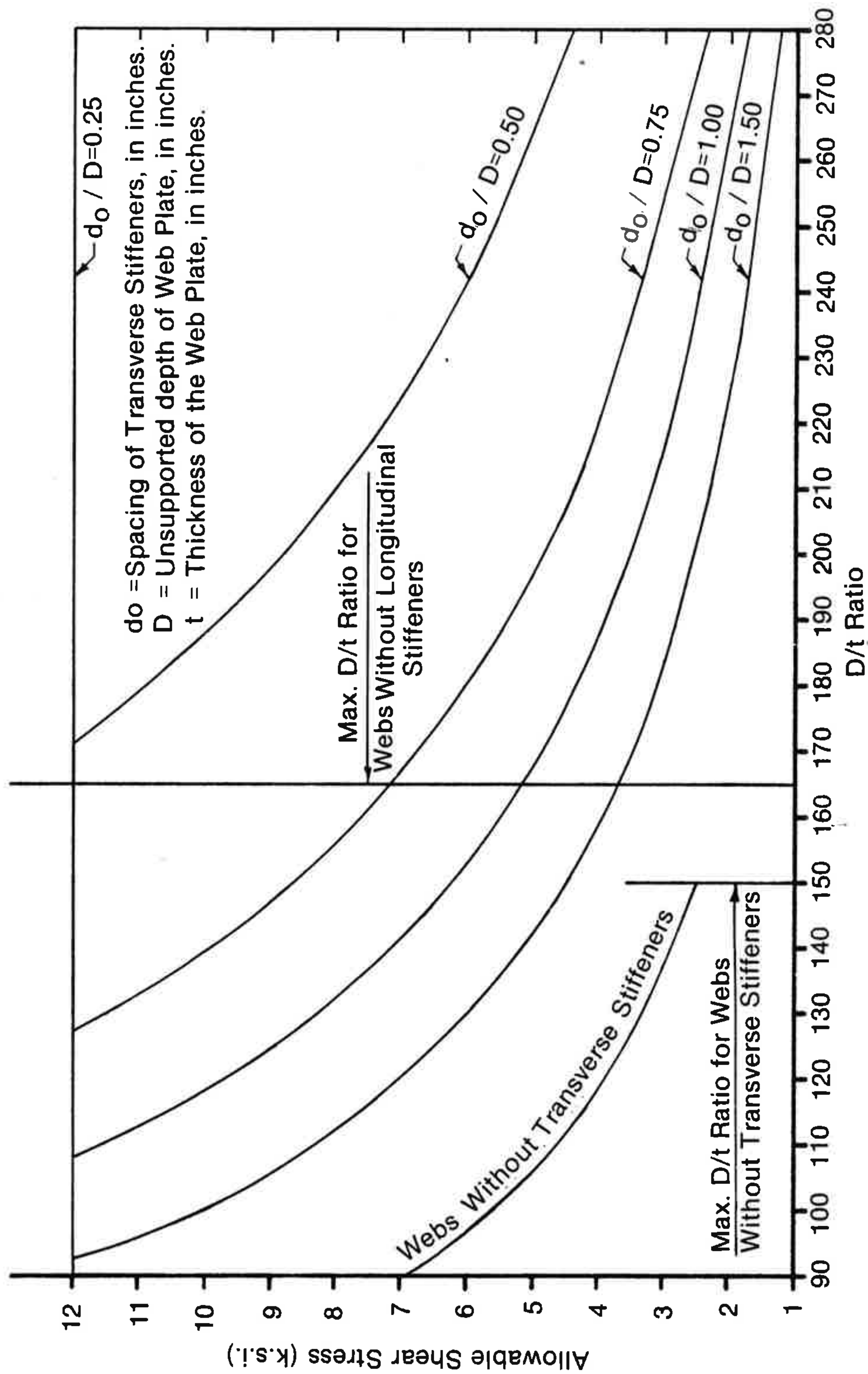

FIGURE 10.40.2.2A

Allowable Shear Stress vs. D/t Ratio

(M183 Steel, F_y = 36 ksi)

and the limitation of the first stiffener spacing shall not apply.

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.

10.41.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.41.1 through 10.41.4.10

An appropriate method of elastic analysis, such as the equivalent-orthotropic-slab method or the equivalent-grid method, shall be used in designing the deck. The equivalent stiffness properties shall be selected to correctly simulate the actual deck. An appropriate method of elastic analysis, such as the thin-walled-beam method, that accounts for the effects of torsional distortions of the cross-sectional shape shall be used in designing the girders of orthotropic-deck box-girder bridges. The box-girder design shall be checked for lane or truck loading arrangements that produce maximum distortional (torsional) effects.

10.41.1.3 For an alternate design method (Strength Design) see Article 10.60.

10.41.2 WHEEL LOAD CONTACT AREA

The wheel loads specified in Article 3.7 shall be uniformly distributed to the deck plate over the rectangular area defined below:

Wheel Load, kip	Width Perpendicular to Traffic, inches	Length in Direction of Traffic, inches
8	20+2t	8+2t
12	20+2t	8+2t
16	24+2t	8+2t

In the above table, t is the thickness of the wearing surface in inches.

10.41.3 EFFECTIVE WIDTH OF DECK PLATE

10.41.3.1 RIBS AND BEAMS

The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam may be calculated by accepted approximate methods.*

10.41.3.2 GIRDERS

10.41.3.2.1 The full width of deck plate may be considered effective in acting as the top flange of the girders if the effective span of the girders is not less than: (1) 5 times the maximum distance between girder webs and (2) 10 times the maximum distance from edge of the deck to the nearest girder web. The effective span shall be taken as the actual span for simple spans and the distance between points of contraflexure for continuous spans. Alternatively, the effective width may be determined by accepted analytical methods.

10.41.3.2.2 The effective width of the bottom flange of a box girder shall be determined according to the provisions of Article 10.39.4.1.

10.41.4 ALLOWABLE STRESSES

10.41.4.1 LOCAL BENDING STRESSES IN DECK PLATE

The term local bending stresses refers to the stresses caused in the deck plate as it carries a wheel load to the ribs and beams. The local transverse bending stresses caused in the deck plate by the specified wheel load plus 30 percent impact shall not exceed 30,000 psi unless a higher allowable stress is justified by a

*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.

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 Articles 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. 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:

$$\left(\frac{L}{r}\right)_{max} = 1,000 \sqrt{\frac{1,500}{F_y} - \frac{2,700F}{F_y^{\,2}}} \qquad (10\text{-}90)$$

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 an alternate method for design of simple and continuous beam and girder structures of moderate length. It is a method of proportioning structural members for multiples of the design loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings and to the control of live load deflections under service loadings.

10.43 LOADS

10.43.1 Service live loads are vehicles which may operate on a highway legally without special load permit.

10.43.2 For design purposes, the service loads are taken as the dead, live and impact loadings described in Section 3.

10.43.3 Overloads are the live loads that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes the maximum overload is taken as 5(L+I)/3.

10.43.4 The maximum loads are the loadings specified in Article 10.47.

10.44 DESIGN THEORY

10.44.1 The moments, shears and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 10.48.1.3.

10.44.2 The members shall be proportioned by the methods specified in Articles 10.48 through 10.56 so that their computed maximum strengths shall be at least equal to the total effects of design loads multiplied by their respective load factors specified in Article 3.2

10.44.3 Service behavior shall be investigated as specified in Articles 10.57 through 10.59.

10.45 ASSUMPTIONS

10.45.1 Strain in flexural members shall be assumed directly proportional to the distance from the neutral axis.

10.45.2 Stress in steel below the yield strength, F_y, of the grade of steel used shall be taken as 29,000,000 psi times the steel strain. For strain greater than that corresponding to the yield strength, F_y, the stress shall be considered independent of strain and equal to the yield strength, F_y. This assumption shall apply also to the longitudinal reinforcement in the concrete floor slab in the region of negative moment when shear developers are provided to secure composite action in this region.

10.45.3 At maximum strength the compressive stress in the concrete slab of a composite beam shall be assumed independent of strain and equal to $0.85f'_c$.

10.45.4 Tensile strength of concrete shall be neglected in flexural calculations.

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 from the formulas shown in Article 3.2. Each part of the structure shall be proportioned for the group loads that are applicable and the maximum design required by the group loading combinations shall be used.

10.48 SYMMETRICAL BEAMS AND GIRDERS

10.48.1 COMPACT SECTIONS

Symmetrical I-shaped beams 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 which rotate at near constant moment.

Rolled or fabricated I-shaped beams meeting the requirements of Article 10.48.1.1 below shall be considered compact sections and the maximum strength shall be as computed:

$$M_u = F_y Z \tag{10-91}$$

Where F_y is the specified yield point of the steel being used, Z is the plastic section modulus.*

10.48.1.1 Beams designed as compact sections shall meet the following requirements: (for certain frequently used steels these requirements are listed in Table 10.48.1.2A.)

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*Values for rolled sections are listed in the "Manual of Steel Construction," Eighth Edition, 1980, American Institute of Steel Construction. Appendix D shows the method of computing Z as presented in the Commentary of AISI Bulletin 15.

(a) Projecting compression flange element

$$\frac{b'}{t} \leq \frac{1,600}{\sqrt{F_y}} \quad (10\text{-}92)$$

where b' is the width of the projecting flange element, t is the flange thickness.

(b) Web thickness

$$\frac{d}{tw} \leq \frac{13,300}{\sqrt{F_y}} \quad (10\text{-}93)$$

where d is the depth of the beam, t_w is the web thickness.

(c) Lateral bracing

$$\frac{L_b}{r_y} \leq \frac{7,000}{\sqrt{F_y}} \quad \text{when } M_2 \geq 0.7M_1 \quad (10\text{-}94)$$

or

$$\frac{L_b}{r_y} \leq \frac{12,000}{\sqrt{F_y}} \quad \text{when } M_2 < 0.7M_1 \quad (10\text{-}95)$$

where L_b is the distance between points of bracing of the compression flange,

r_y is the radius of gyration with respect to the Y-Y axis, M_1 and M_2 are the moments at the two adjacent braced points. In no case shall L_b exceed the value given in Article 10.48.2.1(C).

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

$$P \leq 0.15F_yA \quad (10\text{-}96)$$

where A is the area of the cross section.

(e) Maximum shear force

$$V \leq 0.55F_ydt_w \quad (10\text{-}97)$$

10.48.1.2 Article 10.48.1 is applicable to steels with stress-strain diagrams which exhibit a yield plateau followed by a strain hardening range. Steels such as AASHTO M 183 (ASTM A 36), AASHTO M 223 (ASTM A 572) and AASHTO M 222 (ASTM A 588) meet these requirements. The limitations set forth in Article 10.48.1 are given in Table 10.48.1.2A.

Table 10.48.1.2A

F_y(psi)	36,000	50,000
b'/t	8.4	7.2
d/t	70	59
L_b/r_y $M_2 \geq 0.7M_1$	37	31
L_b/r_y $M_2 < 0.7M_1$	63	54

10.48.1.3 In the design of a continuous beam of compact section complying with the provision of Article 10.48.1.1 negative moments over supports determined by elastic analysis may be reduced by a maximum of 10 percent. Such reduction shall be accompanied by an increase in maximum positive moment in the span equal to the average decrease of the negative moments in the span. The reduction shall not apply to negative moments produced by cantilever loading.

10.48.2 BRACED NON-COMPACT SECTIONS

For rolled or fabricated I-shaped beams 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:

$$M_u = F_y S \tag{10-98}$$

where S is the section modulus.

10.48.2.1 The above equation is applicable to beams meeting the following requirements:

(a) Projecting compression flange element

$$\frac{b'}{t} \leq \frac{2,200}{\sqrt{F_y}} \tag{10-99}$$

when $M < M_u$, b'/t may be increased by the ratio $\sqrt{M_u/M}$

(b) Web thickness

$$\frac{D}{t_w} \leq 150 \qquad (10\text{-}100)$$

where D is the clear unsupported distance between flange components.

(c) Spacing of lateral bracing for compression flange

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

where d is the depth of beam or girder,

A_f is the flange area.

(d) Maximum axial compression

Axial compression shall not exceed $P \leq 0.15\ F_y A$. (10-102)

(e) Maximum shear force

$$V_u \leq 1.015 \times 10^8 \frac{t_w^3}{D} \qquad (10\text{-}103)$$

but not more than $0.58F_y Dt_w$

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

Table 10.48.2.1A

F_y (psi)	36,000	50,000	90,000	100,000
b'/t	11.6	9.8	7.3	7.0
$\frac{L_b d}{A_f}$	556	400	222	200

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) the maximum strength shall be computed as:

$$M_u = F_y S \left[1 - \frac{3F_y}{4\pi^2 E} \left(\frac{L_b}{b'} \right)^2 \right] \qquad (10\text{-}104)$$

When the ratio of stresses at the two ends of the braced length, L_b, is less than 0.7, the maximum strength, M_u, as computed by the above formula may be increased 20 percent but not to exceed $F_y S$.

10.48.4.2 In members not meeting the requirements of Article 10.48.2.1(e) the web shall be provided with transverse stiffeners as specified in Article 10.48.5.

10.48.4.3 Members with axial loads in excess of $0.15F_y A$ should be designed as beam-columns as specified in Article 10.54.

10.48.5 TRANSVERSELY STIFFENED GIRDERS

10.48.5.1 For girders not meeting the shear requirements of Articles 10.48.1.1(e) and 10.48.2.1(e) transverse stiffeners are required for the web. For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement:

$$\frac{D}{t_w} \leq \frac{36{,}500}{\sqrt{F_y}} \qquad (10\text{-}105)$$

For different grades of steel this limit is:

D/t_w	F_y (psi)
192	36,000
163	50,000
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 10.48.2 or 10.48.4.1, as applicable subject to the requirements of Article 10.48.5.4.

10.48.5.3 The shear capacity of beams and girders with webs fulfilling the requirements of Article 10.48.5.1 shall be computed as:

$$V_u = V_p \left[C + \frac{0.87\ (1-C)}{\sqrt{1+(d_o/D)^2}} \right] \qquad (10\text{-}106)$$

where

$$V_p = 0.58 F_y D t_w \qquad (10\text{-}107)$$

$$C = 18{,}000(t_w/D)\sqrt{\frac{1+(D/d_o)^2}{F_y}} - 0.3 \leq 1.0 \qquad (10\text{-}108)$$

D = clear, unsupported distance between flange components
d_o = distance between transverse stiffeners

10.48.5.4 If a girder panel is subjected to simultaneous action of shear and bending moment with the magnitude of the shear higher than $0.6V_u$, then the moment shall be limited to not more than:

$$M/M_u = 1.375 - 0.625V/V_u \qquad (10\text{-}109)$$

10.48.5.5 Transverse stiffeners shall be spaced at a distance, d_o, according to shear capacity as specified in Article 10.48.5.3 but not more than 1.5D. Transverse stiffeners may be omitted in those portions of the girders where the maximum shear force is less than the value given by Article 10.48.2.1(e).

The first stiffener space at the simple support end of a girder shall be such that the shear force in the end panel will not exceed the value given by the following equation. The maximum spacing is limited to D/2

$$V = 1.2\text{x}10^8 \left[1 + \left(\frac{D}{d_o} \right)^2 \right] \frac{t_w^3}{D} \leq V_p \qquad (10\text{-}110)$$

where V_p equals the shear yielding strength of the web or V_p = 0.58 $F_y D t_w$. The width-to-thickness ratio of transverse stiffeners shall be such that

$$\frac{b'}{t} \leq \frac{2{,}600}{\sqrt{F_y}} \qquad (10\text{-}111)$$

where b' is the projecting width of the stiffener.

The gross cross-sectional area of intermediate transverse stiffeners shall not be less than:

$$A = [0.15BDt_w(1-C)(V/V_u)-18t_w^2]Y \qquad (10\text{-}112)$$

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,
2.4 for single plates.
C is computed by Article 10.48.5.3

The moment of inertia of transverse stiffeners with reference to the midplane of the web shall be not less than:

$$I = d_o t_w^3 J \qquad (10\text{-}113)$$

where:

$$J = 2.5(D/d_o)^2-2, \text{but not less than } 0.5. \qquad (10\text{-}114)$$

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 $4t_w$ nor 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, consideration shall be given to the need for this attachment if the location of the stiffener or its use as a connector plate for a diaphragm or cross frame will produce out-of-plane movements in a welded web to flange connection.

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:

$$\frac{D}{t_w} \leq \frac{73{,}000}{\sqrt{F_y}} \qquad (10\text{-}115)$$

For different grades of steel, this limit is:

D/t_w	F_y (psi)
385	36,000
326	50,000
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 Articles 10.48.2 or Article 10.48.4.1 as applicable, subject to the requirement of Article 10.48.5.4.

10.48.6.3 The shear capacity of girders with one longitudinal stiffener shall be computed by Article 10.48.5.3.

The dimensions of the longitudinal stiffener shall be such that:

(a) the width-to-thickness ratio is not greater than that given by Article 10.48.5.5.

(b) the rigidity of the stiffener is not less than:

$$I \geq D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \qquad (10\text{-}116)$$

(c) the radius of gyration of the stiffener is not less than:

$$r \geq \frac{d_o \sqrt{F_y}}{23{,}000} \qquad (10\text{-}117)$$

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 10.48.5.5 except that the depth of subpanels shall be used instead of the total panel depth, D. In addition, the section modulus of the transverse stiffener shall be not less than:

$$S_s = \frac{1}{3}(D/d_o)S_t \qquad (10\text{-}118)$$

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 and 10.34.

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 except that in computing the maximum strength by Article 10.48.4.1 the term b' is replaced by 0.9b'.

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:

$$\frac{D_c}{t_w} \leq \frac{18,250}{\sqrt{F_y}} \qquad (10\text{-}119)$$

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:

$$\frac{D_c}{t_w} \leq \frac{36,500}{\sqrt{F_y}} \qquad (10\text{-}120)$$

10.50 COMPOSITE BEAMS AND GIRDERS

Composite beams shall be so proportioned that the following criteria are satisfied:

(a) The maximum strength of any section shall not be less than the sum of the computed moments at that section multiplied by the appropriate load factors.

(b) The web of the steel section shall be designed to carry the total external shear and must satisfy the applicable provisions of Articles 10.48 and 10.49. 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:

$$\frac{b'}{t} = \frac{2,200}{\sqrt{1.3f_{d\ell 1}}} \qquad (10\text{-}121)$$

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

10.50.1 POSITIVE MOMENT SECTIONS OF COMPOSITE BEAMS AND GIRDERS

10.50.1.1 COMPACT SECTIONS

When the steel section satisfies the compactness requirements of Article 10.50.1.1.2, the maximum strength shall be computed as the resultant moment of the fully plastic stress distribution acting on the section (Figure 10.50A).

10.50.1.1.1 The resultant moment of the fully plastic stress distribution may be computed as follows:

(a) the compressive force in the slab, C, is equal to the smallest of the values given by the following Equations:

$$(1)\ C = 0.85f'_c bt_s + (AF_y)_c \qquad (10\text{-}122)$$

where b is the effective width of slab, t_s is the slab thickness.

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

Figure 10.50A

(2) $C=(AF_y)_{bf} + (AF_y)_{tf} + (AF_y)_w$ (10-123)

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,

$(AF_y)_w$ is the product of area and yield point for web of steel section.

(3) $C = \Sigma Q_u$ (10-124)

where ΣQ_u is the sum of ultimate strengths of shear connectors between the section under consideration and the section of zero moment.

(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) above the top portion of the steel section will be subjected to the following compressive force:

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

(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 $C' \geq (AF_y)_{tf}$

$$\bar{y} = t_{tf} + \frac{C'-(AF_y)_{tf}}{(AF_y)_w} d_w \qquad (10\text{-}128)$$

(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 qualify as compact when their steel section meets the requirements of Articles 10.48.1.1(b) and 10.48.1.1(e), and the stress-strain diagram of the steel exhibits a yield plateau followed by a strain hardening range.

10.50.1.2 NON-COMPACT SECTIONS

10.50.1.2.1 When the steel section does not satisfy the compactness requirements of Article 10.50.1.1.2 the maximum strength of the section shall be taken as the moment at first yielding.

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 which 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 strength of 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.

10.51 COMPOSITE BOX GIRDERS

This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single-cell composite box girders. The distance center-to-center flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of the flanges of each box. In addition to the above, when non-parallel 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

$$M_u = F_{cr}S \qquad (10\text{-}129)$$

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:

$$V_w = V/\cos\theta \qquad (10\text{-}130)$$

where V = one half of the total vertical shear force on one box girder, 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:

$$\frac{b}{t} = \frac{6{,}140}{\sqrt{F_y}} \qquad (10\text{-}131)$$

where b = flange width between webs in inches
t = flange thickness in inches.

