Column Bases

1. BASIC REQUIREMENTS

Base plates are required on the ends of columns to distribute the concentrated compressive load (P) of the column over a much larger area of the material which supports the column.

The base plate is dimensioned on the assumption that the overhanging portion of the base plate acts as a cantilever beam with its fixed end just inside of the column edges. The upward bending load on this cantilever beam is considered to be uniform and equal to the bearing pressure of the supporting material.

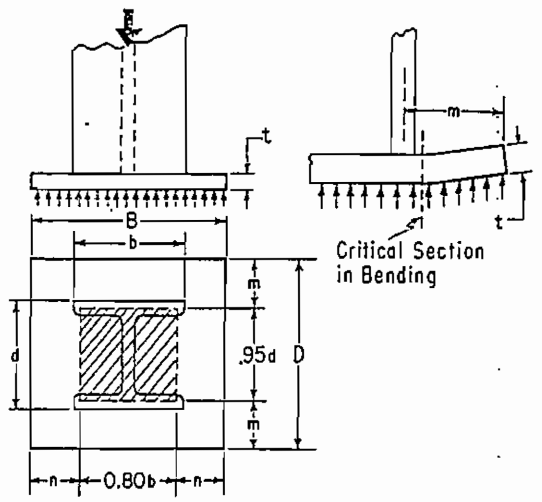


FIGURE 1

AISC suggests the following method to determine the required thickness of bearing plate, using a maximum bending stress of .75 σ_y psi (AISC Sec 1.5.1.4.8):

1. Determine the required minimum base plate area, A = P/p. The column load (P) is applied uniformly to the base plate within a rectangular area (shaded). The dimensions of this area relative to the column section's dimensions are .95 d and .80 b.

The masonry foundation is assumed to have a uniform bearing pressure (p) against the full area (A = B × D) of the base plate. See Table 1 for allowable values of p.

2. Determine plate dimensions B and D so that dimensions m and n are approximately equal. As a guide, start with the square root of required plate area (A). Table 2 lists standard sizes of rolled plate used for bearing plates.

3. Determine overhanging dimensions m and n, the projection of the plate beyond the assumed (shaded) rectangle against which the load (P) is applied.

$$m = \tfrac{1}{2}\,(D - .95\ d)$$
$$n = \tfrac{1}{2}\,(B - .80\ b)$$

4. Use the larger value of m or n to solve for required plate thickness (t) by one of the following formulas:

$$\boxed{t = m\sqrt{\frac{3p}{\sigma}}} \quad \boxed{t = n\sqrt{\frac{3p}{\sigma}}} \quad \ldots\ldots\ldots(1)$$

Derivation of Formula #1

The primary function of the plate thickness is to provide sufficient resistance to the bending moment (M) on the overhang of the plate just beyond the rectangular area contacted by the column. Treating this over-

TABLE 1—Masonry Bearing Allowables

(AISC Sec 1.5.5)

On sandstone and limestone	p = 400 psi
On brick in cement mortar	p = 250 psi
On full area of concrete support	p = 0.25 f'_c
On ⅓ area of concrete support	p = 0.375 f'_c

where f'_c is the specified compression strength of the concrete at 28 days (In this text, σ'_c is used as equivalent to AISC's f'_c.)

TABLE 2—Standard Sizes of Rolled Plate For Bearing Plates

14 × 1¼	28 × 3	44 × 6	60 × 7	72 × 9½
14 × 1½	28 × 3½	48 × 5½	60 × 7½	72 × 10
16 × 1½	32 × 3½	48 × 6	60 × 8	78 × 9
16 × 2	32 × 4	48 × 6½	66 × 7½	78 × 10
20 × 2	36 × 4	52 × 6	66 × 8	84 × 9½
20 × 2½	36 × 4½	52 × 6½	66 × 8½	84 × 10
20 × 3	40 × 4½	52 × 7	66 × 9	
24 × 2	40 × 5	56 × 6½	72 × 8	
24 × 2½	44 × 5	56 × 7	72 × 8½	
24 × 3	44 × 5½	56 × 8	72 × 9	

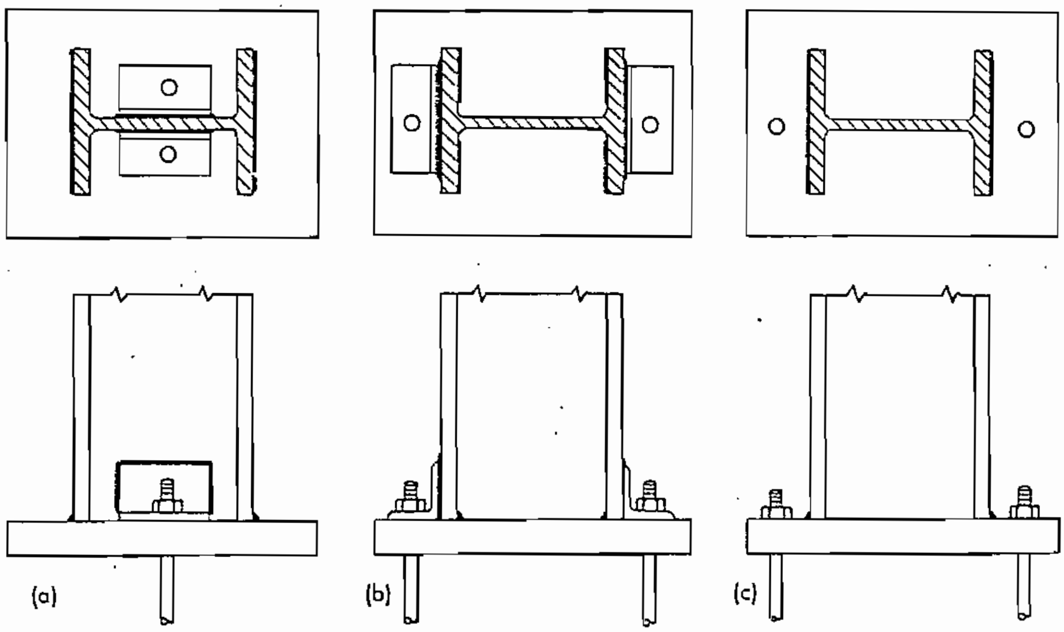


FIGURE 2

hang (m or n) as a cantilever beam with M being maximum at the fixed or column end:

bending moment

$$M = \frac{p\ m^2}{2} \text{ parallel to the column's x-x axis and}$$

$$M = \frac{p\ n^2}{2} \text{ parallel to the column's y-y axis}$$

bending stress in plate

$$\sigma = \frac{M}{S}$$

where, assuming a 1″ strip:

$$S = \frac{(1'')\ t^2}{6}$$

$$\therefore t^2 = 6\ S$$

and by substitution:

$$t^2 = 6\,\frac{M}{\sigma}$$

$$= \frac{6\ p\ m^2}{2\ \sigma} = \frac{3\ p\ m^2}{\sigma} \text{ and}$$

$$t = m\sqrt{\frac{3\ p}{\sigma}} \text{ or Formula \#1.}$$

(similarly for dimension n)

Finishing of Bearing Surfaces

AISC Sec 1.21.3 prescribes that column base plates be finished as follows:

"1. Rolled steel bearing plates, 2″ or less in thickness, may be used without planing, provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 2″ but not over 4″ in thickness may be straightened by pressing; or, if presses are not available, by planing for all bearing surfaces (except as noted under requirement 3) to obtain a satisfactory contact bearing; rolled steel bearing plates over 4″ in thickness shall be planed for all bearing surfaces (except as noted under requirement 3).

"2. Column bases other than rolled steel bearing plates shall be planed for all bearing surfaces (except as noted under requirement 3).

"3. The bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be planed."

The above requirements assume that the thinner base plates are sufficiently smooth and flat as rolled, to provide full contact with milled or planed ends of column bases. Thicker plates (exceeding 2″) are likely to be slightly bowed or cambered and thus need to be straightened and/or made smooth and flat.

2. STANDARD DETAILING PRACTICE

Figure 2 shows typical column bases. Note the simplicity of these designs for arc-welded fabrication.

Designs *a* and *b* are intended for where column and base plate are erected separately. The angles are shop welded to the column, and the column field welded to the base plate after erection. Design *c* is a standard of fabrication for light columns. Here the base plate is first punched for anchor bolts, then shop welded to the column.

If the end of the column is milled, there must be just sufficient welding to the base plate to hold all parts securely in place (AISC Sec 1:15.8). If the end of the column is not milled, the connecting weld must be large enough to carry the compressive load.

Welding Practices

In most cases, during fabrication, the columns are placed horizontally on a rack or table with their ends overhanging. The base plate is tack welded in place (Fig. 3), using a square to insure proper alignment, and is then finish welded.

As much as possible of the welding is done in the downhand position because of the increased welding speed through higher welding currents and larger electrodes. After completing the downhand welding, along the outside of the top flange, the column is rolled over and the downhand welding is applied to the other flange.

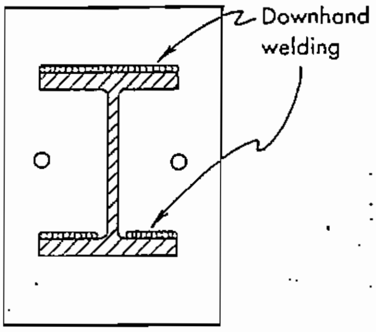


FIGURE 4

It is possible to weld the base plate to the column without turning. See Figure 4. With the web in the vertical position and the flanges in the horizontal position, the top flange is welded on the outside and the lower flange is welded on the inside. This will provide sufficient welding at the flanges without further positioning of the column.

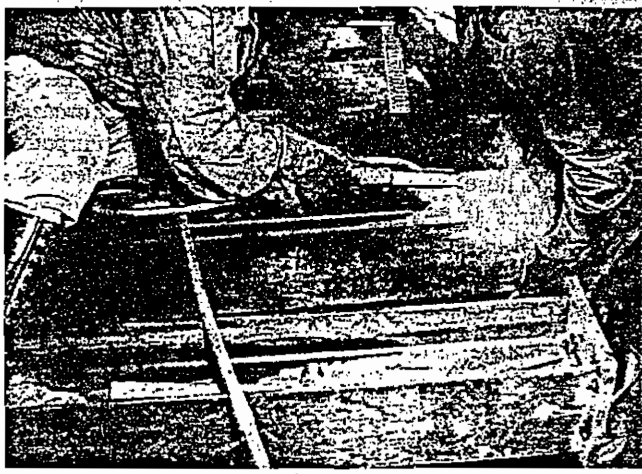


FIGURE 3

3. ANCHOR ATTACHMENTS TO COLUMN BASES

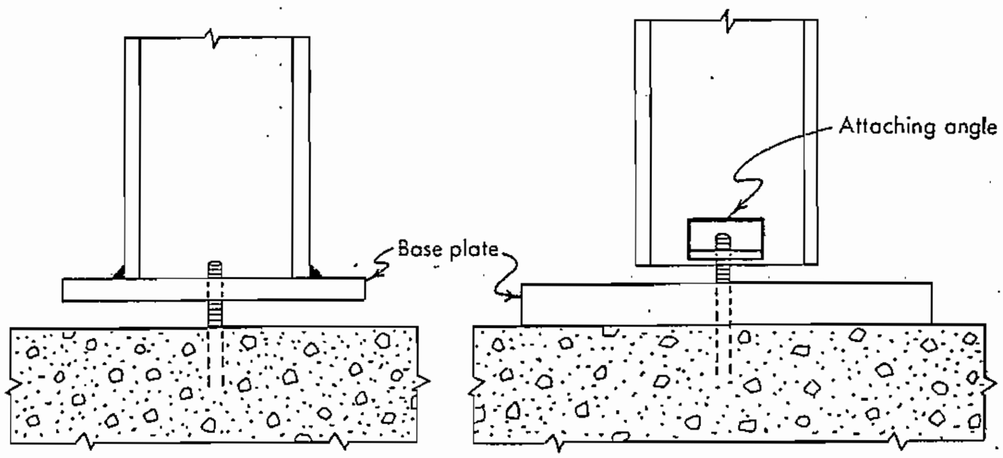
Anchor bolt details can be separated into two general classes.

First, those in which the attachments serve only for erection purposes and carry no important stresses in the finished structure. These include all columns that have no uplift. The design of these columns is governed by direct gravity loads and slenderness ratios set up by specifications for a given column formula.

Here the columns can be shop welded directly to the base plate, unless the detail is too cumbersome for shipment. The anchor bolts preset in the masonry are made to engage the base plate only. See Figure 5a. Large base plates are usually set and levelled separately before beginning column erection. In this case clip angles may be shop welded to the column web or flanges, and in field erection the anchor bolts engage both base plate and clip angle. See Figure 5b.

Secondly, those in which the attachments are designed to resist a direct tension or bending moment, or some combination in which the stability of the

FIGURE 5



(a) Base plate shop welded to column.

(b) Base plate shipped separate—attaching angles shop welded to column.

finished structure is dependent on the anchor attachments. These include all columns having direct loads combined with bending stresses, caused by the eccentric applications of gravity loads or horizontal forces; for example, wind, cable reactions, sway or temperature, etc. These are found in everyday practice in such structures as mill buildings, hangers, rigid frames, portals and towers, crane columns, etc.

In large structures that extend several hundred feet between expansion joints in each direction, the columns at ends and corners of the structure may be plumb only at normal temperature. As temperatures rise and fall, milled-end bearing conditions at edges or corners of the column base may prove very unsatisfactory, even though shop work were perfect. Such columns should have anchor bolt details designed to hold the column firmly fixed, in square contact with the base plate.

The combined effects of the direct load and overturning moments (due to wind, crane runway, etc.) can always be considered by properly applying the direct load at a given eccentricity, even though the bending stresses sometimes occur in two directions simultaneously. Design of the anchor bolts resolves itself into a problem of bending and direct stress.

4. HOLD-DOWN ANGLES

If there is any appreciable uplift on the column, angles may be welded to the base of the column and anchored by means of hold-down bolts. Under load, the angle is subject to a bending action, and its thickness may be determined from this bending moment.

Treating the cross-section of the angle as a frame, the problem is to know the end conditions.

Some engineers treat the horizontal leg as a cantilever beam, fixed at one end by the clamping action of the hold-down bolts. See Figure 6. This is not quite a true picture because there is some restraint offered by the other leg of the angle.

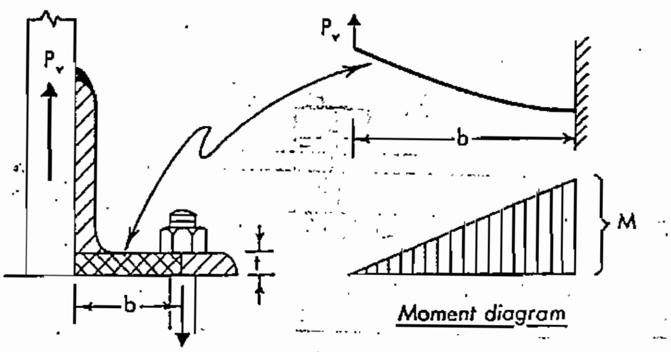


FIGURE 6

$$\boxed{M = P_v\, b} \quad \dots\dots\dots (2)$$

Other engineers have assumed the horizontal leg of the angle acts as a beam with both ends fixed. In this case the resulting moment at either end of the portion being considered, the heel of the angle or the end at the bolt, is only half that indicated by the previous approach. See Figure 7.

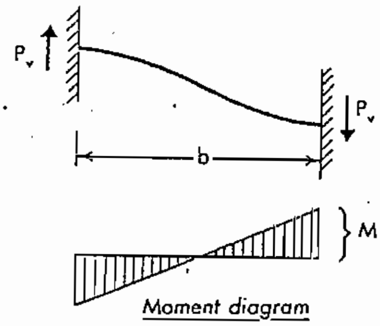


FIGURE 7

$$\boxed{M = \frac{P_v\, b}{2}} \quad \dots\dots\dots (3)$$

However, it might be argued that the vertical leg is not completely fixed and that this will increase the moment in the horizontal leg near the bolt. The following analysis, made on this basis, is probably more nearly correct. See Figure 8.

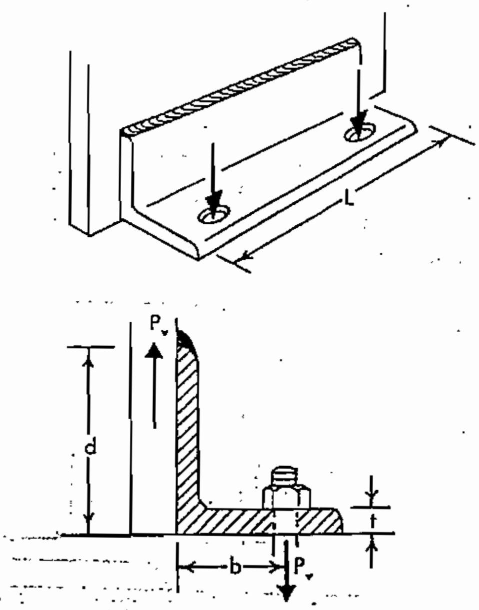
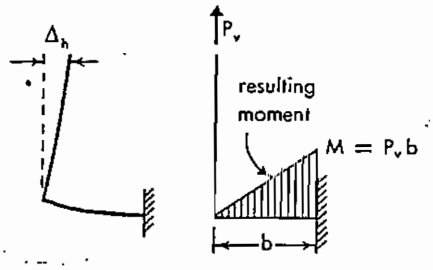


FIGURE 8

1. Considering first just one angle and temporarily ignoring the effect of the other, the upper end of the vertical leg if not restrained would tend to move in horizontally (Δ_h) when an uplift force (P_v) is applied to the column.



