

vided that the same safety factors in tangential and longitudinal directions are used. Then

$$\sigma_t/k(\sigma_w/S.F.) + \sigma_L/k(\sigma_{Lc}/S.F.) \leq 1,$$

where $\sigma_{Lc}/S.F.$ and $\sigma_{Lc}/S.F.$ can be taken as the Code allowable compressive stresses; k is the Code allowable factor equal to 1.2 when applicable. S.F. is the safety factor in the strength design i.e., the number of times the load might be increased before the structure will fail due to stress, usually established with respect to yield stress, buckling stress etc.

Since the S.F. in axial Code allowable stress is higher than in the tangential direction ($B = S_c/2$), the above formula gives too conservative results.

It has also been pointed out in reference 164 that the method needs more experimental verification, particularly on large welded steel cylindrical vessels. However, based on up-to-date test results, the straight line equation seems adequate and conservative. The interaction curve method eliminates the need for theoretical derivation of the maximum combined stress.

Example 4.1. From reference 27 a tall vacuum vessel, 10 feet in diameter and 100 feet high with stiffeners 6 feet apart has a shell thickness 0.375 in. of SA285B. The total vertical weight is 200,000 lbs and the moment of external forces at the bottom t.l. is 2,000,000 ft-lb. Design temperature is 200°F. Check if the shell thickness is adequate under combined loadings by the interaction method.

$$\begin{aligned}\sigma_L &= (PD/4 + 4M/\pi D^2 + W/\pi D)/t \\ &= (15 \times 120/4 + 2000000 \times 12/\pi 120^2 + 200000/\pi 12)/0.375 \\ &= 8280 \text{ psi in compression}\end{aligned}$$

$$\sigma_t = PR/t = 15 \times 60/0.375 = 2400 \text{ psi in compression}$$

$$\sigma_{Lc}/S.F. \text{ for } A = 0.125/(R_o/t) = 0.125/(60.375/0.375) = 0.00078$$

$$B = 10500 \text{ psi (in plastic elastic range), S.Y.} = 27,000 \text{ psi, S.Y./2} = 13500 \text{ psi}$$

$$\sigma_{rc}/S.F. \text{ for } L/D = 0.6 \text{ and } D_o/t = 320 \quad A = 0.00042$$

$$B = 6300 \text{ psi in elastic range}$$

$$\sigma_t/(k\sigma_{rc}/S.F.) + \sigma_L/(k\sigma_{Lc}/S.F.) = 1.0 \quad \text{and taking } k = 1.2$$

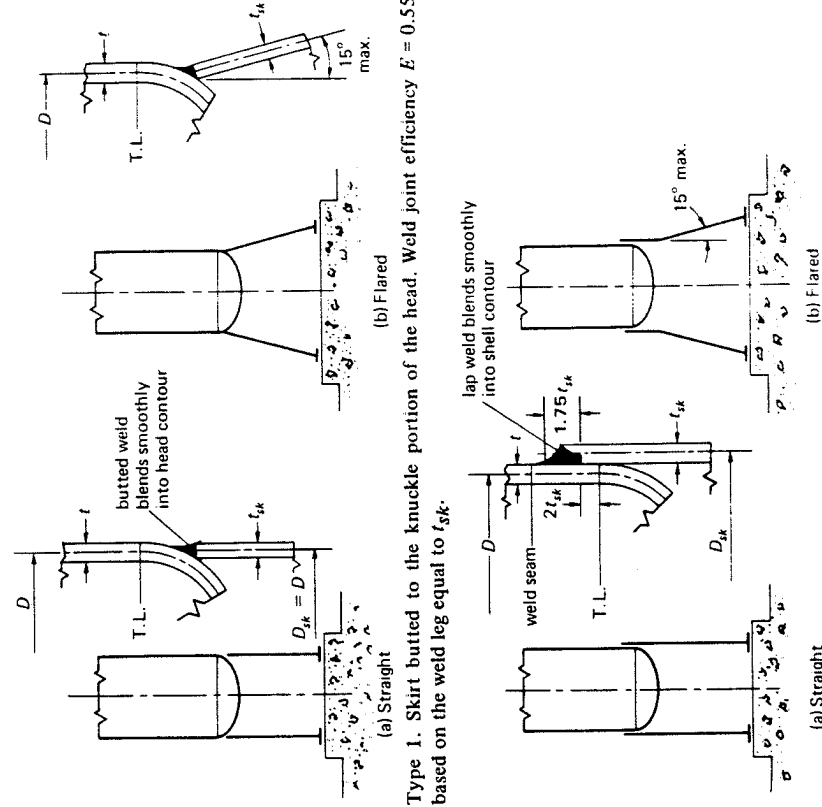
$$8280/(1.2 \times 10500) + 2400/(1.2 \times 6300) = 0.98 < 1.0$$