10.51.5.2 For greater b/t ratios,

$$\frac{6,140}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{13,300}{\sqrt{F_y}} \qquad (10\text{-}132)$$

the buckling stress of an unstiffened bottom flange is given by the formula:

$$F_{cr} = 0.592F_y \left(1+0.687 \sin \frac{c\pi}{2}\right) \qquad (10\text{-}133)$$

in which c shall be taken as

$$c = \frac{13,300 - \frac{b}{t}\sqrt{F_y}}{7,160} \qquad (10\text{-}134)$$

10.51.5.3 For values of

$$\frac{b}{t} > \frac{13,300}{\sqrt{F_y}} \qquad (10\text{-}135)$$

the buckling stress of the flange is given by the formula:

$$F_{cr} = 105(t/b)^2 \times 10^6 \qquad (10\text{-}136)$$

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:

$$I_s = \phi\, t^3 w \qquad (10\text{-}137)$$

where

$\phi = 0.07k^3n^4$ when n equals 2, 3, 4, or 5.
$\phi = 0.125k^3$ 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:

$$\frac{w}{t} = \frac{3,070\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}138)$$

10.51.5.4.2 For greater values of w/t,

$$\frac{3,070\sqrt{K}}{\sqrt{F_y}} < \frac{w}{t} \leq \frac{6,650\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}139)$$

the buckling stress of the flange, including stiffeners is given by Article 10.51.5.2 in which c shall be taken as:

$$c = \frac{6,650\sqrt{k} - \frac{w}{t}\sqrt{F_y}}{3,580\sqrt{k}} \qquad (10\text{-}140)$$

10.51.5.4.3 For values of

$$\frac{w}{t} > \frac{6,650\sqrt{k}}{\sqrt{F_y}} \qquad (10\text{-}141)$$

the buckling stress of the flange, including stiffeners, is given by the formula:

$$F_{cr} = 26.2k(t/w)^2 \times 10^6 \qquad (10\text{-}142)$$

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:

$$\frac{b'}{t'} = \frac{2,600}{\sqrt{F_y}} \qquad (10\text{-}143)$$

where

b' = width of any outstanding stiffener element,
t' = thickness 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 Articles 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 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), 10.48.5.1, 10.48.6.1, 10.49.2, and 10.49.3.2(b) use the F_y of the compression flange.

(2) In Articles 10.48.6.3(a), 10.48.6.3(c) use the F_y of the adjacent flange.

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

$$M_u = F_{yf}Z \qquad (10\text{-}144)$$

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 F_{yf}.

10.53.1.2 BRACED NON-COMPACT SECTIONS

The equation of Article 10.48.2 for the maximum strength of non-compact sections shall be replaced by the expression

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

For symmetrical sections,

$$R = \frac{12+\beta(3\rho - \rho^3)}{12+2\beta} \qquad (10\text{-}146)$$

where

$$\rho = F_{yw}/F_{yf}$$

$$\beta = A_w/A_f$$

For unsymmetrical sections,

$$R = 1 - \left[\frac{\beta\psi(1-\rho)^2(3-\psi+\rho\psi)}{6+\beta\psi(3-\psi)}\right] \qquad (10\text{-}147)$$

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 NON-COMPACT SECTIONS

The equation of Article 10.48.4.1 for the maximum strength of unbraced non-compact sections shall be replaced by the expression

$$M_u = F_{yf}S\left[1 - \frac{3F_{yf}}{4\pi^2 E}\left(\frac{L_b}{b'}\right)^2\right]R \qquad (10\text{-}148)$$

where the appropriate R is determined from 10.53.1.2 above.

10.53.1.4 TRANSVERSELY STIFFENED GIRDERS

The equation of Article 10.48.5.3 for the shear capacity of transversely stiffened girders shall be replaced by the expression

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

The equation for A in Article 10.48.5.5 is 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:

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

where A_s is the gross effective area of the column cross section and F_{cr} is determined by one of the following two formulas:

$$F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_c}{r}\right)^2\right] \qquad (10\text{-}151)$$

$$\text{for } \frac{KL_c}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}} \qquad (10\text{-}152)$$

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2} \qquad (10\text{-}153)$$

$$\text{for } \frac{KL_c}{r} > \sqrt{\frac{2\pi^2 E}{F_y}} \qquad (10\text{-}154)$$

where

- K is effective length factor in the plane of buckling
- L_c is length of the member between points of support, in inches
- r is radius of gyration in the plane of buckling, in inches
- F_y is yield stress of the steel, in psi
- E is 29,000,000 psi
- F_{cr} is buckling stress, in psi

10.54.1.2 EFFECTIVE LENGTH

The effective length factor K shall be determined as follows:

(a) For members having lateral support in both directions at its ends:

K = 0.75 for riveted, bolted, or welded end connections
K = 0.875 for pinned ends.

(b) For members having ends not fully supported laterally by diagonal bracing or an attachment to an adjacent structure, the effective length factor shall be determined by a rational procedure.*

10.54.2 COMBINED AXIAL LOAD AND BENDING

10.54.2.1 MAXIMUM CAPACITY

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

$$\frac{P}{0.85A_s F_{cr}} + \frac{MC}{M_u\left(1 - \dfrac{P}{A_sF_e}\right)} \leq 1.0 \qquad (10\text{-}155)$$

$$\frac{P}{0.85A_sF_y} + \frac{M}{M_p} \leq 1.0 \qquad (10\text{-}156)$$

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

where

F_{cr} is buckling stress as determined by the equations of Article 10.54.1.1.

M_u is the 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 is the equivalent moment factor, as defined below.

$M_p = F_y Z$ the full plastic moment of the section,

Z is the plastic section modulus,

$\frac{KL_c}{r}$ is the 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

$$A_F = \frac{1}{1 - \frac{1.18T}{AF_e}} \tag{10-159}$$

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

10.55.2 WEB PLATES

No longitudinal stiffener, $D/t_w = \frac{6,750}{\sqrt{f_a}}$ (10-161)

One longitudinal stiffener, $D/t_w = \frac{10,150}{\sqrt{f_a}}$ (10-162)

Two longitudinal stiffeners, $D/t_w = \frac{13,500}{\sqrt{f_a}}$ (10-163)

The b'/t_s ratio for the stiffeners shall be:

$$\frac{b'}{t_s} = \frac{2,200}{\sqrt{f_a + \frac{f_b}{3}}} \quad \text{maximum } \frac{b'}{t_s} = 12 \tag{10-164}$$

10.55.3 FLANGE PLATES

$$\frac{b}{t_f} = \frac{5,700}{\sqrt{f_a + f_b}} \text{ for width between webs} \tag{10-165}$$

$$\frac{b'}{t_f} = \frac{2,200}{\sqrt{f_a + f_b}} \text{ for overhang widths, maximum } b'/t_f = 12 \tag{10-166}$$

10.56 SPLICES, CONNECTIONS, AND DETAILS

10.56.1 CONNECTORS

10.56.1.1 GENERAL

Connectors shall be proportioned so that their maximum strength multiplied by the reduction factor, Ø shall be at least equal to the effects of design loads multiplied by their respective load factors specified in Article 3.2. The maximum strengths multipled by the reduction factors are listed in Table 10.56A.

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 insure sound welds. The effective weld area shall be taken as defined in AWSD1.1, Articles 2.3 and 10.8.

10.56.1.3 BOLTS AND RIVETS

10.56.1.3.1 In proportioning fasteners, the nominal diameter shall be used except when a shear plane intersects the threads.

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

For combined tension and shear in bearing-type connections, bolts and rivets shall be proportioned so that the shear stress does not exceed:

$$F_{vc} \leq \sqrt{F_v^2 - (0.6f_t)^2} \qquad (10\text{-}167)$$

where

F_v = shear strength of the fastener, ϕF, as given in Table 10.56A.

f_t = tensile stress due to the applied load

10.56.1.4 FRICTION JOINTS

Friction joints shall be designed to prevent slip at the overload in accordance with Article 10.57.3. Maximum strength of the bolts need not be considered in the design of such joints.

Table 10.56A

Type of Fastener	Strength (ϕF)
Groove Weld[a]	1.00 F_y
Fillet Weld[b]	0.45 F_u
Low-Carbon Steel Bolts AASHTO M--- (ASTM A 307)	
Tension	27 ksi
Shear[c]	25 ksi
Power-Driven Rivets AASHTO M 228 (ASTM A 502)	
Shear--Grade 1	25 ksi
Shear--Grade 2	30 ksi
High-Strength Bolts AASHTO M 164 (ASTM A 325)	
Tension[e]	70 ksi
Shear (Bearing-Type)[c,d]	60 ksi
Bearing[f]	$\frac{LF_u}{1.18d}$ or 3.0 F_u whichever is smaller
AAHSTO M 253 (ASTM A 490)	
Tension	87 ksi
Shear (Bearing-Type)[c,d]	80 ksi
Bearing[f]	$\frac{LF_u}{1.18d}$ or 3.0 F_u whichever is smaller

[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]When a shear plane intersects the bolts threads, the root area shall be used.

[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]For M 164 (A 325) bolts the tensile strength decreases for diameters greater than 1 inch. The design value listed is for bolts up to 1 inch diameter. For diameters greater than 1 inch the design value shall be computed as $0.58F_u$ where F_u is the minimum tensile strength of the bolts.

[f]L is the distance in inches measured in the line of the force from the center line of a bolt to the nearest edge of the hole for an adjacent bolt or to the end of the connected part toward which the force is directed; d is the diameter of the bolt in inches; and F_u is the lowest specified minimum tensile strength of the connected parts. The ratio of L/d shall not be less than 1.5.

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 = 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,
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 = is the column moment,

d_b = the beam depth,

d_c = the 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

$$t_w < \frac{A_f}{t_b+5k} \tag{10-170}$$

and opposite the tension flange of the first member when

$$t_c < 0.4\sqrt{A_f} \tag{10-171}$$

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

For noncomposite beams the moment caused by D+5(L+I)/3 shall not exceed 0.8 F_yS. For such beams designed for Group IA loading, the moment caused by D+2.2(L+I) shall not exceed 0.8 F_yS. 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

For composite beams the moment caused by D+5(L+I)/3 shall not exceed 95 percent of the moment at first yielding in the section. For such beams designed for Group IA loading, the moment caused by D+2.2(L+I) shall not exceed 95 percent of the moment at first yielding in the section. In computing dead load stresses the presence or absence of temporary supports during the construction shall be considered.

10.57.3 FRICTION JOINTS

10.57.3.1 The shear caused by the loading D+5(L+I)/3 in friction-type high-strength bolted joints shall not exceed 1.33 x F_v, where F_v is the allowable shear stress as listed in Tables 10.32.3B and 10.32.3C.

10.57.3.2 For combined shear and tension in friction-type joints where applied forces reduce the total clamping force on the friction plane, the maximum shear stress shall not exceed the values obtained from the following equations:

For AASHTO M 164 (ASTM A 325) bolts

$$f_v = 1.33F_v(1 - 1.59 \times 10^{-5}f_t) \qquad (10\text{-}172)$$

For AASHTO M 253 (ASTM A 490) bolts

$$f_v = 1.33F_v(1 - 1.27 \times 10^{-5}f_t) \qquad (10\text{-}173)$$

where f_t is the applied tensile load in pounds per square inch

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 10.31 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.

SECTION 11 - ALUMINUM DESIGN

11.1 GENERAL

The purpose of this section is to provide a location for indexing aluminum design and material specifications.

11.2 BRIDGES

The "Specifications for Aluminum Bridge and Other Highway Structures," Second Edition, issued April 1976 by the Aluminum Association, Inc., are intended to serve as a standard or guide for the preparation of plans and specifications and as a reference for designers, fabricators, and erectors.

11.3 SOIL-METAL PLATE INTERACTION SYSTEMS

The design of aluminum soil-metal plate interaction systems shall be in accordance with Section 12. Fabrication and installation shall be in accordance with Section 23 - Division II.

11.4 STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS

The AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals" shall be used for the design and preparation of plans and specifications, fabrication, and erection of aluminum sign supports, luminaires and traffic signals.

11.5 BRIDGE RAILING

The design and fabrication of aluminum bridge railing shall be governed by Article 2.7. The AASHTO Guide for Selecting, Locating, and Designing Traffic Barriers, Part I, Section V, should be consulted for guidance on the safety considerations in the design of bridge rail.

SECTION 12 - SOIL-CORRUGATED METAL STRUCTURE INTERACTION SYSTEMS

12.1 GENERAL

12.1.1 SCOPE

The specifications of this section are intended for the structural design of corrugated metal structures. It must be recognized that a buried flexible structure is a composite structure made up of the metal ring and the soil envelope; and that both materials play a vital part in the structural design of flexible metal structures.

12.1.2 NOTATIONS

A = required wall area (Article 12.2.1)

A = area of pipe wall (Article 12.3.1)

E_m = modulus of elasticity of metal (Articles 12.2.2 and 12.3.2)

E_m = modulus of elasticity of pipe material (Articles 12.2.4 and 12.3.4)

FF = flexibility factor (Articles 12.2.4 and 12.3.4)

f_a = allowable stress - specified minimum yield point divided by safety factor (Article 12.2.1)

f_{cr} = critical buckling stress (Articles 12.2.2 and 12.3.2)

f_u = specified minimum tensile strength (Articles 12.2.2 and 12.3.2)

f_y = specified minimum yield point (Article 12.3.1)

I = moment of inertia, per unit length, of cross section of the pipe wall (Articles 12.2.4 and 12.3.4)

k = soil stiffness factor (Articles 12.2.2 and 12.3.2)

P = design load (Article 12.1.4)

r = radius of gyration of corrugation (Articles 12.2.2 and 12.3.2)

S = diameter or span (Articles 12.1.4 and 12.2.2)

s = pipe diameter or span (Articles 12.2.4, 12.3.2 and 12.3.4)

SF = safety factor (Article 12.2.3)

SS = required seam strength (Articles 12.2.3 and 12.3.3)

T = thrust (Article 12.1.4)

T_L = thrust, load factor (Articles 12.3.1 and 12.3.3)

T_s = thrust, service load (Articles 12.2.1 and 12.2.3)

$\emptyset$ = capacity modification factor (Articles 12.3.1 and 12.3.3)

12.1.3 LOADS

Design load, P, shall be the pressure acting on the structure. For earth pressures see Article 3.20. For live load see Articles 3.4 to 3.7, 3.11, 3.12 and 6.4, except that the words "When the depth of fill is 2 feet or more" in Article 6.4.1 need not be considered. For loading combinations see Article 3.22.

12.1.4 DESIGN

12.1.4.1 The thrust in the wall shall be checked by three criteria. Each considers the mutual function of the metal wall and the soil envelope surrounding it. The criteria are:

(a) Wall area
(b) Buckling stress
(c) Seam strength (structures with longitudinal seams)

12.1.4.2 The thrust in the wall is:

$$T = P \times \frac{S}{2} \qquad (12\text{-}1)$$

Where P = Design load, lbs./sq.ft.
S = Diameter or Span, ft.
T = Thrust, lbs./ft.

12.1.4.3 Handling and installation strength shall be sufficient to withstand impact forces when shipping and placing the pipe.

12.1.4.4 Height of cover over the structure shall be sufficient to prevent damage to the buried structure. A minimum of 2 feet is suggested.

12.1.5 MATERIALS

The materials shall conform to the AASHTO specifications referenced herein.

12.1.6 SOIL DESIGN

12.1.6.1 SOIL PARAMETERS

The performance of a flexible culvert is dependent on soil structure interaction and soil stiffness.

The following must be considered:

(a) Soils

(1) The type and anticipated behavior of the foundation soil must be considered; i.e., stability for bedding and settlement under load.

(2) The type, compacted density and strength properties of the soil envelope immediately adjacent to the pipe must be established.

Good side fill is obtained from a granular material with little or no plasticity and free of organic material, i.e., AASHTO classification groups A-1, A-2 and A-3, compacted to a minimum 90 percent of standard density based on AASHTO Specifications T99 (ASTM D 698).

(3) The density of the embankment material above the pipe must be determined. See Article 6.2.

(b) Dimensions of soil envelope

The general recommended criteria for lateral limits of the culvert soil envelope are as follows:

(1) Trench installations - 2 feet minimum each side of culvert. This recommended limit should be modified as necessary to account for variables such as poor in-situ soils.

(2) Embankment installations - one diameter or span each side of culvert.

(3) The minimum upper limit of the soil envelope is one foot above the culvert.

12.1.6.2 PIPE ARCH DESIGN

The design of the corner backfill shall account for corner pressure which shall be considered to be approximately equal to thrust divided by the radius of the pipe arch corner. The soil envelope around the corners of pipe arches shall be capable of supporting this pressure.

12.1.6.3 ARCH DESIGN

12.1.6.3.1 Special design considerations may be applicable; a buried flexible structure may raise two important considerations. The first is that it is undesirable to make the metal arch relatively unyielding or fixed compared with the adjacent sidefill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended.

Where poor materials are encountered, consideration should be given to removing some or all of this poor material and replacing it with acceptable material.

The footing should be designed to provide uniform longitudinal settlement, of acceptable magnitude from a functional aspect. Providing for the arch to settle will protect it from possible drag down forces caused by the consolidation of the adjacent sidefill.

The second consideration is bearing pressure of soils under footings. Recognition must be given to the effect of depth of the base of footing and the direction of the footing reaction from the arch.

Footing reactions for the metal arch are considered to act tangential to the metal plate at its point of connection to the footing. The value of the reaction is the thrust in the metal arch plate at the footing.

12.1.6.3.2 Invert slabs and other appropriate measures shall be provided to anticipate scour.

12.1.7 ABRASIVE OR CORROSIVE CONDITIONS

Extra metal thickness, or coatings, may be required for resistance to corrosion and abrasion. For highly abrasive conditions, a special design may be required.

12.1.8 MINIMUM SPACING

When multiple lines of pipes or pipe arches greater than 48 inches in diameter or span are used, they shall be spaced so that the sides of the pipe shall be no closer than one-half diameter or three feet whichever is less, to permit adequate compaction of backfill material. For diameters up to and including 48 inches, the minimum clear spacing shall not be less than two feet.

12.1.9 END TREATMENT

Protection of end slopes may require special consideration where backwater conditions may occur, or where erosion and uplift could be a problem. Culvert ends constitute a major run-off-the-road hazard if not properly designed. Safety treatment such as structurally adequate grating that conforms to the embankment slope, extension of culvert length beyond the point of hazard, or provision of guard rail are among the alternatives to be considered. End walls on skewed alignment require a special design.

12.1.10 CONSTRUCTION AND INSTALLATION

The construction and installation shall conform to Section 23 - Division II.

12.2 SERVICE LOAD DESIGN

Service Load Design is a working stress method, as traditionally used for culvert design.

12.2.1 WALL AREA

$$A = T_s/f_a \qquad (12\text{-}2)$$

Where A = Required wall area, $in^2/ft.$

T_s = Thrust, Service Load, lbs./ft.

f_a = Allowable stress-specified minimum yield point, psi divided by safety factor (f_y/SF)

12.2.2 BUCKLING

Corrugations with the required wall area, A, shall be checked for possible buckling. If the allowable buckling stress, f_{cr}/SF, is less than f_a, then the required area must be recalculated using f_{cr}/SF in lieu of f_a. Formulae for buckling are:

$$\text{If } S < \frac{r}{k}\sqrt{\frac{24E_m}{f_u}} \text{ then } f_{cr} = f_u - \frac{f_u^2}{48E_m}\left(\frac{kS}{r}\right)^2 \qquad (12\text{-}3)$$

$$\text{If } S > \frac{r}{k}\sqrt{\frac{24E_m}{f_u}} \text{ then } f_{cr} = \frac{12E_m}{(kS/r)^2} \qquad (12\text{-}4)$$

Where f_u = Specified minimum tensile strength, psi

f_{cr} = Critical buckling stress, psi

k = Soil stiffness factor = 0.22

S = Diameter or span, inches

r = Radius of gyration of corrugation, in.

E_m = Modulus of elasticity of metal, psi

12.2.3 SEAM STRENGTH

For pipe fabricated with longitudinal seams (riveted, spot-welded, bolted), the seam strength shall be sufficient to develop the thrust in the pipe wall.

The required seam strength shall be:

$$SS = T_s(SF) \tag{12-5}$$

Where SS = Required seam strength in pounds per foot

T_s = Thrust in pipe wall, lbs./ft.

SF = Safety Factor

12.2.4 HANDLING AND INSTALLATION STRENGTH

Handling and installation rigidity is measured by a Flexibility Factor, FF, determined by the formula

$$FF = s^2/E_mI \tag{12-6}$$

Where FF = Flexibility Factor, inches per pound

s = Pipe diameter or maximum span, inches

E_m = Modulus of elasticity of the pipe material, psi

I = Moment of inertia per unit length of cross section of the pipe wall, inches to the 4th power per inch

12.3 LOAD FACTOR DESIGN

Load Factor Design is an alternative method of design based on ultimate strength principles.

12.3.1 WALL AREA

$$A = T_L/\phi f_y \tag{12-7}$$

Where A = Area of pipe wall, in^2/ft

T_L = Thrust, load factor, lbs./ft

f_y = Specified minimum yield point, psi

ϕ = Capacity modification factor

12.3.2 BUCKLING

If f_{cr} is less than f_y then A must be recalculated using f_{cr} in lieu of f_y.

$$\text{If } s < \frac{r}{k}\sqrt{\frac{24E_m}{f_u}} \text{ then } f_{cr} = f_u - \frac{f_u^{\,2}}{48E_m}(ks/r)^2 \tag{12-8}$$

$$\text{If } s > \frac{r}{k}\sqrt{\frac{24E_m}{f_u}} \text{ then } f_{cr} = \frac{12E_m}{(ks/r)^2} \qquad (12\text{-}9)$$

Where f_u = Specified minimum metal strength, psi

f_{cr} = Critical buckling stress, psi

k = Soil stiffness factor = 0.22

s = Pipe diameter or span, inches

r = Radius of gyration of corrugation, inches

E_m = Modulus of elasticity of metal, psi

12.3.3 SEAM STRENGTH

For pipe fabricated with longitudinal seams (riveted, spot-welded, bolted), the seam strength shall be sufficient to develop the thrust in the pipe wall. The required seam strength shall be:

$$SS = \frac{T_L}{\emptyset} \qquad (12\text{-}10)$$

Where SS = Required seam strength in pounds/ft.

T_L = Thrust multiplied by applicable factor, in pounds/lin. ft.

