3. Solid cover plates and webs, supported on two edges, connecting main segments of box sections: $5000/\sqrt{f_a}$ but not to exceed 50.

4. Perforated cover plates supported on two edges: $6000/\sqrt{f_a}$ but not to exceed 55.

For cases 2 and 3 above, AREA prescribes limiting values of 32 and 40, respectively, for F_y = 36,000 psi, and $6000/\sqrt{F_y}$ and $7500/\sqrt{F_y}$ for high-strength steels.

Plate elements of built-up members may buckle locally between adjacent fasteners. Typical specification requirements to control this are those of the AISC, where for nonstaggered fasteners, the maximum pitch is $127/\sqrt{F_y}$ times the plate thickness but not to exceed 12 in., and for staggered fasteners, $190/\sqrt{F_y}$ times the plate thickness but not to exceed 18 in.

14. Lacing and Perforated Cover Plates The open sides of built-up compression members must be provided with lacing (Fig. 8*a*,*b*,*c*), battens (tie or stay plates, Fig 8*e*), or perforated cover plates (Fig. 8*f*) to prevent local buckling of the components and to ensure

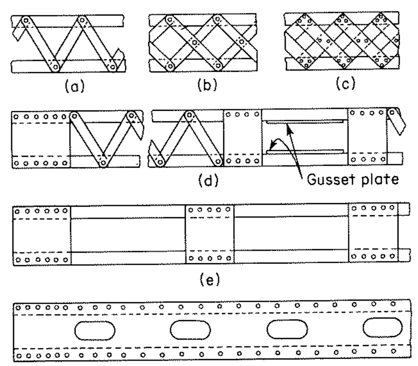

Fig. 8

that they act as a unit. According to the AISC, lacing must be proportioned to resist a shear, normal to the axis of the member, equal to 2 percent of the axial compression. The AASHTO specifies a shear given by

$$V = \frac{P}{100}\left(\frac{100}{L/r + 10} + \frac{L/r}{3{,}300{,}000/F_y}\right) \qquad (17)$$

to which must be added shear due to weight of the member and any external force, other than P, acting on it. In this equation, P is the allowable axial load and r is the radius of gyration about the axis normal to the plane of the lacing or perforated plate. The shear V is divided equally among all parallel planes of shear-resisting elements.

TABLE 7 Geometrical Requirements for Tie Plates

	AISC	AREA and AASHTO
Min length of end plate	B	$1\frac{1}{4}B$
Min length of intermediate plate	$\frac{1}{2}B$	$\frac{3}{4}B$
Min thickness	$\frac{B}{50}$	$B/50$: main members $B/60$: secondary members

B = clear distance between centerlines of fasteners or welds connecting the plate.

The shear specified by AREA is given by Eq. (17) with the denominator of the second term in parentheses equal to 100 for F_y = 36,000 psi and $3{,}600{,}000/F_y$ for high-strength steels.

Lacing bars must be spaced so that the segments of the member will not buckle locally. To assure this, AISC requires that the slenderness ratio of the portion of the flange included between adjacent connections of the bars not exceed the slenderness ratio of the member itself. According to AASHTO and AREA, the slenderness ratio of the segment

must not exceed 40 or two-thirds the slenderness ratio of the member. The inclination of lacing bars to the axis of the member should be about 45° for double lacing and 60° for single lacing. Lacing bars must be designed to resist both tension and compression.

Tie plates are required at the ends of lacing planes and at any other point where the lacing must be interrupted (Fig. 8*d*). Tie plates fulfill the dual function of spacing the components of the member and of distributing the load between them. Dimensions of tie plates are given in Table 7.

TABLE 8 Geometrical Requirements for Perforated Cover Plates

	AISC	AASHTO
Ratio of length of hole in direction of load to width	2	2
Clear distance between holes	B	B
Min radius of hole corners	$1\frac{1}{2}$ in.	$1\frac{1}{2}$ in.
Min distance from edge of end perforation to end of cover plate	......	$1\frac{1}{4}B$
Min distance from edge of perforation, measured at its centerline, to nearest line of connecting fasteners	......	Art. 13, Case 1

B = clear distance between centerlines of fasteners or welds connecting the plate.