The result is more conservative than the result in reference 27. A more acceptable result would be obtained by using S.Y./2 for $\sigma_{Lc}/S.F.$

4.3. SUPPORT SKIRTS

Design of the Support Skirt Shell

The support skirts are welded directly to the vessel bottom head or shell, as shown in Fig. 4.1. The factors determining the skirt thickness t_{sk} can be summarized as follows.



Type 1. Skirt butted to the knuckle portion of the head. Weld joint efficiency $E = 0.55$, based on the weld leg equal to t_{sk} .

Type 2. Skirt lapped to the cylindrical portion of the shell. Weld joint efficiency $E = 0.80$ based on the weld leg equal to t_{sk} .

Fig. 4.1. Types of support skirts and skirt-to-head welds.

1. The maximum longitudinal stress due to the external moment M and weight W at the base is

$$\sigma_L = - (W/\pi D_{sk} t_{sk}) \pm (4M/\pi D_{sk}^2 t_{sk}) \text{ psi.}$$

2. The longitudinal compressive stress at the base under test conditions, if the vessel is tested in vertical position, is

$$\sigma_L = - W_T/\pi D_{sk} t_{sk}$$

3. The maximum stress in the skirt-to-head weld, with the weld joint efficiency E depending on the type of the skirt and weld used (see Fig. 4.1), quite often determines the support skirt thickness,

$$t_{sk} = [(W/\pi D_{sk}) + (4M/\pi D_{sk}^2)]/E \times \text{Code allowable stress}$$

where E = weld efficiency.

4. The skirt thickness t_{sk} should be satisfactory for the allowable column deflection; usually t_{sk} for tall towers is chosen not less than the corroded bottom shell section plate thickness.

5. Support skirts for large-diameter vessels, which have to be stress-relieved in the field in a vertical position, must be checked to determine whether the thickness will withstand the weight under high-temperature conditions.

6. If a large access or pipe opening is located in the skirt shell the maximum stress at a section through the opening can be checked:

$$\sigma_L = \pm M/t_{sk} [(\pi D_{sk}^2/4) - (YD_{sk}/2)] - W/(\pi D_{sk} - Y)t_{sk}.$$

If σ_L is too high the opening has to be reinforced (see Fig. 4.2).

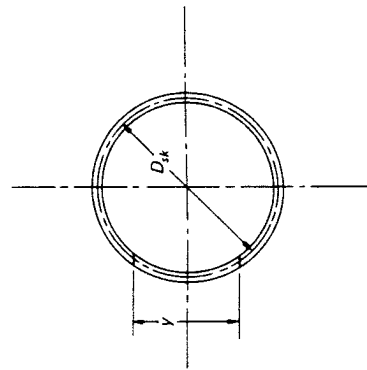


Fig. 4.2. Plan section through a large access opening in the support skirt.

In Fig. 4.1 two typical constructions of support skirts and their attachment welds are shown. Skirt type 1(a) is the most often used design for tall vessels. The centerlines of the cylindrical skirt plate and the corroded shell plate are approximately coincident. If the skirt plate is thicker than the bottom shell plate, the outside diameter of the skirt is made equal to the outside diameter of the bottom shell. If the uplift under the imposed external moment is too high and the anchor bolt spacing becomes too small for the bolt size, the skirt is designed as flared Type 1(b). The localized bending stresses induced in the head by a Type 1(a) skirt are considered acceptable. However, with a Type 1(b) skirt support, the stresses in the head can become excessive and may have to be analyzed more thoroughly.

In Type 2(a) the skirt is attached to the flanged portion of the bottom head in such a way that it does not obstruct an inspection of the head-shell weld seam. This type is more difficult to fabricate and is used mainly for high external loads, high design temperatures, or cyclic operating temperatures. A good fit between the outside diameter of the shell and the inside diameter of the skirt is essential. A flared skirt of Type 2(b) is used for very high columns with extra high external moments.