$\emptyset$ = Capacity modification factor

12.3.4 HANDLING AND INSTALLATION STRENGTH

Handling rigidity is measured by a Flexibility Factor, FF, determined by the formula

$$FF = s^2/E_m I \qquad (12\text{-}11)$$

Where FF = Flexibility Factor, inches per pound

s = Pipe diameter or maximum span, inches

E_m = Modulus of elasticity of the pipe material, psi

I = Moment of inertia per unit length of cross section of the pipe wall, inches to the 4th power per inch

12.4 CORRUGATED METAL PIPE

12.4.1 GENERAL

12.4.1.1 Corrugated metal pipe and pipe-arches may be of riveted, welded or lock seam fabrication with annular or helical corrugations. The specifications are:

Aluminum	Steel
AASHTO M190, M196	AASHTO M36, M245, M190

12.4.1.2 Service load design - safety factor, SF:

Seam Strength	=	3.0
Wall area	=	2.0
Buckling	=	2.0

12.4.1.3 Load factor design - capacity modification factor, Ø. Helical pipe with lock seam or fully welded seam

$$\phi = 1.00$$

Annular pipe with spot welded, riveted or bolted seam

$$\phi = 0.67$$

12.4.1.4 Flexibility Factor

(a) For steel conduits, FF should generally not exceed the following values:

1/4" and 1/2" depth corrugation FF = 4.3×10^{-2}

1" depth corrugation FF = 3.3×10^{-2}

(b) For aluminum conduits, FF should generally not exceed the following values:

1/4" and 1/2" depth corrugation FF = 9.5×10^{-2}

1" depth corrugation FF = 6×10^{-2}

12.4.1.5 Minimum Cover

The minimum cover for design loads shall be Span/8 but not less than 12-inches. (The minimum cover shall be measured from the top of a rigid pavement or the bottom of a flexible pavement). For construction requirements, see Article 23.10 - Division II.

12.4.2 SEAM STRENGTH

Minimum Longitudinal Seam Strength

2 x 1/2 and 2-2/3 x 1/2 Corrugated Steel Pipe Riveted or Spot Welded				3 x 1 Corrugated Steel Pipe Riveted or Spot Welded		
Thickness (inches)	Rivet Size (inch)	Single Rivets (Kips/foot)	Double Rivets (Kips/foot)	Thickness (inches)	Rivet Size (inch)	Double Rivets (Kips/foot)
0.064	5/16	16.7	21.6	0.064	3/8	28.7
0.079	5/16	18.2	29.8	0.079	3/8	35.7
0.109	3/8	23.4	46.8	0.109	7/16	53.0
0.138	3/8	24.5	49.0	0.138	7/16	63.7
0.168	3/8	25.6	51.3	0.168	7/16	70.7

2 x 1/2 and 2-2/3 x 1/2 Corrugated Aluminum Pipe Riveted			
Thickness (inches)	Rivet Size (inch)	Single Rivets (Kips/foot)	Double Rivets (Kips/foot)
0.060	5/16	9.0	14.0
0.075	5/16	9.0	18.0
0.105	3/8	15.6	31.5
0.135	3/8	16.2	33.0
0.164	3/8	16.8	34.0

3 x 1 Corrugated Aluminum Pipe Riveted			6 x 1 Corrugated Aluminum Pipe Riveted		
Thickness (inches)	Rivet Size (inch)	Double Rivets (Kips/foot)	Thickness (inches)	Rivet Size (inch)	Double Rivets (Kips/foot)
0.060	3/8	16.5	0.060	1/2	16.0
0.075	3/8	20.5	0.075	1/2	19.9
0.105	1/2	28.0	0.105	1/2	27.9
0.135	1/2	42.0	0.135	1/2	35.9
0.164	1/2	54.5	0.167	1/2	43.5

12.4.3 SECTION PROPERTIES

12.4.3.1 STEEL CONDUITS

	1-1/2 x 1/4 Corrugation			2-2/3 x 1/2 Corrugation		
Thickness (inches)	A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)	A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)
0.028	0.304					
0.034	0.380					
0.040	0.456	0.0816	0.253	0.465	0.1702	1.121
0.052	0.608	0.0824	0.344	0.619	0.1707	1.500
0.064	0.761	0.0832	0.439	0.775	0.1712	1.892
0.079	0.950	0.0846	0.567	0.968	0.1721	2.392
0.109	1.331	0.0879	0.857	1.356	0.1741	3.425
0.138	1.712	0.0919	1.205	1.744	0.1766	4.533
0.168	2.098	0.0967	1.635	2.133	0.1795	5.725

	3 x 1 Corrugation			5 x 1 Corrugation		
Thickness (inches)	A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)	A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)
0.064	0.890	0.3417	8.659	0.794	0.3657	8.850
0.079	1.113	0.3427	10.883	0.992	0.3663	11.092
0.109	1.560	0.3488	15.459	1.390	0.3677	15.650
0.138	2.008	0.3472	20.183	1.788	0.3693	20.317
0.168	2.458	0.3499	25.091	2.186	0.3711	25.092

12.4.3.2 ALUMINUM CONDUITS

	1-1/2 x 1/4 Corrugation		
Thickness (inches)	A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)
0.048	0.608	0.0824	0.344
0.060	0.761	0.0832	0.349

1-2/3 x 1/2 Corrugation		
A_s (sq.in/ft)	r (inch)	$I \times 10^{-3}$ (in^4/in)
0.775	0.1712	1.892
0.968	0.1721	2.392
1.356	0.1741	3.425
1.745	0.1766	4.533
2.130	0.1795	5.725

	3 x 1 Corrugation			6 x 1			
Thickness (inches)	A_s (sq. in/ft)	r (inch)	I x 10^{-3} (in^4/in)	A_s (sq. in/ft)	Effective Area (sq.in/ft)	r (inch)	I x 10^{-3} (in^4/in)
0.060	0.890	0.3417	8.659	0.775	0.387	0.3629	8.505
0.075	1.118	0.3427	10.883	0.968	0.484	0.3630	10.631
0.105	1.560	0.3488	15.459	1.356	0.678	0.3636	14.340
0.135	2.088	0.3472	20.183	1.744	0.872	0.3646	19.319
0.164	2.458	0.3499	25.091	2.133	1.066	0.3656	23.760

12.4.4 CHEMICAL AND MECHANICAL REQUIREMENTS

12.4.4.1 Aluminum-Corrugated Metal Pipe and Pipe-Arch Material requirements - AASHTO M 197

Mechanical properties for design		
Minimum Tensile Strength psi	Minimum Yield Point psi	Mod. of Elast. psi
31,000	24,000	10 X 10^6

12.4.4.2 Steel - Corrugated Metal Pipe and Pipe-Arch Material requirements - AASHTO M 218
M 246

Mechanical properties for design		
Minimum Tensile Strength psi	Minimum Yield Point psi	Mod. of Elast. psi
45,000	33,000	29 X 10^6

12.4.5 SMOOTH LINED PIPE

Corrugated metal pipe composed of a smooth liner and corrugated shell attached integrally at helical seams spaced not more than 30 inches apart may be designed in accordance with Article 12.1 on the same basis as a standard corrugated metal pipe having the same corrugations as the shell and a weight per foot equal to the sum of the weights per foot of liner and helically corrugated shell. The shell shall be limited to corrugations having a maximum pitch of 3 inches and a thickness of not less than 60 percent of the total thickness of the equivalent standard pipe.

12.5 STRUCTURAL PLATE PIPE STRUCTURES

12.5.1 GENERAL

12.5.1.1 Structural plate pipe, pipe-arches, and arches shall be bolted with annular corrugations only.

The specifications are:

Aluminum	Steel
AASHTO M219	AASHTO M167

12.5.1.2 Service load design - safety factor, SF
Seam strength = 3.0
Wall area = 2.0
Buckling = 2.0

12.5.1.3 Load factor design - capacity modification factor, Ø

$$Ø = 0.67$$

12.5.1.4 Flexibility Factor

(a) For steel conduits, FF should generally not exceed the following values:

6" X 2" corrugation FF = 2.0 X 10^{-2} (Pipe)

6" X 2" corrugation FF = 3.0 X 10^{-2} (Pipe-arch)

6" X 2" corrugation FF = 3.0 X 10^{-2} (Arch)

(b) For aluminum conduits, FF should generally not exceed the following values:

9" X 2-1/2" corrugation FF = 2.5 X 10^{-2} (Pipe)

9" X 2-1/2" corrugation FF = 3.6 X 10^{-2} (Pipe-arch)

9" x 2-1/2" corrugation FF = 7.2 X 10^{-2} (Arch)

12.5.1.5 Minimum Cover

The minimum cover for design loads shall be Span/8 but not less than 12 inches. (The minimum cover shall be measured from the top of a rigid pavement or the bottom of a flexible pavement.) For Construction requirements see Article 23.10 - Division II.

12.5.2 SEAM STRENGTH

Minimum Longitudinal Seam Strengths 6 x 2 Steel Structural Plate Pipe				
Thickness (inches)	Bolt Size (inch)	4 Bolts/ft. (Kips/foot)	6 Bolts/ft. (Kips/foot)	8 Bolts/ft. (Kips/foot)
0.109	3/4	43.0		
0.138	3/4	62.0		
0.168	3/4	81.0		
0.188	3/4	93.0		
0.218	3/4	112.0		
0.249	3/4	132.0		
0.280	3/4	144.0	180	194

9 x 2-1/2 Aluminum Structural Plate Pipe			
Thickness (inches)	Bolt Size (inch)	Steel Bolts 5-1/2 Bolts Per ft. (Kips/foot)	Aluminum Bolts 5-1/2 Bolts Per ft. (Kips/foot)
0.100	3/4	28.0	26.4
0.125	3/4	41.0	34.8
0.150	3/4	54.1	44.4
0.175	3/4	63.7	52.8
0.200	3/4	73.4	52.8
0.225	3/4	83.2	52.8
0.250	3/4	93.1	52.8

12.5.3 SECTION PROPERTIES

12.5.3.1 STEEL CONDUITS

6" X 2" Corrugations			
Thickness (inches)	A_s (sq.in/ft)	r (inch)	I X 10^{-3} (in^4/in)
0.109	1.556	0.682	60.411
0.138	2.003	0.684	78.175
0.168	2.449	0.686	96.163
0.188	2.739	0.688	108.000
0.218	3.199	0.690	126.922
0.249	3.650	0.692	146.172
0.280	4.119	0.695	165.836

12.5.3.2 ALUMINUM CONDUITS

9" X 2-1/2" Corrugations			
Thickness (inches)	A_s (sq.in/ft)	r (inch)	I X 10^{-3} (in^4/in)
0.100	1.404	0.8438	83.065
0.125	1.750	0.8444	103.991
0.150	2.100	0.8449	124.883
0.175	2.449	0.8454	145.895
0.200	2.799	0.8460	166.959
0.225	3.149	0.8468	188.179
0.250	3.501	0.8473	209.434

12.5.4 CHEMICAL AND MECHANICAL PROPERTIES

12.5.4.1 Aluminum - Structural plate pipe, pipe-arch, and arch Material requirements - AASHTO M 219, Alloy 5052.

Mechanical properties for design			
Thickness (inches)	Minimum Tensile Strength psi	Minimum Yield Point psi	Mod. of Elast. psi
0.100 to 0.175	35,000	24,000	10 x 10^6
0.176 to 0.250	34,000	24,000	10 x 10^6

12.5.4.2 Steel - Structural plate pipe, pipe-arch, and arch Material requirements - AASHTO M 167

Mechanical properties for design		
Minimum Tensile Strength psi	Minimum Yield Point psi	Mod. of Elast. psi
45,000	33,000	29 X 10^6

12.5.5 STRUCTURAL PLATE ARCHES

The design of structural plate arches should be based on ratios of a rise to span of 0.3 minimum.

12.6 LONG SPAN STRUCTURAL PLATE STRUCTURES

12.6.1 GENERAL

Long span structural plate structures are short span bridges defined as:

12.6.1.1 Structural Plate Structures (pipe, pipe arch, and arch) which exceed the maximum sizes imposed by Article 12.5.

12.6.1.2 Special shapes of any size which involve a relatively large radius of curvature in crown or side plates. Vertical ellipses, horizontal ellipses, underpasses, low profile arches, high profile arches, and inverted pear shapes are the terms describing these special shapes.

12.6.1.3 Wall Strength and Chemical and Mechanical Properties shall be in accordance with Article 12.5. The construction and installation shall conform to Section 23 - Division II.

12.6.2 DESIGN

12.6.2.1 GENERAL

Long span structures shall be designed in accordance with Articles 12.1 and 12.5 and 12.2 or 12.3 except that the requirements for buckling and flexibility factor shall not apply. The span in the formulae for thrust shall be replaced by twice the top arc radius. Long span structures shall include acceptable special features. Minimum requirements are detailed in Table 12.6.1A.

12.6.2.2 ACCEPTABLE SPECIAL FEATURES

(a) Continuous longitudinal structural stiffeners connected to the corrugated plates at each side of the top arc. Stiffeners may be metal or reinforced concrete either singly or in combination.

(b) Reinforcing ribs formed from structural shapes curved to conform to the curvature of the plates, fastened to the structure as required to ensure integral action with the corrugated plates, and spaced at such intervals as necessary to increase the moment of inertia of the section to that required by the design.

12.6.2.3 DESIGN FOR DEFLECTION

Soil design and placement requirements for long span structures limit deflection satisfactorily. However, construction procedures must be such that severe deformations do not occur during construction.

12.6.2.4 SOIL DESIGN

12.6.2.4.1 Granular type soils shall be used as structure backfill (the envelope next to the metal structure). The order of preference of acceptable structure backfill materials is as follows:

(a) Well-graded sand and gravel; sharp, rough or angular if possible.

(b) Uniform sand or gravel.

(c) Approved stabilized soil shall be used only under direct supervision of a competent, experienced soils engineer. Plastic soils shall not be used.

12.6.2.4.2 The structure backfill material shall conform to one of the following soil classifications from AASHTO Specification M 145, Table 2: for height of fill less than 12 feet, A-1, A-3, A-2-4 and A-2-5; for height of fill of 12 feet and more, A-1, A-3. Structure backfill shall be placed and compacted to not less than 90 percent density per AASHTO T 180.

12.6.2.4.3 The extent of the select structural backfill about the barrel is dependent on the quality of the adjacent embankment. For ordinary installations, with good quality, well compacted embankment or in situ soil adjacent to the structure backfill, a width of structural backfill six feet beyond the structure is sufficient. The structure backfill shall also extend to an elevation two to four feet over the structure.

12.6.2.4.4 It shall not be necessary to excavate native soil at the sides if the quality of the native soil is as good as the proposed compacted side fill except to create the minimum width that can be compacted. The soil over the top shall also be select and shall be carefully and densely compacted.

TABLE 12.6.1A

MINIMUM REQUIREMENTS FOR LONG SPAN STRUCTURES WITH ACCEPTABLE SPECIAL FEATURES

I. TOP ARC MINIMUM THICKNESS

	TOP RADIUS IN FT				
	15	15-17	17-20	20-23	23-25
6 X 2 Corrugated Steel Plates	.109"	.138"	.168"	.218"	.249"

II. MINIMUM COVER IN FEET

	TOP RADIUS IN FT.				
Steel Thickness[1] in inches	15	15-17	17-20	20-23	23-25
.109	2.5				
.138	2.5	3.0			
.168	2.5	3.0	3.0		
.188	2.5	3.0	3.0		
.218	2.0	2.5	2.5	3.0	
.249	2.0	2.0	2.5	3.0	4.0
.280	2.0	2.0	2.5	3.0	4.0

III. GEOMETRIC LIMITS

A. Maximum Plate Radius - 25 Ft.
B. Maximum Central Angle of Top Arc = 80^{o}
C. Minimum Ratio, Top Arc Radius to Side Arc Radius = 2
D. Maximum Ratio, Top Arc Radius to Side Arc Radius = 5*

*Note: Sharp radii generate high soil bearing pressures Avoid high ratios when significant heights of fill are involved.

IV. SPECIAL DESIGNS

Structures not described herein shall be regarded as special designs.

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[1]When reinforcing ribs are used the moment of inertia of the composite section shall be equal to or greater than the moment of inertia of the minimum plate thickness shown.

12.6.3 STRUCTURAL PLATE SHAPES

STANDARD TERMINOLOGY OF STRUCTURAL PLATE SHAPES
INCLUDING LONG SPAN STRUCTURES

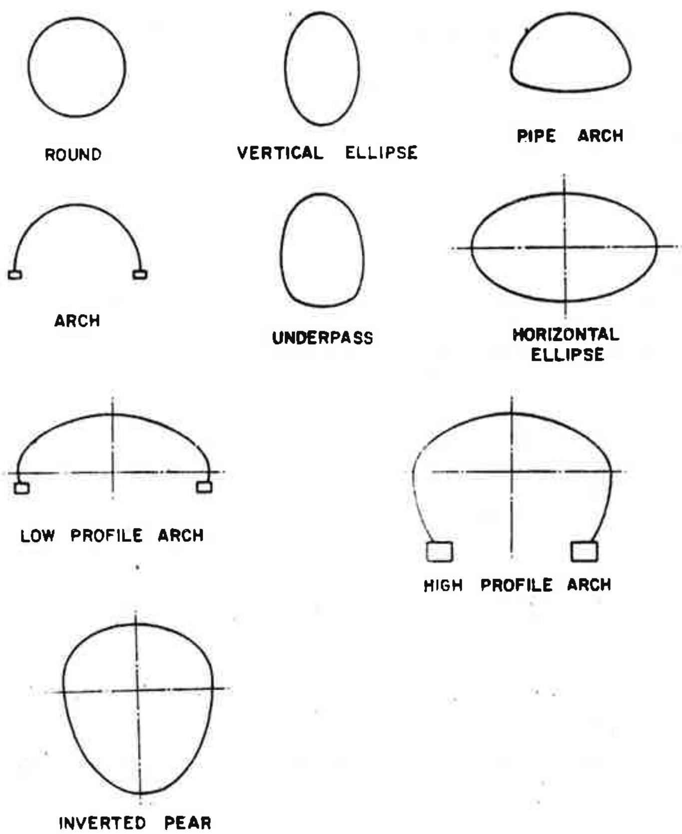


FIGURE 12.6.1A

12.6.4 END TREATMENT

When headwalls are not used, special attention may be necessary at the ends of the structure. Severe bevels and skews are not recommended. For hydraulic structures, additional reinforcement of the end is recommended to secure the metal edges at inlet and outlet against hydraulic forces. Reinforced concrete or structural steel collars, tension tie-backs or anchors in soil, partial headwalls and cut-off walls below invert elevation are some of the methods which can be used. Square ends may have side plates beveled up to a maximum 2:1 slope. Skew ends up to 15^{0} with no bevel, are permissible but when this is done on spans over 20 feet the cut edge must be reinforced with a reinforced concrete or structural steel collar. When full headwalls are used and they are skewed, the offset portion of the metal structure shall be supported by the headwall. A special headwall shall be designed for skews exceeding 15^{0}. The maximum skew shall be limited to 35^{0}.

12.6.5 MULTIPLE STRUCTURES

Care must be exercised on the design of multiple, closely spaced structures to control unbalanced loading. Fills should be kept level over the series of structures when possible. Significant roadway grades across a series of structures require checking of the stability of the flexible structures under the resultant unbalanced loading.

13.1. GENERAL AND NOTATIONS

13.1.1 GENERAL

The following information on timber design is based on the National Design Specification for Wood Construction (NDS), 1982 Edition. See the latest edition of the NDS for additional information.

All wood used in timber structures shall be preservatively treated as provided in Section 20 - Division II unless otherwise specified.

The hardware for structures on the seacoast shall be galvanized or cadmium plated.

13.1.2 NOTATIONS

"a" = fixity condition in which the centroid of connectors or a connector group lies within one-twentieth of ℓ_1, from the column end (Article 13.6.3)

"b" = fixity condition in which the centroid of connectors or a connector group lies between one-twentieth and one-tenth of ℓ_1, from the column end (Article 13.6.3)

b = width of beam or glued laminated deck panel (Article 13.3.1)

b = width of cross section (Article 13.3.2.2)

b = breadth (width) of rectangular member (Article 13.3.4)

C_F = size factor (Article 13.3.7)

C_k = limiting value of slenderness factor (Article 13.3.8)

C_s = slenderness factor (Article 13.3.8)

C_x = constant for fixity condition "a" or "b" (Article 13.6.4)

d = depth of beam or glued laminated deck panel (Article 13.3.1)

d = depth of cross section (Article 13.3.2.2)

d = depth of beam above notch (Article 13.3.4)

d = depth of member (Article 13.3.7)

d = depth of beam (Article 13.3.8)

d = least dimension of a compression member (Article 13.4.6)

d_1 = dimension of column in plane of lateral support corresponding to length ℓ_1 (Article 13.5)

d_2 = dimension of column in plane of lateral support corresponding to length ℓ_2 (Article 13.5)

E = modulus of elasticity (Article 13.2)

F_b = allowable unit stress in bending (Article 13.2)

F_b' = allowable unit stress in bending based on slenderness considerations (Article 13.3.8.3)

F_c = allowable unit stress in compression parallel to grain (Article 13.5.2)

$F_{c\perp}$ = allowable unit stress in compression perpendicular to grain (Article 13.2)

F_c' = allowable unit stress in compression parallel to grain adjusted for slenderness ratio (Article 13.4.4)

F_t = allowable unit stress in tension parallel to grain (Article 13.2)

F_v = allowable unit stress in horizontal shear (Article 13.3.4)

f_r = actual radial stress (Article 13.3.2.2)

f_v = actual horizontal shear stress (Article 13.3.1)

h = total depth of beam (Article 13.3.4)

K = limiting value of slenderness ratio (Article 13.5.2)

L = length of bearing measured along the grain of the wood (Article 13.3.3)

ℓ = unsupported length (Article 13.3.8 and 13.5)

ℓ_e = effective length of beam (Article 13.3.8)

ℓ_1 = distance between points of lateral support of column in plane 1 (Article 13.5)

ℓ_2 = distance between points of lateral support of column in plane 2 (Article 13.5)

ℓ_1 & ℓ_2 = distance from center to center of lateral supports of continuous spaced columns from end to end of simple spaced columns (Article 13.6)

ℓ_3 = distance from center to center of connectors in end blocks to center of spacer block (Article 13.6)

M = bending or resisting moment (Article 13.3.2.2)

N = unit bearing on an inclined surface (Article 13.3.5)

P = unit stress in compression parallel to grain (Article 13.3.5)

Q = unit stress in compression perpendicular to grain (Article 13.3.5)

R = radius of curvature of lamination (Article 13.3.2.1)

R = radius of curvature at centerline of member (Article 13.3.2.2)

R = maximum end load (Article 13.3.4)

t = thickness of a lamination (Article 13.3.2.1)

V = vertical shear (Article 13.3.1)

θ = angle in degrees between the direction of load and the direction of grain (Article 13.3.5)

13.2 ALLOWABLE STRESSES

13.2.1 STRESS-GRADE LUMBER

13.2.1.1 The allowable unit stresses given in Table 13.2.1A are for normal duration of loading for stress grades of sawn lumber used under continuously dry conditions as in most covered structures. For lumber used under service conditions in which the moisture content of the wood is above 19 percent, as when continuously submerged, the allowable unit stresses shall be reduced as indicated in the footnotes to Table 13.2.1A.

