5.5 Base Design

To analyze the stresses between the base of the stack and its foundation, it is important to consider the degree of fixity of the base, which depends on the connection details. ϵ is a factor which allows for this degree of fixity and is graphically explained in Figure 5.8.

5.5.1 Design of Anchor Bolts

In designing anchor bolts for a self-supporting steel stack, the weight of the lining is not considered as the steel work is usually built first and the masonry lining added afterwards. Sometimes a considerable portion of the lining is removed and renewed during the life of the stack. This means that the anchor bolts must be large enough to keep the stack from overturning before the lining is removed and renewed during the life of the stack.

The load acting on the stack is transferred either as a compressive or tensile load to the concrete footing through the anchor bolts. The bending moment M and the weight of the stack W results in a loading condition in the concrete footing similar to that as shown in Figure 5.9.

In the calculation it is assumed that the bolt ring is the center of the bearing plate. The moment and weight of the

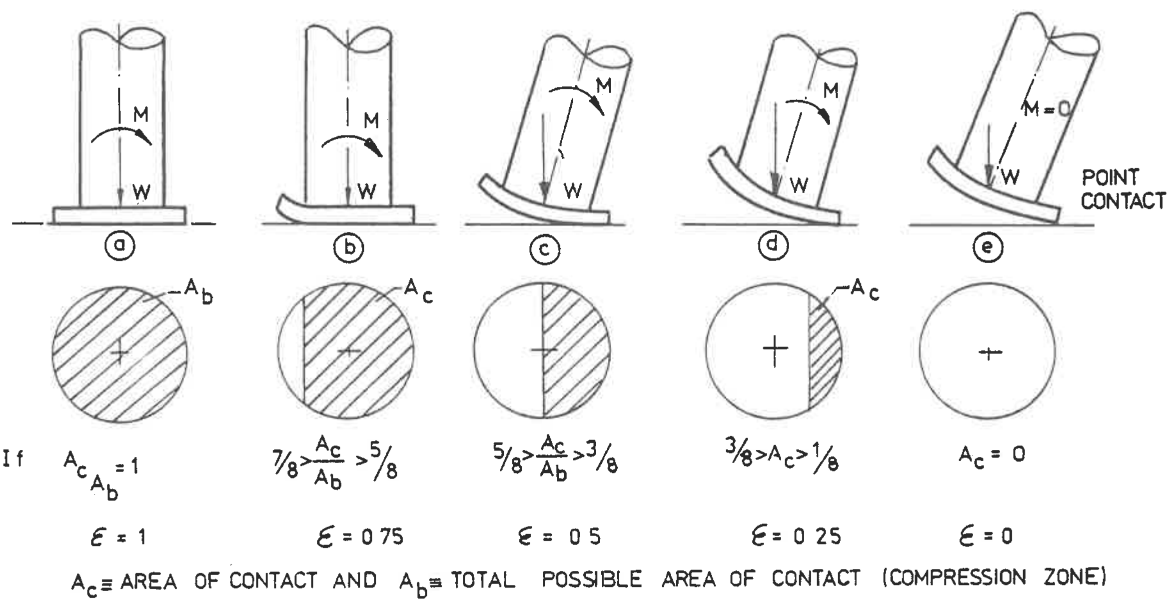

FIGURE 5.8 — ϵ as a function of base area contact of stack.

stack result in a tensile load on the left-side anchor bolts and a compression load on the right-side [5.52].

Calling κD the distance between the neutral axis and the mean circumference on compression side, as shown in Figure 5.10, we have by similar triangles

$$\kappa = \frac{nf_c}{nf_c + f_s} = \frac{1}{1 + \frac{f_s}{nf_c}} \tag{5.44}$$

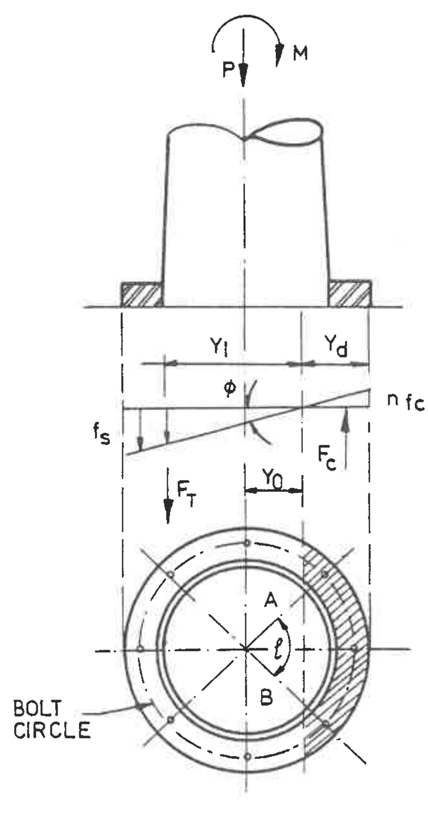

FIGURE 5.9 — Stress distribution at stack base.

where

$n = \frac{E_s}{E_c}$ = the ratio of moduli or elasticities of steel to the concrete.

