TABLE 10.3.1B (Continued)

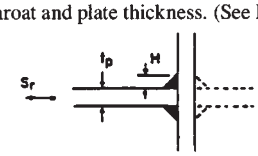
General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness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

TABLE 10.3.1B (Continued)

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

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

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



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

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

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

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

10.7 LIMITING LENGTHS OF MEMBERS

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

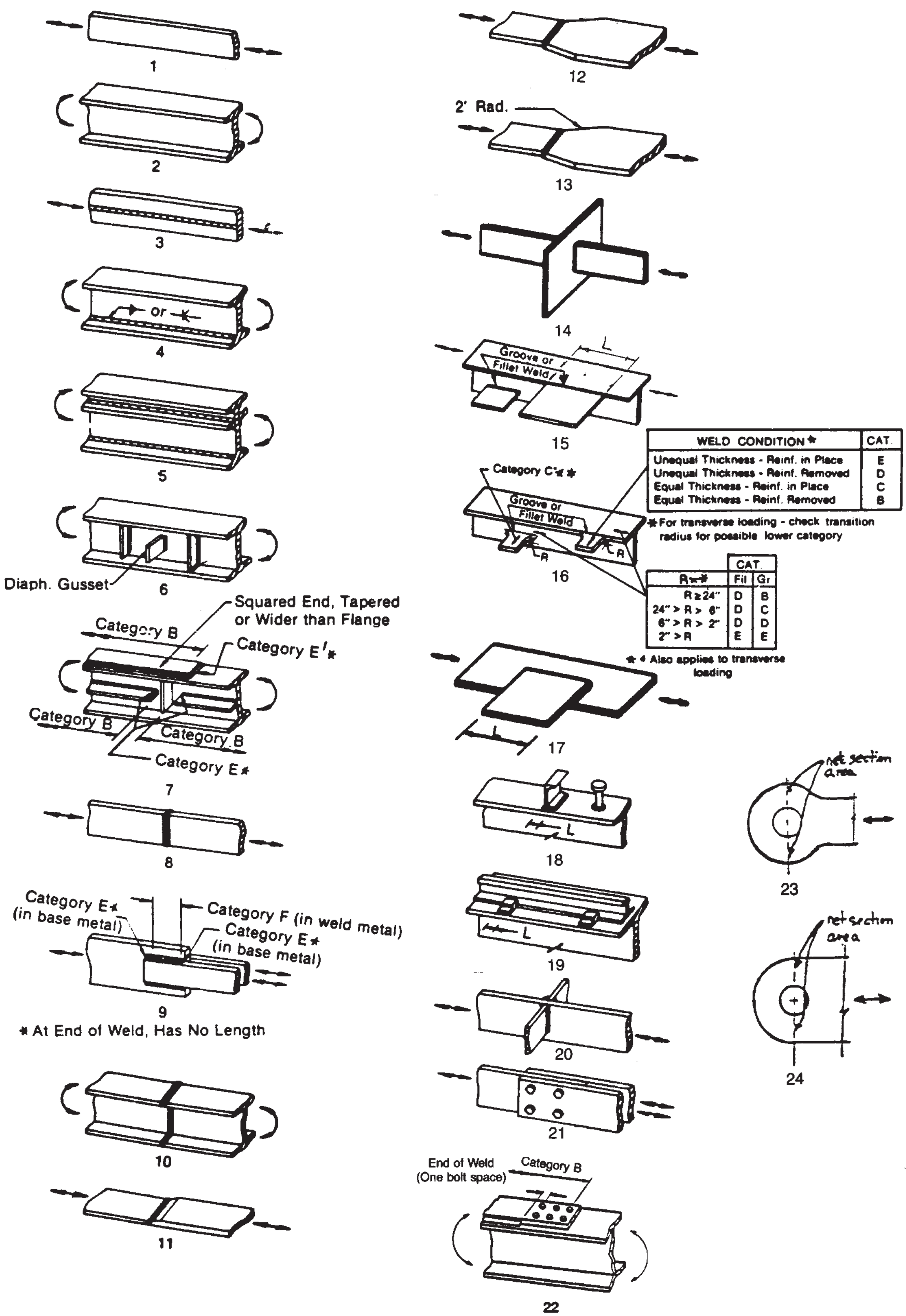


FIGURE 10.3.1C Illustrative Examples

TABLE 10.3.2A Stress Cycles

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

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

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

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

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

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

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

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

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

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

10.8 MINIMUM THICKNESS OF METAL

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

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

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

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

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

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

10.9 EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION

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

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

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

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

10.10 OUTSTANDING LEGS OF ANGLES

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

In main members carrying axial stress, 12 times the thickness.

In bracing and other secondary members, 16 times the thickness.

For other limitations, see Article 10.35.2.

10.11 EXPANSION AND CONTRACTION

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

10.12 FLEXURAL MEMBERS

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

10.13 COVER PLATES

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

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

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

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

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

10.14 CAMBER

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

10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

10.15.1 Scope

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

10.15.2 Minimum Radius of Curvature

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

$$R = \frac{14bD}{\sqrt{F_y \psi t_w}} \quad (10\text{-}1)$$

$$R = \frac{7{,}500b}{F_y \psi} \quad (10\text{-}2)$$

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

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

10.15.3 Camber

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

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

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

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

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

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

10.16 TRUSSES

10.16.1 General

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

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

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

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

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

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

10.16.2 Truss Members

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

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

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

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

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

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

10.16.3 Secondary Stresses

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

10.16.4 Diaphragms

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

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

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

10.16.5 Camber

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

10.16.6 Working Lines and Gravity Axes

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

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

10.16.7 Portal and Sway Bracing

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

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

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

10.16.8 Perforated Cover Plates

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

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

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

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

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

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

10.16.8.6 For thickness of metal, see Article 10.35.2.

10.16.9 Stay Plates

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

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

10.16.9.3 The separate segments of tension members composed of shapes may be connected by perforated plates or by stay plates or end stay plates and lacing. End stay plates shall have the same minimum length as specified for end stay plates on main compression members, and intermediate stay plates shall have a minimum length of $3/4$ 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% of that distance for double lacing.

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

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

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

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

10.16.11 Gusset Plates

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

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

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

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

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

10.16.12 Half-Through Truss Spans

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

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

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

10.16.13 Fastener Pitch in Ends of Compression Members

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

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

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

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

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

where:

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

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

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

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

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

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

10.17 BENTS AND TOWERS

10.17.1 General

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

10.17.2 Single Bents

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

10.17.3 Batter

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

10.17.4 Bracing

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

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

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