Perforated cover plates have two advantages over lacing: (1) the net width of the plate may be considered to be part of the cross section, and (2) they permit easy access to the interior of the member for maintenance. Geometrical requirements are given in Table 8.

Example 3 Design a square column consisting of four angles (Fig. 9). The effective length KL is 30 ft and the load P = 740 kips. A36 steel, AISC specification.

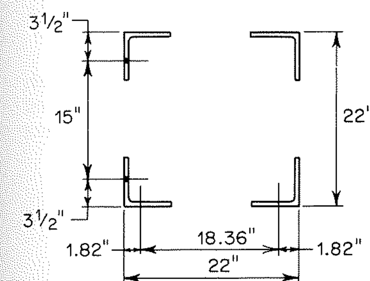

Fig. 9 Example 3.

From Table 6,

$$r = 0.42 \times 22 = 9.24 \text{ in.}$$

$$\frac{L}{r} = \frac{360}{9.24} = 39$$

$$F_a = 19.27 \text{ ksi} \qquad A = \frac{740}{19.27} = 38.4 \text{ in.}^2$$

Try four angles $6 \times 6 \times \frac{7}{8}$, A = 38.92 in.²

$$I_x = I_y = 4(31.9 + 9.73 \times 9.18^2) = 3407 \text{ in.}^4$$

$$r_x = r_y = \sqrt{\frac{3407}{38.92}} = 9.35 \approx 9.24 \text{ in.} \qquad \text{Column O.K.}$$

LACING. For $3\frac{1}{2}$-in. gage on angles, the distance between rows of fasteners is 15 in. For single lacing at 60° with the axis of the member, the distance between lacing points is $15 \times 2 \cot 60° = 17.3$ in. For the 6×6 angle between lacing points,

$$\frac{L}{r} = \frac{17.3}{1.17} = 14.8 < 39$$

The shear on each plane of lacing is

$$V = 0.02 \times 740/2 = 7.4 \text{ kips}$$

The corresponding force in one lacing bar is

$$\frac{7.4}{\cos 30°} = 8.55 \text{ kips}$$

For a maximum permissible L/r of 140

$$r = \frac{17.3}{140} = 0.29t$$

$$t = 0.43 \text{ in. Use } 7/16$$

$$r = 0.29 \times 0.437 = 0.126 \qquad \frac{L}{r} = \frac{17.3}{0.126} = 137$$

$$F_a = 8.70 \qquad A = \frac{8.55}{8.70} = 0.982 \text{ in.}^2$$

$$b = \frac{0.982}{0.437} = 2.25 \text{ in.}$$

Use $2\frac{1}{4} \times \frac{7}{16}$ lacing, $\frac{3}{4}$-in. rivets.

$$A_n = (2.25 - 0.88)0.437 = 0.6 \text{ in.}^2$$

$$\text{Allowable tension} = 0.6 \times 22 = 13.2 > 8.55$$

TIE PLATES. Minimum length = 15 in., minimum thickness = $15/50$ = 0.3. Use 15 × $\frac{3}{8}$ tie plates.

15. Tapered Columns The elastic-buckling load for a tapered column can be found by multiplying the Euler buckling load by a factor

$$P = \frac{\pi^2 E I_0 \mu}{(KL)^2} = \frac{\pi^2 E A_0 r_0^2 \mu}{(KL)^2} \tag{18a}$$

where I_0 = moment of inertia at small end (Fig. 10)
A_0 = area at small end
r_0 = radius of gyration at small end

Values of μ for I and box sections of uniformly tapered depth d and constant flange width and flange and web thickness, and of four-legged tower or boom sections with constant leg area and uniformly tapered depth d are given in Table 9. Buckling is about the strong axis.[8]