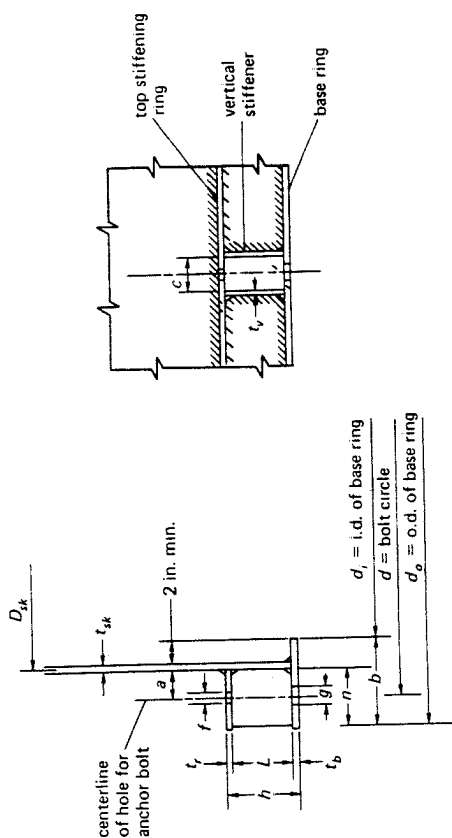
Skirt Base Design

In Fig. 4.3 two most frequently used skirt base types are illustrated. In Type A the anchor bolts are located off the centerline of the skirt plate. A continuous top stiffening ring is provided to reinforce the support skirt shell against undesirable localized bending stresses.

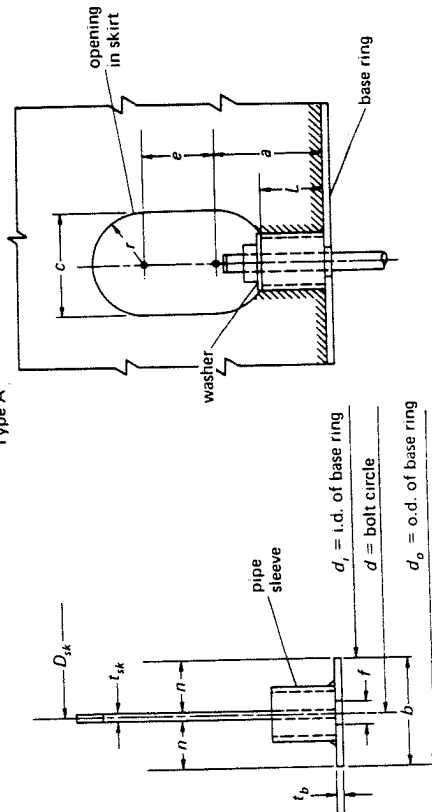
The disadvantages of the Type B skirt base are the weakening of the skirt shell by the access openings for the bolts and the necessity to check the plate between the openings for buckling. Both dimensions e and c should be kept to a minimum. The bolt circle is equal to the mean diameter of the skirt shell, and the weld connecting the pipe sleeve to the skirt is stressed in shear only because of the holding-down bolt force.

Base Ring Design. To distribute the vertical load over a sufficient area of the concrete foundation a plate base ring is used. In addition it serves also to accommodate the anchor bolts. For the determination of the base ring thickness of both Types A and B the method in the AISC Manual is generally applied. The load is assumed to be uniformly distributed over the entire width b (see Fig. 4.3). The effect of bolt holes and any reinforcement by the vertical stiffeners is disregarded. If the bearing pressure p due to the dead-weight load W combined with the external moment M is

$$p = [(W/\pi D_{sk}) + (4M/\pi D_{sk}^2)]/b \text{ kips/in}^2.$$



Type A



Type B

c = wrench o.d. + clearance

Fig. 4.3. Types of skirt base.

then the maximum bending stress in the base ring plate is given by

$$\sigma_b = (pn^2/2)/(t_b^2/6) = 3pn^2/t_b^2 \leq S_b$$

and the required base plate thickness is

$$t_b = (3pn^2/S_b)^{1/2} \text{ in.},$$

where n is the dimension in Fig. 4.3 and S_b is the allowable bending stress. For $S_b = 20 \text{ kips/in.}^2$, the formula becomes

$$t_b = (0.15 pn^2)^{1/2} \text{ in.}$$

The above formula can be corrected for a lower allowable stress S'_b (psi) as follows:

$$t_b = (0.15 pn^2 \times 20000/S'_b)^{1/2} \text{ in.}$$

The maximum allowable bearing F_b pressure between the base ring and the concrete depends on the compressive strength f'_c of the grade of concrete used. If f'_c is the specified compressive strength (usually between 2 and 5 kips), then, per AISC Manual, Section 1.5.5, the allowable bearing pressure F_b for direct dead load is

$$F_b = 0.25 f'_c$$

when the entire area of the concrete support is covered and

$$F_b = 0.375 f'_c$$

when only one-third of the area of the concrete base is covered. A value of $F_b = 0.3 f'_c$ is generally used and recommended, and the maximum bearing pressure $p \leq F_b$. The above value of F_b can be increased by one third for bearing pressures produced by combined dead loads plus wind or earthquake loads (AISC Manual, Section 1.5.6).