From Figure 5.10, the location of the neutral axis n-n may be defined in terms of angle α

$$\cos\alpha = \frac{D/2 - \kappa D}{D/2} = 1 - 2\kappa \tag{5.45}$$

The total force, T, on the tension side of the section is

$$T = 2 \int_0^{(\pi-\alpha)} t_s r f_s \frac{(\cos\theta + \cos\alpha)}{(1+\cos\alpha)} d\theta$$

$$= f_s r t_s \frac{2}{(1+\cos\alpha)} [\sin\alpha + (\pi-\alpha)\cos\alpha] \tag{5.46}$$

Since any given position of the neutral axis determines α, this equation may take the form

$$T = C_T f_s r t_s \tag{5.47}$$

in which C_T is a constant for a given position of the neutral axis and shown in Table 5.2.

The moment of the total tensile force, T, about the neutral axis is

$$M_T = 2 \int_0^{(\pi-\alpha)} t_s r f_s \frac{r(\cos\theta + \cos\alpha)^2}{(1+\cos\alpha)} d\theta$$

$$= t_s r^2 f_s \frac{2}{(1 \ \cos\alpha)} [(\pi-\alpha)\cos^2\alpha + $$

$$+ \frac{3}{2} \sin\alpha\cos\alpha + \frac{1}{2}(\pi-\alpha)] \tag{5.48}$$

Dividing M_T by T, we have as the distance of the center of tension from the neutral axis

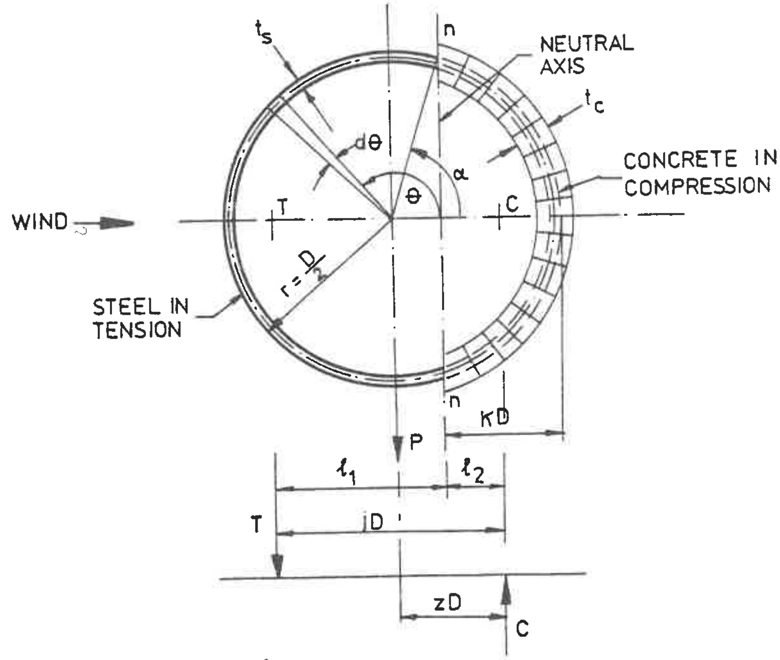

FIGURE 5.10 — Load on anchor bolts and bearing plate.

$$\ell_1 = \frac{[(\pi-\alpha)\cos^2\alpha + \frac{3}{2}\sin\alpha\cos\alpha + \frac{1}{2}(\pi-\alpha)]}{[\sin\alpha + (\pi-\alpha)\cos\alpha]} r \qquad (5.49)$$

Similarly, we determine the total compression force

$$C = C_P f_c r \ (t_c + nt_s) \qquad (5.50)$$

in which Cp is a constant for a given position of the neutral axis shown in Table 5.2.