13.2.1.2 Use of stress grades in flexure:

13.2.1.2.1 The allowable unit stresses in flexure for Joist and Plank grades apply to material with the load applied to either the narrow or wide face.

13.2.1.2.2 The allowable unit stresses in flexure for Beam and Stringer grades apply only to material with the load applied to the narrow face.

13.2.1.2.3 Beam grades ordinarily are graded for use on simple spans. When used as a continuous beam the grading provisions customarily applied to the middle third of the length of simple spans shall be applied to the middle two-thirds of the lengths of pieces to be used over double spans and to the entire length of pieces to be used over three or more spans.

Table 13.2.1A

ALLOWABLE UNIT STRESSES FOR STRUCTURAL LUMBER - VISUALLY GRADED

Species and Commercial Grade	Size Classification	Allowable Unit Stress in Pounds per Square Inch[1] — Extreme Fiber in Bending "F_b" — Single-Member Uses	Extreme Fiber in Bending "F_b" — Repetitive Member Uses	Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"	Modulus of Elasticity "E"	Grading Rules Agency
DOUGLAS FIR-LARCH (Surfaced dry or surfaced green. Used at 19% max. m.c.)									
Dense Select Structural	2" to 4" thick 2" to 4" wide	2,450	--	1,400	95	455	1,850	1,900,000	West Coast Lumber Inspection Bureau and Western Wood Products Association (See footnotes 2 through 9)
Select Structural		2,100	--	1,200	95	385	1,600	1,800,000	
Dense No. 1		2,050	--	1,200	95	455	1,450	1,900,000	
No. 1		1,750	--	1,050	95	385	1,250	1,800,000	
Dense No. 2		1,700	--	1,000	95	455	1,150	1,700,000	
No. 2		1,450	--	850	95	385	1,000	1,700,000	
No. 3		800	--	475	95	385	600	1,500,000	
Dense Select Structural	2" to 4" thick 5" and wider	2,100	--	1,400	95	455	1,650	1,900,000	
Select Structural		1,800	--	1,200	95	385	1,400	1,800,000	
Dense No. 1		1,800	--	1,200	95	455	1,450	1,900,000	
No. 1		1,500	--	1,000	95	385	1,250	1,800,000	
Dense No. 2		1,450	--	775	95	455	1,250	1,700,000	
No. 2		1,250	--	650	95	385	2,050	1,700,000	
No. 3		725	--	375	95	385	675	1,500,000	
Dense Select Structural	Beams and Stringers	1,900	--	1,100	85	455	1,300	1,700,000	West Coast Lumber Inspection Bureau (See footnotes 2 through 9)
Select Structural		1,600	--	950	85	385	1,100	1,600,000	
Dense No. 1		1,550	--	775	85	455	1,100	1,700,000	
No. 1		1,200	--	825	85	385	1,000	1,600,000	
Dense Select Structural	Posts and Timbers	1,750	--	1,150	85	455	1,350	1,700,000	
Select Structural		1,500	--	1,000	85	385	1,150	1,600,000	
Dense No. 1		1,400	--	950	85	455	1,200	1,700,000	
No. 1		1,200	--	825	85	385	1,000	1,600,000	
Select Dex	Decking	1,750	2,000	--	--	385	--	1,800,000	
Commercial Dex		1,450	1,650	--	--	385	--	1,700,000	

Table 13.2.1A

ALLOWABLE UNIT STRESSES FOR STRUCTURAL LUMBER - VISUALLY GRADED
(continued)

Species and Commercial Grade	Size Classification	Allowable Unit Stress in Pounds per Square Inch[1]						Modulus of Elasticity "E"	Grading Rules Agency
		Extreme Fiber in Bending "F_b"		Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"		
		Single-Member Uses	Repetitive Member Uses						
Dense Select Structural	Beams and Stringers	1,900	--	1,250	85	455	1,300	1,700,000	
Select Structural		1,600	--	1,050	85	385	1,100	1,600,000	
Dense No. 1		1,550	--	1,050	85	455	1,100	1,700,000	
No. 1		1,350	--	900	85	385	925	1,600,000	
Dense Select Structural	Posts and Timbers	1,750	--	1,150	85	455	1,350	1,700,000	Western Wood Products Association
Select Structural		1,500	--	1,000	85	385	1,150	1,600,000	
Dense No. 1		1,400	--	950	85	455	1,200	1,700,000	(See footnotes 2 through 9)
Selected Decking	Decking	--	2,000	--	--	--	--	1,800,000	
Commercial Decking		--	1,650	--	--	--	--	1,700,000	
Selected Decking	Decking	--	2,150	--	--	(Stress apply at 15% moisture content)		1,800,000	
Commercial Decking		--	1,800	--	--			1,700,000	
HEM-FIR (Surfaced dry or surfaced green. Used at 19% max. m.c.)									
Select Structural	2" to 4" thick 2" to 4" wide	1,650	--	975	75	245	1,300	1,500,000	West Coast Lumber Inspection Bureau and Western Wood Products Association
No. 1		1,400	--	825	75	245	1,050	1,500,000	
No. 2		1,150	--	675	75	245	825	1,400,000	
No. 3		650	--	375	75	245	500	1,200,000	
Select Structural	2" to 4" thick 5" and wider	1,400	--	950	75	145	1,150	1,500,000	(See footnotes 2 through 9)
No. 1		1,200	--	800	75	245	1,060	1,500,000	
No. 2		1,000	--	525	75	245	875	1,400,000	
No. 3		575	--	300	75	245	550	1,200,000	
Select Structural	Beams and Stringers	1,300	--	750	70	245	925	1,300,000	
No. 1		1,050	--	525	70	245	750	1,300,000	West Coast Lumber Inspection Bureau
Select Structural	Posts and Timbers	1,200	--	800	70	245	975	1,300,000	
No. 1		975	--	650	70	245	850	1,300,000	(See footnotes 2 through 9)
Select Dex	Decking	1,400	1,600	--	--	245	--	1,500,000	
Commercial Dex		1,150	1,350	--	--	245	--	1,400,000	
Select Structural	Beams and Stringers	1,250	--	850	70	245	925	1,300,000	
No. 1		1,050	--	725	70	245	775	1,300,000	
Select Structural	Posts and Timbers	1,200	--	800	70	245	975	1,300,000	Western Wood Products Association
No. 1		950	--	650	70	245	850	1,300,000	
Selected Decking	Decking	--	1,600	--	--	--	--	1,500,000	(See footnotes 2 through 9)
Commercial Decking		--	1,350	--	--	--	--	1,400,000	
Selected Decking	Decking	--	1,700	--	--	(Stresses apply at 15% moisture content)		1,600,000	
Commercial Decking		--	1,450	--	--			1,400,000	

Table 13.2.1A

ALLOWABLE UNIT STRESSES FOR STRUCTURAL LUMBER - VISUALLY GRADED
(continued)

Species and Commercial Grade	Size Classification	Allowable Unit Stress in Pounds per Square Inch[1]: Extreme Fiber in Bending "F_b" Single-Member Uses	Extreme Fiber in Bending "F_b" Repetitive Member Uses	Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"	Modulus of Elasticity "E"	Grading Rules Agency
SOUTHERN PINE (Surfaced dry. Used at 19% max. m.c.)									
Selected Structural	2" to 4" thick	2,000	--	1,150	100	405	1,550	1,700,000	Southern Pine Inpsection Bureau
Dense Select Structural	2" to 4" wide	2,350	--	1,350	100	475	1,800	1,800,000	
No. 1		1,700	--	1,000	100	405	1,250	1,700,000	
No. 1 Dense		2,000	--	1,150	100	475	1,450	1,800,000	
No. 2		1,400	--	825	90	405	975	1,600,000	
No. 2 Dense	2" to 4" thick	1,650	--	975	90	475	1,150	1,600,000	
No. 3	2" to 4" wide	775	--	450	90	405	575	1,400,000	
No. 3 Dense		925	--	525	90	475	675	1,500,000	
Select Structural	2" to 4" thick	1,750	--	1,150	90	405	1,350	1,700,000	Southern Pine Inspection Bureau
Dense Select Structural	5" and wider	2,050	--	1,300	90	475	1,600	1,800,000	(See footnotes 2 through 9)
No. 1		1,450	--	975	90	405	1,250	1,700,000	
No. 1 Dense		1,700	--	1,150	90	475	1,450	1,800,000	
No. 2		1,200	--	625	90	405	1,000	1,600,000	
No. 2 Dense		1,400	--	725	90	475	1,200	1,600,000	
No. 3		700	--	350	90	405	625	1,400,000	
No. 3 Dense		825	--	425	90	475	725	1,500,000	
No. 1 SR	5" and thicker	1,350	--	875	110	270	775	1,500,000	
No. 1 Dense SR		1,550	--	1,050	110	315	925	1,600,000	
No. 2 SR		1,100	--	725	95	270	625	1,400,000	
No. 2 Dense SR		1,250	--	850	95	315	725	1,400,000	
Dense Standard Decking	2-1/2" to 4" thick	--	1,800	--	--	--	--	1,600,000	
Select Decking	2" and wider decking	--	1,300	--	--	--	--	1,400,000	
Dense Select Decking		--	1,500	--	--	--	--	1,400,000	
Commercial Decking		--	1,300	--	--	--	--	1,400,000	
Dense Commercial Decking		--	1,500	--	--	--	--	1,400,000	

(The allowable unit stresses above are for normal loading conditions. See other provisions of Article 13.2 for adjustments of these tabulated allowable unit stresses.)

NOTE: This represents only a partial listing of available species and grades taken from the National Design Specification for Wood Construction (NDS), 1982 Edition. For a complete listing, see the latest edition of the NDS, NFPA.

FOOTNOTES FOR TABLE 13.2.1A

[1]The allowable unit stresses shown are for selected species and commercial grades. For stresses for other species and commercial grades not shown, the designer is referred to the grading rules of the appropriate grading rules agency, or the NDS.

[2]The recommended design values shown in Table 13.2.1A are applicable to lumber that will be used under dry conditions such as in most covered structures. For 2" to 4" thick lumber the DRY surfaced size should be used. In calculating design values, the natural gain in strength and stiffness that occurs as lumber dries has been taken into consideration as well as the reduction in size that occurs when unseasoned lumber shrinks. The gain in load carrying capacity due to increased strength and stiffness resulting from drying more than offsets the design effect of size reductions due to shrinkage. For 5" and thicker lumber, the surfaced sizes also may be used because design values have been adjusted to compensate for any loss in size by shrinkage which may occur.

[3]Tabulated tension parallel to grain values for all species 5" and wider, 2" to 4" thick size classifications apply to 5" and 6" widths only, for grades of Select Structural, No. 1, No. 2, and No. 3 (including dense grades). For lumber wider than 6" in these grades, the tabulated "F_t" values shall be multiplied by the following factors:

Grade	Multiply Tabulated "F_t" values By		
2" to 4" thick, 5" and wider (includes "Dense" grades)	5" and 6" wide	8" wide	10" and wider
Select Structural	1.00	0.90	0.80
No. 1, No. 2, and No. 3	1.00	0.80	0.60

[4]The values in Table 13.2.1A are based on edgewide use. For dimension 2" to 4" in thickness, when used flatwise, the recommended design values for fiber stress in bending may be multiplied by the following factors:

Dimension Lumber Used Flatwise			
Width	Thickness		
	2"	3"	4"
2" to 4"	1.10	1.04	1.00
5" and wider	1.22	1.16	1.11

[5]When 2" and 4" thick lumber is manufactured at a maximum moisture content of 15 percent and used in a condition where the moisture content does not exceed 15 percent, the design values shown in Table 13.2.1A may be multiplied by the following factors: (For Southern Pine use tabulated design values without adjustment.)

FOOTNOTES FOR TABLE 13.2.1A
(continued)

Extreme Fiber in Bending "F_b"	Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"	Modulus of Elasticity "E"
1.08	1.08	1.05	1.00	1.17	1.05

[6]When 2" to 4" thick lumber is designed for use where the moisture content will exceed 19 percent for an extended period of time, the values shown in Table 13.2.1A should be multiplied by the following factors:

Extreme Fiber in Bending "F_b"	Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"	Modulus of Elasticity "E"
Douglas Fir-Larch and Hem-Fir					
0.86	0.84	0.97	0.67	0.70	0.97
Southern Pine (factors apply to 2" to 4" thick, 5" and wider values)					
0.80	0.80	0.95	0.67	0.66	0.83

[7]When lumber 5" and thicker is designed for use where the moisture content will exceed 19 percent for an extended period of time, the values shown in Table 13.2.1A except for Southern Pine should be multiplied by the following factors:

Extreme Fiber in Bending "F_b"	Tension Parallel to Grain "F_t"	Horizontal Shear "F_v"	Compression Perpendicular to Grain "$F_{c\perp}$"	Compression Parallel to Grain "F_c"	Modulus of Elasticity "E"
1.00	1.00	1.00	0.67	0.91	1.00

[8]When lumber 4" and thinner is manufactured unseasoned, the tabulated values should be multiplied by a factor of 0.92 (except Southern Pine).

[9]Stress rated boards of nominal 1", 1-1/4", and 1-1/2", thickness, 2" and wider of most species are permitted the recommended design values shown for Select Structural No. 1, No. 2, and No.3 grades as shown in 2" to 4" thick categories when graded in accordance with the stress board provisions in the applicable grading rules. Information on stress rated board grades applicable to the various species is available from the respective grading standards.

13.2.1.3 Modification for bearing perpendicular to grain:

The allowable unit stresses for compression perpendicular to the grain assume that the material will be surface seasoned when installed. When used under continuously wet conditions, the tabulated values should be reduced one-third.

13.2.2 GLUED LAMINATED TIMBER

13.2.2.1 The allowable unit stresses for softwood species shall be as recommended in American Institute of Timber Construction (AITC) 117-79 "Standard Specifications for Structural Glued Laminated Timber of Softwood Species," and as given in Tables 13.2.2A and 13.2.2B of this Specification. For hardwood species, the allowable unit stresses shall be as given in Table 2.11 of the "Timber Construction Manual," 1974 Edition, by the American Institute of Timber Construction published by John Wiley & Sons, New York City, New York.

13.2.2.2 The stress tables given in AITC 117-82 are for dry-use conditions; factors are included for wet-use conditions. Allowable unit stresses for dry-use conditions are applicable when the moisture content in service is less than 16 percent as in most covered structures. Allowable unit stresses for wet-use conditions are applicable when the moisture content in service is 16 percent or more, as may occur in exterior or submerged construction, and in some structures housing wet processes or otherwise having constant high relative humidities.

13.2.2.3 The stress tables in AITC 117-82 give stresses for members stressed primarily in bending (load applied perpendicular to the wide face of the lamination) and for members stressed primarily in axial tension, axial compression or loaded in bending parallel or perpendicular to the wide face of lamination.

13.2.2.4 The type and location of end joints, and other requirements, together with certain manufacturing requirements, must be met for these allowable unit stresses to apply. The requirements for slope of grain for softwoods are given in AITC 117-82; those for hardwoods are incorporated in Table 2.11 of the AITC "Timber Construction Manual," 1974 Edition. Other requirements are given in U.S. Product Standard PS 56-73.

13.2.2.5 Species other than those specifically included in these Specifications may be used provided allowable unit stresses are established for them in accordance with U.S. Product Standard PS 56-73.

13.2.3 ALLOWABLE STRESSES FOR NORMAL LOADING CONDITIONS

The allowable unit stresses are for normal load duration in which a member is fully stressed by the application, either continuously or cumulatively, of the full design load for a duration of approximately ten years. For other loading conditions, adjustments should be made as given in the following sections.

Table 13.2.2A

ALLOWABLE UNIT STRESSES FOR STRUCTURAL GLUED LAMINATED TIMBER, MEMBERS STRESSED PRINCIPALLY IN BENDING, LOADING PERPENDICULAR TO THE WIDE FACE OF THE LAMINATIONS[1,2,3]

Allowable Unit Stresses for Dry Conditions of Use[4], psi								
					Compression ⊥ to Grain			
Combination Symbol	Number of Laminations	Extreme Fiber in Bending F_b	Tension Parallel to Grain F_t	Compression Parallel to Grain F_c	Tension Face $F_{c\perp}$	Compression Face $F_{c\perp}$	Horizontal Shear F_v	Modulus of Elasticity E
(1) Douglas Fir and Western Larch[5]								
20F-V3	4 or more	2,000	1,000	1,550	450	385	165	1,600,000
24F-V4	4 or more	2,400	1,150	1,650	450	450	165	1,800,000
(2) Southern Pine								
20F-V2	4 or more	2,000	1,050	1,550	450	385	200	1,600,000
24F-V3	4 or more	2,400	1,150	1,700	450	450	200	1,800,000

(Stresses shown above are for normal conditions of loading. See other provisions of Article 13.2 for adjustments of these tabulated allowable unit stresses.)

NOTE: The following allowable unit stresses are selected from AITC 117-82. For more complete information see the latest edition of AITC 117.

FOOTNOTES FOR TABLE 13.2.2A

[1]The tabulated stresses in this table are primarily applicable to members stressed in bending due to a load applied perpendicular to the wide faces of the laminations. For combinations and stresses applicable to members loaded primarily axially or parallel to the wide faces of the laminations, see Table 13.2.2B.

[2]The tabulated bending stresses are applicable to members 12 inches or less in depth. For members greater than 12 inches in depth, the requirements of Article 13.3 on Size Factor apply.

[3]The tabulated combinations are applicable to arches, compression members, tension members and bending members.

[4]To obtain wet-use stresses, the following factors are applied to the dry-use stresses.

Type of Stress	Wet-Use Factor
Bending and tension parallel to grain	0.8
Compression parallel to grain	0.73
Compression perpendicular to grain	0.667
Shear	0.875
Modulus of Elasticity	0.833

[5]If these combinations are to be preservatively treated, AWPA C28 permits only Pacific Coast Douglas Fir to be used.

Table 13.2.2B

ALLOWABLE UNIT STRESSES FOR STRUCTURAL GLUED LAMINATED TIMBER, MEMBERS STRESSED PRINCIPALLY IN AXIAL TENSION OR AXIAL COMPRESSION, OR A COMBINATION OF AXIAL LOADING PLUS BENDING PARALLEL TO OR PERPENDICULAR TO THE WIDE FACE OF THE LAMINATIONS[1]

Allowable Unit Stresses for Dry Conditions of Use[5], psi									
		Tension	Compression	Extreme Fiber in Bending when Loaded		Compression	Horizontal Shear F_v when Loaded		
Combination Symbol	Number of Laminations	Parallel to Grain F_t	Parallel to Grain F_c	Parallel to Wide Face[3]	Perpendicular to Wide Face[2,4]	Perpendicular to Grain F_c	Parallel to Wide Face[3]	Perpendicular to Wide Face[4]	Modulus of Elasticity E
(1) Douglas Fir and Western Larch[6]									
1	4 or more	900	1,550	1,450	1,250	385	145	165	1,500,000
2		1,250	1,900	1,800	1,700	385	145	165	1,700,000
3		1,450	2,300	2,100	2,000	450	145	165	1,800,000
4		1,400	2,100	2,200	1,900	410	145	165	1,900,000
5		1,600	2,400	2,400	2,200	450	145	165	2,000,000
(2) Southern Pine									
46	4 or more	900	1,500	1,450	1,000	385	175	200	1,300,000
47		1,200	1,900	1,750	1,400	385	175	200	1,400,000
48		1,400	2,200	2,000	1,600	450	175	200	1,700,000
49		1,350	2,100	1,950	1,800	385	175	200	1,700,000
50		1,550	2,300	2,300	2,100	450	175	200	1,900,000

(Stresses shown above are for normal conditions of loading. See other provisions of Article 13.2 for adjustments of these tabulated allowable unit stresses.)

NOTE: The above allowable unit stresses are selected from AITC 117-82. For more complete information see the latest edition of AITC 117.

FOOTNOTES FOR TABLE 13.2.2B

[1]The tabulated stresses in this table are primarily applicable to members loaded axially or parallel to the wide face of the laminations. For combinations and stresses applicable to members stressed principally in bending due to a load applied perpendicular to the wide face of the laminations, see Table 13.2.2A.

[2]It is not intended that these combinations be used for deep bending members, but if bending members 16-1/4" or deeper are used, the applicable AITC tension lamination requirements must be followed.

[3]The tabulated stresses are applicable to members containing three (3) or more laminations.

[4]The tabulated stresses are applicable to members containing four (4) or more laminations.

[5]To obtain wet-use stresses, the following factors are applied to the dry-use stresses:

Type of Stress	Wet-Use Factor
Bending and tension parallel to grain	0.8
Compression parallel to grain	0.73
Compression perpendicular to grain	0.667
Shear	0.875
Modulus of Elasticity	0.833

[6]If these combinations are to be preservatively treated, AWPA C28 permits only Pacific Coast Douglas Fir to be used.