The resulting moment is

$$M = P_v\, b \text{ and}$$

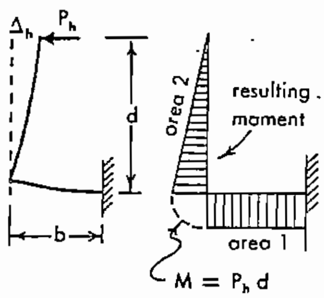
$$\Delta_{hv} = \frac{\text{area of moment diagram}}{E\, I} \times \text{moment arm}$$

$$= \frac{\tfrac{1}{2}\,(P_v\, b)(b)(d)}{E\, I}$$

$$= \frac{P_v\, b^2\, d}{2\, E\, I}$$

2. Since the opposite angle does provide restraint, a horizontal force (P_h) is applied to pull the vertical leg back to its support position. The resulting moment is

$$M = P_h\, d \text{ and}$$



$$\Delta_{hh} = \frac{\text{area 1} \times \text{moment arm 1}}{E\, I} + \frac{\text{area 2} \times \text{moment arm 2}}{E\, I}$$

$$= \frac{(P_h\, d)b\, d}{E\, I} + \frac{\tfrac{1}{2}(P_h\, d)(d)\tfrac{2}{3}\, d}{E\, I}$$

$$= \frac{P_h\, d}{3\, E\, I}(3b + d)$$

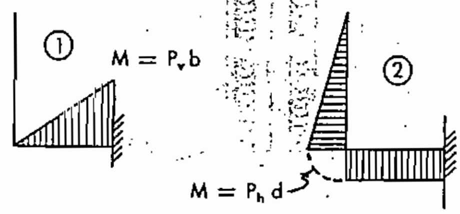
Since the horizontal movement is the same in each direction:

$$\Delta_{hh} = \Delta_{hv}$$

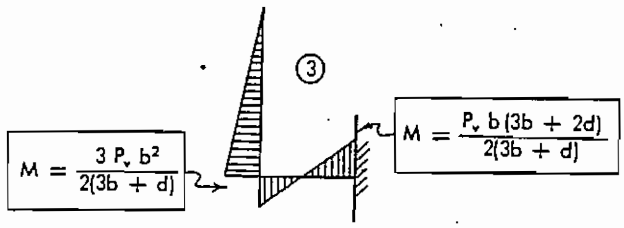
$$\therefore \frac{P_h\, d}{3\, E\, I}(3b + d) = \frac{P_v\, b^2\, d}{2\, E\, I} \text{ or}$$

$$P_h = \frac{3\, P_v\, b^2}{2\, d(3b + d)}$$

3. Combining the initial moment resulting from the uplift force (1) and the secondary moment resulting from the restraint offered by the opposite angle (2):



gives—



Substituting into the previous equations:

$$\boxed{M = \frac{3\, P_v\, b^2}{2(3b + d)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

at the heel of the angle, and

$$\boxed{M = \frac{P_v\, b\,(3b + 2d)}{2(3b + d)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

which is the critical moment and is located at the hold-down bolts.

Required Thickness of Angle

The leg of the angle has a section modulus of—

$$S = \frac{L\, t^2}{6}$$

or required thickness of

$$t = \sqrt{\frac{6\, S}{L}} \qquad \text{where: } S = \frac{M}{\sigma}$$

or, see Figure 9, where the vertical leg of the angle is welded its full length to the column providing a fixed-end condition (Case A); here formula #3 applies—

$$\boxed{t = \sqrt{\frac{3\, P_v\, b}{L\, \sigma}} \qquad \text{Case (A)}} \quad \ldots\ldots\ldots\ldots (6)$$

or where, the vertical leg of the angle is welded only

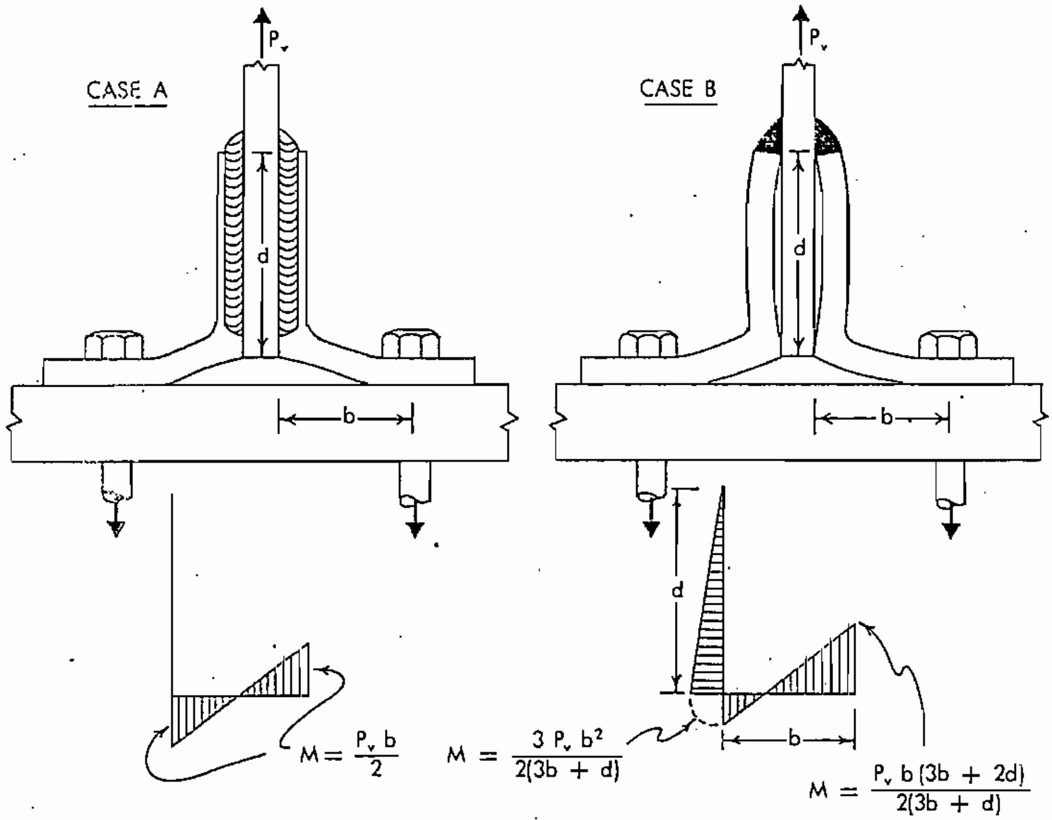


FIGURE 9

at its toe to the column (Case B); here formula #5 applies—

$$t = \sqrt{\frac{3\,P_v\,b\,(3b + 2d)}{L\,(3b + d)\,\sigma}} \qquad \text{Case (B)} \qquad \ldots(7)$$

Allowable Stresses

Table 3 presents the allowable stresses for holddown bolts used in building (AISC) and in bridge (AASHO) construction. Also included are dimensions of standard bols. (Table 3A).

TABLE 3—Allowable Stresses for Hold-Down Bolts

Allowable unit tension and shear stresses on bolts and threaded parts (psi of unthreaded body area):

	Tension psi	Shear psi
AISC 1.5.2.1 (Building)		
A307 bolts and threaded parts of A7 and A373 steel	14,000	10,000
A325 bolts when threading is not excluded from shear planes	40,000	15,000
A325 bolts when threading is excluded from shear planes	40,000	22,000
A354, Grade BC, bolts when threading is not excluded from shear planes	50,000	20,000
A354, Grade BC, when threading is excluded from shear planes	50,000	24,000
AASHO 1.4.2 (Bridge)		psi
tension — bolts at root of thread		13,500
shear — turned bolts		11,000
bearing — turned bolts		20,000

Effective bearing area of a pin or bolt shall be its diameter multiplied by the thickness of the metal on which it bears.

5. BASE PLATE FOR COLUMN LOADED WITH MOMENT

When a moment (M) is applied to a column already subjected to an axial compressive force (P_c), it is more convenient to express this combined load as the same axial force (P_c) applied at some eccentricity (e) from the neutral axis of the column.

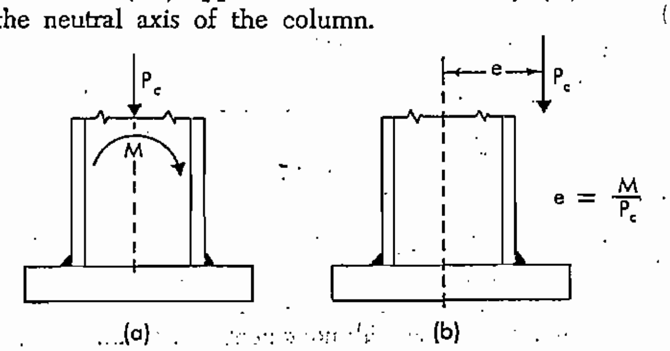


FIGURE 10

In either representation, there is a combination of axial compressive stress and bending stress acting on a cross-section of the column. See Figure 11.

Multiplying this stress by the width of the flange (or the thickness of the web) over which the stresses are applied, gives the following force distribution

TABLE 3A—Standard Bolt Dimensions

Bolt Diameter	No. of threads per inch	Area of bolt	Net area at root of thread	Bolt Diameter	No. of threads per inch	Area of bolt	Net area at root of thread
¼"	20	.049	.026	2"	4½	3.142	2.302
5/16"	18	.076	.045	2¼"	4½	3.976	3.023
⅜"	16	.110	.068	2½"	4	4.909	3.719
7/16"	14	.150	.093	2¾"	4	5.940	4.620
½"	13	.196	.126				
9/16"	12	.248	.162	3"	3½	7.069	5.428
⅝"	11	.307	.202	3¼"	3½	8.296	6.510
¾"	10	.442	.302	3½"	3¼	9.621	7.548
⅞"	9	.601	.419	3¾"	3	11.045	8.641
1"	8	.785	.551	4"	3	12.566	9.963
1⅛"	7	.994	.694	4¼"	2⅞	14.186	11.340
1¼"	7	1.227	.893	4½"	2¾	15.904	12.750
1⅜"	6	1.485	1.057	4¾"	2⅝	17.721	14.215
1½"	6	1.767	1.295				
1⅝"	5½	2.074	1.515	5"	2½	19.635	15.760
1¾"	5	2.405	1.746	5¼"	2½	21.648	17.570
1⅞"	5	2.761	2.051	5½"	2⅜	23.758	19.260
				5¾"	2⅜	25.967	21.250
				6"	2¼	28.274	23.090

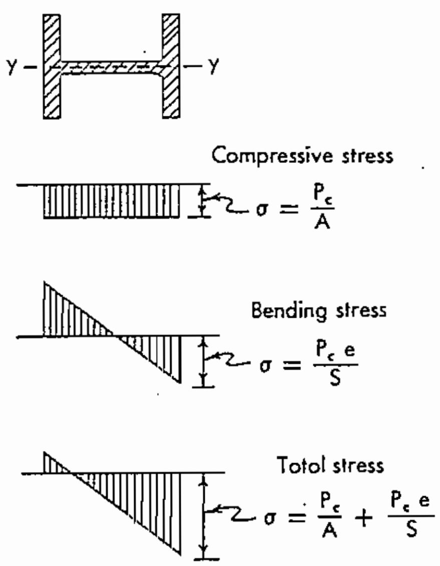


FIGURE 11

across the depth of the column. This force is transferred to the base plate. See Figure 12. This assumes that the column flanges are welded directly to the base plate.

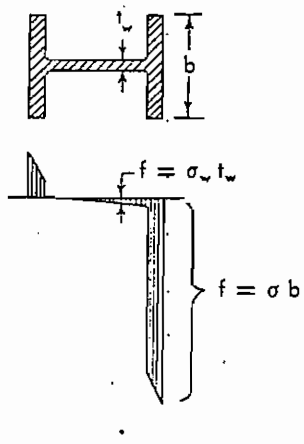


FIGURE 12

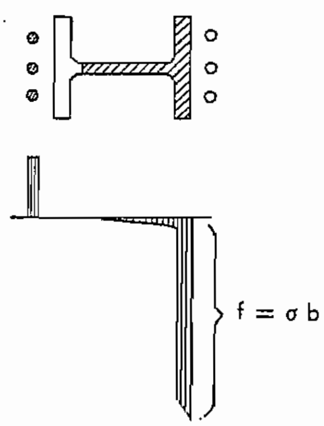


FIGURE 13

If anchor hold-down bolts transfer the tensile forces, then—

The column is usually set with the eccentricity (e) lying within the plane of the column web (axis y-y), as in Figure 11. Thus the column flanges will carry most of the resulting forces because of their having relatively greater cross-sectional area, and being located in areas of higher stress. See Figure 14.

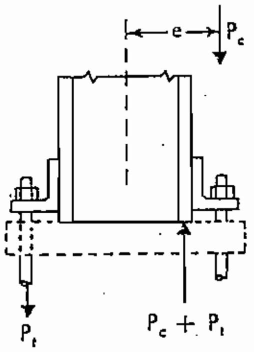


FIGURE 14

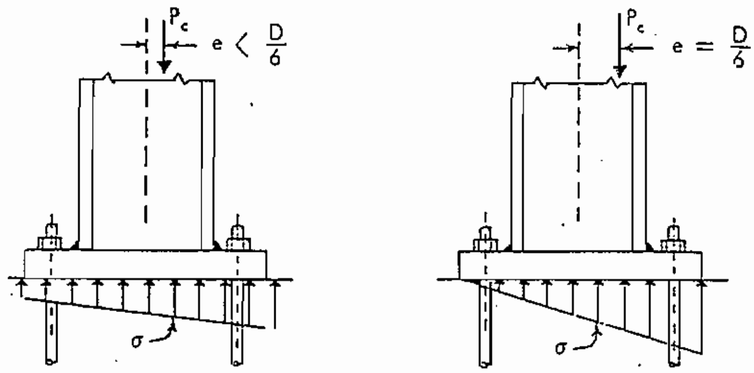
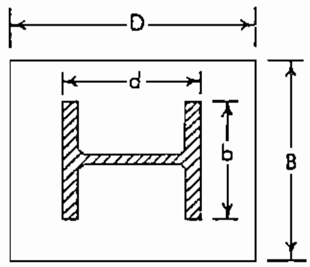


FIGURE 15



If the eccentricity (e) *is less than* ⅙ D, there is no uplift of the base plate at the surface of the masonry support (Figure 15):

section modulus of base plate

$$S = \frac{B\ D^2}{6} \qquad A = B \times D$$

stress in base plate

$$\sigma_T = \sigma_1 \text{ compression} \pm \sigma_2 \text{ bending}$$

$$= \frac{P_c}{A} \pm \frac{P_c\ e}{S}$$

When the eccentricity (e) *exceeds* ⅙ D, there is uplift on the base plate which is resisted by the anchor hold-down bolts. The bearing stress on the masonry support is maximum at the extreme edge of the bearing plate. It is assumed this stress decreases linearly back along the plate for a distance (Y); however, there is some question as to how far this extends. One problem analysis approach treats this section as a reinforced concrete beam.

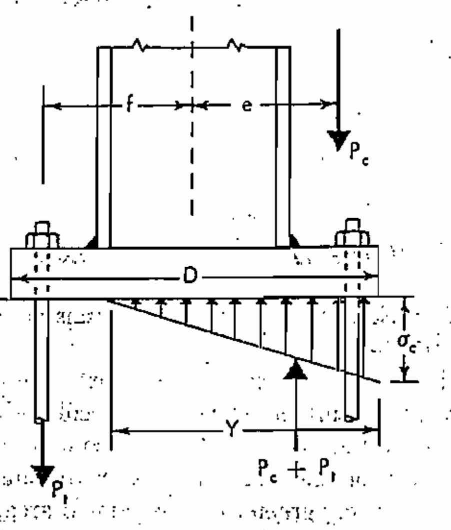


FIGURE 16

Basic Method (If Uplift)

There are three equations, and three unknowns (P_t), (Y), and (σ_c):

1. $\Sigma\ V = 0$

$$\tfrac{1}{2}\ Y\ \sigma_c\ B - P_t - P_c = 0$$

or

$$\boxed{P_c + P_t = \frac{\sigma_c\ Y\ B}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8a)$$

and

$$\boxed{\sigma_c = \frac{2(P_c + P_t)}{Y\ B}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8b)$$

where: σ_c = pressure supplied by masonry supporting material

2. $\Sigma\ M = 0$ (About N.A. of column)

$$P_t\ f + (P_c + P_t)\left(\frac{D}{2} - \frac{Y}{3}\right) - P_c\ e = 0$$

or

$$\boxed{P_c = -P_t \left[\frac{\frac{D}{2} - \frac{Y}{3} + f}{\frac{D}{2} - \frac{Y}{3} - e}\right]} \quad \ldots\ldots\ldots (9a)$$

and

$$\boxed{P_t = -P_c \left[\frac{\frac{D}{2} - \frac{Y}{3} - e}{\frac{D}{2} - \frac{Y}{3} + f}\right]} \quad \ldots\ldots\ldots (9b)$$

3. Representing the elastic behavior of the concrete support and the steel hold-down bolt (see Figure 17):

$$\frac{a}{b} = \frac{\epsilon_s}{\epsilon_c} = \frac{\dfrac{\sigma_s}{E_s}}{\dfrac{\sigma_c}{E_c}}$$

$$= \frac{\sigma_s\, E_c}{\sigma_c\, E_s}$$

since: $E_s = \dfrac{\sigma_s}{\epsilon_s}$

$E_c = \dfrac{\sigma_c}{\epsilon_c}$

Also

$$\sigma_s = \frac{P_t}{A_s}$$

and letting

$$n = \frac{E_s}{E_c}$$

then

$$\frac{a}{b} = \frac{\dfrac{P_t}{A_s}}{\sigma_c\, n} = \frac{P_t}{A_s\, \sigma_c\, n}$$

and from similar triangles

$$\frac{a}{b} = \frac{\dfrac{D}{2} - Y + f}{Y}$$

where:

A_s = total area of steel hold-down bolts under tension

σ_s = stress in steel bolt

ϵ_s = strain in steel bolt

E_s = modulus of elasticity of steel bolt

and:

σ_c = stress in concrete support

ϵ_c = strain in concrete support

E_c = modulus of elasticity of concrete support

n = modular ratio of elasticity, steel to concrete

so

$$\frac{P_t}{A_s\, \sigma_c\, n} = \frac{\dfrac{D}{2} - Y + f}{Y}$$

or

$$\boxed{\sigma_c = \frac{P_t\, Y}{A_s\, n\left(\dfrac{D}{2} - Y + f\right)}} \quad \cdots\cdots\cdots\cdots (10)$$

* * *

Substituting formula #10 into formula #8a:

$$P_c + P_t = \frac{P_t\, Y}{A_s\, n\left(\dfrac{D}{2} - Y + f\right)}\left(\frac{Y\,B}{2}\right)$$

or

$$\boxed{P_c + P_t = \frac{P_t\, Y^2\, B}{2\, A_s\, n\left(\dfrac{D}{2} - Y + f\right)}} \quad \cdots\cdots (11)$$

Substituting formula #9a into formula #11:

$$-P_t\left[\frac{\dfrac{D}{2} - \dfrac{Y}{3} + f}{\dfrac{D}{2} - \dfrac{Y}{3} - e}\right] + P_t = \frac{P_t\, Y^2\, B}{2\, A_s\, n\left(\dfrac{D}{2} - Y + f\right)}$$

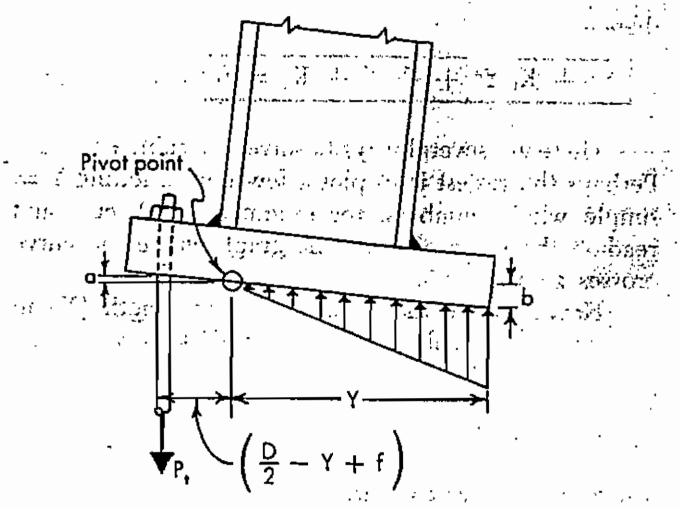


FIGURE 17

Solve for Y:

$$-2\,n\,A_s\left(\frac{D}{2} - Y + f\right)\left(\frac{D}{2} - \frac{Y}{3} + f\right)$$

$$+\left(\frac{D}{2} - \frac{Y}{3} - e\right)\left(2\,n\,A_s\right)\left(\frac{D}{2} - Y + f\right)$$

$$= Y^2\,B\left(\frac{D}{2} - \frac{Y}{3} - e\right)$$

or

$$-\frac{n\,A_s\,D^2}{2} + \frac{4\,n\,A_s\,D\,Y}{3} - 2\,n\,A_s\,D\,f - \frac{2\,n\,A_s\,Y^2}{3}$$

$$+ \frac{8\,n\,A_s\,f\,Y}{3} - 2\,n\,A_s\,f^2 + \frac{n\,A_s\,D^2}{2} - \frac{4\,n\,A_s\,D\,Y}{3}$$

$$-n\,A_s\,D\,e + \frac{2\,n\,A_s\,Y^2}{3} + 2\,n\,A_s\,e\,Y + n\,A_s\,D\,f$$

$$-\frac{2\,n\,A_s\,f\,Y}{3} - 2\,n\,A_s\,e\,f = \frac{B\,D}{2}\,Y^2 - \frac{B\,Y^3}{3} - B\,e\,Y^2$$

This reduces to—

$$\boxed{Y^3 + 3\left(e - \frac{D}{2}\right)Y^2 + \frac{6\,n\,A_s}{B}\left(f + e\right)Y - \frac{6\,n\,A_s}{B}\left(\frac{D}{2} + f\right)(f + e) = 0} \quad ..(12)$$

or to express it in a manner to facilitate repetitive use, let—

$$K_1 = 3\left(e - \frac{D}{2}\right)$$

$$K_2 = \frac{6\,n\,A_s}{B}\left(f + e\right)$$

$$K_3 = -\,K_2\left(\frac{D}{2} + f\right)$$

then—

$$\boxed{Y^3 + K_1 Y^2 + K_2 Y + K_3 = 0} \quad \ldots\ldots\ldots (13)$$

There are several ways to solve this cubic equation. Perhaps the easiest is to plot a few points, letting Y = simple whole numbers, for example, 9, 10, etc., and reading the value of Y on the graph where the curve crosses zero.