The moment of the total compressive force C about the neutral axis is

$$M_c = (t_c + nt_s) f_c r^2 \frac{2}{(1-\cos\alpha)} [\alpha\cos^2\alpha - \frac{3}{2}\sin\alpha\cos\alpha + \frac{1}{2}\alpha] \qquad (5.51)$$

TABLE 5.2 — Values of k, C_P, C_T, z and j.

	Function of k			
k	C_P	C_T	z	j
0.050	0.600	3.003	0.490	0.760
0.100	0.852	2.887	0.480	0.766
0.150	1.049	2.772	0.469	0.771
0.200	1.218	2.661	0.459	0.776
0.250	1.370	2.551	0.443	0.779
0.300	1.510	2.412	0.433	0.781
0.350	1.640	2.333	0.427	0.783
0.400	1.765	2.224	0.416	0.784
0.450	1.884	2.113	0.404	0.785
0.500	2.000	2.000	0.393	0.786
0.550	2.113	1.884	0.381	0.785
0.600	2.224	1.765	0.369	0.784

and the distance of the center of compression from the neutral axis is

$$\ell_2 = \frac{M_c}{C} = \frac{[\alpha\cos^2\alpha - \frac{3}{2}\sin\alpha\cos\alpha + \frac{1}{2}\alpha)]}{(\sin\alpha - \alpha\cos\alpha)} r \qquad (5.52)$$

The system of forces, as shown in Figure 5.10, must be in equilibrium. Hence, taking moment ab out the force P, we may write

$$TjD = M - PzD$$

But

$$T = C_T f_s r t_s$$

Therefore

$$C_T f_s r t_s jD = M - PzD$$

Whence

$$rt_s = \frac{M - PzD}{C_T f_s jD}$$

The total area of steel required is

$$A_s = 2\pi r t_s$$

Therefore

$$A_s = \frac{2\pi(M-PzD)}{C_T f_s jD} \qquad (5.53)$$

From Table 5.1, it may be seen that the constant j changes but slightly for a considerable variation in the position of the neutral axis. Taking $\frac{2\pi}{j} = 8$ for all cases. Equation (5.53) may be

$$A_s = \frac{8(M - PzD)}{C_T f_s jD} \tag{5.54}$$

Applying now the condition that the summation of all vertical forces must be zero, we have

$$C - T = P$$

Substituting the values of C and T as previously found, we

$$C_P f_c r\,(t_c + nt_s) - C_T f_s r t_s = P$$

Solving for t_c, we obtain

$$t_c = \frac{P + (C_T f_s - C_P f_c n) r t_s}{C_P f_c r} \tag{5.55}$$

The number of anchor bolts for a self-supporting steel stack should never be less than 8 and should preferably be 10 or 12 or more depending on the size of the stack.

Generally, for given values of M and P, and assumed number and cross-sections of anchor bolts it is required to determine the maximum stresses in the anchor bolts f_s and the concrete under compression f_c. The problem is solved by a method of successive trials, since the position of the neutral axis is not known. The procedure is, as follows:

1. Assume a position of the neutral axis, select the constants accordingly, substitute into Equations (5.54) and (5.55) and solve them for f_s and f_c.
2. Check the position of the neutral axis as fixed by these values of f_s and f_c is the same as the position assumed at the start. If the two positions agree, then f_s and f_c as found are the actual stresses.
3. If not, a new position of the neutral axis must be assumed, new constants selected, and new values of f_s and f_c computed. Thus a series of trials must be made until the location of the neutral axis as assumed is consistent with the computed values of f_c and f_s.

5.5.2 Base Plate

5.5.2.1 Base Plates Without Gussets

A base plate without gussets may be assumed to be a uniformly loaded cantilever beam with f_c the uniform load. The maximum bending moment for such a beam occurs at the junction of the stack shell and the base plate for unit circumferential length b = 1 in and is equal to

$$M_{max} = f_c b \ell \left(\frac{\ell}{2}\right) = \frac{f_c \ell^2}{2} \tag{5.56}$$

where ℓ is the base plate minus the other radius of the stack shell.

The maximum stress in an elemental strip of unit width is

$$f_{max} = \frac{6M_{max}}{bt_1^2} = \frac{3f_c \ell^2}{t_1^2} \tag{5.57}$$

where t_1 is the base-plate thickness in inches. Letting $f_{max} = f_{all}$ and solving for t_1 gives

$$t_1 = \ell\sqrt{3f_c/f_{all}} \tag{5.58}$$

5.5.2.2 Base Plates With Gussets

If gussets are used to stiffen the base plates, the loading conditions on the section of the plate between two gussets may be considered to act similarly to that of a rectangular uniformly loaded plate with two opposite edges simply supported by the gussets, the third edge joined to the shell, and the fourth and outer edge free. For this particular case Timoshenko and Woinowsky-Kreiger **[5.53]** have tabulated the deflections and bending moments as shown in Table 5.3.