13.2.4 ALLOWABLE STRESSES FOR PERMANENT LOADING

When a member is fully stressed for periods greater than ten years, either continuously or cumulatively, 90 percent of the tabulated allowable unit stresses shall be used. The provisions of this paragraph do not apply to modulus of elasticity.

13.2.5 ALLOWABLE STRESSES FOR WIND, EARTHQUAKE OR SHORT TERM LOADING

13.2.5.1 When full maximum load resulting from the short term loads specified below has a duration that does not exceed the period indicated, the tabulated unit stresses shall be increased as follows:

15 percent for static load for 2 months duration, such as snow or ice
25 per cent for static load for 7 days duration, such as snow or ice
33 1/3 percent for wind or earthquake
65 percent for 5 minute duration (railing loads only)

13.2.5.2 The above increases are not cumulative. The resulting structural members shall not be smaller than required for a longer duration of loading. The provisions of this paragraph do not apply to modulus of elasticity, but do apply to mechanical fastenings except as otherwise noted.

13.2.6 COMBINED STRESSES

13.2.6.1 These specifications do not cover the application of loads which produce combined axial and bending stresses, or the effective reductions in the tabulated stresses as a result of these loads.

13.2.6.2 For this condition, attention is directed to Section 4, page 10, of the "Timber Construction Manual," 1974, by the American Institute of Timber Construction, published by John Wiley & Sons, New York, N.Y.

13.3 FORMULAS FOR THE COMPUTATION OF STRESSES IN TIMBER

In calculating live load stresses in timber, impact shall be neglected. See Article 3.8.1.

13.3.1 HORIZONTAL SHEAR IN BEAMS

Horizontal shear in beams shall be computed from the maximum shear occurring at a distance from the support equal to three times the depth of the beam, or at the quarter point, whichever is the lesser distance from the support. The live load used in computing horizontal shear shall be placed to produce maximum external shear at this distance from the support. This external live load shear shall be one-half the sum of 60 percent of the shear from the undistributed wheel loads and the shear from the wheel loads distributed laterally as specified for moment in Article 3.23. For undistributed wheel loads, one line of wheels is assumed to be carried by one beam.

The shear shall be calculated according to the following formula:

$$f_v = \frac{3V}{2bd} \tag{13-1}$$

where

f_v = horizontal shear stress in pounds per square inch
b = width of beam or glued laminated deck panel in inches
d = depth of beam or glued laminated deck panel in inches
V = vertical shear in pounds

13.3.2 SECONDARY STRESSES IN CURVED GLUED LAMINATED MEMBERS

13.3.2.1 CURVATURE FACTOR

13.3.2.1.1 For the curved portion of members, the allowable stress in bending shall be modified by multiplication by the following curvature factor:

$$1-2{,}000(t/R)^2 \tag{13-2}$$

in which

t = thickness of lamination in inches
R = radius of curvature of a lamination in inches

13.3.2.1.2 t/R should not exceed 1/125 for Douglas fir and larch and 1/100 for Southern Pine.

13.3.2.1.3 No curvature factor shall be applied to stress in the straight portion of an assembly regardless of curvature elsewhere.

13.3.2.2 RADIAL TENSION OR COMPRESSION

13.3.2.2.1 The radial stress, f_r, induced by a bending moment in a curved member of constant cross section, shall be defined by the following equation:

$$f_r = 3M/2Rbd \tag{13-3}$$

where

M = bending moment in inch pounds
R = radius of curvature at centerline of member in inches
b = width of cross section in inches
d = depth of cross section in inches

13.3.2.2.2 When M is in the direction tending to decrease curvature (increase the radius), the stress is tension across the grain. For this condition, the tensile stress across the grain shall be limited to 1/3 the allowable unit stress in horizontal shear for Southern pine under all load conditions and for Douglas fir and larch under wind or earthquake loadings. The limit shall be 15 psi for Douglas fir and larch under other types of load. These values are subject to modifications for duration of load. If these values are exceeded, mechanical reinforcing shall be used and shall be sufficient to resist all radial tension stresses.

13.3.2.2.3 When M is in the direction tending to increase curvature (decrease the radius), the stress is compression across the grain and shall be limited to the allowable unit stress in compression perpendicular to the grain for all species included herein.

13.3.3 COMPRESSION OR BEARING PERPENDICULAR TO GRAIN

13.3.3.1 The allowable unit stresses for compression perpendicular to the grain shall apply to bearings of any length at the ends of the beam, and to all bearings 6 inches or more in length at any other location. When calculating the bearing area at the ends of beams, no allowance shall be made for the fact that, as the beam bends, the pressure upon the inner edge of the bearing is greater than at the end of the beam.

13.3.3.2 For bearings of less than 6 inches in length and not nearer than 3 inches to the end of a member, the maximum allowable load per square inch shall be the allowable unit stresses in compression perpendicular to grain multiplied by the following factor:

$$\frac{L+3/8}{L} \qquad (13\text{-}4)$$

where L is the length of bearing in inches measured along the grain of the wood which for round washers or other round bearing areas shall be the diameter.

13.3.3.3 The multiplying factors for lengths of bearing on such small areas as plates and washers are given in Table 13.3.1A.

Table 13.3.1A

Length of Bearing (in inches)	1/2	1	1 1/2	2	3	4	6 or more
Factor	1.75	1.38	1.25	1.19	1.13	1.10	1.00

13.3.4 NOTCHED BEAMS

Beams notched in the bearing face on supports shall be limited to maximum end load R as determined by the formula:

$$R = \frac{2bd^2F_v}{3h} \tag{13-5}$$

where

R = maximum end load
F_v = allowable unit horizontal shear stress
b = breadth of beam
d = depth of beam above the notch
h = total depth of beam

13.3.5 BEARING ON INCLINED SURFACES

$$N = \frac{PQ}{P \sin^2\theta \ +Q \cos^2\theta} \tag{13-6}$$

where

N = unit bearing on an inclined surface
P = unit stress in compression parallel to the grain
Q = unit stress in compression perpendicular to the grain
θ = angle in degrees between the direction of load and the direction of grain

13.3.6 TIMBER CONNECTORS

13.3.6.1 Timber connectors shall consist of devices to be used at surfaces of contact in bolted timber joints, to increase the strength or shear resistance of wood-to-wood or wood-to-steel connections.

13.3.6.2 Allowable loads, spacing, edge and end distances, bolt and washer sizes and other details of design shall be those recommended or approved by the "National Design Specification for Wood Construction," National Forest Products Association. Alternatively, the allowable loads may be determined by actual tests of full size joints for each condition of connector used in accordance with standard procedure.

13.3.7 SIZE FACTOR

13.3.7.1 When the depth of a rectangular beam exceeds 12 inches the tabulated unit stress in bending, F_b, shall be reduced by multiplying the tabulated stress by the size factor, C_F, as determined from the following relationship

$$C_F = (12/d)^{1/9} \qquad (13\text{-}7)$$

where

C_F = size factor
d = depth of member in inches

13.3.7.2 The size factor relationship as given above is applicable to a bending member satisfying the following basic assumptions: (a) simply supported beam, (b) uniformly distributed load, and (c) span to depth ratio (ℓ/d) of 21. This factor can thus be applied with reasonable accuracy to most commonly encountered design situations. Where greater accuracy is desired for other sizes and conditions of loading, the percentage changes given in the following table may be applied directly to the size factor calculated for the basic assumptions. Straight line interpolation may be used for other ℓ/d ratios.

Span to Depth Ratio ℓ/d	Percent Change	Loading Condition for Simply Supported Beams	Percent Change
14	+2.3	Center Point	+7.8
28	-1.6	Third Point	-3.2

13.3.7.3 For more detailed analysis of the size factor and its application to the design of bending members, the designer is referred to the AITC "Timber Construction Manual." The reduction in bending stresses for deep members based on the size factor is applicable to both glued laminated members and sawn lumber.

13.3.8 LATERAL STABILITY

13.3.8.1 GENERAL

The tabulated allowable unit bending stresses given under Article 13.2 are applicable to members which are adequately braced. When deep, slender members which are not adequately braced are used, a reduction based on a computation of the slenderness factor of the member must be applied to the allowable unit bending stresses. In the check of lateral stability, the slenderness factor is computed by the relationship:

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} \qquad (13\text{-}8)$$

where

C_s = slenderness factor
ℓ_e = effective length of beam, in. (See Table 13.3.8A)
d = depth of beam, in.
b = breadth of beam, in.

Table 13.3.8A

EFFECTIVE LENGTH OF GLUED LAMINATED BEAMS

Type of Beam Span and Nature of Load	Value of Effective Length, ℓ_e[1]
Single span beam, load concentrated at center	1.61ℓ
Single span beam, uniformly distributed load	1.92ℓ
Single span beam, equal end moments	1.84ℓ
Cantilever beam, load concentrated at unsupported end	1.69ℓ
Cantilever beam, uniformly distributed load	1.06ℓ
Single span or cantilever beam, any load (conservative value)	1.92ℓ

[1]Where ℓ = unsupported length.

13.3.8.2 BEAMS WITH VARIOUS LATERAL SUPPORT CONDITIONS

13.3.8.2.1 When the depth of a beam does not exceed its breadth, no lateral support is required and the allowable unit stress is determined by applying the appropriate provisions of Articles 13.2 and 13.3.

13.3.8.2.2 If lateral movement of the compression flange is prevented by a continuous support, there is no danger of lateral buckling, and the allowable stresses require no reduction based on a slenderness ratio concept. Also, there is no need to limit the depth-breadth ratio to 5 or 6.

13.3.8.2.3 When the depth of a beam exceeds the breadth, bracing must be provided at the points of bearing to prevent rotation of the beam at those points in a plane perpendicular to its longitudinal axis. The allowable stresses are calculated by the formulae contained in the following paragraphs for short, intermediate and long beams.

13.3.8.3 ALLOWABLE STRESSES

The allowable unit stresses are determined from the following equations:

(a) Short beams. When the slenderness factor, C_s, does not exceed 10, the tabular allowable unit stress in bending, F_b, adjusted in accordance with the applicable provisions of Articles 13.2.4, 13.2.5, 13.3.2, and 13.3.7 is used for design.

(b) Intermediate beams. When the slenderness factor, C_s, is greater than 10 but does not exceed C_k, a unit stress in bending based on slenderness considerations, F'_b, is calculated by the formula:

$$F'_b = F_b\left[1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4\right] \qquad (13\text{-}9)$$

where

F_b = tabular allowable unit stress in bending, psi

$$C_k = \sqrt{\frac{3E}{5F_b}} \qquad (13\text{-}10)$$

E = modulus of elasticity, psi

(c) Long Beams. When the slenderness factor, C_s, is greater than C_k, but less than 50, the unit stress in bending is calculated by the formula:

$$F'_b = \frac{0.40E}{(C_s)^2} \qquad (13\text{-}11)$$

In no case shall C_s be greater than 50.

For both intermediate and long beams, the allowable unit stress for design based on slenderness considerations is obtained by adjusting F'_b in accordance with the applicable provisions of Articles 13.2.4, 13.2.5, and 13.3.2.

Regardless of the slenderness classification into which a beam may be categorized, in no case shall the allowable unit stress in bending used for design exceed the value as obtained by adjusting the tabular allowable unit stress based on the applicable provisions of Articles 13.2.4, 13.2.5, 13.3.3, and 13.3.7.

13.3.9 BOLTS

The center to center spacing of bolts shall be greater than 4 times the bolt diameter. The distance from the center of a bolt to the end of any timber shall be not less than 7 times the bolt diameter if loaded in tension parallel to grain, and 4 times the bolt diameter if loaded in compression parallel to the grain or in tension or compression perpendicular to the grain. For tension or compression, loading parallel to grain, the distance from any edge of the timber to the center of the nearest bolt shall be at least 1-1/2 times the bolt diameter, except that for ℓ/d ratios greater than 6, the distance shall be one-half the distance between rows of bolts. For loading perpendicular to grain, the edge distance toward which the load is acting shall be at least 4 times the bolt diameter and the edge distance on the opposite edge shall be at least 1-1/2 bolt diameters.

13.3.10 WASHERS

13.3.10.1 A washer shall be used under all bolt heads (except timber bolts or bolts with button type heads) and nuts which would otherwise come in contact with wood. Either cast or plate washers may be used and they shall be designed to prevent excessive crushing of the wood when the bolts are tightened. For bolts or rods in tension, washers shall be of sufficient size to develop the tension stress in the bolt or rod without exceeding the allowable unit stress in compression perpendicular to grain for the species and grade of lumber used.

13.3.10.2 A standard circular washer shall be used under the heads of all lag screws.

13.4 COMPRESSION MEMBERS

13.4.1 GENERAL

13.4.1.1 No column shall have an unsupported length greater than 50 times its least dimension.

13.4.1.2 The strength of built-up columns composed of two or more pieces bolted together, either with or without packing blocks, shall be considered as equal to the combined strength of the single pieces each considered as an independent column.

13.4.2 DEFINITIONS

13.4.2.1 The term "column" refers to all types of compression members, including members forming part of a truss or other structural components.

13.4.2.2 Simple Solid Wood Columns. Simple columns consist of a single piece or of pieces properly glued together to form a single member. (See Figure 13.5.1A).

13.4.2.3 Connector-Joined Spaced Columns. Spaced columns are formed of two or more individual members with their longitudinal axes parallel, separated at the ends and middle points of their length by blocking and joined at the ends by timber connectors capable of developing the required shear resistance. (See Figure 13.6.1A)

13.4.2.4 Built-up Columns. Built-up columns other than connector-joined spaced columns and glued laminated columns, shall not be designed as solid columns but shall be designed in accordance with NDS, Appendix I, including the reduction factors therein, except that the design values provided herein shall apply.

13.4.3 DESIGN VALUES FOR INTERMEDIATE AND LONG COLUMNS

Intermediate and long columns shall be designed using the allowable stress in compression parallel to grain adjusted for slenderness ratio, F'_c, and the net column section for lengths most liable to buckling or the full column section elsewhere. In addition, the stress, based on the net section, at any point of any column shall not exceed the design value in compression parallel to grain for a short column.

13.4.4 MODIFICATIONS FOR CONDITIONS OF USE

When the allowable unit stress in compression parallel to grain, F_c, and the modulus of elasticity, E, are used in the column formulas, they shall incorporate modifications for the conditions under which the wood is used, as given in Article 13.2.1 and Tables 13.2.1A, 13.2.2A, and 13.2.2B. The allowable unit stress in compression parallel to grain adjusted for slenderness ratio, F'_c, obtained from the column formulas in Articles 13.5 and 13.6 are not subject to further modifications for moisture service conditions, duration of loading, temperature or types of treatment.

13.4.5 MODIFICATIONS FOR DURATION OF LOAD

In the column formulae given in Articles 13.5 and 13.6, the duration of load modifications in Articles 13.2.4 to 13.2.5 shall apply to the allowable unit stress in compression parallel to grain, F_c, but not to the modulus of elasticity, E.

13.4.6 BRACING

13.4.6.1 Column bracing shall be installed where necessary to resist wind or other lateral forces.

13.4.6.2 Where roof joists are used between arches or at the top chords of trusses instead of purlins, the depth, rather than the breadth, of the arch or top chord compression member may be taken as the least dimension, d, in determining the ratio of ℓ to d. The roof joists should be placed so that their upper edges are at least one-half inch above the tops of the arch or chord, but also placed low enough to provide adequate lateral support.

13.4.6.3 When roof joists or planks are placed on top of an arch or top chord of a truss, and are well spiked or otherwise securely fastened to the arch or top chord and to blocking placed between the joists, or when sheathing is nailed properly to the top chord of trussed rafters, the depth of the arch or individual chord members may be used as the least dimension in determining ℓ/d. When not so fastened or blocked, the thickness of the arch or individual chord member shall be used as the least dimension d in determining ℓ/d.

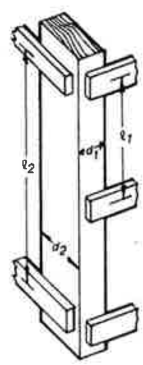
13.5 DESIGN OF SIMPLE SOLID COLUMNS

13.5.1 SLENDERNESS RATIO

13.5.1.1 These formulas for simple solid columns are based on pin-end conditions but may be applied also to square-end conditions.

13.5.1.2 When column end conditions provide less stability than pin-end conditions, the effective length of the column for design purposes shall be increased in accordance with good engineering practice. A reduction in effective length may be made when column end conditions provide greater stability than pin-end conditions.

13.5.1.3 For simple solid columns, the slenderness ratio, ℓ/d, shall be taken as the larger of the ratios, ℓ_1/d_1 and ℓ_2/d_2, and shall not exceed 50.



ℓ_1 and ℓ_2 =
Distances between points of lateral support of column in planes 1 and 2, inches.

d_1 and d_2 =
Dimensions of column in planes of lateral support, inches.

Figure 13.5.1A
Simple Solid Column

13.5.2 MAXIMUM DESIGN COMPRESSIVE STRESS PARALLEL TO GRAIN, F'_c

13.5.2.1 The maximum values of F'_c for square or rectangular simple solid columns shall be determined in accordance with the following formulas.

13.5.2.1.1 For columns having an ℓ/d ratio of 11 or less

$$F'_c = F_c \tag{13-12}$$

13.5.2.1.2 For columns having an ℓ/d ratio greater than 11 but less than K, where:

$$K = 0.671\sqrt{\frac{E}{F_c}} \tag{13-13}$$

$$F'_c = F_c\left[1 - \frac{1}{3}\left(\frac{\ell/d}{K}\right)^4\right] \tag{13-14}$$

13.5.2.1.3 For columns having an ℓ/d ratio of K or greater:

$$F'_c = \frac{0.30E}{(\ell/d)^2} \tag{13-15}$$

13.5.2.2 For especially severe service conditions or extraordi-nary hazardous conditions, the use of lower design values than those obtained above may be necessary.

13.5.2.3 The design load for a column of round cross section shall not exceed that for a square column of the same cross sectional area.

13.5.3 TAPERING COLUMNS

For a design of a column of rectangular cross section, tapered at one or both ends, the dimension d in each plane of the column shall be taken as the sum of the minimum d in that plane plus one-third the difference between the minimum and maximum d in that plane. The design value, F'_c, determined for the tapered column shall apply to the cross-sectional area of the column, corresponding to the dimension d used in the calculation of F'_c. The maximum load for a column of round cross section, tapered on one or both ends, shall not exceed that for a square column of the same cross-sectional area and having the same degree of taper. The design value in compression parallel to grain, F_c, shall not be exceeded in any cross-section in a tapered column.

13.6 DESIGN OF SPACED COLUMNS

13.6.1 GENERAL

13.6.1.1 The increased load capacity of a spaced column due to the end fixity developed by the connectors and end blocks is effective only in the direction perpendicular to the wide faces of the individual members (direction parallel to dimension d_1 in Figure 13.6.1A).

13.6.1.2 The capacity of a spaced column in the direction parallel to the wide faces of the individual members (direction parallel to dimension d_2 in Figure 13.6.1A is subject to the provisions for simple solid columns.

13.6.2 SLENDERNESS RATIO

13.6.2.1 When column end conditions provide less stability than pin-end conditions, the effective length of the column for design purposes shall be increased in accordance with good engineering practice. No reduction in effective length shall be made when column ends provide greater stability than pin-end conditions.

13.6.2.2 For individual members of a spaced column,

(a) ℓ_1/d_1 shall not exceed 80, where ℓ_1 is the distance between lateral supports that provide restraint perpendicular to the wide faces of the individual members.

(b) ℓ_2/d_2 shall not exceed 50, where ℓ_2 is the distance between lateral supports that provide restraint in a direction parallel to the wide faces of the individual members.

(c) ℓ_3/d_1 shall not exceed 40, where ℓ_3 is the distance between the centroid of connectors in an end block and the center of the spacer block.

See Figure 13.6.1A for dimensions d_1 and d_2.

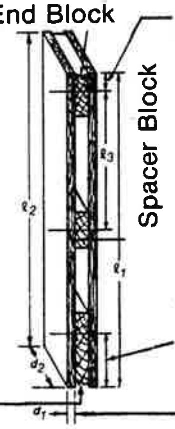


Figure 13.6.1A

13.6.3 FIXITY

13.6.3.1 The end fixity of spaced columns shall be classified either as condition "a" or condition "b" (see Figure 13.6.1A) as follows:

(a) For condition "a", the centroid of connectors or a connector group in the end block shall be within one-twentieth of the length ℓ_1 from the column end.

(b) For condition "b", the centroid of connectors or a connector group in the end block shall be between one-twentieth and one-tenth of the length ℓ_1 from the column end.

13.6.3.2 Connectors are not required for a single spacer block located within the middle tenth of the column length. If there are two or more spacer blocks, connectors are required and the distance between two adjacent blocks shall not exceed one-half the distance between centers of connectors in the end blocks.

13.6.3.3 For spaced columns used as compression members of a truss, a panel point which is stayed laterally shall be considered as the end of the spaced column, and the portion of the web members, between the individual pieces making up a spaced column, may be considered as the end blocks.

13.6.3.4 The thickness of spacer and end blocks shall not be less than that of individual members of the spaced column nor shall thickness, width, and length of spacer and end blocks be less than required for connectors of a size and number capable of carrying the load computed as indicated in the following paragraph. Blocks thicker than a side member do not appreciably increase load capacity.

13.6.3.5 To obtain spaced-column action, the connectors in each mutually contacting surface of end block and individual member at end of a spaced column shall be of a size and number to provide a load capacity in pounds equal to the required cross-sectional area in square inches of one of the individual members times the appropriate end-spacer block constant in Table 13.6.1.