Having found the effective bearing length (Y) in this manner, formula #9b can be used to solve for the tensile force (P_t) in the hold-down bolts. Formula #10 then gives the amount of bearing stress in the masonry support.

Alternative Shorter Method

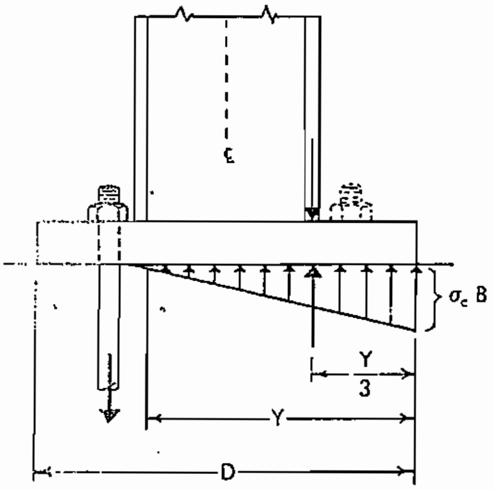


FIGURE 18

Another approach to determining the effective bearing length, involving less work, assumes the same triangular distribution of bearing forces from the supporting masonry against the bearing plate. However, the center of gravity of the triangle, or the concentrated force representing this triangle, is assumed to be fixed at a point coinciding with the concentrated compressive force of the column flange. See Figure 18.

From this assumption, the overhang of the bearing plate, i.e. the distance from the column flange to the plate's outer edge, is seen to equal ⅓ the effective bearing length.

Problem 1

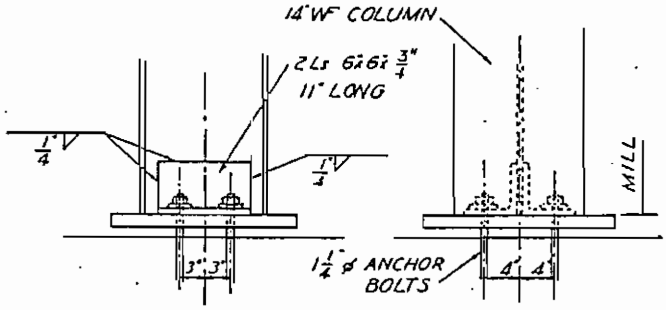


FIGURE 19

Figure 19 shows a column base detail. The columns have a maximum load of 186 kips, and receive no uplift under normal wind. See Figure 19. Under heavier wind load and in combination with temperature, they may receive up to 20 kips direct uplift. See Figure 20. Four bolts are provided, attached by means of 6″ × 6″ × ¾″ clip angles, 11″ long on a 4″ gauge.

To be effective, the angles must carry this load on the anchor bolts into the column web. This causes a bending moment on the outstanding legs of the angles. Analysis follows that for formula #3. The bolt tension fixes the toe of the angle against the base plate and causes an inflection point between the bolts and the vertical leg of the angle, so that the bolt load is cantilevered only about halfway.

$$M_{max} = \frac{P\ b}{2}$$

To compute the bending stress in the angles:

$$\sigma_b = \frac{M\ c}{I}$$

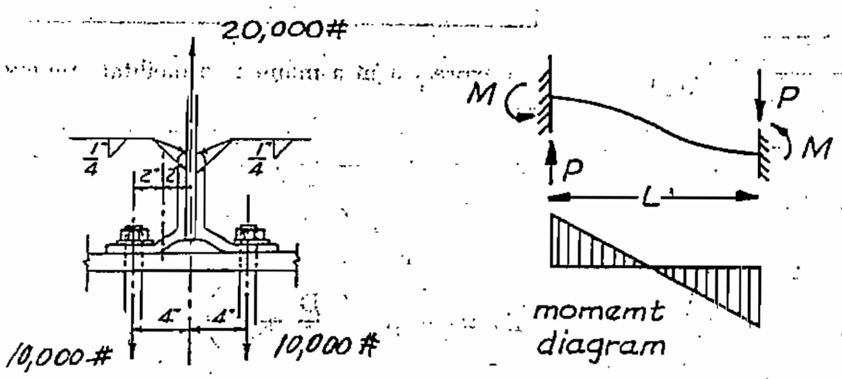


FIGURE 20

where:

σ_b = stress in outer fibers

M = bending moment

c = distance to neutral axis

I = moment of inertia

Since:

$$I = \frac{(11'')(3/4'')^3}{12}$$

$$= .386 \text{ in.}^4$$

$$\therefore \sigma_b = \frac{M\ c}{I}$$

$$= \frac{(10{,}000\ \# \times 2'')(3/8'')}{(.386)}$$

$$= 19{,}400 \text{ psi}$$

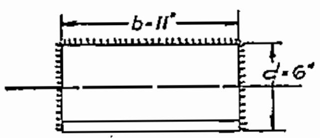
Hence, the detail with ¾″ angles is OK for this load.

Check Welds to Column Web

The angles are welded to the column web with ¼″ fillet welds; this will now be checked.

The heel of the angle is in compression against the web of the column and is equivalent to an additional weld across the bottom for resisting moment. On this basis, the section modulus of the weld is calculated. For simplicity, the weld is treated as a line without any cross-sectional area. From Table 5, Sect. 7.4, the section modulus of a rectangular connection is:

$$S_w = b\ d + \frac{d^2}{3}$$



and here:

$$S_w = (11)(6) + \frac{(6)^2}{3}$$

$$= 78 \text{ in.}^2$$

Normally, section modulus is expressed as inches to the third power; however, here where the weld has no area, the resulting section modulus is expressed as inches squared.

When a standard bending formula is used, the answer (σ) is stress in lbs/in.²; however, when this new section modulus is used in the bending formula, the answer (f) is force on the weld in lbs/linear in.

bending

$$f_b = \frac{M}{S_w}$$

$$= \frac{(10{,}000\# \times 4'')}{(78 \text{ in.}^2)}$$

$$= 513 \text{ lbs/in.}$$

shear

$$f_s = \frac{P}{L_w}$$

$$= \frac{(10{,}000\#)}{(23'')}$$

$$= 435 \text{ lbs/in.}$$

resultant force on weld

$$f_r = \sqrt{f_b^2 + f_s^2}$$

$$= \sqrt{(513)^2 + (435)^2}$$

$$= 673 \text{ lbs/in.}$$

leg size of (E70) fillet weld

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$

$$= \frac{(673)}{(11{,}200)}$$

$$= .06''$$

but ¾″-thick angle requires a minimum of ¼″ ◺ (Table 3, Section 7.4).

If it is desired to increase the anchor bolt capacity of the clip angle detail, thicker angles should be used with large plate washers on top of the angle. The attachments should be made to the column flanges, since the welds are more accessible there and the bolts have better leverage.

Problem 2

To illustrate how the column flange can be checked to determine whether or not it is too thin, consider a clip angle anchored with two 1¼″ bolts centered 2½″ out from the face of the column flange; see Figure 21. The angle is attached to the column flange by fillet welds across the top and down each side.

The capacity of the two bolts at 14,000 psi allowable stress on unthreaded area (AISC Sec 1.5.2) is—

2 (1.227)(14,000) = 34,400 lbs > 28,500 lbs OK

The bending moment on the weld is—

(28,500 lbs)(2½″) = 71,250 in.-lbs

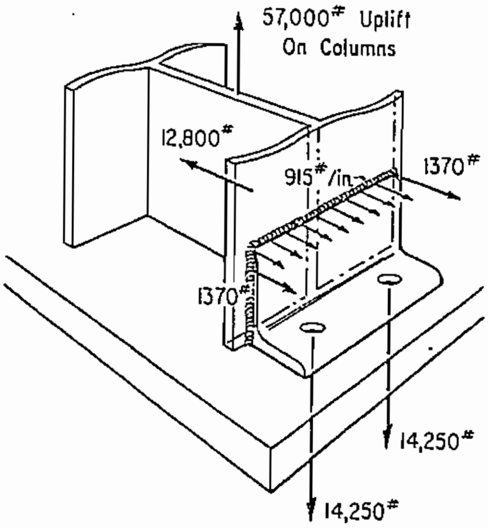


FIGURE 21

As in the previous example, the heel of the angle is in compression against the web of the column and is replaced with an equivalent weld. The welds are treated as a line, and the section modulus of the welded connection is found to be—

$$S_w = b\ d + \frac{d^2}{3}$$

$$= (11)\ 6) + \frac{(6)^2}{3}$$

$$= 78 \text{ in.}^2 \quad \text{(See Problem 1)}$$

The bending force is—

$$f_b = \frac{M}{S_w}$$

$$= \frac{71{,}250 \text{ in.-lbs}}{78 \text{ in.}^2}$$

$$= 915 \text{ lbs/in.}$$

all along the top edge of the angle, pulling outward on the column flange. This is the force on the horizontal top weld. At the ends of the angle, the force couple is $\frac{(915)(3)}{2} = 1370$ lbs centered 1″ below the top toe of the angle. See Figure 22.

This is the force on each of the vertical welds at ends of the angle. Since these forces are not resisted by anything but the flange, they have to be carried transversely by bending stresses in the flange until they reach the resistance in the column web.

The bending moment in the column flange is computed as follows:

$$\text{Force along top of angle} = 915 \times 5.5 = 5040 \text{ lbs}$$

$$M_h = 5040 \times 2.75 = 13{,}860 \text{ in.-lbs}$$

$$M_v = 1370 \times 5.5 = \underline{7{,}535} \text{ in.-lbs}$$

$$\text{Total } M = 21{,}395 \text{ in.-lbs}$$

If we assume a 6″ wide strip of the column flange to resist this load, this moment will cause a bending stress of 45,300 psi in the 14″ WF 87-lb column with a flange $^{11}/_{16}$″ thick.

This is calculated as follows:

$$I = \frac{(6'')(^{11}/_{16}'')^3}{12}$$

$$= .1625 \text{ in.}^4$$

$$\sigma_b = \frac{M\ c}{I}$$

$$= \frac{(21{,}395)(^{11}/_{32})}{(.1625)}$$

$$= 45{,}300 \text{ psi}$$

Obviously, since this stress distribution along the welds is capable of bending the column flange beyond the yield point, the column flange will deflect outward sufficiently to relieve these stresses and cause a redistribution. The resultant stresses in the weld metal on the toe of the clip angle will be concentrated opposite the column web.

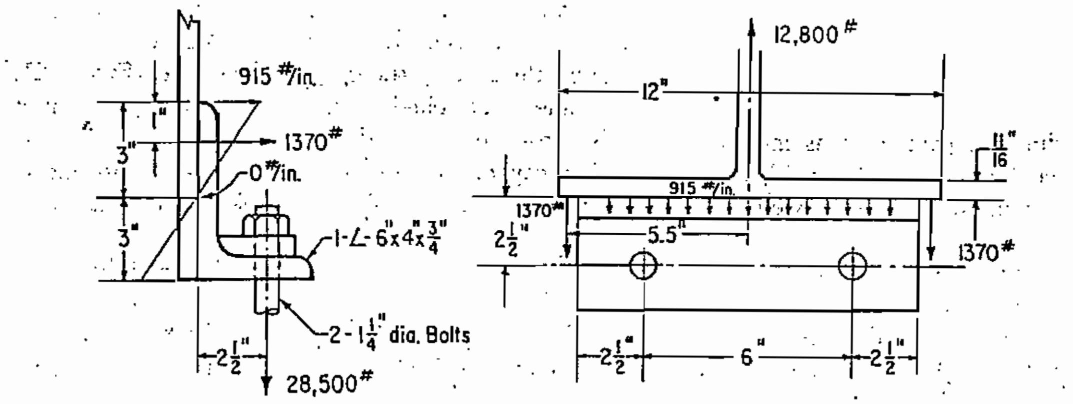


FIGURE 22

Thus, the capacity of this anchor bolt detail is limited by the bending strength of the column flange even after the clip angle has been satisfactorily stiffened.

The force back through the column web is:

$$F = (915 \text{ lbs/in.})\ (11'') + 2\ (1370 \text{ lbs})$$
$$= 12{,}800 \text{ lbs}$$

A ½″ fillet weld 3 inches long on the top of the angle opposite the column web will satisfactorily resist the force couple:

$$F = (3'')\ (5600 \text{ lbs/in.}) \qquad \text{E70 welds}$$
$$= 16{,}800 \text{ lbs.} \qquad \underline{\text{OK}}$$

For greater anchor bolt capacities than shown in Figure 22, either horizontal stiffeners or diaphragms should be provided to prevent bending of the column flanges.

Problem 3

A rather simple detail, whereby a wide-flanged channel serves as a stiffener, is shown in Figure 23.

This detail was used with three 1⅝″-dia anchor bolts on a 14″ × 87-lb mill building column designed to resist a wind bending moment of 175,000 ft-lbs combined with a direct load downward of 130,000 lbs.

The tension on the bolts is determined by taking moments about the right-hand compression flange of the column after first determining the eccentricity at which the direct load will cause a moment of 175,000 ft-lbs about the centerline of the column. The eccentricity is—

$$e = \frac{(175{,}000)(12)}{(130{,}000)}$$
$$= 16.15''$$

The load on the bolts is—

$$F = \frac{(130{,}000)(9.49)}{(15.66)}$$
$$= 78{,}800 \text{ lbs}$$

The area of the three 1⅝″ dia. bolts in the unthreaded body area is—

$$A = (3)(2.074)$$
$$= 6.22 \text{ in.}^2$$

The tensile stress in the bolts is:

$$\sigma = \frac{(78{,}800)}{(6.22)}$$
$$= 12{,}700 \text{ psi} < 14{,}000 \text{ psi} \qquad \underline{\text{OK}}$$

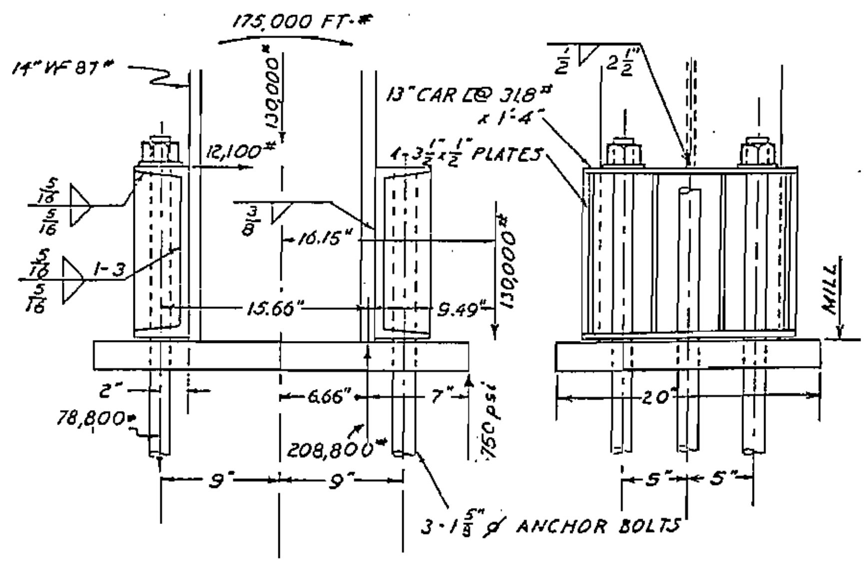
(AISC Sec 1.5.2)

The compression flange reaction (R) is the sum of the 130,000-lb column load plus the 78,800-lb pull of the anchor bolts, or 208,800 lbs. The 13″ ship channels are set up just clear of the bearing on the base plate so that the end of the column will take the compressive load of 208,800 lbs without overloading channels.

Bearing stress on masonry

The bearing stress on the masonry support is maximum at the extreme edge of the bearing plate, and is assumed to decrease linearly back along the plate. This bearing stress would resemble a triangle in which

FIGURE 23



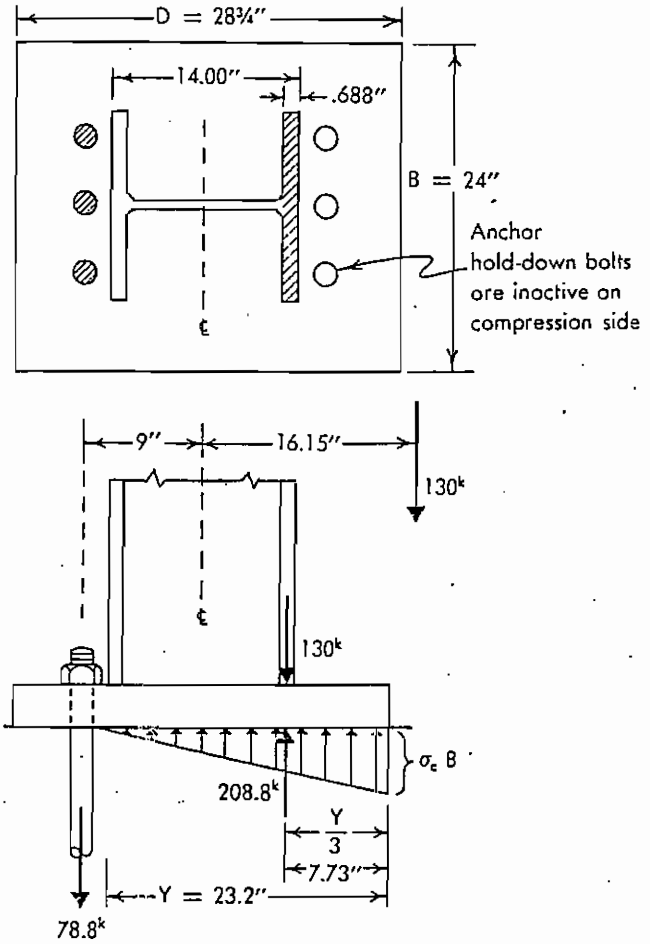


FIGURE 24

the altitude is the maximum bearing stress at the edge of the plate, and the base of the triangle is the effective bearing length (Y) against the plate. (See short method described on page 10.) Since the area of this triangle has a center of gravity ⅓ Y back from the altitude, the bearing pressure may be resolved into a concentrated force at this point. This point will be assumed to lie where the column flange's concentrated compressive load of 208,800 lbs is applied.