To determine the base-plate thickness from the bending moment the following formula should be used

$$t_1 = \sqrt{\frac{6M_{max}}{f_{all}}} \tag{5.59}$$

Note that in Table 5.3, for the case where L/b = 0 (no gussets or gusset spacing b = ∞) the bending moment is reduced to Equation (5.56), and the thickness of the base plate is determined by Equation (5.58). Also when L/b is equal to or less than 3/2, the maximum bending moment occurs at the junction with the shell because of cantilever action. If L/b is greater than 3/2, the maximum bending moment occurs at the middle of the free edge.

5.5.2.3 Practical Considerations in Designing Base Plates

Rolled-angle base plates may be used for stacks if the calculated thickness of the base plates is 1/2 in. or less. The steel angle is rolled to fit as shown in Figure 5.11 **[5.54]**.

If the required base-plate thickness is 1/2 in. to 3/4 in., a design using a single-ring base plate may be used as shown in Figure 5.12.

TABLE 5.3 — Maximum Bending Moments in a Base Plate with Gussets.

1/b	$\left(M_x\ \begin{matrix} x = b/2 \\ y = \ell \end{matrix}\right)$	$\left(M_y\ \begin{matrix} x = b/2 \\ y = 0 \end{matrix}\right)$
0	0	$-0.500 f_c \ell$
1/3	$0.0078 f_c b^2$	$-0.428 f_c \ell$
1/2	$0.0293 f_c b^2$	$-0.319 f_c \ell$
2/3	$0.0558 f_c b^2$	$-0.227 f_c \ell$
1	$0.0972 f_c b^2$	$-0.119 f_c \ell$
3/2	$0.123 f_c b^2$	$-0.124 f_c \ell$
2	$0.131 f_c b^2$	$-0.125 f_c \ell$
3	$0.133 f_c b^2$	$-0.125 f_c \ell$
∝	$0.133 f_c b^2$	$-0.125 f_c \ell$

b = gusset spacing (x direction) inches.
δ = ℓ = bearing-plate outside radius minus skirt outside radius (y direction) inches.

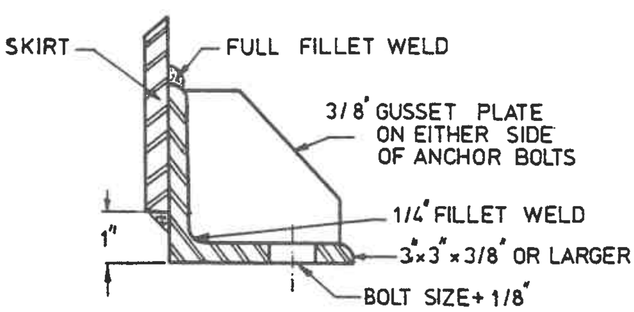

FIGURE 5.11 — Rolled-angle base plate.

If the required base-plate thickness is ¾ in. or greater a bolting "chair" may be used as shown in Figures 5.13 and 5.14.

Although the number and size of bolts required should be checked for each individual design, some typical values of maximum numbers of chairs can be obtained from Table 5.4 for a given stack base diameter.

When checking the base-plate thickness for a centered chair, Figure 5.13, the plate inside the stiffeners is considered to act as a concentrated loaded beam with fixed ends. The concentrated load, P, produced by the bolt is equal to the maximum bolt stress multiplied by the bolt root thread area.

The maximum moment in the base plate occurs on the line of symmetry centered inside the chair and is given by

$$M_{max} = \frac{Pb}{8} \qquad (5.60)$$

The anchor bolt hole reduces the effective width of the plate. Taking this into consideration, the base-plate thickness, t_2, is

$$t_2 = \sqrt{\frac{6M_{max}}{(b_1 - bhd)\,f_{all}}} \qquad (5.61)$$

where b_1 is the width of the base plate, bhd is the bolt-hole diameter and f_{all} is the allowable stress in psi.

If the number of bolts required is greater than that given in Table 5.4, an external bolting chair may be used, as shown in Figure 5.14.