Table 13.6.1

END SPACER BLOCK CONSTANTS FOR CONNECTOR JOINED SPACED COLUMNS

ℓ_1/d_1 Ratio of Individual Member in the Spaced Column[1]	End Spacer Block Constant			
	Group A Connector Loads	Group B Connector Loads	Group C Connector Loads	Group D Connector Loads
0 to 11	0	0	0	0
15	38	33	27	21
20	86	73	63	48
25	134	114	94	75
30	181	155	128	101
35	229	195	162	128
40	277	230	195	154
45	325	277	229	181
50	372	318	263	208
55	420	358	296	234
60 to 80	468	399	330	261

[1]Constants for intermediate ℓ/d ratios may be obtained by straight line interpolation.

13.6.4 MAXIMUM DESIGN COMPRESSIVE STRESS PARALLEL TO GRAIN, F'_c

13.6.4.1 The design load for a spaced column is the sum of the design loads for each of its individual members based on the unit design values determined in accordance with Article 13.5 but subject to the limitations relative to stress at the net section.

13.6.4.2 The maximum values of F'_c for individual members of spaced columns shall be determined in accordance with the following formulas.

(a) Short Columns. For columns with individual members having an ℓ_1/d_1 ratio of 11 or less:

$$F'_c = F_c \qquad (13\text{-}16)$$

(b) Intermediate Columns. For columns with individual members having an ℓ_1/d_1 ratio greater than 11 but less than K:

$$F'_c = F_c\left[1 - \frac{1}{3}\left(\frac{\ell_1/d_1}{K}\right)^4\right] \qquad (13\text{-}17)$$

where

$$K = 0.671\ \sqrt{C_x E/F_c} \tag{13-18}$$

(c) Long Columns. For columns with individual members having an ℓ_1/d_1 ratio greater than K:

$$F'_c = \frac{0.30 C_x E}{(\ell_1/d_1)^2} \tag{13-19}$$

where:

C_x = 2.5 for fixity condition "a"
C_x = 3.0 for fixity condition "b"

13.6.4.3 Where the design values, F'_c, for the two individual members of a spaced column are different because the members are of different species, grades or thicknesses, the lesser value of F'_c shall apply to both members.

13.6.4.4 The design values determined using this Article shall not exceed the design values for the individual members taken as simple solid columns without regard to fixity, when determined in accordance with Article 13.5 by using dimensions d_2 and length ℓ_2 as the distance between the lateral supports which provide restraint in a direction parallel to dimension d_2.

13.6.4.5 For especially severe service conditions or extraordinary hazardous conditions, the use of lower design values than those obtained above may be necessary. See Appendix G, NDS, for an explanation of the basis for column design procedures.

13.7 PILE AND FRAMED BENTS

13.7.1 PILE BENTS

Pile bents generally shall not exceed 40 feet in height. Pile bents over 10 feet high shall be sway-braced transversely with diagonal braces on each side of the bent, and shall be adequately braced longitudinally. In general, pile bents shall contain not less than four piles each and the outside piles, preferably, shall be battered. The piles shall be designed for safe bearing and for column action.

13.7.2 FRAMED BENTS

Framed bents may be supported on piles, concrete pedestals or mud sills. All bents shall be sway-braced transversely and shall be adequately braced longitudinally. In general, framed bents shall contain not less than four posts each and the outside posts of the bent shall be battered. The posts shall be designed as columns.

13.7.3 SILLS AND MUD SILLS

13.7.3.1 When possible, sills shall be located clear of all earth so that there may be a free circulation of air around them. Sills shall be fastened to mud sills or piles with drift bolts of not less than 3/4 inch diameter and extending at least 6 inches into the mud sills or piles. Sills shall be fastened to pedestals with dowels of not less than 3/4 inch diameter, set in the pedestals and extending at least 6 inches into the sills.

13.7.3.2 Posts shall be fastened to sills by dowels of not less than 3/4 inch diameter, extending at least 6 inches into the posts and sills, or by drift bolts of not less than 3/4 inch diameter driven diagonally through the base of the post and extending at least 9 inches into the sill. Posts shall be fastened to pedestals with dowels of not less than 3/4 inch diameter and extending into the posts at least 6 inches.

13.7.4 CAPS

Timber caps shall not be less in size than 10 by 10 inches. They shall be fastened with drift bolts of not less than 3/4 inch diameter, extending at least 9 inches into the piles or posts.

13.7.5 BRACING

Single-story bracing shall not exceed 20 feet in height. The minimum size of the transverse sway braces shall be 3 by 8 inches. All bracing shall be bolted through the piles, posts or caps at the ends and at all intermediate intersections. The bolts used shall not be less than 5/8 inch diameter.

13.7.6 PILE BENT ABUTMENTS

Pile bent abutments shall be adequately braced or anchored to resist earth pressure. Bulkhead planks shall be not less than 3 inches thick and shall be fastened to the piles with spikes, which shall be at least 3 inches longer than the thickness of the plank.

13.8 TRUSSES

13.8.1 JOINTS AND SPLICES

13.8.1.1 Joints shall be detailed to shed water to the maximum degree practicable. Joints and splices shall be designed to develop the computed stresses in the members being connected and, preferably, to develop the full strength of the members. Posts or struts bearing against the sides of timber members shall preferably be provided with metal end bearings. End bearing on inclined surfaces shall be avoided, preference being given to square-cut ends of timbers bearing against blocks.

13.8.1.2 Bearing surfaces of castings connecting timber members shall be milled to provide smooth, even surfaces permitting accurate fitting and complete contact of the wood and metal bearing surfaces. Rolled plates, bars and shapes used in chord splice plates, or other parts bearing upon wood surfaces, shall be true and even. The wood surfaces taking bearing upon metal parts shall be not less than 5/8 inch in width. Bolts engaging castings and structural parts shall hold them rigidly in position so that bending on the parts in contact will be reduced to a minimum. The joint details at truss panel points shall provide definite lines of action and shall be simple and as susceptible as possible to definite strength analysis. When included bolts are used to connect end posts or web members with chord members, they shall be placed approximately at an angle of not more than 60 degrees with the latter and when used in conjunction with joint castings, the holes in one of the connected members shall be bored 1/8 inch larger than the nominal diameter of the bolts. For truss joints using wood or steel side members, bolt holes shall be 1/16 inch larger than the bolt diameter for bolts 1/2 inch and larger; they should be 1/32 inch larger than the bolt diameter for smaller bolts. No daps in chords for butt blocks shall be less than 3/4 inch deep.

13.8.1.3 Splices for tension members shall be designed to reduce to a minimum the effects of cross shrinkage of the timber. Neither steel splice plates of the batten type nor shear pin splices shall be used when the timbers to be spliced are more than 8 inches thick, since the shrinkage will permit the joint to become loose. Shear pin joints shall be used only with fully seasoned timber.

13.8.2 FLOOR BEAMS

Floor beams shall be sized at bearing points. In floor beams composed of two or more timbers, the timbers shall preferably be separated by at least 2 inches for air circulation. Floor beams shall be connected to the main truss members by means of rods or structural shapes.

13.8.3 HANGERS

Hangers generally shall be rods having upset ends with a suitably designed washer or bearing plate at each end. Upset ends shall conform to the requirements specified for Structural Steel Design, Division I.

13.8.4 EYEBARS AND COUNTERS

The requirements specified for Structural Steel Design, Division I, for counters, eyebars and eyebar packing shall apply to such members when used in timber trusses.

13.8.5 BRACING

Timber trusses shall be provided with a rigid system of laterals in the plane of the loaded chord. When the details will permit, this lateral bracing shall be securely fastened to all longitudinal stringers. Lateral bracing, preferably rigid, in the plane of the unloaded chord, and rigid portal and sway-bracing shall be provided in all trusses having sufficient headroom. Outrigger brackets connected to extensions of the floor beams shall be used for bracing through-trusses having headroom insufficient for a top lateral system.

13.8.6 CAMBER

Camber, in addition to that required to provide for dead load and shrinkage, shall be provided in timber trusses in sufficient amount to give the structure a good appearance.

13.9 FLOORS AND RAILINGS

13.9.1 STRINGERS

Stringers shall be of sufficient length to obtain the required bearing at supports. Preferably, they shall be of two panel lengths placed with staggered joints. The lapped ends of untreated stringers in temporary structures shall be separated at least 1/2 inch for air circulation. Stringers shall be secured to caps or floor beams.

13.9.2 BRIDGING

Stringers shall be braced by cross bridging in each panel. The bridging shall be not less in size than 2 by 4 inches.

13.9.3 NAILING STRIPS

13.9.3.1 When timber floors are supported by steel joists, the joists shall be provided with nailing strips or some other acceptable means of securing the timber floor to the joists. If nailing strips are used they shall be bolted either to the top flanges or to the webs.

13.9.3.2 When nailing strips are bolted to the flanges, they shall be used on all joists. They shall be not less than 4 inches deep and shall be wider than the supporting flange. They shall be secured with 5/8 inch bolts through the flanges, spaced not more than 4 feet apart and not more than 18 inches from the ends of the strips.

13.9.3.3 Nailing strips bolted to the webs shall be not less than 4 inches thick and shall be fastened with bolts spaced not farther apart than 5 feet. They shall be held clear of the flanges by blocks between the web and strip, and bolted through the web with 5/8 inch bolts spaced not more than 4 feet apart and not more than 18 inches from the ends of the strips.

13.9.4 FLOORING

13.9.4.1 Roadway floor plank shall have a nominal thickness of not less than 3 inches. Sidewalk floor plank shall have a nominal thickness of not less than 2 inches.

13.9.4.2 The minimum size of material used for laminated or strip floors shall be 2 by 4 inches.

13.9.5 RETAINING PIECES

Retaining pieces, where required, shall be not less than 6 inches in width. In general, they shall be secured in place by 5/8 inch bolts at 3-foot intervals and spiked at 1-foot intervals.

13.9.6 DRAINAGE

Adequate provision shall be made for the proper drainage of timber floors.

13.9.7 RAILINGS

Rails, rail posts and fastenings shall be designed for the loads specified in Article 2.7.4.

13.10 FIRE STOPS

13.10.1 Timber floors or trestles of any considerably length shall preferably be provided with fire stops to inhibit the spread of fire along the length of the structure.

13.10.2 In timber floors, these fire stops should be provided at span ends and at least at one intermediate location for spans over 50 feet. They may consist of diaphragms of wood or fire-resistant material at least as thick as the flooring, located over caps or floorbeams and completely filling the opening between the joists down to within 8 inches of the bottom of the beam.

13.10.3 In timber trestle bridges, in addition to the fire stops in the floor, fire curtains should be provided at intervals of 100 feet or more. These curtains may consist of plank or asbestos-covered metal spiked to the bents. They should extend downward at least 5 feet from the bottom of the joists and horizontally at least to the ends of the caps. A fire stop between the joists should be located over each curtain.

SECTION 14 - ELASTOMERIC BEARINGS

14.1 GENERAL

Elastomeric bearings shall be subject to the requirements of this section and to the sections applicable to the particular types of construction with which they are used.

The elastomers to be used shall conform to requirements given in Section 25 - Division II of this specification.

14.2 DESIGN

14.2.1 Bearings may be plain (consisting of elastomer only) or laminated (consisting of layers of elastomer restrained at their interfaces by bonded laminates). Elastomer compounds of nominal 70 durometer hardness shall not be used in laminated bearings. Plain bearings generally will be restricted by the requirements of this specification to conditions where little movement is anticipated.

14.2.2 The following terms will be used:

L = Length of a rectangular bearing parallel to the direction of translation
W = Width of a rectangular bearing perpendicular to the direction of translation
R = Radius of a circular bearing
t = Average thickness of a plain bearing or the thicknesss of any individual layer of elastomer in a laminated bearing (including the top and bottom layer)
T = Total effective elastomer thickness (summation of t's)
S = Shape factor (the area of the loaded face divided by the side area free to bulge)

$$S = \frac{LW}{2t(L+W)} \quad \text{for rectangular bearings} \qquad (14\text{-}1)$$

$$S = \frac{R}{2t} \quad \text{for circular bearings} \qquad (14\text{-}2)$$

14.2.3 The size of the elastomeric pad shall be such that both surfaces are in complete contact with the bearing areas.

14.2.4 The compressive strain of a plain bearing or of any individual layer of a laminated bearing is a function of the average unit compressive stress, the hardness of the elastomer and the shape factor. The compressive deflection of each layer is the product of the strain and the thickness of the layer. The total deflection of the bearing is the sum of the layer deflections. The shear strain of a bearing is a function of the temperature, hardness of the elastomer and average unit shearing stress. The shear deflection of a bearing is the product of the shear strain and the total effective elastomer thickness. These relationships may be taken from existing test reports, but for large bearings or groups of standard designs they shall preferably be verified by tests of the particular designs involved.

14.2.5 The average unit pressure on elastomeric bearings shall not exceed 800 psi under a combination of dead load plus live load, not including impact. The average unit pressure due to dead load only shall not exceed 500 psi. When dead load plus live load uplift reduce the average pressure to less than 200 psi the bearing shall be secured against horizontal crawling preferably by positive attachment to the top surface or to the top and bottom surfaces. When secured to the top and bottom surfaces the bearing may be subject to momentary light tension.

14.2.6 The initial compressive deflection in a plain bearing, or in any layer of a laminated bearing, under dead load plus live load, not including impact, shall not exceed .07 t. The deflection can be determined from a plot showing the relationship of shape factor, load and the durometer hardness of the elastomer under consideration. These curves are generally available from manufacturers for their product.

14.2.7 Bearings shall have built in taper when nonparallel load surfaces would otherwise produce a compressive deflection of .06T under dead load. Such taper shall be limited to 5/8" per foot.

14.2.8 The total of the positive and negative movements caused by anticipated temperature change shall not exceed .5T.

14.2.9 To ensure stability, the following limits shall be observed:

Plain Bearings	- Minimum L = 5T
	Minimum W = 5T
	Minimum R = 3T
Laminated Bearings	- Minimum L = 3T
	Minimum W = 2T
	Minimum R = 2T

SECTION 15 - TFE BEARING SURFACE

15.1 GENERAL

15.1.1 Proprietary makes of polytetrafluoroethylene (TFE) fixed and expansion bearings may be used if in the opinion of the Engineer, and substantiated either by tests or experience, they meet design requirements.

15.1.2 Bearings having sliding surfaces of TFE shall be subject to the requirements of this Section and to Sections applicable to the particular types of construction with which they are used.

15.1.3 The TFE material consisting of filled or unfilled sheet, fabric containing TFE fibers, interlocked bronze and filled TFE structures, TFE-perforated metal composites together with adhesive materials, stainless steel mating surface and manufacturing processes shall conform to the requirements given in Section 27 - Division II.

15.2 DESIGN

15.2.1 TFE sliding surfaces are designed to translate or rotate by sliding of a self-lubricating polytetrafluoroethylene surface across a smooth hard mating surface preferably of stainless steel or other equally corrosion resistant material.

15.2.2 Expansion bearings having sliding surfaces of TFE shall not be used without provision for rotation which shall be not less than .015 radians, to prevent excessive local stresses on the TFE sliding surface. Rotation shall be considered the sum of live load rotation, changes in camber during construction, and misalignment of the bearing seats due to construction tolerances. The design shall include compensating provision for grade. Provision for rotation may be accomplished with a hinge, radiused sliding surfaces, elastomeric pads, preformed fabric pads or other means.

15.2.3 TFE sliding surfaces shall have the following minimum and maximum thickness:

Unfilled or filled TFE	1/32" minimum to 3/32" maximum
Fabric containing TFE fibers	1/80" minimum to 1/8" maximum
Interlocked bronze and filled TFE structures	1/32" minimum to 1/8" maximum
TFE-Perforated metal composite	1/16" minimum to 1/8" maximum

15.2.4 The TFE sliding surface must be either bonded under factory controlled conditions or mechanically connected to a rigid back-up material capable of resisting any bending stresses to which the sliding surfaces may be subjected. Alternatively, TFE material of twice the thickness specified above may be recessed for half its thickness in the back-up material and shall not be less than 1/8" thick. If the other side of the back-up material is to be bonded to an elastomeric pad, the back-up material must have sufficient tensile strength to restrain the elastomeric pad. The elastomeric pad must be sufficiently hard to allow sliding of the contact surfaces, preferably at least 70 durometer hardness.

15.2.5 The mating surface to the TFE should be an accurate flat, cylindrical or spherical surface as required by the design and shall have minimum Brinell hardness of 125 and a surface finish of less than 20 micro inches root mean square (rms). The mating surface shall completely cover the TFE surface in all operating positions of the bearing. Wherever possible, the mating surface shall be oriented so that sliding movements will cause dirt and dust accumulation to fall from the mating surface.

15.2.6 The minimum coefficient of friction used for design shall be as specified by the bearing manufacturer or as follows:

Material	Bearing Pressure		
	500 psi	2000 psi	3500 psi
Unfilled TFE, Fabric containing TFE fibers, TFE-Perforated Metal Composite	.08	.06	.04
Filled TFE	.12	.10	.08
Interlocked Bronze and Filled TFE Structures	.10	.07	.05

15.2.7 The average bearing pressure on the TFE sliding surface due to all loads shall not exceed:

Filled TFE	3,500 psi
Unfilled TFE (Recessed)	3,500 psi
Unfilled TFE (Not Recessed)	2,000 psi
Fabric containing TFE fibers	3,500 psi
Interlocking Bronze and Filled TFE Structures	6,000 psi
TFE-Perforated Metal Composite	5,000 psi

15.2.8 The edge load pressure due to all loads and rotation shall not exceed:

Unfilled and Filled TFE	5,000 psi
Fabric containing TFE fibers	10,000 psi
Interlocked Bronze and Filled	10,000 psi
TFE-Perforated Metal Composite	5,000 psi

15.2.9 Holes or slots shall not be used in the sliding surfaces.

15.2.10 Welding to steel plate which has a bonded TFE surface may be permitted providing welding procedures are established which restrict the maximum temperature reached by the bond area to less than 300 F (150 C) as determined by temperature indicating wax pencils or other suitable means.

15.2.11 Means shall be provided in the design to locate positively all elements of the bearing. Where a thin non-corrosive smooth facing material is used as a mating sliding surface, it shall be structurally bonded by an approved adhesive system and may also be mechanically fastened by means of either screws or rivets to the back-up material, or if the materials permit, seal-welded around the entire perimeter of the facing material.

SECTION 16 - STEEL TUNNEL LINER PLATES

16.1 GENERAL AND NOTATIONS

16.1.1 GENERAL

16.1.1.1 These criteria cover the design of cold-formed panel steel tunnel liner plates. The minimum thickness shall be as determined by design in accordance with Articles 16.2, 3, 4, 5 and 6 and the construction shall conform to Section 26 - Division II. The supporting capacity of a non-rigid tunnel lining such as a steel liner plate results from its ability to deflect under load so that side restraint developed by the lateral resistance of the soil constrains further deflection. Deflection thus tends to equalize radial pressures and to load the tunnel liner as a compression ring.

16.1.1.2 The load to be carried by the tunnel liner is a function of the type of soil. In a granular soil, with little or no cohesion, the load is a function of the angle of internal friction of the soil and the diameter of the tunnel being constructed. In cohesive soils such as clays and silty clays the load to be carried by the tunnel liner is dependent on the shearing strength of the soil above the roof of the tunnel.

16.1.1.3 A subsurface exploration program and appropriate soil tests should be performed at each installation before undertaking a design.

16.1.1.4 Nothing included in this section shall be interpreted as prohibiting the use of new developments where usefulness can be substantiated.

16.1.2 NOTATIONS

A = cross-sectional area of liner plates (Article 16.3.4)

C_d = coefficient for tunnel liner, used in Marston's formula (Article 16.2.4)

D = horizontal diameter or span of the tunnel (Article 16.2.4)

D = pipe diameter (Article 16.3.3)

D_c = critical pipe diameter (Article 16.3.4)

E = modulus of elasticity (Article 16.3.3)

FS = factor of safety for buckling (Article 16.3.4)

f_c = buckling stress (Article 16.3.4)

f_u = minimum specified tensile strength (Article 16.3.4)

H = height of soil over the top of the tunnel (Article 16.2.4)

I = moment of inertia (Article 16.3.3)

k = parameter dependent upon the value of the friction angle (Article 16.3.4)

P = external load on tunnel liner (Article 16.2.1)

P_d = vertical load at the level of the top of the tunnel liner due to dead load (Article 16.2.1)

P_1 = vertical load at the level of the top of the tunnel liner due to live load (Article 16.2.1)

r = radius of gyration (Article 16.3.4)

T = thrust per unit length (Article 16.3.4)

W = total (moist) unit weight of soil (Article 16.2.4)

$\emptyset$ = friction angle of soil (Article 16.2.3A)

16.2 LOADS

16.2.1 External load on a circular tunnel liner made up of tunnel liner plates may be predicted by various methods including actual tests. In cases where more precise methods of analysis are not employed, the external load P can be predicted by the following:

(a) If the grouting pressure is greater than the computed external load, the external load P on the tunnel liner shall be the grouting pressure.

(b) In general the external load can be computed by the formula

$$P=P_1+P_d \qquad (16\text{-}1)$$

where:

P = the external load on the tunnel liner;

P_1 = the vertical load at the level of the top of the tunnel liner due to live loads;

P_d = the vertical load at the level of the top of the tunnel liner due to dead load.

16.2.2 For an H-20 load, values of P_1 are approximately the following:

H(ft)	4	5	6	7	8	9	10
P_1(lbs. per sq.ft.)	375	260	190	140	110	90	75

16.2.3 Values of P_d may be calculated using Marston's formula for load or any other suitable method.

16.2.4 In the absence of adequate borings and soil tests, the full overburden height should be the basis for P_d in the tunnel liner plate design.