Hence, the distance from the compressive force of the flange out to the edge of the bearing plate (in other words, the overhang of the bearing plate) equals ⅓ the effective distance of the bearing support. See Figure 24.

area of triangle

$$A = \tfrac{1}{2}\,\sigma_c\,Y$$
$$= P_c + P_t$$

effective bearing length of base plate (from formula #8)

$$Y = \frac{2(P_c + P_t)}{\sigma_c\,B}$$
$$= \frac{2(130^k + 78.8^k)}{(750)(24)}$$
$$= 23.2''$$

Let:

$$B = 24''$$
$$\sigma_c = .25\,\sigma'_c$$
$$= .25\ (3000\ \text{psi})$$
$$= 750\ \text{psi}$$

$$\text{and } \frac{Y}{3} = 7.73''\ \text{overhang}$$

$$\therefore D = 7.73'' + 13.31'' + 7.73''$$
$$= 28.77''\ \text{or use}\ \underline{28\tfrac{3}{4}''}$$

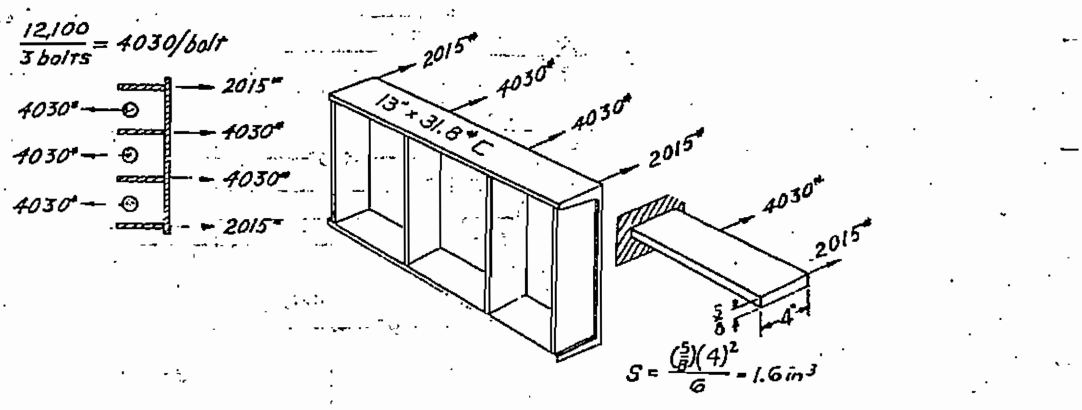
Bolt load

The load on the bolts is supported by the top flange of the 13″ channel, reinforced by four 3⅝″ × ½″ stiffener plates welded between the channel flanges. See Figure 23.

The two interior plates each support a full bolt load of ⅓ (78,800 lbs) or 26,300 lbs. These stiffeners are attached to the channel web with four 1″ × $\frac{5}{16}$″ intermittent fillet welds on each side of the plate, and to both flanges by continuous $\frac{5}{16}$″ fillet welds on each side of the plate. See Figure 25. The welds at the channel flanges transmit the moment to the channel flanges, and the welds at the channel web support most of the shearing load.

The 2″ eccentricity of the bolt load to column flange is transposed to a force couple acting on the channel flanges. This couple is obtained by dividing

FIGURE 25



the moment by the depth of the stiffeners:

$$C = \frac{(78{,}800)(2)}{(13)}$$

$$= 12{,}100 \text{ lbs}$$

This is a horizontal load acting at right angles to the column flange. It is delivered as four concentrated loads at the tops of stiffeners and then carried horizontally by the channel flange to a point opposite the column web where it is attached to the column with a 2½" × ½" fillet weld.

2½" × 5600 lbs/in. = 14,000 lbs.

The concentrated load values are 2015 lbs at each end stiffener for one-half a bolt load, and 4030 lbs at each interior stiffener.

The total moment on the flanges is:

$$(2{,}015)(7.5) = 15{,}200 \text{ in.-lbs}$$
$$(4{,}030)(2.5) = \underline{10{,}100} \text{ in.-lbs}$$
$$M = 25{,}300 \text{ in.-lbs}$$

It causes a bending stress in the channels 4" × ⅝" top flange section of approximately—

$$\sigma_b = \frac{M}{S}$$

$$= \frac{(25{,}300)}{(1.6)}$$

$$= 15{,}800 \text{ psi}$$

To keep the channel section from sliding parallel to the column flange, the direct vertical pull of the bolts is supported by two 13" × 5⁄16" continuous fillet welds between the edge of the column flanges and the web of the 13" channel section. The shear on these welds is—

$$f_s = \frac{(78{,}800)}{(2)(13)}$$

$$= 3030 \text{ lbs/in.}$$

$$\omega = \frac{(3030)}{(11{,}200)} \qquad \text{E70-weld allowable}$$

$$= .276'' \text{ or use } \underline{5/16'' \text{ fillet}}$$

The problem in Figure 23 has been analyzed on the basis of simple levers with the compression load concentrated on the column flange. It ignores the compression area under the web of the column and illustrates the problem where the channel flange of the anchor bolt attachment does not bear against the base plate.

For simplicity, this analysis has assumed that the effective bearing length (Y) was such that the center of gravity of the triangular bearing stress distribution, C.G. at ⅓ Y, lies along the centerline of the column flange where the compressive force of the column is applied.

Problem 4

With the same column base detail as in Problem 3, we will now use the original derivation for this effective bearing length (Y), treating the analysis as a reinforced concrete beam and solving the resulting cubic equation. The work may take longer, but results are more accurate. See Figure 26, temporarily ignoring the anchor-bolt channel attachments.

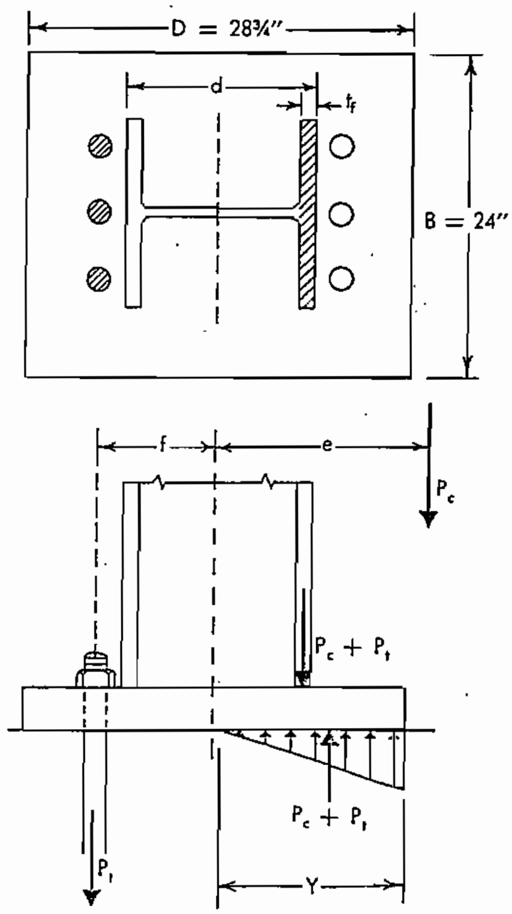


FIGURE 26

Here:

$e = 16.15''$

$f = 9''$

$D = 28¾''$

$B = 24''$

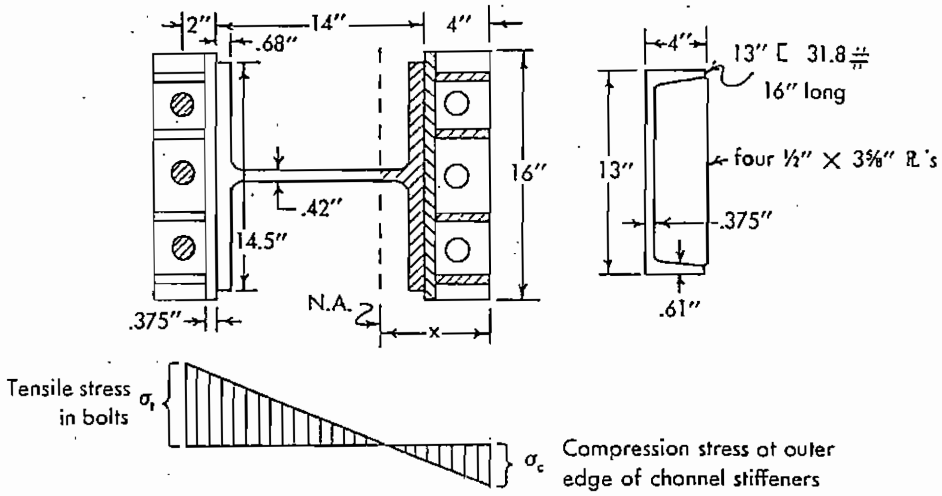


FIGURE 27

$$n = \frac{E_s}{E_c} = 10 \quad (E_c = 3000 \text{ psi})$$

1⅝" bolts

$$A_s = 3\ (2.074)$$
$$= 6.22 \text{ in.}^2 \text{ (bolts under tension)}$$
$$P_c = 130 \text{ kips}$$

from formula #13 (cubic equation)

$$Y^3 + K_1\, Y^2 + K_2\, Y + K_3 = 0$$

where:

$$K_1 = 3\left(e - \frac{D}{2}\right)$$
$$= 3\left(16.15 - \frac{28¾}{2}\right)$$
$$= 5.33$$
$$K_2 = \frac{6\ n\ A_s}{B}(f + e)$$
$$= \frac{6\ (10)(6.22)}{24}(9 + 16.15)$$
$$= 392$$
$$K_3 = -K_2\left(\frac{D}{2} + f\right)$$
$$= -392\left(\frac{28¾}{2} + 9\right)$$
$$= -9160$$

Therefore, substituting into formula #13:

$$Y^3 + 5.33\ Y^2 + 392\ Y - 9160 = 0$$

Letting $Y = +10$, $+12$, and $+15$ provides the following solutions to the cubic equation as the function of Y:

$$Y = +10 \rightarrow -3707$$
$$Y = +12 \rightarrow -1960$$
$$Y = +15 \rightarrow +1294$$

Plotting these three points, the curve is observed to pass through zero at—

$$Y = \underline{13.9''}$$

which is the effective bearing length.

from formula #9b

$$P_t = -P_c\left[\frac{\dfrac{D}{2} - \dfrac{Y}{3} - e}{\dfrac{D}{2} - \dfrac{Y}{3} + f}\right]$$
$$= -130^k\left[\frac{\dfrac{28¾}{2} - \dfrac{13.9}{3} - 16.15}{\dfrac{28¾}{2} - \dfrac{13.9}{3} + 9}\right]$$
$$= +44.5^k$$

which is the tensile load on the hold-down bolts.

from formula #8b

$$\sigma_c = \frac{2(P_c + P_t)}{Y\ B}$$
$$= \frac{2(130^k + 44.5^k)}{(13.9)(24)}$$
$$= 1050 \text{ psi}$$

which is the bearing pressure of the masonry support against the bearing plate.

If the anchor hold-down bolt detail is milled with the column base so that it bears against the base plate, it must be made strong enough to support the portion of the reaction load ($P_c + P_t$) which tends to bear upward against the portions of the bolt detail outside the column flange. This upward reaction on the compression side ($P_c + P_t$) is much larger than the downward load of the bolts on the tension side (P_t).

The area of section effective in resisting this reaction includes all the area of the compression material—column flange, portion of column web, the channel web, and stiffeners—plus the area of the anchor bolts on the tension side. See shaded area in Figure 27.

The anchor bolts on the compression side do not act because they have no way of transmitting a compressive load to the rest of the column. In like manner, the column flange and web on the tension side do not act because they have no way of transmitting a tensile stress across the milled joint to the base plate. The tension flange simply tends to lift off the base plate and no stress is transmitted in the tensile area except by the hold-down bolts attached to the column.

Determining moment of inertia

To determine the moment of inertia of this effective area of section, the area's neutral axis must be located. Properties of the elements making up this effective area are entered in the table shown here. Moments are taken about a reference axis (y-y) at the outermost edge of the channel stiffeners on the compression side (Fig. 27). See Section 2.2 for method.

Having obtained the 1st totals of area (A) and moment (M), solve for the location (n) of the neutral axis relative to the reference axis:

$$n = \frac{\Sigma\ M}{\Sigma\ A}$$

$$= \frac{(199.98 + .21\ n^2)}{(27.36 + .42\ n)}$$

$$199.98 + .21\ n^2 = 27.36 + .42\ n^2$$

$$n^2 + 130.28\ n - 952.47 = 0$$

$$n = \frac{-130.28 \pm \sqrt{130.28^2 + 4(952.47)}}{2}$$

$$= 6.93'' \text{ distance of N.A. to ref. axis y-y}$$

$$\therefore\ c = 6.93'' \text{ distance of N.A. to outer fiber}$$

Now, having the value of n, properties of the effective portion of the column web can be fixed and the table completed. With the 2nd totals of area (A), moment (B), and also moments of inertia ($I_y + I_g$), solve for the moment of inertia about the neutral axis (I_n):

$$I_n = I_y + I_g - \frac{M^2}{A}$$

$$= (2789.93) - \frac{(210.07)^2}{(30.27)}$$

$$= 1326 \text{ in.}^4$$

Since the concentrated compressive load (P_c) is applied at an eccentricity (e) of 16.15" to provide for the wind moment of 175,000 kips, the moment arm of the 130-kip load is—

9.15" from face of column flange

5.15" from outer edge of channel stiffeners

12.08" from neutral axis of effective area

compressive stress at outer edge of channel stiffeners

$$\sigma_c = \frac{M\ c}{I} + \frac{P_c}{A}$$

$$= \frac{(130^k \times 12.08)(6.93)}{(1326)} + \frac{130^k}{30.27}$$

$$= 8220 + 4300 = 12{,}150 \text{ psi}$$

	Distance: C.G. to ref. axis y-y (y)	Area (A)	Moment (M)	Moment of inertia (I_y)	Moment of inertia (I_g)
3 bolts	20.0	6.22	124.40	2448.0	
Portion of web	$\frac{4.688 + n}{2} =$ $= 2.344 + .5n$	$(n - 4.688)(.42) =$ $= .42n - 1.969$	$(2.344 + .5n)(.42n - 1.969) =$ $= .21n^2 - 4.615$		
	= 5.809	= .94	= + 5.47	31.77	
Column flange	4.344	9.86	42.83	186.05	
Channel web	3.812	6.00	22.87	87.19	
Channel stiffeners	2.00	7.25	14.50	29.00	7.92
	First Total →	27.36 + .42 n	199.98 + .21 n^2		
By substituting value of n = 6.93":	Second Total →	30.27	210.07	2789.93	

tensile stress in hold-down bolts

$$\sigma_t = \frac{M\ c}{I} \quad \frac{P_c}{A}$$

where c is distance of N.A. from extreme fiber of tensile area

$$= \frac{(130^k \times 12.08)(13.07)}{(1326)} - \frac{130^k}{30.27}$$

$$= 15{,}500 \text{ psi} - 4{,}300 = 11{,}200 \text{ psi}$$

total force in hold-down bolts

$$P_t = A_s\ \sigma_t$$

$$= (6.22)(11{,}200)$$

$$= 69.6 \text{ kips}$$

Size of Welds Attaching Stiffeners to Channel Web

Compressive force is carried by each of the four channel stiffeners. The average compressive stress on these stiffeners is—

$$\sigma_c = \frac{5.15''}{6.93''}\ (8220 \text{ psi}) + 4300 \text{ psi}$$

$$= 6110 \text{ psi} + 4300 \text{ psi} = 10{,}410 \text{ psi}$$

$$F = \sigma_c\ A$$

$$= (10{,}410)(\tfrac{1}{2} \times 3\tfrac{5}{8}'')$$

$$= 18{,}850 \text{ lbs}$$

This compressive force on each channel stiffener is transferred to the channel web by two vertical fillet welds, each 11″ long. The force on each weld is thus—

$$f = \frac{F}{2\ L}$$

$$= \frac{(18{,}850 \text{ lbs})}{2\,(11'')}$$

$$= 856 \text{ lbs/linear inch}$$

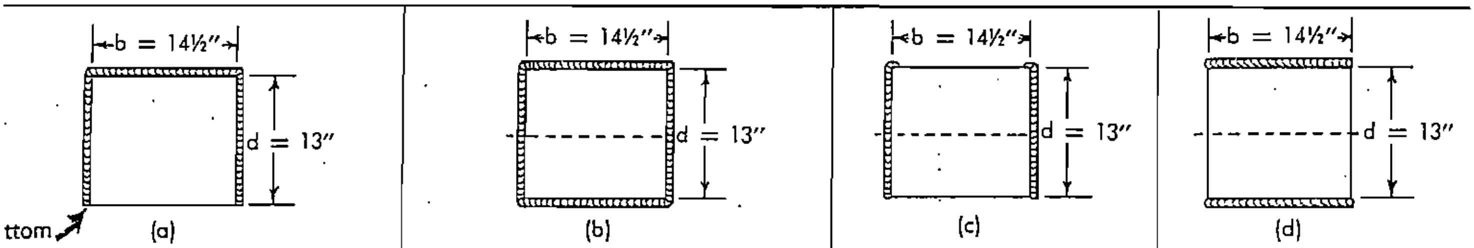
and the required fillet weld leg size is—

$$\omega = \frac{856}{11{,}200} \leftrightarrow \text{for E70 welds (Table 5, Sect. 7.4)}$$

$$= .076'' \text{ or use } \tfrac{3}{16}'' \triangle \qquad \text{(Table 2, Sect. 7.4)}$$

With this leg size, intermittent welds can be used instead of continuous welding—