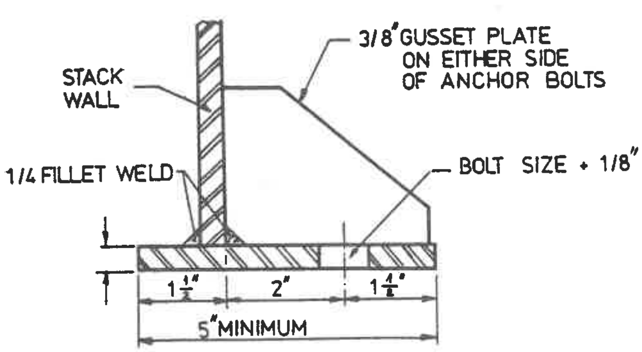

FIGURE 5.12 — Single ring base plate.

TABLE 5.4 — Number of Chairs for Various Size Stack Diameter.

Stack Diameter, ft	No. of Chairs
3	4
4	8
5	8
6	12
7	16
8	16
9	20
10	24

With reference to Figure 5.14, plan view, with y in the radial direction and z in the circumferential direction, the maximum bending moments M_Z and M_X are given by

$$M_z = \frac{P}{4\pi}\left[(1+\mu)\,\ell n\left(\frac{2\ell\sin\frac{\pi a}{\ell}}{\pi e}\right) + 1\right] - \left[\frac{\gamma_1 P}{4\pi}\right] \qquad (5.62)$$

$$M_x = \frac{P}{4\pi}\left[(1+\mu)\,\ell n\left(\frac{2\ell\sin\frac{\pi a}{\ell}}{\pi e}\right) + 1\right] - \left[(1-\mu-\gamma_2)\frac{P}{4\pi}\right] \qquad (5.63)$$

where

μ	=	Poisson's ratio (0.30 for steel)
ℓn	=	natural logarithm
α	=	radial distance from outside of skirt to bolt circle, inches
ℓ	=	radial distance from outside of skirt to outer edge of compression plate, inches
b	=	gusset spacing, inches
e	=	radius, of action of concentrated load, inches or one-half distance across flats of bolting nut, inches
γ_1, γ_2	=	constants from Table 5.5

when b – unity, Mx = Mz, and when b > 1, Mz > Mx and therefore Mz controls.

For the case in which a = $\ell/2$ and Mz is controlling, Equation (5.62) reduced to

$$M_z = \frac{P}{4\pi}\left[(1+\mu)\;\ell n\frac{2\ell}{\pi e} + (1-\gamma_1)\right] \qquad (5.64)$$

The maximum stress in the compression ring of unit width is

$$f_{max} = \frac{6M_y}{t_3^2} \qquad (5.65)$$

TABLE 5.5 — Constants for Moment Calculations.

b/ℓ	1.0	1.2	1.4	1.6	1.8	2.0	∝
γ_1	0.565	0.350	0.211	0.125	0.073	0.042	0
γ_2	0.135	0.115	0.085	0.057	0.037	0.023	0

Note: For a, b, ℓ less than 1.0 invert b,ℓ and rotate axes 90².