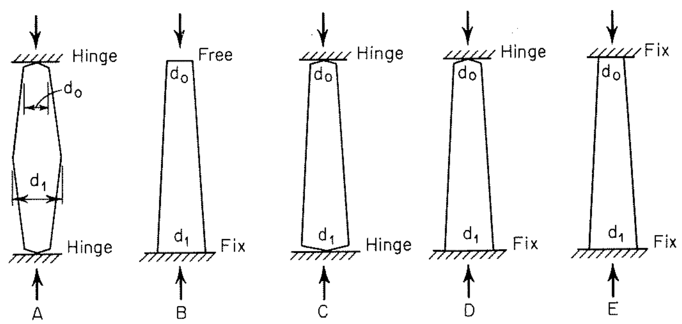

Fig. 10

A formula for inelastic buckling is obtained by replacing E in Eq. (18a) by the tangent modulus E_t:

$$P = \frac{\pi^2 E_t A_0 r_0^2 \mu}{(KL)^2} \tag{18b}$$

This equation is correct if the area is constant, as in the four-legged tower section, but it is approximate for the I and box sections since the area increases with increase in depth of the section. The increase is relatively small, however, since it is confined to the web (or webs). Furthermore, the formula is on the safe side for the latter sections since the member yields progressively from the smaller end to the larger, rather than uniformly over the whole length.

Equations (18a) and (18b) show that the buckling stress P/A_0 for a tapered member can be found by using an equivalent radius of gyration $r_0\sqrt{\mu}$ to determine the slenderness ratio. Furthermore, since specification allowable-stress formulas are obtained by dividing buckling-stress formulas by a factor of safety, the equivalent slenderness ratio $KL/(r_0\sqrt{\mu})$ can be used in allowable-stress formulas.

TABLE 9 Values of μ and K in Eqs. (18)*

Case (Fig. 10)	K	d_1/d_0 = 1	2	3	4	5	6
A	1	1	2.6	4.9	7.7	11.1	14.9
B	2	1	2.6	4.9	7.7	11.1	14.9
C	1	1	2.1	3.4	4.9	6.6	8.3
D	0.7	1	2.0	3.3	4.7	6.2	7.9
E	0.5	1	2.0	3.3	4.6	6.0	7.6

*From Ref. 8.

Based on work reported in Ref. 9, AISC gives a procedure for determining the allowable stress for uniformly tapered I-shaped columns, in which the rotational restraint of connecting beams can be accounted for in determining an effective length KL. Charts for determining the effective-length coefficient (denoted by K_γ in the specification) are given.

Buckling about the weak axis for the tapered members discussed in this article should be computed for the weak-axis radius of gyration at the small end, using $\mu = 1$.

Example 4 Compute the allowable load for the A36-steel column shown in Fig. 11 if it is supported against buckling in the weak direction. Use the AISC allowable-stress formula with the slenderness ratio based on K and μ from Table 9.

$$I_0 = 2 \times 6 \times 3.62^2 + \frac{1}{4} \times \frac{6.5^3}{12} = 157 + 6 = 163 \text{ in.}^4$$

$$A_0 = 2 \times 6 + \tfrac{1}{4} \times 6.5 = 13.6 \text{ in.}^2$$

$$r_0 = \sqrt{\frac{163}{13.6}} = 3.46$$

$$\frac{d_1}{d_0} = 2 \qquad \text{Case } C,\ \mu = 2.1$$

$$\frac{KL}{r_0\sqrt{\mu}} = \frac{1 \times 240}{3.46\sqrt{2.1}} = 48$$

$$F_a = 18.5 \qquad P = 18.5 \times 13.6 = 252 \text{ kips}$$

The AISC procedure based on K_γ gives the same result.

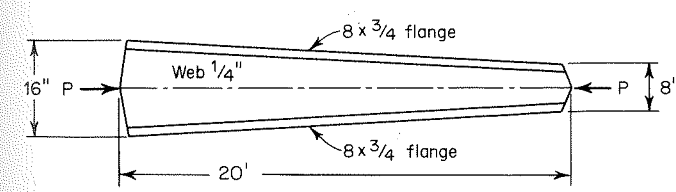

Fig. 11 Example 4.

16. Slender Compression Elements Compression elements whose slenderness exceeds the limits given in Art. 13 are not fully effective. This is because they buckle at stresses below the yield point. Elements of this type are common in cold-formed construction (Sec. 9). To provide for the occasional situation in which they may be used in structural members covered by the AISC specification, formulas and procedures similar to those discussed in Sec. 9 are given in Appendix C of the specification.