TABLE 4—Four Methods of Welding Channel Assembly to Column Flange



(a)	(b)	(c)	(d)
$S_w = \frac{d^2(2b+d)}{3\,(b+d)}$ $= \frac{(13)^2(2 \times 14.5 + 13)}{3(14.5+13)}$ $= 86.1 \text{ in.}^2$	$S_w = bd + \frac{d^2}{3}$ $= (14.5)(13) + \frac{(13)^2}{3}$ $= 242.2 \text{ in.}^2$	$S_w = \frac{d^2}{3}$ $= \frac{(13)^2}{3}$ $= 56.3 \text{ in.}^2$	$S_w = bd$ $= (14.5)(13)$ $= 185.9 \text{ in.}^2$
$f_b = \frac{M}{S_w}$ $= \frac{(174{,}200)}{(86.1)}$ $= 2020$ lbs/in.	$f_b = \frac{M}{S_w}$ $= \frac{(174{,}200)}{(242.2)}$ $= 720$ lbs/in.	$f_b = \frac{M}{S_w}$ $= \frac{(174{,}200)}{(56.3)}$ $= 3100$ lbs/in.	$f_b = \frac{M}{S_w}$ $= \frac{(174{,}200)}{(185.9)}$ $= 935$ lbs/in.
$f_s = \frac{V}{L}$ $= \frac{(123{,}400)}{2(13) + (14.5)}$ $= 3050$ lbs/in.	$f_s = \frac{V}{L}$ $= \frac{(123{,}400)}{2(13 + 14.5)}$ $= 2240$ lbs/in.	$f_s = \frac{V}{L}$ $= \frac{(123{,}400)}{2\ (13)}$ $= 4750$ lbs/in.	$f_s = \frac{V}{L}$ $= \frac{(123{,}400)}{2\ (14.5)}$ $= 4260$ lbs/in.
$f_r = \sqrt{f_b^2 + f_s^2}$ $= \sqrt{(2020)^2 + (3050)^2}$ $= 3670$ lbs/in.	$f_r = \sqrt{f_b^2 + f_s^2}$ $= \sqrt{(720)^2 + (2240)^2}$ $= 2350$ lbs/in.	$f_r = \sqrt{f_b^2 + f_s^2}$ $= \sqrt{(3100)^2 + (4750)^2}$ $= 5680$ lbs/in.	$f_r = \sqrt{f_b^2 + f_s^2}$ $= \sqrt{(935)^2 + (4260)^2}$ $= 4360$ lbs/in.
$\omega = \frac{\text{actual force}}{\text{allowable force}}$ $= \frac{(3670)}{(11{,}200)} \leftarrow$ E70 $= .328''$ or $5/16'' \triangle$	$\omega = \frac{\text{actual force}}{\text{allowable force}}$ $= \frac{(2350)}{(11{,}200)}$ $= .210''$ or $\tfrac{1}{4}'' \triangle$	$\omega = \frac{\text{actual force}}{\text{allowable force}}$ $= \frac{(5680)}{(11{,}200)}$ $= .506''$ or $\tfrac{1}{2}'' \triangle$	$\omega = \frac{\text{actual force}}{\text{allowable force}}$ $= \frac{(4360)}{(11{,}200)}$ $= .389''$ or $7/16'' \triangle$

$$L = \frac{\frac{1}{2}\,(18{,}850\ \text{lbs})}{2100}$$
$$= 4.49''$$

or a total length of 4½" of 3/16" fillet welds on each side of each stiffener.

Size of Weld Connecting Channel Assembly to Column Flange

The average compressive stress on the channel web is—

$$\sigma_c = \frac{3.12''}{6.93''}\ 8220 + 4300$$
$$= 3700 + 4300 = 8000\ \text{psi}$$

$$\therefore\ F = \sigma A$$
$$= 8000$$
$$= 48{,}000\ \text{lbs}$$

total compressive force on channel assembly

$$F = 48{,}000 + 4(18{,}850)$$
$$= 123{,}400\ \text{lbs}$$

The fillet welds connecting the assembly to the column flange must transfer this total compressive force into the column flange. There are four ways to weld this, as shown in Table 4. Assume the welds carry all of the compressive force, and ignore any bearing of the channel against the column flange.

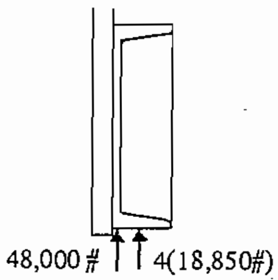


FIGURE 28

First find the moment applied to the weld, Figure 28, which applies in each case of Table 4:

$$M = 4(18{,}850\ \text{lbs})\ (2.187'') + (48{,}000\ \text{lbs})\ (3/16'')$$
$$= 174{,}200\ \text{in.-lbs}$$

Then, making each weld pattern in turn, treat the weld as a line to find its section modulus (S_w), the maximum bending force on the weld (f_b), the vertical shear on the weld (f_v), the resultant force on the weld (f_r), and the required weld leg size (ω).

Perhaps the most efficient way to weld this is method (d) in which two transverse ¼" fillet welds are placed across the column flange and channel flange, with no longitudinal welding along the channel web.

5. USE OF WING PLATES

When large wing plates are used to increase the leverage of an anchor bolt, the detail should always be checked for weakness in bearing against the side of the column flange.

Problem 5

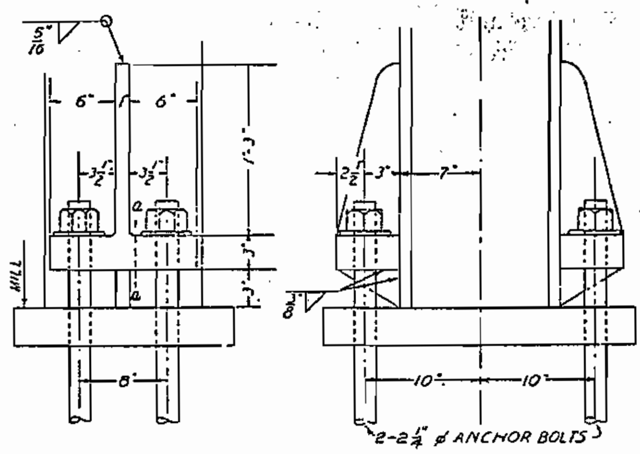


FIGURE 29

Figure 29 illustrates a wing-plate type of column base detail that is not limited with respect to size of bolts or strength of column flange. A similar detail, with bolts as large as 4½" diameter, has been used on a large terminal project.

The detail shown is good for four 2¼"-dia. anchor bolts. Two of these bolts have a gross area of 6.046 in.² and are good for 84,600 lbs tension at a stress of 14,000 psi.

In this detail, the bolt load is first carried laterally to a point opposite the column web by the horizontal bar which is 5½" wide by 3" thick.

section modulus of section a-a

$$S = \frac{5\tfrac{1}{2}''\ (3'')^2}{6}$$
$$= 8.25\ \text{in.}^3$$

bending moment on bar

$$M = 42{,}300\# \times 3\tfrac{1}{2}''$$
$$= 148{,}000\ \text{in.-lbs.}$$

resulting bending stress

$$\sigma = \frac{M}{S}$$
$$= \frac{(148{,}000)}{(8.25)}$$
$$= 18{,}000\ \text{psi}$$

At the center of the 3″ bar, the bolt loads are supported by tension and compression forces in the 1″ thick web plates above and below the bar. The web plates are attached to the column flange, opposite the column web, by welds that carry this moment and shear into the column.

The shear and moment caused by the anchor bolt forces, which are not in the plane of the weld, determine the size of the vertical welds. The welds extend 15″ above and 3″ below the 3″ transverse bar.

The properties and stresses on the vertical welds are figured on the basis of treating the welds as a line, having no width. See Figure 30.

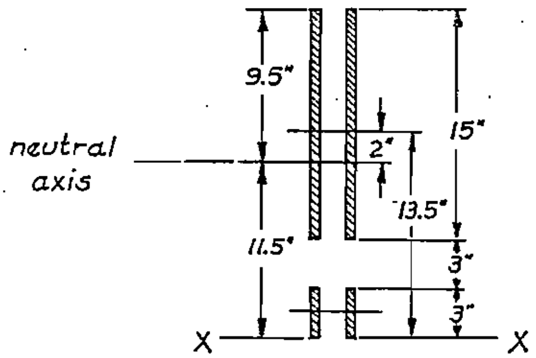


FIGURE 30

Take area moments about the base line (y-y):

	A	y	M	I_y	I_g
2 welds × 3″	6	1.5	9.0	13.5	4.5
2 welds × 15″	30	15.3	405.0	5467.5	562.5
Total	36		414.0	6048	

moment of inertia about N.A.

$$I_n = I_y + I_g - \frac{M^2}{A}$$

$$= (6048) - \frac{(414)^2}{(36)}$$

$$= 1288 \text{ in.}^3$$

$$n = \frac{M}{A}$$

$$= \frac{(414)}{(36)}$$

$$= 11.5'' \text{ (up from base line y-y)}$$

distance of N.A. from outer fiber

$$c_{bottom} = 11.5''$$

$$c_{top} = 9.5''$$

section modulus of weld

$$S_{bottom} = \frac{(1288)}{(11.5)}$$

$$= 112 \text{ in.}^2$$

$$S_{top} = \frac{(1288)}{(9.5)}$$

$$= 135.5 \text{ in.}^2$$

maximum bending force on weld

$$\text{(top) } f_b = \frac{M}{S_w}$$

$$= \frac{(84{,}600)(3)}{(135.5)}$$

$$= 1870 \text{ lbs/in.}$$

shear force on weld

$$f_s = \frac{V}{L_w}$$

$$= \frac{(84{,}600)}{(36)}$$

$$= 2340 \text{ lbs/in.}$$

resultant force on weld

$$f_r = \sqrt{f_b^2 + f_s^2}$$

$$= \sqrt{(1870)^2 + (2340)^2}$$

$$= 3000 \text{ lbs/in.}$$

required fillet weld size

$$\omega = \frac{3000}{11{,}200} \leftarrow \text{E70 allowable}$$

$$= .268'' \text{ or use } \tfrac{5}{16}''$$

This requires continuous 5/16″ fillet welds on both sides for the full length of the 1″ vertical web plate. If greater weld strength had been required, the 1″ web plate could be made thicker or taller.

For bolts of ordinary size, the upper portion of the plates for this detail can be cut in one piece from column sections of 14″ flanges. This insures full continuity of the web-to-flange in tension for carrying the bolt loads. By welding across the top and bottom edges of the horizontal plate to the column flange, the required thickness of flange plate in bending is reduced by having support in two directions.

6. TYPICAL COLUMN BASES

In (a) of Figure 31, small brackets are groove butt

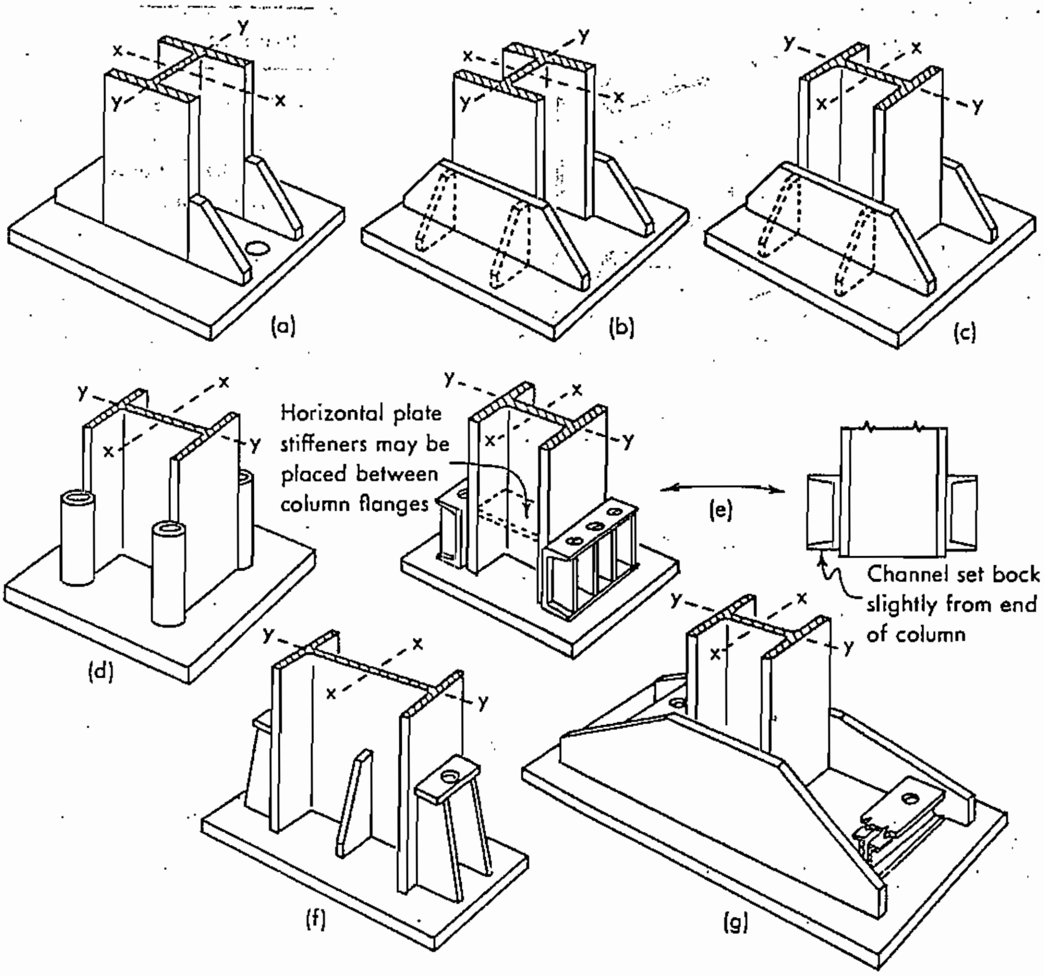


FIGURE 31

welded to the outer edges of the column flanges to develop greater moment resistance for the attachment to the base plate. This will help for moments about either the x-x or the y-y axis. A single bevel or single V joint is prepared by beveling just the edge of the brackets; no beveling is done on the column flanges.

For column flanges of nominal thickness, it might be easier to simply add two brackets, fillet welded to the base of the column; see (b) and (c). No beveling is required, and handling and assembling time is reduced because only two additional pieces are required.

In (b) the bracket plates are attached to the face of the column flange; in (c) the plates are attached to the outer edge of the column flange. In any rolled section used as a column, greater bending strength and stiffness is obtained about the x-x axis. If the moment is about the x-x axis, it would be better to attach the additional plates to the face of the column as in (b). This will provide a good transverse fillet across the column flange and two longitudinal fillet welds along the outer edge of the column flange with good accessibility for welding. The attaching plates and the welds connecting them to the base plate are in the most effective position and location to transfer this moment. The only slight drawback is that the attaching plates will not stiffen the overhung portion of the base plate for the bending due to tension in the hold-down bolts, or due to the upward bearing pressure of the masonry support. However if this is a problem, small brackets shown in dotted lines may be easily added.

The plates can be fillet welded to the outer edges of the column flange as in (c), although there is not good accessibility for the welds on the inside. Some of these inside fillet welds can be made before the unit is assembled to the base plate.

For thick flanges, detail (a) might represent the least amount of welding and additional plate material.

Short lengths of pipe have been welded to the outer edge of the column flange to develop the necessary moment for the hold-down bolts; see (d). The length and leg size of the attaching fillet welds are sufficient for the moment.

In (e) two channels with additional stiffeners are welded to the column flanges for the required moment from the hold-down bolts. By setting this channel assembly back slightly from the milled end of the column, it does not have to be designed for any bear-

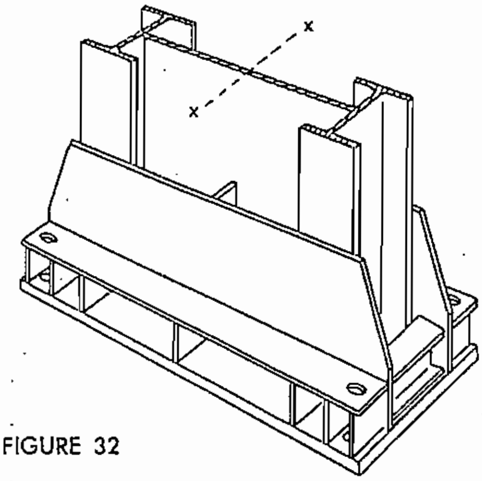


FIGURE 32

ing, but just the tension from the hold-down bolts. If this assembly is set flush with the end of the column and milled to bear, then this additional bearing load must be considered in its design. Any vertical tensile load on the assembly from the holddown bolts, or vertical bearing load from the base plate (if in contact), will produce a horizontal force at the top which will be applied transverse to the column flange. If the column flange is too thin, then horizontal plate stiffeners must be added between the column flanges to effectively transfer this force. These stiffeners are shown in (e) by dotted lines.

In (f) built-up, hold-down bolt supports are welded to the column flanges. These may be designed to any size for any value of moment.

In (g), the attaching plates have been extended out farther for very high moments. This particular detail uses a pair of channels with a top plate for the hold-down bolts to transfer this tensile force back to the main attaching plates, and in turn back to the column.

One of the many possible details for the base of a built-up crane runway girder column in a steel mill is shown in Figure 32. Two large attaching plates are fillet welded to the flanges of the rolled sections of the column. This is welded to a thick base plate. Two long narrow plates are next welded into the assembly, with spacers or small diaphragms separating them from the base plate. This provides additional strength and stiffness of the base plate through beam action for the forces from the hold-down bolts. Short sections of I beam can also be welded across the ends between the attaching plates.

7. HIGH-RISE REQUIREMENTS

Columns for high-rise buildings may use brackets on their base plates to help distribute the column load out over the larger area of the base plate to the masonry support.

Problem 6

A 14″ WF 426# column of A36 steel is to carry a compressive load of 2,000 kips. Using a bearing load of 750 psi, this would require a 50″ × 60″ base plate. Use E70 welds.

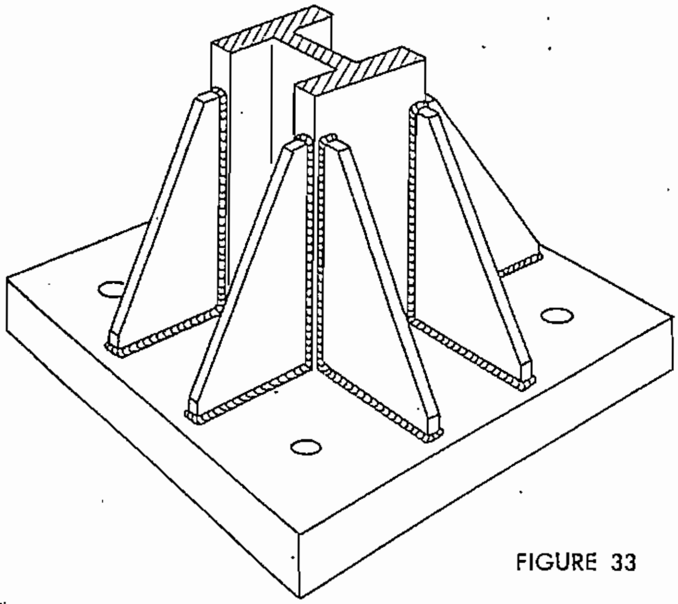


FIGURE 33

For simplicity, each set of brackets together with a portion of the base plate formed by a diagonal line from the outer corner of the plate back to the column flange, will be assumed to resist the bearing pressure of the masonry support; see Figure 34. This is a conservative analysis because the base plate is not cut along these lines and these portions do not act independently of each other.