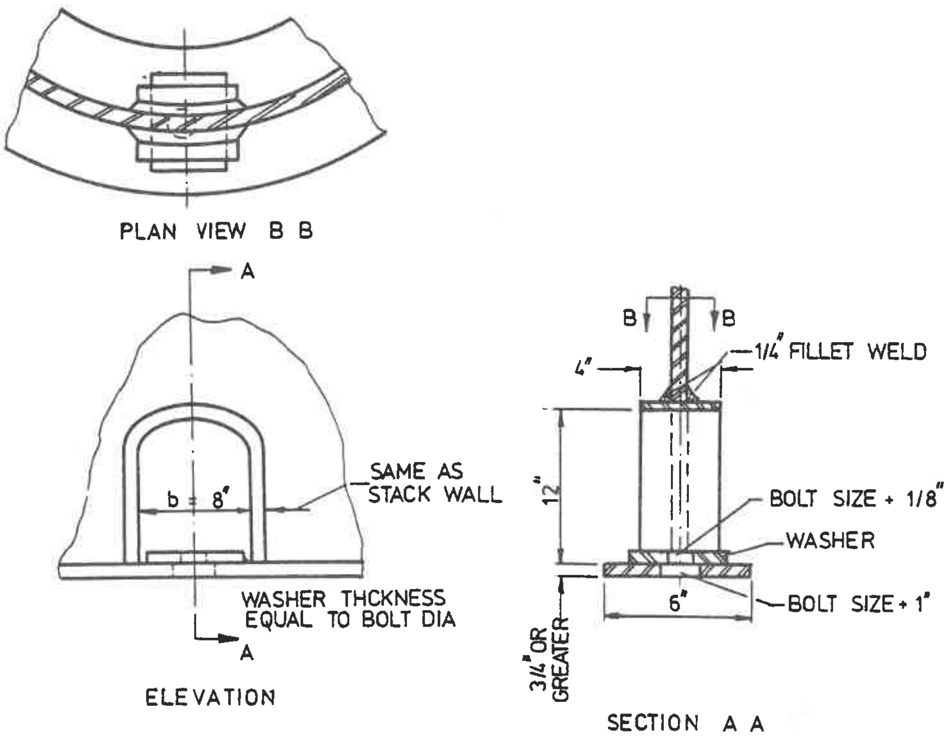

FIGURE 5.13 — Center anchor-bolt chair.

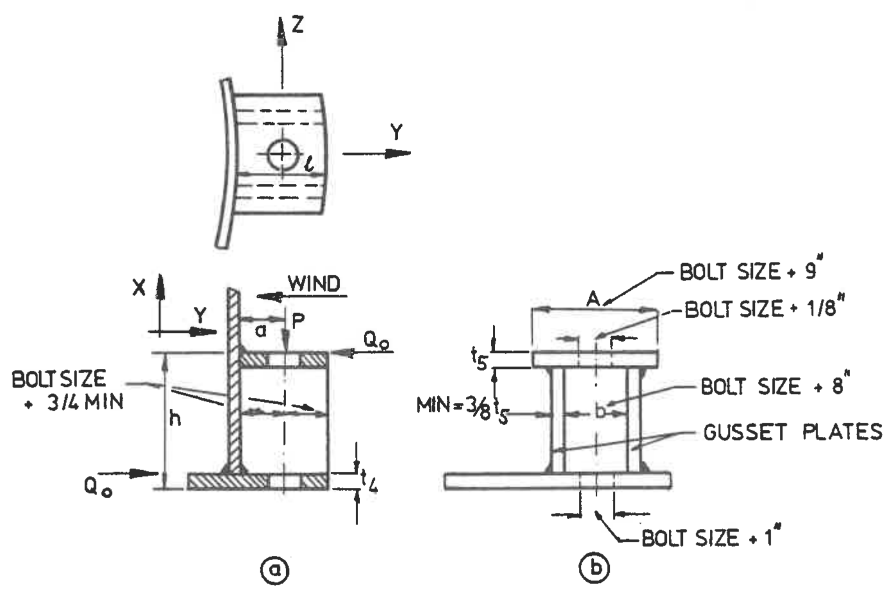

FIGURE 5.14 — External bolting chair.

where t_3 is the thickness of the compression ring.

5.5.2.4 Design of Gusset Plates for Compression Rings

If the gussets are spaced evenly as shown in Figure 5.15, they may be considered to behave as vertical columns [5.56].

The moment of inertia of the gusset about the axis having the radius of gyration is given by

$$I = \frac{\ell t_4^3}{12} = ar^2 = \ell t_4 r^2 \tag{5.66}$$

or

$$r^2 = \frac{t_4^2}{12} \tag{5.67}$$

where

a = area of cross-section, in

r = radius of gyration, in

t_4 = gusset-plate thickness, in

ℓ = width of gusset, in

and

$$f = \frac{P}{a} = \frac{18000}{1 + (h^2/18000r^2)} \text{ if } 60 < h/r < 200 \tag{5.68}$$

where

h = height of gusset, in

From Equations (5.66), (5.67) and (5.68) we may obtain

$$18{,}000\ \ell t_4^3 - (\text{bolt load})t_4^2 - \frac{h^2(\text{bolt load})}{1{,}500} = 0 \tag{5.69}$$

If h is small the third term in the equation may be disregarded, therefore simplifying Equation (5.69) to

$$t_4 = \frac{\text{bolt load}}{18{,}000\ \ell} \tag{5.70}$$

When an external bolting chair is used the thickness of the stack shell, t, at the base should be checked. To determine the thickness Equation (5.71) can be used

$$t = 1.76\left(\frac{P_a}{mh\ f_{all}}\right)^{2/3} r^{1/3} \tag{5.71}$$

where

r = radius of the stack at the point under consideration, inches

P = maximum bolt load, pounds

a = radial distance from outside of stack shell to the anchor bolt circle, inches

h = gusset height, inches

m = 2A (see Figure 5.14) or bolt spacing

TUBULAR STEEL STRUCTURES —

Theory and Design

SECOND EDITION

by

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