The following is one form of Marston's formula:

$$P_d = C_d W D \qquad (16\text{-}2)$$

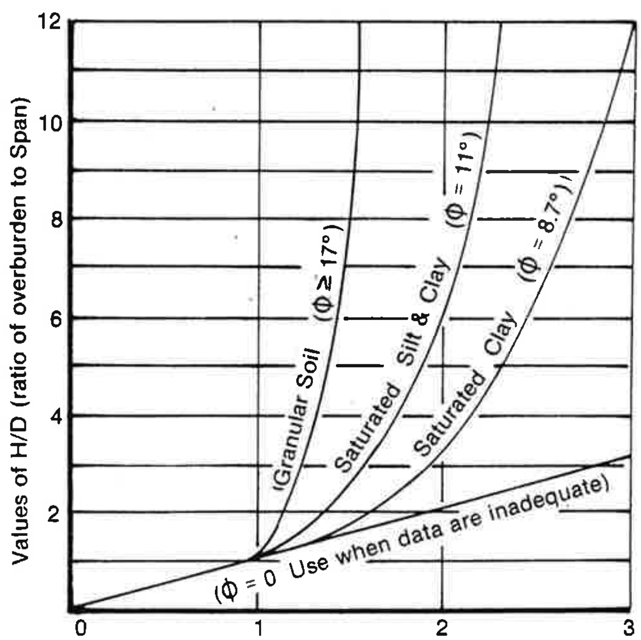
Where:

C_d = coefficient for tunnel liner, Figure 16.2.3A

W = total (moist) unit weight of soil

D = horizontal diameter or span of the tunnel

H = height of soil over the top of the tunnel



Values of coefficient C_d

DIAGRAM FOR COEFFICIENT C_d FOR TUNNELS IN SOIL

(Ø=FRICTION ANGLE)

16.2.3A

16.3 DESIGN

16.3.1 CRITERIA

The following criteria must be considered in the design of liner plates:

(a) Joint strength
(b) Minimum stiffness for installation
(c) Critical buckling of liner plate wall
(d) Deflection or flattening of tunnel section.

16.3.2 JOINT STRENGTH

16.3.2.1 The seam strength of liner plates must be sufficient to withstand the thrust developed from the total load supported by the liner plate. This thrust, T, in pounds per linear foot is

$$T=PD/2 \qquad (16\text{-}3)$$

Where:

P= load as defined in Article 16.2
D= diameter or span in feet

16.3.2.2 The ultimate design longitudinal seam strengths are:

Table 16.3.2.2

ULTIMATE SEAM STRENGTH OF LINER PLATES

Plate thickness, inches	Ultimate strength, kips/ft	
	2 Flange	4 Flange
0.075	20.0	
0.105	30.0	26.0
0.135	47.0	43.0
0.164	55.0	50.0
0.179	62.0	54.0
0.209	87.0	67.0
0.239	92.0	81.0
0.313		115.0
0.375		119.0

16.3.2.3 The thrust, T, multiplied by the safety factor, should not exceed the ultimate seam strength.

16.3.3 MINIMUM STIFFNESS FOR INSTALLATION

16.3.3.1 The liner plate ring shall have enough rigidity to resist the unbalanced loads of normal construction: grouting pressure, local slough-ins, and miscellaneous concentrated loads.

The minimum stiffness required for these loads can be expressed for convenience by the formula below. It must be recognized, however, that the limiting values given here are only recommended minima. Actual job conditions may require higher values (greater effective stiffness). Final determination on this factor should be based on intimate knowledge of the project and practical experience.

16.3.3.2 The minimum stiffness for installation is determined by the formula:

$$\text{Minimum stiffness} = EI/D^2 \qquad (16\text{-}4)$$

Where

D = Diameter in inches

E = Modulus of elasticity, psi (29×10^6)

I = Moment of inertia, inches to the fourth power per inch

For 2-Flange (EI/D^2) = 50 minimum

For 4-Flange (EI/D^2) = 111 minimum

16.3.4 CRITICAL BUCKLING OF LINER PLATE WALL

16.3.4.1 Wall buckling stresses are determined from the following formulas:

For diameters less than D_c, the ring compression stress at which buckling becomes critical is:

$$f_c = f_u - \left[\frac{f_u^{\,2}}{48E} \times \left(\frac{kD}{r}\right)^2\right] \text{ in psi} \qquad (16\text{-}5)$$

For diameters greater than D_c:

$$f_c = \frac{12E}{(kD/r)^2} \text{ in psi} \qquad (16\text{-}6)$$

Where:

D_c = $(r/k)\sqrt{24E/f_u}$ = critical pipe diameter, inches. (16-7)

f_u = minimum specified tensile strength, psi

f_c = buckling stress, psi., not to exceed minimum specified yield strength

D = pipe diameter, inches

r = radius of gyration of section, inches per foot

E = modulus of elasticity, psi

k = will vary from 0.22 for soils with ø > 15 to 0.44 for soils ø < 15

16.3.4.2 Design for buckling is accomplished by limiting the ring compression thrust T to the buckling stress multiplied by the effective cross section area of the liner plate divided by the factor of safety.

$$T = \frac{f_c A}{FS} \tag{16-8}$$

Where:

T = thrust per linear foot from Article 16.3.2
A = cross-section area of liner plate, sq. in. per foot
FS = factor of safety for buckling

16.3.5 DEFLECTION OR FLATTENING

16.3.5.1 Deflection of a tunnel depends significantly on the amount of over excavation of the bore and is affected by delay in backpacking or inadequate backpacking. The magnitude of deflection is not primarily a function of soil modulus or the liner plate properties, so it cannot be computed with usual deflection formulae.

16.3.6.2 Where the tunnel clearances are important, the designer should oversize the structure to provide for a normal deflection. Good construction methods should result in deflections of not more than 3 percent of the normal diameter.

16.4 CHEMICAL AND MECHANICAL REQUIREMENTS

16.4.1 CHEMICAL COMPOSITION

Base metal shall conform to ASTM A 569.

16.4.2 MINIMUM MECHANICAL PROPERTIES OF FLAT PLATE BEFORE COLD FORMING

Tensile strength	= 42,000 psi
Yield strength	= 28,000 psi
Elongation, 2 inches	= 30 percent

16.4.3 DIMENSIONS AND TOLERANCES

Nominal plate dimensions shall provide the section properties shown in Article 16.5. Thickness tolerances shall conform to Paragraph 14 of AASHTO M 167.

16.5 SECTION PROPERTIES

The section properties per inch of plate width, based on the average of one ring of liner plates, shall conform to the following:

Table 16.5a

SECTION PROPERTIES FOR FOUR FLANGE LINER PLATE				
Gage	Thickness Decimal inches	Area in^2/inch	Effective Area in^2/inch	Moment of Inertia in^4/inch
12	0.105	0.133	0.067	0.042
11	0.1196	0.152	0.076	0.049
10	0.135	0.170	0.085	0.055
8	0.164	0.209	0.105	0.070
7	0.179	0.227	0.114	0.075
5	0.209	0.264	0.132	0.087
3	0.239	0.300	0.150	0.120
1/4	0.250	0.309	0.155	0.101
5/16	0.3125	0.386	0.193	0.123
3/8	0.375	0.460	0.230	0.143

Table 16.5b

SECTION PROPERTIES FOR TWO FLANGE LINER PLATE		
Thickness inches	Effective Area in^2/inch	Moment of Inertia in^4/inch
0.075	0.096	0.034
0.105	0.135	0.049
0.135	0.174	0.064
0.164	0.213	0.079
0.179	0.233	0.087
0.209	0.272	0.103
0.239	0.312	0.118

16.6 COATINGS

Steel tunnel liner plates shall be of heavier gage or thickness or protected by coatings or other means when required for resistance to abrasion or corrosion.

16.7 BOLTS

16.7.1 Bolts and nuts used with lapped seams shall be not less than 5/8 inch in diameter. The bolts shall conform to the specifications of ASTM A449 for plate thickness equal to or greater than 0.209 inches and A307 for plate thickness less than 0.209 inches. The nut shall conform to ASTM A 307, Grade A.

16.7.2 Bolts and nuts used with four flanged plates shall be not less than 1/2 inch in diameter for plate thicknesses to and including 0.179 inches and not less than 5/8 inch in diameter for plates of greater thickness. The bolts and nuts shall be quick acting coarse thread and shall conform to ASTM A 307, Grade A.

16.8 SAFETY FACTORS

Longitudinal test seam strength	-3
Pipe Wall Buckling	-2

SECTION 17 - SOIL-REINFORCED CONCRETE STRUCTURE INTERACTION SYSTEMS

17.1 GENERAL

17.1.1 SCOPE

Specifications in this section govern the design of buried reinforced concrete structures. A buried reinforced concrete element becomes part of a composite system comprising the reinforced concrete section and the soil envelope, both of which contribute to the structural behavior of the system.

17.1.2 NOTATIONS

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, sq. in. When the flexural reinforcement consists of several bar sizes or wire 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. (Articles 17.6.4.6 and 17.7.4.7.)

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto. (Articles 17.6.4.6 and 17.7.4.7.)

f_s = maximum service load stress in the reinforcing steel for crack control (Articles 17.6.4.6 and 17.7.4.7).

17.1.3 LOADS

Design loads shall be determined by the forces acting on the structure. For earth loads see Article 3.20. For live loads see Article 3.4 through Article 3.7, and Articles 3.11 and 3.12. For loading combinations see Article 3.22.

17.1.4 DESIGN

Design may be based on working stress or ultimate strength principles. The design criteria shall include structural aspects (e.g. flexure, thrust, shear), handling and installation, and crack control. Footing design for cast-in-place boxes and arches shall be in conformity with Article 4.4.

17.1.5 MATERIALS

The materials shall conform to the AASHTO materials specifications referenced herein.

17.1.6 SOIL

Structural performance is dependent on soil structure interaction. The type and anticipated behavior of the material beneath the structure, adjacent to the structure, and over the structure must be considered.

17.1.7 ABRASIVE OR CORROSIVE CONDITIONS

Where abrasive or corrosive conditions exist, suitable protective measures shall be considered.

17.1.8 END STRUCTURES

Structures may require special consideration where erosion may occur. Skewed alignment may require special end wall designs.

17.1.9 CONSTRUCTION AND INSTALLATION

The construction and installation shall conform to Section 28 - Division II.

17.2 SERVICE LOAD DESIGN

17.2.1 For soil-reinforced concrete structure interaction systems designed with reference to service loads and allowable stresses, the service load stresses shall not exceed the values shown in Article 8.15 except as modified herein.

17.2.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edge bearing tests made in accordance with AASHTO materials specifications may be used in lieu of service load design.

17.3 LOAD FACTOR DESIGN

17.3.1 Soil-reinforced concrete structure interaction systems shall be designed to have design strengths of all sections at least equal to the required strengths calculated for the factored loads as stipulated in Article 3.22, except as modified herein.

17.3.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edge bearing tests made in accordance with AASHTO materials specifications may be used in lieu of load factor design.

17.4 REINFORCED CONCRETE PIPE

(To be provided later.)

17.5 REINFORCED CONCRETE ARCH, CAST-IN-PLACE

17.5.1 APPLICATION

This specification is intended for use in the design of cast-in-place reinforced concrete arches with the arch barrel monolithic with each footing. A separate reinforced concrete invert may be required where the structure is subject to scour.

17.5.2 MATERIALS

17.5.2.1 CONCRETE

Concrete shall conform to Article 8.2

17.5.2.2 REINFORCEMENT

Reinforcement shall meet the requirements of Article 8.3.

17.5.3 DESIGN

17.5.3.1 GENERAL REQUIREMENTS

Design shall conform to these specifications except as provided otherwise in this section. For design loads and loading conditions, see Article 3.2. For reinforced concrete design requirements see Section 8.

17.5.3.2 MINIMUM COVER

The minimum fill over reinforced concrete arches shall be 12 inches or span/8.

17.5.3.3 STRENGTH REDUCTION FACTORS

Strength reduction factors for load factor design of cast-in-place arches may be taken as 0.90 for flexure and 0.85 for shear.

17.5.3.4 SPLICES OF REINFORCEMENT

Reinforcement shall be in conformity with Article 8.33.1.1. If lap splicing is used laps shall be staggered with a minimum of one foot measured along the circumference of the arch. Ties shall be provided connecting the intrados and extrados reinforcement. Ties shall be at twelve-inch maximum spacing, in both longitudinal and circumferential directions, except as modified by shear.

17.5.3.5 FOOTING DESIGN

Design shall include consideration of differential horizontal and vertical movements and footing rotations. Footing design shall conform to Article 4.4.

17.6 REINFORCED CONCRETE BOX, CAST-IN-PLACE

17.6.1 APPLICATION

This specification is intended for use in the design of cast-in-place reinforced concrete box culverts.

17.6.2 MATERIALS

17.6.2.1 CONCRETE

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on test beams.

17.6.2.2 REINFORCEMENT

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

17.6.3 CONCRETE COVER FOR REINFORCEMENT

The minimum concrete cover for reinforcement shall conform to Article 8.22. The top slab shall be considered a bridge slab for concrete cover considerations.

17.6.4 DESIGN

17.6.4.1 GENERAL REQUIREMENTS

Designs shall conform to applicable sections of these specifications except as provided otherwise in this section. For design loads and loading conditions see Section 3. For distribution of concentrated loads through earth for culverts with less than 2 feet of cover see Article 3.24.3, Case B, and for requirements for bottom distribution reinforcement in top slabs of such culverts see Article 3.24.10. For distribution of wheel loads to culverts with 2 feet or more of cover see Article 6.4. For reinforced concrete design requirements see Section 8.

17.6.4.2 DISTRIBUTION OF CONCENTRATED LOAD EFFECTS TO BOTTOM SLAB

The width of top slab strip used for distribution of concentrated wheel loads may be increased by twice the box height and used for the distribution of loads to the bottom slab.

17.6.4.3 DISTRIBUTION OF CONCENTRATED LOADS IN SKEWED CULVERTS

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

17.6.4.4 SPAN LENGTH

For span length see Article 8.8, except when monolithic haunches included at 45 degrees are considered in the design, negative moment reinforcement in walls and slabs may be proportioned based on the bending moment at the intersection of haunch and the uniform depth member.

17.6.4.5 STRENGTH REDUCTION FACTORS

Strength reduction factors for load factor design may be taken at 0.9 for combined flexure and thrust and as 0.85 shear.

17.6.4.6 CRACK CONTROL

The maximum service load stress in the reinforcing steel for crack control shall be:

$$fs = \frac{98}{\sqrt[3]{d_c A}} \text{ ksi} \qquad (17\text{-}1)$$

17.6.4.7 MINIMUM REINFORCEMENT

Minimum reinforcement shall be provided in accordance with Article 8.17.1 at all cross sections subject to flexural tension, including the inside face of walls. Shrinkage and temperature reinforcement shall be provided near the inside surfaces of walls and slabs in accordance with Article 8.20.

17.7 REINFORCED CONCRETE BOX, PRECAST

17.7.1 APPLICATION

This specification is intended for use in design for precast reinforced concrete box sections. Boxes may be manufactured using conventional structural concrete and forms (formed) or with dry concrete and vibrating form pipe making methods (machine-made). Standard dimensions are shown in AASHTO Materials Specification M259, and M273.

17.7.2 MATERIALS

17.7.2.1 CONCRETE

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on cores.

17.7.2.2 REINFORCEMENT

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

17.7.3 CONCRETE COVER FOR REINFORCEMENT

The minimum concrete cover for reinforcement in boxes reinforced with wire fabric shall be three times the wire diameter but not less than one inch. For boxes covered by less than two feet of fill, the minimum cover for reinforcement in the top of the slab shall be two inches.

17.7.4 DESIGN

17.7.4.1 GENERAL REQUIREMENTS

Design shall conform to applicable sections of these specifications except as provided otherwise in this section. For design loads and loading conditions see Section 3. For distribution of wheel loads to culvert slabs under less than 2 feet of cover see Article 3.24.3, Case B, and for requirements for bottom reinforcement in top slabs of such culverts see Article 3.24.10. For distribution of wheel loads to culvert slabs with 2 feet or more of cover, see Article 6.4.

For reinforced concrete design requirements see Section 8. For span length see Article 8.8, except as noted in Article 17.7.4.5.

17.7.4.2 DISTRIBUTION OF CONCENTRATED LOAD EFFECTS IN SIDES AND BOTTOMS

The width of the top slab strip used for distribution of concentrated wheel loads shall also be used for determination of bending moments, shears and thrusts in the sides and bottom.

17.7.4.3 DISTRIBUTION OF CONCENTRATED LOADS IN SKEWED CULVERTS

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

17.7.4.4 SHEAR TRANSFER IN TRANSVERSE JOINTS BETWEEN CULVERT SECTIONS

For boxes with less than 2 feet of cover, the top slab joint shall either conform to the edge beam requirements of Section 3, Part C, or be capable of transferring a minimum shear load of 3,000 pounds per linear foot of top slab joint. If individual shear connectors are used, they shall be spaced at no more than 2.5 feet on center with a minimum of 2 connectors per joint.

17.7.4.5 SPAN LENGTH

When monolithic haunches inclined at 45 degrees are taken into account, negative reinforcement in walls and slabs may be proportioned based on the bending moment at the intersection of haunch and uniform depth member.

17.7.4.6 STRENGTH REDUCTION FACTORS

Strength reduction factors for load factor design of machine-made boxes may be taken as 1.0 for moment and 0.9 for shear.

17.7.4.7 CRACK CONTROL

The maximum service load stress in the reinforcing steel for crack control shall be:

$$f_s = \frac{98}{\sqrt[3]{d_c A}} \text{ ksi} \qquad (17\text{-}2)$$

17.7.4.8 MINIMUM REINFORCEMENT

The primary flexural reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002. Such minimum reinforcement shall be provided at all cross sections subject to flexural tension, at the inside face of walls, and in each direction at the top of slabs of box sections with less than 2 feet of fill. The provisions of Article 8.20 do not apply to precast concrete box sections except, if units of unusual length (over 16 ft.) are fabricated, the minimum longitudinal reinforcement for shrinkage and temperature should be as provided in Article 8.20.

APPENDIX A

LOADING—H 15-44 (M13.5)

TABLE OF MAXIMUM MOMENTS, SHEARS AND REACTIONS.—SIMPLE SPANS, ONE LANE

Spans in feet ; moments in thousands of foot-pounds ; shears and reactions in thousands of pounds .

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	6.0(b)	24.0(b)	42	274.4(b)	29.6
2	12.0(b)	24.0(b)	44	289.3(b)	30.1
3	18.0(b)	24.0(b)	46	304.3(b)	30.5
4	24.0(b)	24.0(b)	48	319.2(b)	31.0
5	30.0(b)	24.0(b)	50	334.2(b)	31.5
6	36.0(b)	24.0(b)	52	349.1(b)	32.0
7	42.0(b)	24.0(b)	54	364.1(b)	32.5
8	48.0(b)	24.0(b)	56	379.1(b)	32.9
9	54.0(b)	24.0(b)	58	397.6	33.4
10	60.0(b)	24.0(b)	60	418.5	33.9
11	66.0(b)	24.0(b)	62	439.9	34.4
12	72.0(b)	24.0(b)	64	461.8	34.9
13	78.0(b)	24.0(b)	66	484.1	35.3
14	84.0(b)	24.0(b)	68	506.9	35.8
15	90.0(b)	24.0(b)	70	530.3	36.3
16	96.0(b)	24.8(b)	75	590.6	37.5
17	102.0(b)	25.1(b)	80	654.0	38.7
18	108.0(b)	25.3(b)	85	720.4	39.9
19	114.0(b)	25.6(b)	90	789.8	41.1
20	120.0(b)	25.8(b)	95	862.1	42.3
21	126.0(b)	26.0(b)	100	937.5	43.5
22	132.0(b)	26.2(b)	110	1,097.3	45.9
23	138.0(b)	26.3(b)	120	1,269.0	48.3
24	144.0(b)	26.5(b)	130	1,452.8	50.7
25	150.0(b)	26.6(b)	140	1,648.5	53.1
26	156.0(b)	26.8(b)	150	1,856.3	55.5
27	162.7(b)	26.9(b)	160	2,076.0	57.9
28	170.1(b)	27.0(b)	170	2,307.8	60.3
29	177.5(b)	27.1(b)	180	2,551.5	62.7
30	185.0(b)	27.2(b)	190	2,807.3	65.1
31	192.4(b)	27.3(b)	200	3,075.0	67.5
32	199.8(b)	27.4(b)	220	3,646.5	72.3
33	207.3(b)	27.5(b)	240	4,266.0	77.1
34	214.7(b)	27.7	260	4,933.5	81.9
35	222.2(b)	27.9	280	5,649.0	86.7
36	229.6(b)	28.1	300	6,412.5	91.5
37	237.1(b)	28.4			
38	244.5(b)	28.6			
39	252.0(b)	28.9			
40	259.5(b)	29.1			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

LOADING—HS 15-44(MS 13.5)