Skirt-to-Base Ring Weld. In Type A the holding-down force of the anchor bolts is transferred into the skirt shell by welds connecting the top ring, vertical stiffeners, and the base ring plate, in Type B by the weld between the pipe sleeves and the base ring plate. However, the weld between the skirt and the base ring is assumed to carry the total load and designed as a primary strength weld, preferably continuous.

On the windward side of the vessel the weld has to resist the full uplift load,

$$I_1 = (4M/\pi D_{sk}^2) - (W/\pi D_{sk}) \text{ lb/lin. in.}$$

On the leeward side for the down-load condition any size of weld would theoretically be sufficient. However, since most skirts are so large that their ends cannot be machined accurately enough to produce a uniform bearing, it

for axially loaded columns per the AISC column formula should not exceed

$$P/2a \leq 17000 - 0.485 (L/r)^2 \text{ psi,}$$

where

$a = t_V(n - 0.25)$, the cross-sectional area of one stiffener, in.

L = length of the stiffener, in.

$r = (I/a)^{1/2} = 0.289 t_V$, the radius of gyration of the stiffener, in.

P = the maximum bolt load, lb.

$(n - 0.25)$ = effective width of the stiffener, in.

The thickness t_V is usually between $\frac{1}{2}$ and $1\frac{1}{4}$ in., depending on bolt size. Where no uplift or only a very small uplift results from external loads the top ring section can be omitted between bolts and the design reduces to a bolting chair. The minimum size for bolts ($\frac{3}{4}$ to 1 in.) is selected and the anchor bolts serve merely to locate the vessel in place.

Skirt Material

The most frequently used materials for support skirts are carbon steels, A283 gr. C for thicknesses up to $\frac{5}{8}$ in. and A285 gr. C for thicknesses $\frac{5}{8}$ in. and above. Since this is a very important structural part the allowable stresses used are the same as for the pressure parts.

The heavy base rings are also fabricated from A285 gr. C with yield strength $S_y = 30000$ psi. Since the allowable bending stress for structural steels with $S_y = 36000$ psi in the AISC base plate formula is 20000 psi, the allowable bending stress for A285C to be used in the AISC formula can be determined as follows:

$$S = 20000 \times 30/36 = 16700 \text{ psi} \quad \text{weight only}$$

$$S = 1.33 \times 16700 = 22200 \text{ psi} \quad \text{weight plus wind}$$

$$S = 1.2 \times 16700 = 20000 \text{ psi} \quad \text{test weight}$$

Higher allowable stresses are acceptable for the base ring than for the skirt shell because minor deformations of the base ring from overstressing would not cause any damage.

4.4. ANCHOR BOLTS

Self-supporting columns must be safely fixed to the supporting concrete foundations with adequately sized anchor bolts embedded in the concrete to prevent

would seem justified to size the weld to take the full design load

$$I_2 = (4M/\pi D_{sk}^2) + (W/\pi D_{sk}) \text{ lb/in. in.}$$

assuming that the skirt is not in contact with the base ring, since it could be difficult to estimate the approximate number of contact points. Then w , the size of the weld leg, is

$$w = I_2 / f_w \text{ in.,}$$

where the allowable weld unit force f_w (lb/in. in. of one-inch weld) is

$$f_w = 1.33 S_q \times 0.55 \quad \text{for wind or earthquake}$$

$$f_w = 1.20 S_q \times 0.55 \quad \text{for the test condition}$$

and S_q is the allowable stress for the skirt base plate or skirt shell plate, which ever is smaller.

Top Stiffening Ring. The continuous top ring with anchor bolt holes and welded to the skirt shell as shown in Fig. 4.3, Type A, helps to distribute the bolt holding down reactions more evenly into the skirt shell. To determine the required thickness t_r would require an involved stress analysis, including the base ring, vertical stiffeners, and skirt section. The thickness t_r could be computed from the concentrated load at the edges of the bolt holes. A rectangular plate with dimensions n and c can be roughly approximated by a beam with the longer ends fixed, load P on the plate, and minimum section modulus $Z = t_r^2 (n - f)/6$. Based on the above simplifying assumptions the thickness t_r can be estimated by

$$t_r = [Pc/4S_b(n - f)]^{1/2} \text{ in.,}$$

where P is the maximum bolt load (approximated by 1.25 times bolt stress area A_b times the bolt allowable stress S_a) and S_b is the allowable stress in bending for the top ring material.