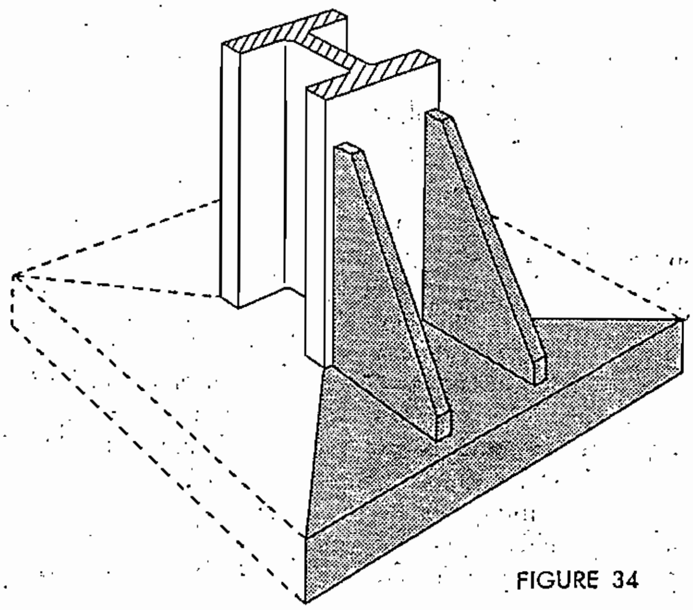


FIGURE 34

This portion of the assembly occupies a trapezoidal area; Figure 35.

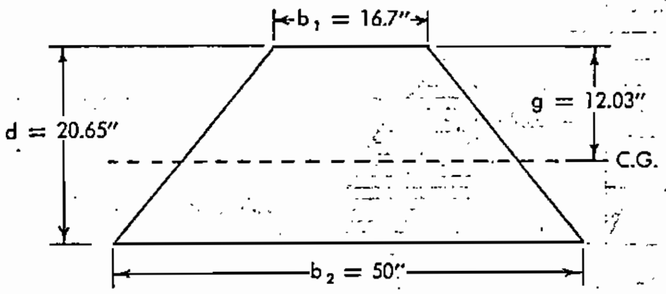


FIGURE 35

$$g = \frac{d(b_1 + 2\ b_2)}{3(b_1 + b_2)}$$

$$= \frac{20.65\ (16.7 + 2 \times 50)}{3\ (16.7 + 50)}$$

$$= 12.03''$$

$$A = (b_1 + b_2)\frac{d}{2}$$

$$= (16.7 + 50)\ \frac{20.65}{2}$$

$$= 690 \text{ in.}^2$$

$$P = A\ \sigma$$

$$= (690 \text{ in.}^2)(750 \text{ psi})$$

$$= 516 \text{ kips}$$

$$M = P\ g$$

$$= (516^k)(12.03'')$$

$$= 6.225 \text{ in.-kips}$$

Determining thickness of base plate

To get an idea of the thickness of the base plate (t), consider a 1″ wide strip as a uniformly loaded, continuous beam supported at two points (the brackets) and overhanging at each end. See Figure 36.

From beam formula #6Bb in Section 8.1:

$$M_{max} \text{ (at support)} = \frac{-w\ a^2}{2}$$

$$= \frac{-(750)(18.4)^2}{2}$$

$$= -126{,}500 \text{ in.-lbs}$$

Since:

$$M = \sigma\ S$$

$$S = \frac{M}{\sigma} \qquad = \frac{1''\ t^2}{6}$$

or:

$$t = \sqrt{\frac{6\ M}{\sigma}}$$

where:
$\sigma = .75\ \sigma_y$ (AISC 1.5.1.4.8)

$$= \sqrt{\frac{6(126{,}500)}{(25{,}000)}}$$

$$= \sqrt{30.4}$$

$= 5.51''$ or use 6″-thick plate

Check bending stresses & shear stresses in base plate bracket section

Start with 1½″-thick brackets (2 × 1½″ = 3″ flange thickness) at right angles to face of column flange. Find moment of inertia of the vertical section through brackets and base plate, Figure 37, using the method of adding areas:

	A	y	M	I_y	I_g
16.7″ × 6″	100.2	+ 3	300.6	902	301
3″ × 24″	72.0	+ 18	1296.0	23,328	3456
Total	172.2		1596.6	27,990	

moment of inertia about N.A.

$$I_n = I_y + I_g - \frac{M^2}{A}$$

$$= (27{,}990) - \frac{(1596.6)^2}{(172.2)}$$

$$= 13{,}190 \text{ in.}^4$$

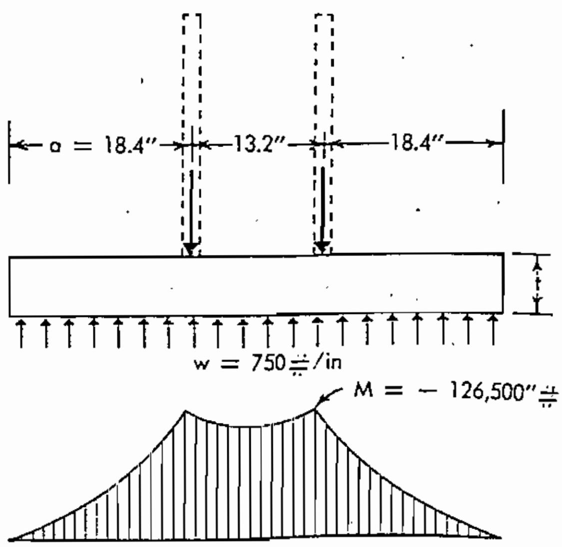


FIGURE 36

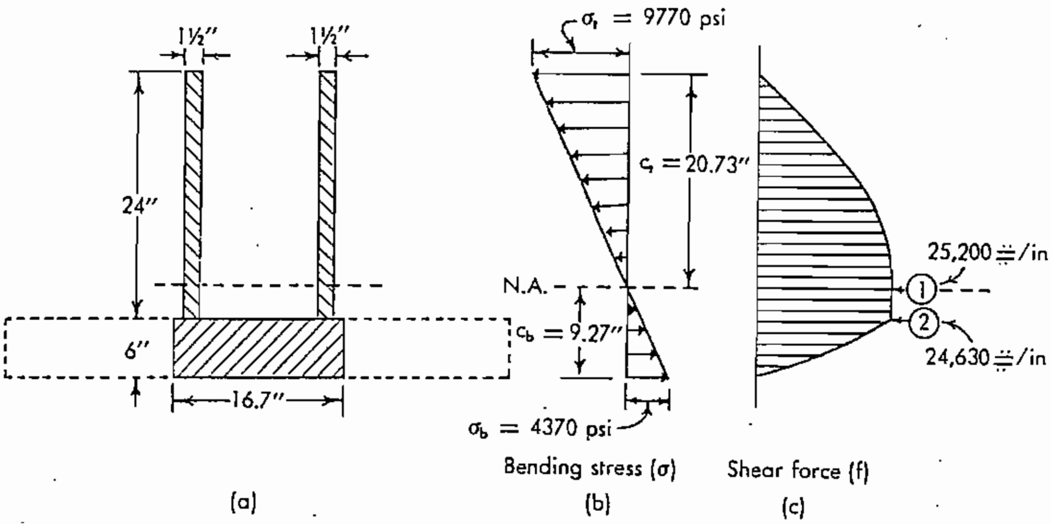


FIGURE 37

$$n = \frac{M}{A} = \frac{(1596.6)}{(172.2)} = 9.27''$$

distance of N.A. to outer fiber

$$c_b = 9.27''$$

$$c_t = 30'' - 9.27'' = 20.73''$$

bending stresses

$$\sigma_b = \frac{M\ c_b}{I} = \frac{(6225)(9.27)}{(13{,}190)} = 4370 \text{ psi}$$

$$\sigma_t = \frac{M\ c_t}{I} = \frac{(6225)(20.73)}{(13{,}190)} = 9770 \text{ psi} \quad \underline{\text{OK}}$$

maximum shear force at neutral axis

$$f_1 = \frac{V\ a\ y}{I} = \frac{(516.5^k)(3'' \times 20.73'')(10.37'')}{(13{,}190)} = 25{,}200 \text{ lbs/in.}$$

corresponding shear stress in brackets

$$\tau = \frac{f}{t} = \frac{(25{,}200 \text{ lbs/in.})}{(3'')} = 8400 \text{ psi} \quad \underline{\text{OK}}$$

shear force at face of 6″ base plate
(to be transferred through fillet welds)

$$f_2 = \frac{V\ a\ y}{I} = \frac{(516.5^k)(6'' \times 16.7'')(6.27'')}{(13{,}190)}$$

$= 24{,}630$ lbs/in. (to be carried by four fillet welds at 1½″ thick brackets)

leg size of each fillet weld joining base plate to brackets

$$\omega = \frac{\frac{1}{4}\ (24{,}630)}{(11{,}200)} \leftarrow \text{E70 allowable}$$

$= .545''$ or use $\underline{9/16''}$

(The minimum fillet weld leg size for 6″ plate is ½″ .)

Determining vertical weld requirements

In determining fillet weld sizes on the usual beam seat bracket, it is often assumed that the shear reaction is uniformly distributed along the vertical length of the bracket. The two unit forces resulting from shear and bending are then resolved together (vectorially added), and the resultant force is then divided by the allowable force for the fillet weld to give the weld size. This is of course conservative, because the maximum unit bending force does not occur on the fillet weld at the same region as does the maximum unit shear force. However the analysis does not take long:

bending force on weld

$$f_b = \sigma \, t$$
$$= (9770 \text{ psi})(1\tfrac{1}{2}'')$$
$$= 14{,}660 \text{ lbs/in. (one bracket and two fillet welds)}$$

or

$$= 7330 \text{ lbs/in. (one fillet weld)}$$

vertical shear force on weld
(assuming uniform distribution)

$$f_s = \frac{516.5^k}{4 \times 30''}$$
$$= 4310 \text{ lbs/in.}$$

resultant force on weld

$$f_r = \sqrt{f_b^2 + f_s^2}$$
$$= \sqrt{(7330)^2 + (4310)^2}$$
$$= 8500 \text{ lbs/in.}$$

required leg size of vertical fillet weld

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$
$$= \frac{(8500)}{(11{,}200)}$$
$$= .758'' \text{ or use } \tfrac{3}{4}''$$

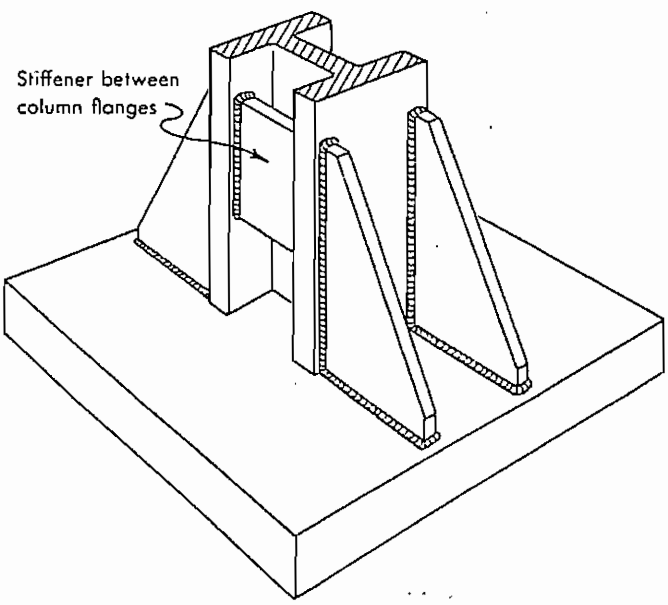


FIGURE 38

Alternate method. In cases where the forces are high, and the requirement for welding is greater, it would be well to look further into the analysis in order to reduce the amount of welding.

In Figure 37, it is seen that the maximum unit force on the vertical weld due to bending moment occurs at the top of the bracket connection (b) in a region of very low shear transfer. Likewise the maximum unit shear force occurs in a region of low bending moment (c). In the following analysis, the weld size is determined both for bending and for shear, and the larger of these two values are used:

vertical shear requirement
(maximum condition at N.A.)

$$f_1 = 25{,}200 \text{ lbs/in.}$$

to be carried by four fillet welds.

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$
$$= \frac{\tfrac{1}{4}\,(25{,}200)}{(11{,}200)}$$
$$= .562'' \text{ or } \tfrac{9}{16}''$$

bending requirement
(maximum condition at top of bracket)

$$f_b = 7330 \text{ lbs/in.}$$

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$
$$= \frac{(7330)}{(11{,}200)}$$
$$= .654'' \text{ or } \tfrac{3}{4}''$$

Hence use the larger of the two, or ¾″ fillet welds. Although this alternate method required a slightly smaller fillet weld (.654″) as against (.758″), they both ended up at ¾″ when they were rounded off. So, in this particular example, there was no saving in using this method.

Column stiffeners

A rather high compressive force in the top portion of these brackets is applied horizontally to the column flange. It would be well to add stiffeners between the column flanges to transfer this force from one bracket through the column to the opposite column flange; Figure 38.

It might be argued that, if the brackets are milled to bear against the column flanges, the bearing area may then be considered to carry the compressive horizontal force between the bracket and the column flange. Also, the connecting welds may then be considered to

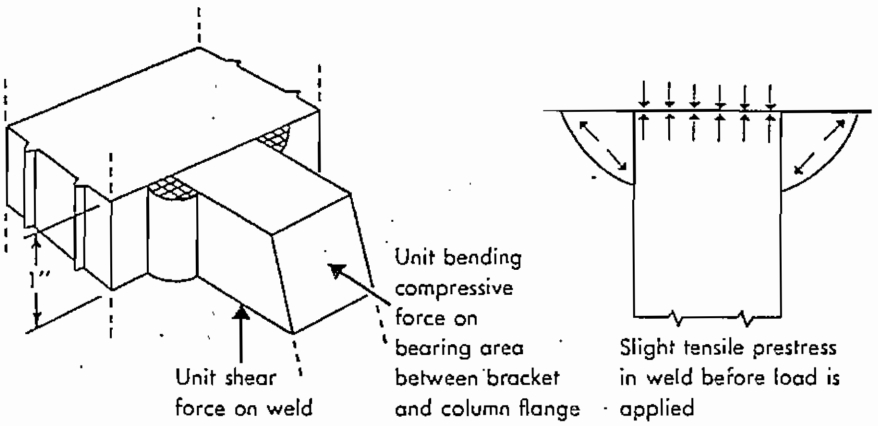


FIGURE 39

carry only the vertical shear forces. See Figure 39, left.

If the designer questions whether the weld would load up in compression along with the bearing area of the bracket, it should be remembered that weld shrinkage will slightly prestress the weld in tension and the end of the bracket within the weld region in compression. See Figure 39, right. As the horizontal compression is applied, the weld must first unload in tension before it would be loaded in compression. In the meantime, the bracket bearing area continues to load up in compression.

This is very similar to standard practice in welded plate girder design. Even though the web is not milled along its edge, it is fitted tight to the flange and simple fillet welds join the two. In almost all cases, these welds are designed just for the shear transfer (parallel to the weld) between the web and the flange; any distributed floor load is assumed to transfer down through the flange (transverse to the weld) into the edge of the web which is in contact with the flange. Designers believe that even if this transverse force is transferred through the weld, it does not lower the capacity of the fillet weld to transfer the shear forces.

Refer to Figure 37(b) and notice that the bending action provides a horizontal compressive force on the vertical connecting welds along almost their entire length. Only a very small length of the welds near the base plate is subjected to horizontal tension, and these forces are very small. The maximum tensile forces occur within the base plate, which has no connecting welds.

shear force on vertical weld
(assuming uniform distribution)

$$f_s = \frac{516.5^k}{4 \times 30''}$$

$$\doteq 4310 \text{ lbs/in. (one weld)}$$

vertical weld size
(assuming it to transfer shear force only)

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$

$$= \frac{(4310)}{(11{,}200)}$$

$$= .385''$$

but 3″ thick column flange would require a minimum ½″ ◺ (Table 2, Sect. 7.4).

If partial-penetration groove welds are used (assuming a tight fit) the following applies:

allowables (E70 welds)

compression: same as plate

shear: $\tau = 15{,}800$ psi

shear force on one weld

$$f_s = 4310 \text{ lbs/in.}$$

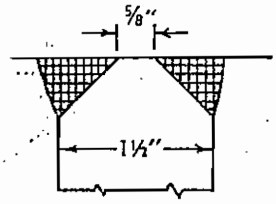
required effective throat

$$t_e = \frac{f_s}{\tau}$$

$$= \frac{(4310)}{(15{,}800)}$$

$$= .273''$$

if using bevel joint



$$t = t_e + ⅛''$$

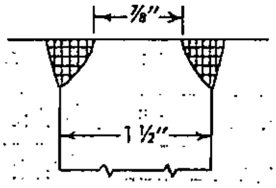
$$= .273'' + ⅛''$$

$$= .398''$$

root face (land) = 1½″ − 2(.398″)

= .704″ or use ⅝″

if using J joint



$t = t_e$

$= .273''$

root face (land) $= 1\frac{1}{2}'' - 2(.273'')$

$= .954''$ or use $\underline{7/8''}$

A portion of the shear transfer represented by the shear force distribution in Figure 37 (c) lies below a line through the top surface of the base plate. It might be reasoned that this portion would be carried by the base plate and not the vertical connecting welds between the bracket and the column flange. If so, this triangular area would approximately represent a shear force of

$\frac{1}{2}$ (24,630#/in.) $6'' = 73.9^k$

to be deducted:

$516.5^k - 73.9^k = 442.6^k$

$$f = \frac{442.6^k}{4 \times 30''} = 3690 \text{ lbs/in.}$$

$$\omega = \frac{3690}{11{,}200} = .33'' \text{ or } \tfrac{3}{8}''$$

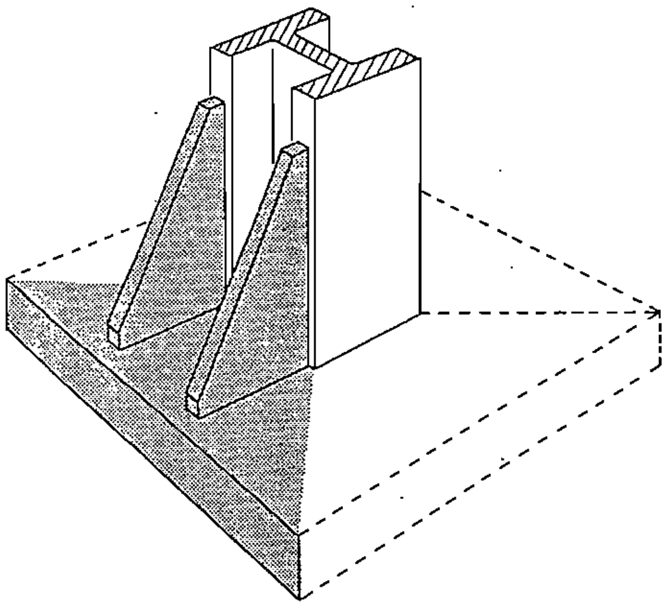


FIGURE 40

However, in this example, the column flange thickness of 3" would require a $\underline{\frac{1}{2}''}$ fillet weld to be used.