TABLE OF MAXIMUM MOMENTS, SHEARS AND REACTIONS.—SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	6.0(b)	24.0(b)	42	364.0(b)	42.0(b)
2	12.0(b)	24.0(b)	44	390.7(b)	42.5(b)
3	18.0(b)	24.0(b)	46	417.4(b)	43.0(b)
4	24.0)b)	24.0(b)	48	444.1(b)	43.5(b)
5	30.0(b)	24.0(b)	50	470.9(b)	43.9(b)
6	36.0(b)	24.0(b)	52	497.7(b)	44.3(b)
7	42.0(b)	24.0(b)	54	524.5(b)	44.7(b)
8	48.0(b)	24.0(b)	56	551.3(b)	45.0(b)
9	54.0(b)	24.0(b)	58	578.1(b)	45.3(b)
10	60.0(b)	24.0(b)	60	604.9(b)	45.6(b)
11	66.0(b)	24.0(b)	62	631.8(b)	45.9(b)
12	72.0(b)	24.0(b)	64	658.6(b)	46.1(b)
13	78.0(b)	24.0(b)	66	685.5(b)	46.4(b)
14	84.0(b)	24.0(b)	68	712.3(b)	46.6(b)
15	90.0(b)	25.6(b)	70	739.2(b)	46.8(b)
16	96.0(b)	27.0(b)	75	806.3(b)	47.3(b)
17	102.0(b)	28.2(b)	80	873.7(b)	47.7(b)
18	108.0(b)	29.3(b)	85	941.0(b)	48.1(b)
19	114.0(b)	30.3(b)	90	1,008.3(b)	48.4(b)
20	120.0(b)	31.2(b)	95	1,074.9(b)	48.7(b)
21	126.0(b)	32.0(b)	100	1,143.0(b)	49.0(b)
22	132.0(b)	32.7(b)	110	1,277.7(b)	49.4(b)
23	138.0(b)	33.4(b)	120	1,412.5(b)	49.8(b)
24	144.5(b)	34.0(b)	130	1,547.3(b)	50.7
25	155.5(b)	34.6(b)	140	1,682.1(b)	53.1
26	166.6(b)	35.1(b)	150	1,856.3	55.5
27	177.8(b)	35.6(b)	160	2,076.0	57.9
28	189.0(b)	36.0(b)	170	2,307.8	60.3
29	200.3(b)	36.6(b)	180	2,551.5	62.7
30	211.6(b)	37.2(b)	190	2,807.3	65.1
31	223.0(b)	37.7(b)	200	3,075.0	67.5
32	234.4(b)	38.3(b)	220	3,646.5	72.3
33	245.8(b)	38.7(b)	240	4,266.0	77.1
34	257.7(b)	39.2(b)	260	4,933.5	81.9
35	270.9(b)	39.6(b)	280	5,649.0	86.7
36	284.2(b)	40.0(b)	300	6,412.5	91.5
37	297.5(b)	40.4(b)			
38	310.7(b)	40.7(b)			
39	324.0(b)	41.1(b)			
40	337.4(b)	41.4(b)			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

LOADING—H 20-44 (M 18)

TABLE OF MAXIMUM MOMENTS, SHEARS AND REACTIONS.—SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	365.9(b)	39.4
2	16.0(b)	32.0(b)	44	385.8(b)	40.1
3	24.0(b)	32.0(b)	46	405.7(b)	40.7
4	32.0(b)	32.0(b)	48	425.6(b)	41.4
5	40.0(b)	32.0(b)	50	445.6(b)	42.0
6	48.0(b)	32.0(b)	52	465.5(b)	42.6
7	56.0(b)	32.0(b)	54	485.5(b)	43.3
8	64.0(b)	32.0(b)	56	505.4(b)	43.9
9	72.0(b)	32.0(b)	58	530.1	44.6
10	80.0(b)	32.0(b)	60	558.0	45.2
11	88.0(b)	32.0(b)	62	586.5	45.8
12	96.0(b)	32.0(b)	64	615.7	46.5
13	104.0(b)	32.0(b)	66	645.5	47.1
14	112.0(b)	32.0(b)	68	675.9	47.8
15	120.0(b)	32.5(b)	70	707.0	48.4
16	128.0(b)	33.0(b)	75	787.5	50.0
17	136.0(b)	33.4(b)	80	872.0	51.6
18	144.0(b)	33.8(b)	85	960.5	53.2
19	152.0(b)	34.1(b)	90	1,053.0	54.8
20	160.0(b)	34.4(b)	95	1,149.5	56.4
21	168.0(b)	34.7(b)	100	1,250.0	58.0
22	176.0(b)	34.9(b)	110	1,463.0	61.2
23	184.0(b)	35.1(b)	120	1,692.0	64.4
24	192.0(b)	35.3(b)	130	1,937.0	67.6
25	200.0(b)	35.5(b)	140	2,198.0	70.8
26	208.0(b)	35.7(b)	150	2,475.0	74.0
27	216.9(b)	35.9(b)	160	2,768.0	77.2
28	226.8(b)	36.0(b)	170	3,077.0	80.4
29	236.7(b)	36.1(b)	180	3,402.0	83.6
30	246.6(b)	36.3(b)	190	3,743.0	86.8
31	256.5(b)	36.4(b)	200	4,100.0	90.0
32	266.5(b)	36.5(b)	220	4,862.0	96.4
33	276.4(b)	36.6(b)	240	5,688.0	102.8
34	286.3(b)	36.9	260	6,578.0	109.2
35	296.2(b)	37.2	280	7,532.0	115.6
36	306.2(b)	37.5	300	8,550.0	122.0
37	316.1(b)	37.8			
38	326.1(b)	38.2			
39	336.0(b)	38.5			
40	346.0(b)	38.8			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

LOADING—HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS AND REACTIONS.—SIMPLE SPANS, ONE LANE

Spans in feet ; moments in thousands of foot-pounds ; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End Shear and end reaction (a)
1	8.0	32.0(b)
2	16.0	32.0(b)
3	24.0	32.0(b)
4	32.0	32.0(b)
5	40.0	32.0(b)
6	48.0	32.0(b)
7	56.0	32.0(b)
8	64.0	32.0(b)
9	72.0	32.0(b)
10	80.0	32.0(b)
11	88.0	32.0(b)
12	96.0	32.0(b)
13	104.0	32.0(b)
14	112.0	32.0(b)
15	120.0	34.1(b)
16	128.0	36.0(b)
17	136.0	37.7(b)
18	144.0	39.1(b)
19	152.0	40.4(b)
20	160.0	41.6(b)
21	168.0	42.7(b)
22	176.0	43.6(b)
23	184.0	44.5(b)
24	192.7	45.3(b)
25	207.4	46.1(b)
26	222.2	46.8(b)
27	237.0	47.4(b)
28	252.0	48.0(b)
29	267.0	48.8(b)
30	282.1	49.6(b)
31	297.3	50.3(b)
32	312.5	51.0(b)
33	327.8	51.6(b)
34	343.5	52.2(b)
35	361.2	52.8(b)
36	378.9	53.3(b)
37	396.6	53.8(b)
38	414.3	54.3(b)
39	432.1	54.8(b)
40	449.8	55.2(b)

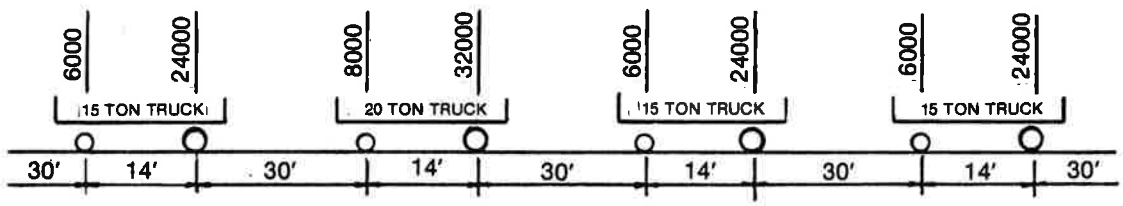
Span	Moment	End shear and end reaction (a)
42	485.3(b)	56.0(b)
44	520.9(b)	56.7(b)
46	556.5(b)	57.3(b)
48	592.1(b)	58.0(b)
50	627.9(b)	58.5(b)
52	663.6(b)	59.1(b)
54	699.3(b)	59.6(b)
56	735.1(b)	60.0(b)
58	770.8(b)	60.4(b)
60	806.5(b)	60.8(b)
62	842.4(b)	61.2(b)
64	878.1(b)	61.5(b)
66	914.0(b)	61.9(b)
68	949.7(b)	62.1(b)
70	985.6(b)	62.4(b)
75	1,075.1(b)	63.1(b)
80	1,164.9(b)	63.6(b)
85	1,254.7(b)	64.1(b)
90	1,344.4(b)	64.5(b)
95	1,434.1(b)	64.9(b)
100	1,524.0(b)	65.3(b)
110	1,703.6(b)	65.9(b)
120	1,883.3(b)	66.4(b)
130	2,063.1(b)	67.6
140	2,242.8(b)	70.8
150	2,475.1	74.0
160	2,768.0	77.2
170	3,077.1	80.4
180	3,402.1	83.6
190	3,743.1	86.8
200	4,100.0	90.0
220	4,862.0	96.4
240	5,688.0	102.8
260	6,578.0	109.2
280	7,532.0	115.6
300	8,550.0	122.0

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

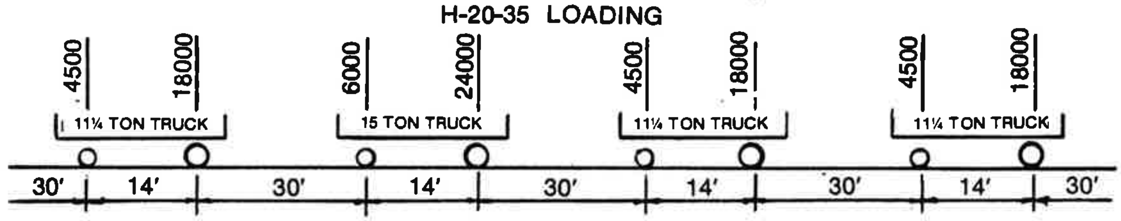
(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

APPENDIX B

TRUCK TRAIN AND EQUIVALENT LOADINGS — 1935 SPECIFICATIONS AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS

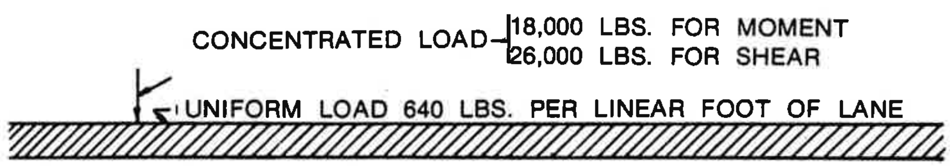


H-20-35 LOADING

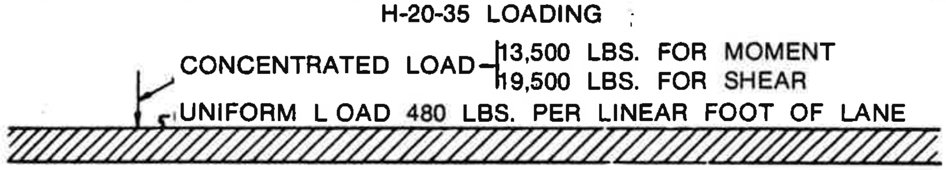


H-15-35 LOADING

TRUCK TRAIN LOADING



H-20-35 LOADING



H-15-35 LOADING

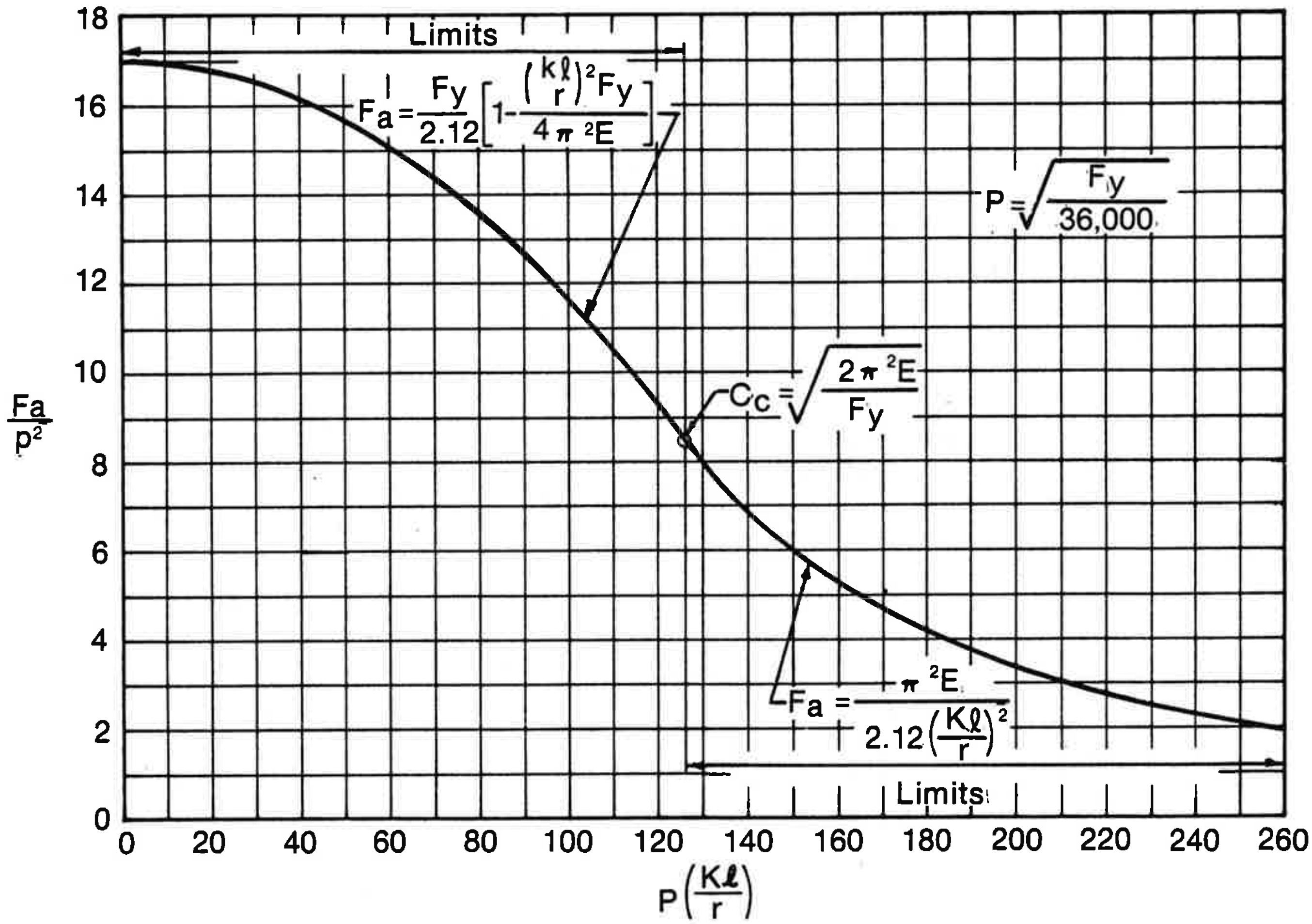
EQUIVALENT LOADING

LANE WIDTH 10 FEET

APPENDIX C

FORMULA FOR COMPRESSION IN CONCENTRICALLY LOADED COLUMNS

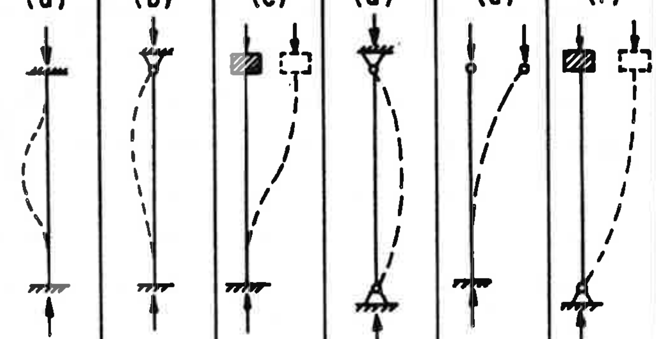
(See Table 10.32.1A for Specific Values)



EFFECTIVE LENGTH FACTOR, K

The Effective Length of a column, KL, has been used in the equations for allowable compression stress in the column. K is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. KL represents the length between inflection points of a buckled column. Restraint against rotation and translation of column ends influences the position of the inflection points in a column. Theoretical values of K for some idealized column end conditions are shown in Table C-1. Since column end conditions seldom comply fully with idealized restraint against rotation and translation, the recommended values suggested by the Column Research Council are higher than the idealized values.

TABLE C-1

EFFECTIVE LENGTH FACTORS, K							
		(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE							
THEORETICAL K VALUE		0.5	0.7	1.0	1.0	2.0	2.0
DESIGN VALUE OF K WHEN IDEAL CONDITIONS ARE APPROXIMATED[1]		0.65	0.80	1.2	1.0	2.1	2.0
END CONDITION CODE		ROTATION FIXED TRANSLATION FIXED ROTATION FREE TRANSLATION FIXED ROTATION FIXED TRANSLATION FREE ROTATION FREE TRANSLATION FREE					

[1] For riveted and bolted truss members (partially restrained) use K = 0.75. For pinned connections in truss members use K = 0.875 (pin friction).

Columns in continuous frames unbraced by adequate attachment to shear walls, diagonal bracing, or adjacent structures depend upon the bending stiffness of the rigidly connected beams for lateral stability. The effective length factor, K, is dependent upon the amount of bending stiffness supplied by the beams at the column ends. If the amount of stiffness supplied by the beams is small, then the value of K could exceed 2.0.

If it is assumed that elastic action occurs and that all columns buckle simultaneously in a frame, then it can be rationally shown that*

$$\frac{G_a G_b(\pi/K)^2 - 36}{6(G_a + G_b)} = \frac{\pi/K}{\tan(\pi/K)} \qquad \text{(C-1)}$$

Where subscripts a and b refer to the two ends of the column.

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} \qquad \text{(C-2)}$$

Σ = Summation of all members rigidly connected to an end of the column in the plane of bending.
I_c = Moment of inertia of column.
L_c = Unbraced length of column.
I_g = Moment of inertia of beam or other restraining member.
L_g = Unsupported length of beam or other restraining member.
K = Effective length factor.

Table C-2 is a graphical representation of the relationship between K, G_a, and G_b, and can be used to obtain the value of K easily. In frames which have columns that fall in the inelastic buckling range, [i.e., $KL/r<C_c = (2\pi^2 E/F_y)^{1/2}$], K may often be reduced. The procedure for reducing K can be found in Effective Length of Columns in Unbraced Frames by Joseph A. Yura, AISC Engineering Journal Published by American Institute of Steel Construction, 101 Park Avenue, New York, New York 10017.

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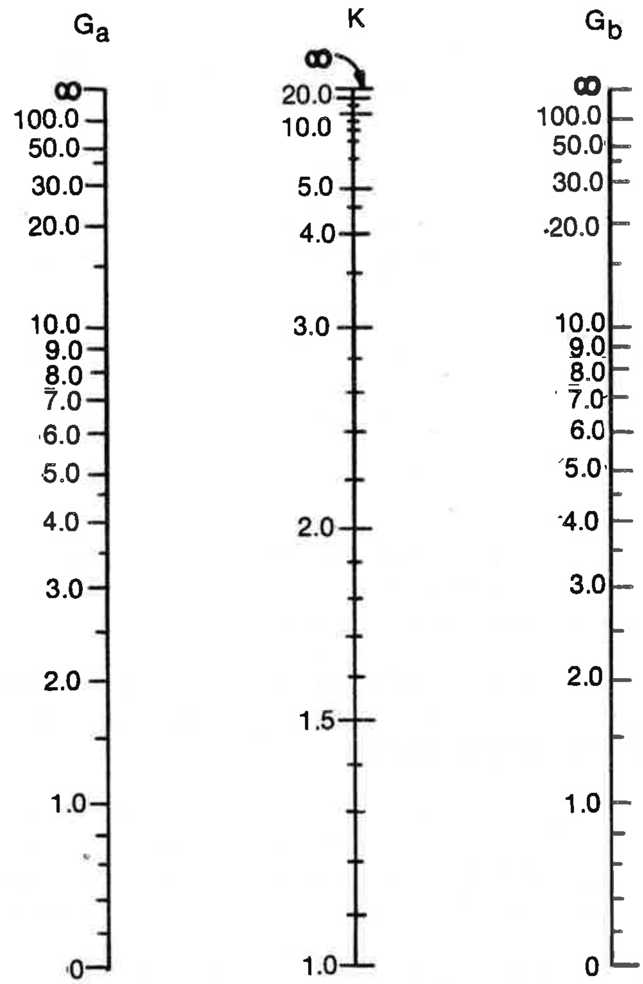
*See "Steel Structures Design and Behavior" by Charles G. Salmon and John E. Johnson, published by International Text Book Company, 1971.

TABLE C-2



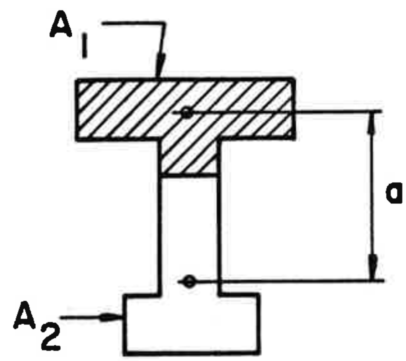
SIDESWAY PERMITTED

FOR COLUMN ENDS SUPPORTED BY BUT NOT RIGIDLY CONNECTED TO A FOOTING OR FOUNDATION, G IS THEORETICALLY EQUAL TO INFINITY, BUT UNLESS ACTUALLY DESIGNED AS A TRUE FRICTIONLESS PIN, MAY BE TAKEN EQUAL TO 10 FOR PRACTICAL DESIGN. IF THE COLUMN END IS RIGIDLY ATTACHED TO A PROPERLY DESIGNED FOOTING, G MAY BE TAKEN EQUAL TO 1.0 SMALLER VALUES MAY BE TAKEN IF JUSTIFIED BY ANALYSIS.

APPENDIX D

COMPUTATION OF PLASTIC SECTION MODULUS Z*

The plastic modulus Z is the statical first moment of one half-area of the cross section about an axis through the centroid of the other half area.



A_1 (shaded) = A_2 (clear) = $A/2$

a = distance between centroid of A_1 and A_2

$Z = aA_1 = aA_2$

When a section is built up from plates or shapes of more than one yield point, the plastic moment should be computed on the basis of equilibrium on the cross section with all fibers stressed to the appropriate yield point in either tension or compression.

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*Information in this Appendix is obtained from the Commentary of AISI Bulletin 15. Values of Z for rolled sections are listed in the "Manual of Steel Construction," Eighth Edition, 1980, American Institute of Steel Construction.