Vertical Stiffeners. The vertical stiffeners in Fig. 4.3, Type A, are welded to the skirt and top and bottom base rings. The distance c between stiffeners is kept to the minimum that fabrication allows for the computed bolt size. On the leeward side the stiffeners are stressed in compression and their thickness can be computed as that of a plate supported on three sides with one side free. A simpler and more conservative approach is to treat the stiffener as a plate column. If the thickness t_V is assumed, then the maximum allowable unit stress

overturning or excessive swaying from lateral wind or earthquake loads. To compute the tension stress in the bolts and their required size and number one of three methods can be applied: (1) a simplified method, using generalized design conditions and ignoring dynamic effects and necessary preloading of bolts; (2) a more complete method, considering initial preload on bolts; and (3) disregarding initial preload on bolts.

Simplified Method

The forces and the moments acting on a tall slender vessel are shown in Fig. 4.4 as assumed for the anchor bolt analysis.

Assuming that the column will rotate about the axis y in Fig. 4.4, the maximum uplift force F per bolt due to the outside moment M is determined as follows. The maximum tension on the bolt circumference in lb. per lin. in. is given by

$$T = (M/Z_L) - (W/C) = (4M/\pi d^2) - (W/\pi d) \text{ lb/in.}$$

If $4M/\pi d^2$ is larger than $W/\pi d$ there is a positive uplift force inducing tension stress, with magnitude depending on the distance x spanned by half the anchor

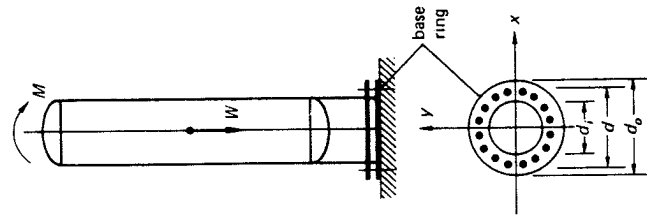


Fig. 4.4.

A_b = bolt tensile stress area, in.²

M = overturning moment at base due to wind or earthquake, lb-in.

W = weight of the vessel, operating (W_o) or erection (W_e), lb

d = bolt circle diameter, in.

d_o = outside diameter of the base ring, in.

d_i = inside diameter of the base ring, in.

N = total number of anchor bolts in multiples of 4

$Z_L = \pi d^2/4$, linear section modulus of the bolt circle, in.²

$C = \pi d$, circumference of the bolt circle, in.

x = distance of an anchor bolt from the neutral axis y , in.

S_a = allowable design stress for the anchor bolts, psi

bolts. The maximum force F on the bolt at distance $x = d/2$ from the axis y is

$$F = T\pi d/N = (4M/dN) - (W/N) \text{ lb/bolt}$$

and the required bolt area is

$$A_b = [(4M/d) - W]/NS_a.$$

The anchor bolts are not designed for the horizontal shear force since it is clearly counteracted by the friction between the base ring and the foundation.

Obviously, this approach is simplified, since it does not try to establish more accurately the actual design conditions, the initial bolt preload, and the dynamic effect of the external overturning moment. However, it is generally accepted for sizing the anchor bolts and, if a relatively conservative allowable stress S_a is used in the design, it gives acceptable results and is very simple to apply.

Initial Preload in Bolts Considered

Since wind and earthquake loads are essentially dynamic, initial tightening of the bolt nuts is required to reduce the variable stress range or any other impact effect on the nut under operating conditions.

The maximum force per bolt resulting from the combined action of the overturning moment M and the initial preload could be found by an approximate analysis with the following design conditions assumed, as follows.

1. The initial bolt preload together with the weight of the vessel is large enough to maintain a compressive pressure between the vessel base plate and the concrete pedestal under design loads.

2. As long as this compression exists at the contact area the skirt base and the pedestal behave as a continuous structure and the support base will rotate about the neutral axis of the contact area (axis y in Fig. 4.4).

Under moment M the maximum and the minimum pressure on the contact area becomes

$$S_c = (NF_i/A_c) + (W/A_c) \pm (Md_o/2I_c),$$

where

$$A_c = \pi(d_o^2 - d_i^2)/4$$

$$I_c = \pi(d_o^4 - d_i^4)/64$$