Brackets to column flange edges

The base section consisting of the brackets attached to the edge of the column flanges, Figure 40, is now considered in a similar manner. From this similar analysis, the brackets will be made of 1¼"-thick plate.

Figure 41 shows the resulting column base detail.

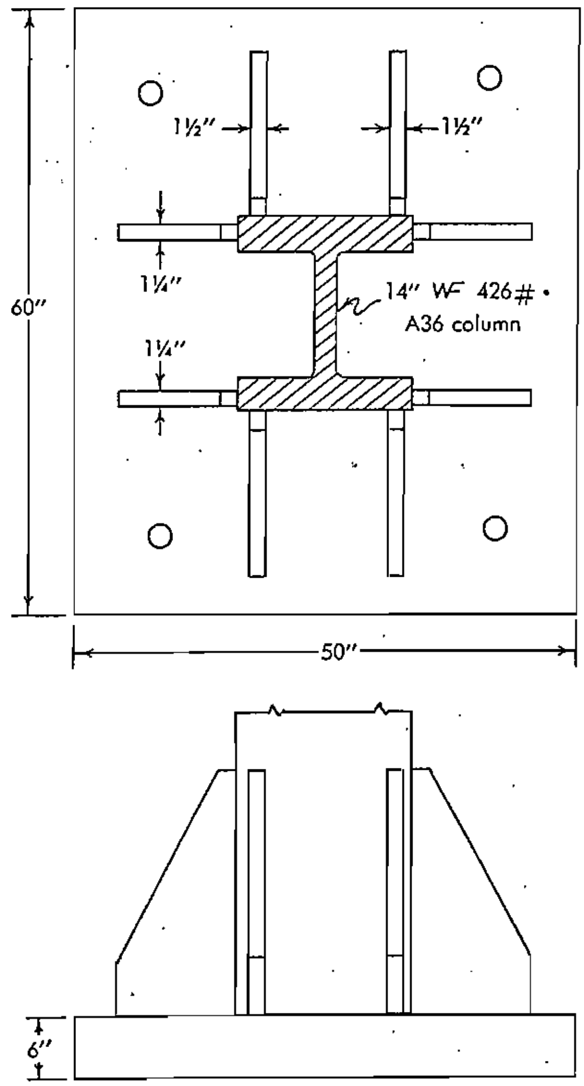
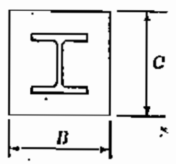


FIGURE 41

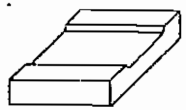
COLUMN BASE PLATE DIMENSIONS (AISC, 1963)



COLUMN BASE PLATES

Dimensions for maximum column loads

For A36 Columns



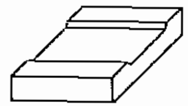
Base plates, ASTM A36, F_y = 27 ksi
Concrete, f'_c = 3000 psi

Column			Unit Pressure on Support $F_p = 0.25 f'_c$ = 750 psi						Unit Pressure on Support $F_p = 0.375 f'_c$ = 1125 psi					
Nom. Size & Designation	Wt. per Ft.	Max. Load	Dimensions		Thickness of Plate			Gross Wt.	Dimensions		Thickness of Plate			Gross Wt.
			B	C	Calc.	Fin.	Rolled		B	C	Calc.	Fin.	Rolled	
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
14 × 16 WF	426	2605	57	61	6.30	6¼	6¾	6649	46	51	5.84	5⅞	6¼	4154
	398	2433	55	59	6.02	6	6½	5976	45	49	5.56	5⅝	6	3748
	370	2261	53	57	5.76	5¾	6¼	5349	43	47	5.28	5¼	5⅝	3221
	342	2090	51	55	5.51	5½	5⅞	4669	41	46	5.14	5⅛	5½	2939
	320	1954	50	53	5.30	5¼	5⅝	4223	40	44	4.92	5	5⅜	2680
	314	1917	49	53	5.25	5¼	5⅝	4138	40	43	4.75	4¾	5⅛	2497
	287	1752	47	50	4.91	5	5⅜	3578	38	41	4.44	4½	4⅞	2152
	264	1611	45	48	4.65	4⅝	5	3060	36	40	4.29	4¼	4⅝	1887
	246	1501	44	46	4.48	4½	4⅞	2795	36	38	4.06	4⅛	4½	1744
	237	1446	43	45	4.36	4⅜	4¾	2604	35	37	3.92	4	4	1467
	228	1391	42	45	4.26	4¼	4⅝	2476	34	37	3.82	3⅞	3⅞	1381
	219	1335	41	44	4.15	4⅛	4½	2300	33	36	3.70	3¾	3¾	1262
	211	1287	40	43	4.04	4	4	1949	33	35	3.58	3⅝	3⅝	1186
	202	1231	40	42	3.91	4	4	1904	32	35	3.52	3½	3½	1111
	193	1176	39	41	3.78	3¾	3¾	1699	31	34	3.39	3⅜	3⅜	1008
	184	1121	38	40	3.65	3⅝	3⅝	1561	31	33	3.22	3¼	3¼	942
	176	1072	37	39	3.52	3½	3½	1431	30	32	3.08	3⅛	3⅛	850
	167	1017	36	38	3.40	3⅜	3⅜	1308	29	32	3.08	3⅛	3⅛	822
	158	963	35	37	3.27	3¼	3¼	1192	28	31	2.94	3	3	738
	150	913	34	36	3.15	3⅛	3⅛	1084	28	29	2.76	2¾	2¾	633
	142	867	33	36	3.13	3⅛	3⅛	1052	27	29	2.63	2⅝	2⅝	582
14 × 14½ WF	136	826	32	35	3.00	3	3	952	26	29	2.61	2⅝	2⅝	561
	127	771	31	34	2.87	2⅞	2⅞	858	25	28	2.47	2½	2½	496
	119	722	30	33	2.74	2¾	2¾	771	24	27	2.33	2⅜	2⅜	436
	111	674	29	31	2.50	2½	2½	637	24	25	2.17	2¼	2¼	382
	103	625	28	30	2.37	2⅜	2⅜	565	23	25	1.99	2	2	326
	95	577	27	29	2.23	2¼	2¼	499	22	24	1.84	1⅞	1⅞	280
	87	527	26	28	2.09	2⅛	2⅛	438	21	23	1.69	1¾	1¾	239
14 × 12 WF	84	503	24	28	2.10	2⅛	2⅛	405	20	23	1.81	1⅞	1⅞	244
	78	467	24	26	2.08	2⅛	2⅛	376	19	22	1.66	1¾	1¾	207
14 × 10 WF	74	435	22	27	1.99	2	2	337	17	23	1.67	1¾	1¾	194
	68	400	21	26	1.85	1⅞	1⅞	290	17	21	1.58	1⅝	1⅝	164
	61	359	20	24	1.73	1¾	1¾	238	16	20	1.41	1½	1½	136
14 × 8 WF	53	302	17	24	1.54	1⅝	1⅝	188	13	21	1.36	1⅜	1⅜	106
	48	273	16	23	1.42	1½	1½	156	13	19	1.15	1¼	1¼	87
	43	245	15	22	1.29	1⅜	1⅜	129	12	19	1.04	1⅛	1⅛	73

Note: Rolled plate thicknesses above 4 inches are based on finished thickness plus suggested allowances for finishing one side, and may be modified to suit fabricating plant practice. When it is required to finish both surfaces of base plates, additional allowance must be made.

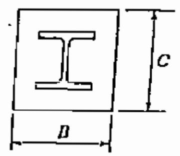
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

For A36 Columns



COLUMN BASE PLATES

Dimensions for maximum column loads



Base plates, ASTM A36, F_y = 27 ksi
Concrete, f'_c = 3000 psi

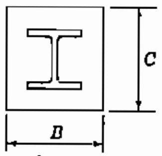
Column			Unit Pressure on Support $F_p = 0.25 f'_c$ = 750 psi						Unit Pressure on Support $F_p = 0.375 f'_c$ = 1125 psi					
Nom. Size & Designation	Wt. per Ft.	Max. Load	Dimensions		Thickness of Plate			Gross Wt.	Dimensions		Thickness of Plate			Gross Wt.
			B	C	Calc.	Fin.	Rolled		B	C	Calc.	Fin.	Rolled	
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
12 × 12 WF	190	1143	38	41	3.97	4	4	1766	30	34	3.59	3⅝	3⅝	1048
	161	969	34	38	3.58	3⅝	3⅝	1327	28	31	3.17	3¼	3¼	799
	133	799	31	35	3.19	3¼	3¼	999	25	29	2.85	2⅞	2⅞	591
	120	721	30	33	2.92	3	3	841	24	27	2.56	2⅝	2⅝	482
	106	636	28	31	2.68	2¾	2¾	676	23	25	2.32	2⅜	2⅜	387
	99	593	27	30	2.55	2⅝	2⅝	602	22	24	2.16	2¼	2¼	337
	92	552	26	29	2.43	2½	2½	534	21	24	2.10	2⅛	2⅛	303
	85	509	25	28	2.29	2⅜	2⅜	471	20	23	1.95	2	2	261
	79	473	24	27	2.17	2¼	2¼	413	20	22	1.79	1⅞	1⅞	234
	72	431	23	25	1.93	2	2	326	19	21	1.62	1⅝	1⅝	184
	65	389	22	24	1.79	1⅞	1⅞	280	18	20	1.47	1½	1½	153
12 × 10 WF	58	342	19	24	1.79	1⅞	1⅞	242	16	19	1.41	1½	1½	129
	53	312	19	22	1.58	1⅝	1⅝	192	15	19	1.32	1⅜	1⅜	111
12 × 8 WF	50	286	17	23	1.63	1⅝	1⅝	180	13	20	1.47	1½	1½	110
	45	257	16	22	1.50	1½	1½	150	13	18	1.15	1¼	1¼	83
	40	229	15	21	1.37	1⅜	1⅜	123	12	17	1.00	1	1	58
10 × 10 WF	112	663	28	32	3.04	3	3	762	23	26	2.67	2¾	2¾	466
	100	593	27	30	2.77	2¾	2¾	631	22	24	2.42	2½	2½	374
	89	527	26	28	2.52	2½	2½	516	21	23	2.23	2¼	2¼	308
	77	456	24	26	2.27	2¼	2¼	398	19	22	2.07	2⅛	2⅛	252
	72	426	23	25	2.16	2¼	2¼	367	19	20	1.92	2	2	215
	66	390	22	24	2.03	2	2	299	18	20	1.76	1¾	1¾	178
	60	355	21	23	1.90	2	2	274	17	19	1.62	1⅝	1⅝	149
	54	319	20	22	1.76	1¾	1¾	218	16	18	1.47	1¼	1½	122
	49	289	19	21	1.63	1⅝	1⅝	184	15	18	1.47	1½	1½	115
10 × 8 WF	45	258	17	21	1.61	1⅝	1⅝	164	13	18	1.47	1½	1½	99
	39	224	16	19	1.38	1⅜	1⅜	118	13	16	1.14	1¼	1¼	74
	33	189	14	18	1.26	1¼	1¼	89	12	14	1.00	1	1	48
8 × 8 WF	67	387	22	24	2.21	2¼	2¼	337	18	20	1.98	2	2	204
	58	335	21	22	2.06	2⅛	2⅛	278	17	18	1.82	1⅞	1⅞	163
	48	277	18	21	1.84	1⅞	1⅞	201	15	17	1.65	1⅝	1⅝	117
	40	230	17	19	1.57	1⅝	1⅝	149	13	16	1.43	1½	1½	88
	35	201	15	18	1.48	1½	1½	115	12	15	1.28	1⅜	1⅜	70
	31	178	14	17	1.36	1⅜	1⅜	93	12	14	1.10	1⅛	1⅛	54
8 × 6½ WF	28	155	13	16	1.20	1¼	1¼	74	10	14	1.11	1⅛	1⅛	45
	24	133	12	15	1.07	1⅛	1⅛	57	10	12	.84	⅞	⅞	30
8 × 5¼ WF	20	102	10	14	.89	1	1	40	8	12	.73	¾	¾	20
	17	86	9	13	.77	⅞	⅞	29	7	11	.60	⅝	⅝	14

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

—This and following tables presented here by courtesy of American Institute of Steel Construction.

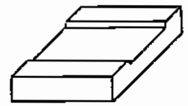
COLUMN BASE PLATE DIMENSIONS (AISC, 1963)

COLUMN BASE PLATES — For A36 Columns



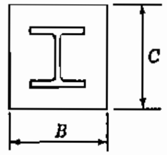
Dimensions for maximum column loads

Base plates, ASTM A36, F_b = 27 ksi
Concrete, f'_c = 3000 psi



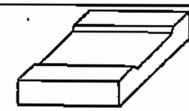
Column			Unit pressure on Support $F_p = 0.25 f'_c$ = 750 psi						Unit Pressure on Support $F_p = 0.375 f'_c$ = 1125 psi					
Nom. Size & Designation	Wt. per Ft.	Max. Load	Dimensions B	Dimensions C	Thickness of Plate Calc.	Fin.	Rolled	Gross Wt.	Dimension B	Dimension C	Thickness of Plate Calc.	Fin.	Rolled	Gross Wt.
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
14 × 14½ BP14	117	709	30	32	2.65	2⅝	2⅝	714	24	27	2.35	2⅜	2⅜	436
	102	618	28	30	2.38	2⅜	2⅜	565	23	24	1.97	2	2	313
	89	539	26	28	2.13	2⅛	2⅛	438	21	23	1.73	1¾	1¾	239
	73	441	24	25	1.76	1¾	1¾	297	19	21	1.41	1½	1½	170
12 × 12 BP12	74	442	23	26	2.08	2⅛	2⅛	360	19	21	1.66	1¾	1¾	198
	53	316	20	22	1.53	1⅝	1⅝	203	16	18	1.19	1¼	1¼	102
10 × 10 BP10	57	335	20	23	1.92	2	2	261	16	19	1.66	1¾	1¾	151
	42	246	18	19	1.41	1½	1½	145	14	16	1.18	1¼	1¼	79
8 × 8 BP8	36	206	16	18	1.46	1½	1½	122	13	15	1.26	1¼	1¼	69
8 × 8 M	34.3	204	16	17	1.39	1⅜	1⅜	106	13	14	1.17	1¼	1¼	64
	32.6	194	16	17	1.36	1⅜	1⅜	106	13	14	1.15	1¼	1¼	64
8 × 6½ M	28	163	13	17	1.35	1⅜	1⅜	86	11	14	1.10	1⅛	1⅛	49
	24	140	12	16	1.20	1¼	1¼	68	10	13	.93	1	1	37
6 × 6 M	25	145	13	15	1.34	1⅜	1⅜	76	11	12	1.10	1⅛	1⅛	42
	22.5	129	13	14	1.17	1¼	1¼	64	10	12	1.09	1⅛	1⅛	38
	20	115	12	13	1.05	1⅛	1⅛	50	10	11	.90	1	1	31
6 × 6 WF	25	146	13	15	1.29	1⅜	1⅜	76	11	12	1.08	1⅛	1⅛	42
	20	117	12	13	1.04	1⅛	1⅛	50	10	11	.89	1	1	31
	15.5	91	11	12	.87	⅞	⅞	33	9	9	.74	¾	¾	17
6 × 4 B	16	87	9	13	1.02	1	1	33	8	10	.83	⅞	⅞	20
	12	64	8	11	.75	¾	¾	19	6	10	.74	¾	¾	13
5 × 5 M	18.9	107	11	13	1.19	1¼	1¼	51	9	11	1.08	1⅛	1⅛	32
5 × 5 WF	18.5	106	11	13	1.17	1¼	1¼	51	9	11	1.06	1⅛	1⅛	32
	16	91	11	12	1.00	1	1	37	9	9	.88	⅞	⅞	20
4 × 4 WF	13	71	9	11	1.00	1	1	28	8	8	.83	⅞	⅞	16
4 × 4 M	13	70	9	11	1.01	1	1	28	7	9	.91	1	1	18
6 × 3⅜ I	17.25	98	10	14	1.16	1¼	1¼	50	8	11	.93	1	1	25
	12.5	71	8	12	.90	1	1	27	7	10	.73	¾	¾	15
5 × 3 I	14.75	83	10	12	1.02	1	1	34	8	10	.91	1	1	23
	10	56	7	11	.89	1	1	22	6	9	.72	¾	¾	11
4 × 2⅝ I	9.5	53	8	9	.82	⅞	⅞	18	6	8	.74	¾	¾	10
	7.7	42	7	8	.70	¾	¾	12	5	8	.72	¾	¾	8
3 × 2⅜ I	7.5	41	7	8	.73	¾	¾	12	6	7	.68	¾	¾	9
	5.7	31	6	7	.59	⅝	⅝	7	5	6	.53	⅝	⅝	5

COLUMN BASE PLATES — For A242-A440-A441 Columns



Dimensions for maximum column loads

Base plates, ASTM A36, F_b = 27 ksi
Concrete, f'_c = 3000 psi

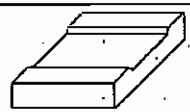


Column			Unit Pressure on Support $F_p = 0.25 f'_c$ = 750 psi						Unit Pressure on Support $F_p = 0.375 f'_c$ = 1125 psi					
Nom. Size & Designation	Wt. per Ft.	Max. Load	Dimensions B	Dimensions C	Thickness of Plate Calc.	Fin.	Rolled	Gross Wt.	Dimensions B	Dimensions C	Thickness of Plate Calc.	Fin.	Rolled	Gross Wt.
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
14 × 16 WF	426	3028	61	67	7.07	7	7½	8684	49	55	6.58	6⅝	7	5344
	398	2827	59	64	6.72	6¾	7¼	7756	48	53	6.26	6¼	6¾	4865
	370	2628	57	62	6.46	6½	7	7008	46	51	5.99	6	6½	4320
	342	2429	55	59	6.10	6⅛	6½	5976	45	48	5.64	5⅝	6	3672
	320	2271	54	57	5.87	5⅞	6¼	5450	43	47	5.48	5½	5⅞	3364
	314	2228	53	57	5.82	5⅞	6¼	5349	43	47	5.37	5⅜	5¾	3292
	287	2036	51	54	5.46	5½	5⅞	4584	41	45	5.08	5⅛	5½	2875
	264	1872	48	52	5.24	5¼	5⅝	3978	39	43	4.81	4⅞	5¼	2494
	246	1744	47	50	4.96	5	5⅜	3578	38	41	4.51	4½	4⅞	2152
	237	1680	46	49	4.85	4⅞	5¼	3352	38	40	4.43	4½	4⅞	2099
	228	1616	45	48	4.73	4¾	5⅛	3136	37	39	4.29	4¼	4⅝	1891
	219	1551	44	47	4.61	4⅝	5	2929	36	39	4.19	4¼	4⅝	1840
	211	1634	46	48	4.78	4¾	5⅛	3206	37	40	4.39	4⅜	4¾	1992
	202	1563	45	47	4.64	4⅝	5	2996	36	39	4.25	4¼	4⅝	1840
	193	1493	44	46	4.50	4½	4⅞	2795	35	38	4.11	4⅛	4½	1696
	184	1423	43	45	4.36	4⅜	4¾	2604	35	37	3.93	4	4	1467
	176	1361	42	44	4.22	4¼	4⅝	2421	34	36	3.78	3¾	3¾	1300
	167	1291	41	42	4.12	4⅛	4½	2195	33	35	3.64	3⅝	3⅝	1186
	158	1222	39	42	4.00	4	4	1856	32	34	3.49	3½	3½	1079
	150	1159	38	41	3.85	3⅞	3⅞	1710	31	34	3.47	3½	3½	1045
	142	1100	37	40	3.73	3¾	3¾	1572	30	33	3.34	3⅜	3⅜	947
14 × 14½ WF	136	1135	38	40	3.77	3¾	3¾	1615	31	33	3.37	3⅜	3⅜	978
	127	1060	37	39	3.61	3⅝	3⅝	1482	30	32	3.20	3¼	3¼	884
	119	993	35	38	3.49	3½	3½	1319	29	31	3.03	3	3	764
	111	926	34	37	3.34	3⅜	3⅜	1203	28	30	2.86	2⅞	2⅞	684
	103	858	33	35	3.08	3⅛	3⅛	1023	27	29	2.70	2¾	2¾	610
	95	792	32	33	2.94	3	3	897	26	28	2.54	2⅝	2⅝	541
	87	725	31	32	2.76	2¾	2¾	773	25	26	2.36	2⅜	2⅜	437
14 × 12 WF	84	689	29	32	2.78	2¾	2¾	723	23	27	2.38	2⅜	2⅜	418
	78	639	28	31	2.63	2⅝	2⅝	645	22	26	2.23	2¼	2¼	365
14 × 10 WF	74	593	26	31	2.57	2⅝	2⅝	599	21	26	2.25	2¼	2¼	348
	68	545	25	30	2.41	2½	2½	531	20	25	2.08	2⅛	2⅛	301
	61	488	23	29	2.25	2¼	2¼	425	19	23	1.94	2	2	248
14 × 8 WF	53	408	20	28	2.10	2⅛	2⅛	337	16	23	1.71	1¾	1¾	182
	48	369	19	26	1.86	1⅞	1⅞	262	15	22	1.57	1⅝	1⅝	152
	43	330	18	25	1.71	1¾	1¾	223	14	21	1.41	1½	1½	125

Note: Rolled plate thicknesses above 4 inches are based on finished thickness plus suggested allowances for finishing one side, and may be modified to suit fabricating plant practice. When it is required to finish both surfaces of base plates, additional allowance must be made.

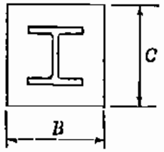
COLUMN BASE PLATE DIMENSIONS (AISC, 1963)

For A242-A440-A441 Columns



COLUMN BASE PLATES
Dimensions for maximum column loads

Base plates, ASTM A36, F_b = 27 ksi
Concrete, f'_c = 3000 psi

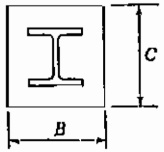


Column: Nom. size & Designation	Column: Wt. per ft.	Column: Max. Load	$F_p = 0.25 f'_c$ = 750 psi: Dimensions B	Dimensions C	Thickness of Plate: Calc.	Fin.	Rolled	Gross Wt.	$F_p = 0.375 f'_c$ = 1125 psi: Dimensions B	Dimensions C	Thickness of Plate: Calc.	Fin.	Rolled	Gross Wt.
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
12 × 12 WF	190	1448	42	46	4.67	4¾	5⅛	2805	34	38	4.29	4¼	4⅝	1693
	161	1226	39	42	4.18	4¼	4⅝	2146	32	35	3.83	3⅞	3⅞	1230
	133	1011	35	39	3.77	3¾	3¾	1450	29	31	3.38	3⅜	3⅜	860
	120	912	34	36	3.47	3½	3½	1214	28	29	3.21	3¼	3¼	748
	106	872	33	36	3.39	3⅜	3⅜	1136	27	29	3.03	3	3	665
	99	813	32	34	3.21	3¼	3¼	1002	26	28	2.86	2⅞	2⅞	593
	92	756	31	33	3.05	3⅛	3⅛	906	25	27	2.69	2¾	2¾	526
	85	698	30	32	2.89	2⅞	2⅞	782	24	26	2.52	2½	2½	442
	79	648	28	31	2.77	2¾	2¾	676	23	25	2.47	2½	2½	424
	72	590	27	30	2.61	2⅝	2⅝	602	22	24	2.18	2¼	2¼	337
	65	533	26	28	2.35	2⅜	2⅜	490	21	23	2.01	2	2	274
12 × 10 WF	58	466	23	28	2.33	2⅜	2⅜	433	19	22	1.93	2	2	237
	53	425	22	26	2.09	2⅛	2⅛	344	18	21	1.77	1¾	1¾	188
12 × 8 WF	50	387	20	26	2.07	2⅛	2⅛	313	16	22	1.82	1⅞	1⅞	187
	45	347	19	25	1.93	2	2	269	15	21	1.67	1¾	1¾	156
	40	309	18	23	1.68	1¾	1¾	205	14	20	1.52	1½	1½	119
10 × 10 WF	112	906	33	37	3.76	3¾	3¾	1297	27	30	3.38	3⅜	3⅜	774
	100	809	31	35	3.52	3½	3½	1076	25	29	3.25	3¼	3¼	668
	89	719	30	32	3.14	3⅛	3⅛	850	24	27	2.93	3	3	551
	77	622	28	30	2.86	2⅞	2⅞	684	23	25	2.59	2⅝	2⅝	428
	72	581	27	29	2.73	2¾	2¾	610	22	24	2.45	2½	2½	374
	66	532	26	28	2.58	2⅝	2⅝	541	21	23	2.30	2⅜	2⅜	325
	60	484	24	27	2.49	2½	2½	459	20	22	2.14	2⅛	2⅛	265
	54	435	23	26	2.33	2⅜	2⅜	402	19	21	1.98	2	2	226
	49	394	22	24	2.09	2⅛	2⅛	318	18	20	1.83	1⅞	1⅞	191
10 × 8 WF	45	349	20	24	2.04	2⅛	2⅛	289	16	20	1.81	1⅞	1⅞	170
	39	302	19	22	1.79	1⅞	1⅞	222	15	18	1.52	1½	1½	115
	33	255	17	20	1.55	1⅝	1⅝	157	14	17	1.34	1⅜	1⅜	93
8 × 8 WF	67	525	25	28	2.81	2⅞	2⅞	570	21	23	2.51	2½	2½	342
	58	454	24	26	2.51	2½	2½	442	20	21	2.33	2⅜	2⅜	283
	48	375	21	24	2.29	2⅜	2⅜	339	17	20	2.09	2⅛	2⅛	205
	40	311	19	22	2.04	2⅛	2⅛	252	16	18	1.76	1¾	1¾	143
	35	272	18	21	1.88	1⅞	1⅞	201	15	17	1.60	1⅝	1⅝	117
	31	241	17	19	1.64	1⅝	1⅝	149	14	16	1.45	1½	1½	95
8 × 6½ WF	28	207	15	19	1.61	1⅝	1⅝	131	12	16	1.44	1½	1½	82
	24	178	14	17	1.36	1⅜	1⅜	93	11	15	1.29	1⅜	1⅜	64
8 × 5¼ WF	20	134	12	15	1.12	1⅛	1⅛	57	9	14	1.08	1⅛	1⅛	40
	17	112	10	15	1.07	1⅛	1⅛	48	8	13	.93	1	1	29

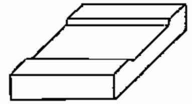
Note: Rolled plate thicknesses above 4 inches are based on finished thickness plus suggested allowances for finishing one side, and may be modified to suit fabricating plant practice. When it is required to finish both surfaces of base plates, additional allowance must be made.

COLUMN BASE PLATES
Dimensions for maximum column loads

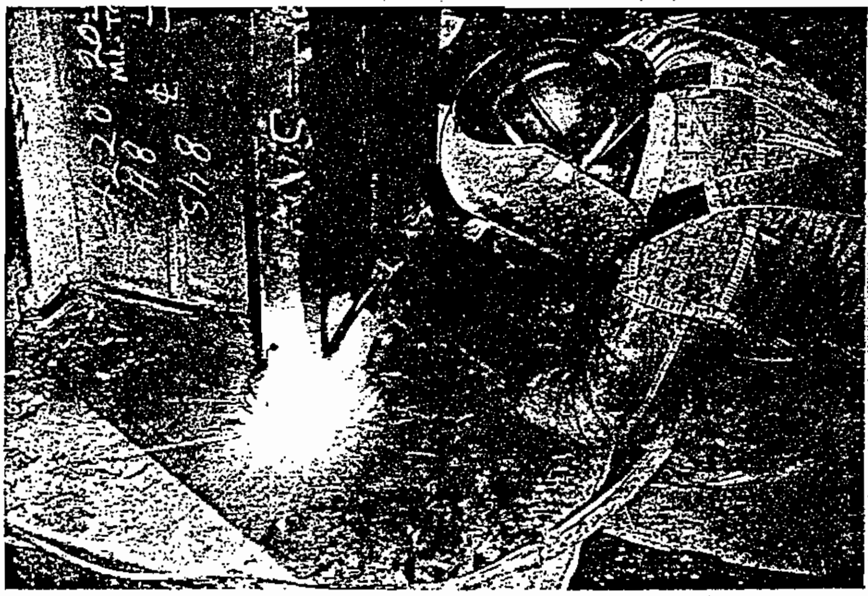
Base plates, ASTM A36, F_b = 27 ksi
Concrete, f'_c = 3000 psi



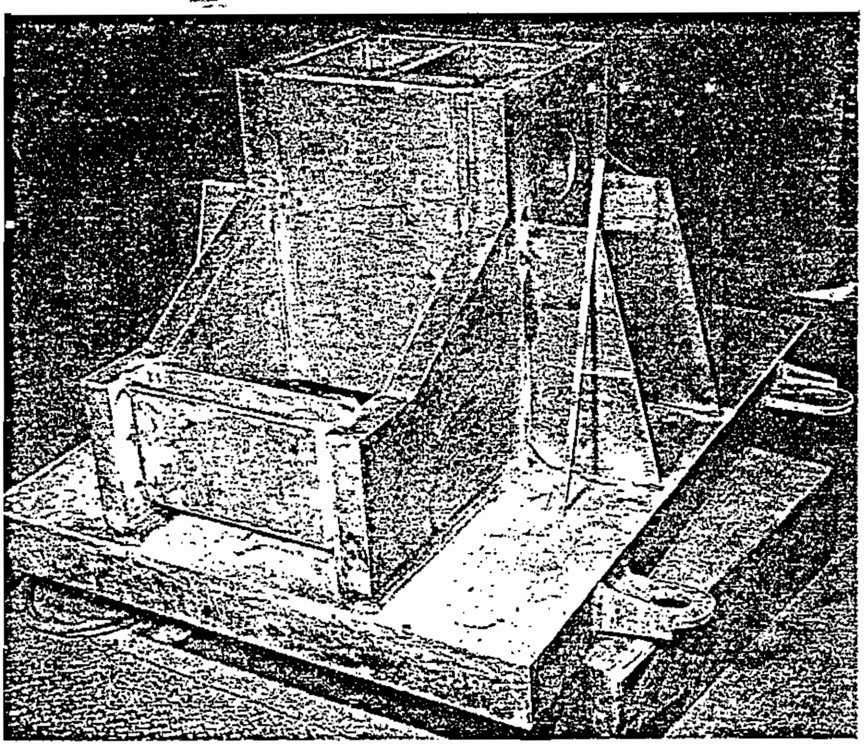
For A242-A440-A441 Columns



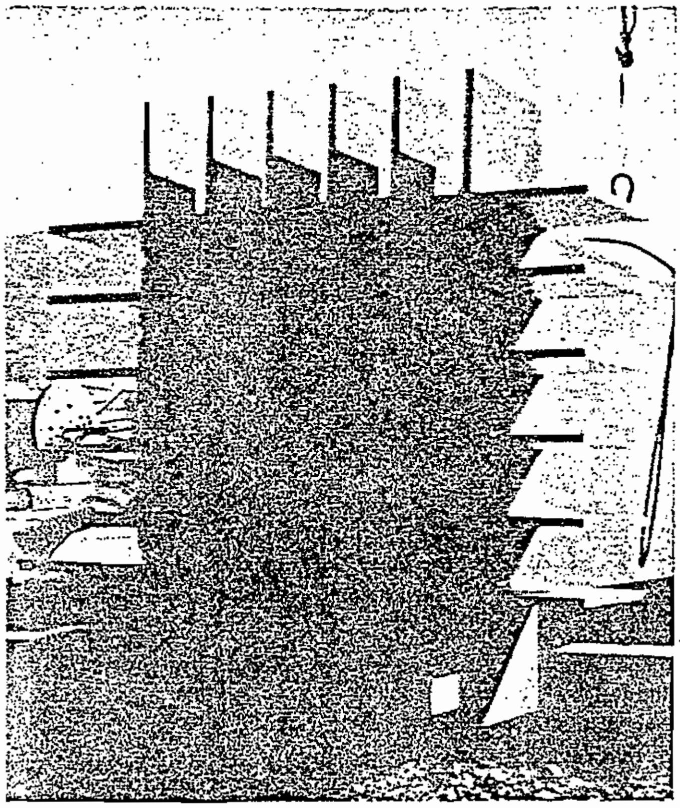
Column: Nom. Size & Designation	Column: Wt. per Ft.	Column: Max. Load	$F_p = 0.25 f'_c$ = 750 psi: Dimensions B	Dimensions C	Thickness of Plate: Calc.	Fin.	Rolled	Gross Wt.	$F_p = 0.375 f'_c$ = 1125 psi: Dimensions B	Dimensions C	Thickness of Plate: Calc.	Fin.	Rolled	Gross Wt.
In.	Lb.	Kips	In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
14 × 14½ BP14	117	974	35	38	3.49	3½	3½	1319	28	31	3.09	3⅛	3⅛	768
	102	849	33	35	3.10	3⅛	3⅛	1023	27	28	2.68	2¾	2¾	589
	89	740	31	32	2.77	2¾	2¾	773	25	27	2.41	2½	2½	478
	73	573	27	29	2.29	2⅜	2⅜	527	22	24	1.92	2	2	299
12 × 12 BP12	74	604	27	30	2.66	2¾	2¾	631	22	25	2.36	2⅜	2⅜	370
	53	422	23	25	1.97	2	2	326	19	20	1.65	1¾	1¾	188
10 × 10 BP10	57	456	24	26	2.35	2⅜	2⅜	420	19	22	2.17	2¼	2¼	266
	42	335	21	22	1.84	1⅞	1⅞	245	17	18	1.56	1⅝	1⅝	141
8 × 8 BP8	36	278	19	20	1.78	1⅞	1⅞	202	15	17	1.63	1⅝	1⅝	117
8 × 8 M	34.3	279	18	21	1.92	2	2	214	15	17	1.64	1⅝	1⅝	117
	32.6	266	18	20	1.78	1⅞	1⅞	191	15	16	1.52	1½	1½	102
8 × 6½ M	28	221	16	19	1.62	1⅝	1⅝	140	13	16	1.44	1½	1½	88
	24	190	15	17	1.41	1½	1½	108	12	15	1.27	1⅜	1⅜	70
6 × 6 M	25	196	16	17	1.60	1⅝	1⅝	125	13	14	1.44	1½	1½	77
	22.5	175	15	16	1.47	1½	1½	102	12	13	1.29	1⅜	1⅜	61
	20	156	14	15	1.34	1⅜	1⅜	82	11	13	1.27	1⅜	1⅜	56
6 × 6 WF	25	199	16	17	1.59	1⅝	1⅝	125	13	14	1.42	1½	1½	77
	20	159	14	16	1.42	1½	1½	95	11	13	1.25	1¼	1¼	51
	15.5	124	12	14	1.19	1¼	1¼	59	10	12	1.07	1⅛	1⅛	38
6 × 4 B	16	115	11	14	1.16	1¼	1¼	55	9	12	1.04	1⅛	1⅛	34
	12	84	9	13	1.03	1⅛	1⅛	37	7	11	.92	1	1	22
5 × 5 M	18.9	144	13	15	1.47	1½	1½	83	11	12	1.26	1¼	1¼	47
5 × 5 WF	18.5	143	13	15	1.45	1½	1½	83	11	12	1.24	1¼	1¼	47
	16	123	12	14	1.32	1⅜	1⅜	65	10	11	1.10	1⅛	1⅛	35
4 × 4 WF	13	94	11	12	1.13	1⅛	1⅛	42	9	10	1.03	1⅛	1⅛	29
4 × 4 M	13	92	11	12	1.14	1¼	1¼	47	9	10	1.05	1⅛	1⅛	29
6 × 3⅜ I	17.25	133	12	15	1.33	1⅜	1⅜	70	10	12	1.25	1¼	1¼	42
	12.5	97	10	13	1.06	1⅛	1⅛	41	8	11	.93	1	1	25
5 × 3 I	14.75	112	11	14	1.32	1⅜	1⅜	60	9	12	1.23	1¼	1¼	38
	10	75	9	12	1.01	1	1	31	7	10	.91	1	1	20
4 × 2⅝ I	9.5	71	9	11	1.02	1	1	28	7	10	1.04	1⅛	1⅛	22
	7.7	57	8	10	.87	⅞	⅞	20	6	9	.89	1	1	15
3 × 2⅜ I	7.5	54	8	9	.89	1	1	20	6	8	.91	1	1	14
	5.7	41	7	8	.73	¾	¾	12	6	7	.68	¾	¾	9



Column base plates for the 32-story Commerce Towers, Kansas City, Mo., were shop-fabricated and shipped separately. At the site they were positioned and bolted to the concrete. The heavy columns were then erected and field welded to base plates. This was facilitated by use of semi-automatic arc welding with self-shielding cored electrode wire. Process quadrupled the speed of manual welding and produced sounder welds.



Ten-ton weldments were required for tower bases on lift bridges along the St. Lawrence Seaway. Edges of attaching members were double-beveled to permit full penetration. Iron powder electrodes were specified for higher welding speeds and lower costs. Because of high restraint, LH-70 (low hydrogen) E7018 electrodes were used on root passes to avoid cracking, while E6027 was used on subsequent passes to fill the joint.



In designing a scenic highway bridge with 700′ arch span, near Santa Barbara, Cal., engineers called for tower columns to be anchored to the concrete skewbacks by means of $1\frac{3}{8}$″ prestressing rods. The bottom of the column is slotted to accommodate the base, an "eggbox" grill made up of vertical plates welded together and to the box column. The towers support heavy vertical girder loads but also safely transmit horizontal wind and seismic loads from the deck system to the foundation.