4

Design of Vessel Supports

Introduction: Support Structures

There are various methods that are used in the support structures of pressure vessels, as outlined below.

- Skirt Supports
  1. Cylindrical
  2. Conical
  3. Pedestal
  4. Shear ring
- Leg Supports
  1. Braced
     a. Cross braced (pinned and unpinned)
     b. Sway braced
  2. Unbraced
  3. Stub columns
- Saddle Supports
- Lug Supports
- Ring Supports
- Combination Supports
  1. Lugs and legs
  2. Rings and legs
  3. Skirt and legs
  4. Skirt and ring girder

Skirt Supports

One of the most common methods of supporting vertical pressure vessels is by means of a rolled cylindrical or conical shell called a skirt. The skirt can be either lap-, fillet-, or butt-welded directly to the vessel. This method of support is attractive from the designer's standpoint because it minimizes the local stresses at the point of attachment, and the direct load is uniformly distributed over the entire circumference. The use of conical skirts is more expensive from a fabrication standpoint, and unnecessary for most design situations.

The critical line in the skirt support is the weld attaching the vessel to the skirt. This weld, in addition to transmitting the overall weight and overturning moments, must also resist the thermal and bending stresses due to the temperature drop in the skirt. The thinner the skirt, the better it is able to adjust to temperature variations. A "hot box" design is used for elevated temperatures to minimize discontinuity stresses at the juncture by maintaining a uniform temperature in the region. In addition, skirts for elevated temperature design will normally be insulated inside and outside for several feet below the point of attachment.

There are various methods of making the attachment weld of the skirt to the shell. The preferred method is the one in which the center line of the shell and skirt coincide. This method will minimize stresses at the juncture. Probably the most common method, however, is to make the OD of the skirt match the OD of the shell. Other methods of attachment include lap-welding, pedestal type, or a shear ring arrangement. The joint efficiency of the attachment weld also varies by the method of attachment and is usually the governing factor in determining the skirt thickness. This weld may be subject to cracking in severe cyclic service.

Because the skirt is an attachment to the pressure vessel, the selection of material is not governed by the ASME Code. Any material selected, however, should be compatible with the vessel material in terms of weldability. Strength for design is also not specified for support material by the ASME Code. Usually, in the absence of any other standard, the rules of the AISC Steel Construction Manual will be utilized. Nonmandatory Appendix G in the ASME Code, Section VIII, Division 1 contains general guidelines on skirt supports (and other types of supports). Additionally, Part 4 in the ASME Code, Section VIII, Division 2 contains rules regarding applied forces, localized stresses, and thermal gradients for skirt supports for vessels designed to Division 2, but may be used for good practice of skirt supports for vessels designed to Division 1. For elevated temperature design of a vessel with a support skirt made of different materials, the upper portion of the skirt should be the same material of the shell, however, the upper portion should also extend below the hotbox. A thermal analysis should be performed to determine the temperature gradient along the length of the skirt and the location where another material may be used for the skirt support.

The most common governing conditions for determining the thickness of the skirt are as follows:

1. Weight + overturning moment
2. Imposed loads from anchor chairs
3. Vessel erection

Leg Supports

A wide variety of vessels, bins, tanks, and hoppers may be supported on legs. The designs can vary from small vessels supported on 3 or 4 legs, to very large vessels and spheres up to 80 feet in diameter, supported on 16 or 20 legs. Sometimes the legs are also called columns or posts.

Almost any number of legs can be used, but the most common variations are 3, 4, 6, 8, 12, 16, or 20. Legs should be equally spaced around the circumference.

Leg supports may be braced or unbraced. Braced legs are those which are reinforced with either cross-bracing or sway-bracing. Sway braces are the diagonal members which transfer the horizontal loads, but unlike cross braces, they operate in tension only. The diagonal members in a sway-braced system are called tie rods, which transfer the load to each adjacent panel. Turnbuckles may be used for adjustments of the tie rods.

Cross braces, on the other hand, are tension and compression members. Cross braces can be pinned at the center or unpinned, and transfer their loads to the legs via wing plates or can be welded directly to the legs.

Bracing is used to reduce the number or size of legs required by eliminating bending in the legs. The bracing will take the horizontal loads, thus reducing the size of the legs to those determined by compression or buckling. The additional fabrication costs of bracing may not warrant the savings in the size of the legs, however. Bracing may also cause some additional difficulties with the routing of any piping connected to nozzles on the bottom of the vessel.

Legs may be made out of pipe, channels, angles, rectangular tubing, or structural sections. Legs may be welded directly to the vessel shell or head or may be bolted or welded to clips which are directly attached to the shell. It is preferable if the centroid of the leg coincides with the center line of the vessel shell to minimize the eccentric action. However, this may be more expensive from a welding and fit up viewpoint due to the coping and contouring necessary to accomplish this.

Very large vessels and tanks may require a circumferential box girder, compression ring, or ring girder at or near the attachment point of the legs to distribute the large localized loads induced by the columns and bracing. These localized stresses at the attachment point should be analyzed for the eccentric action of the legs, overturning moments, torsion of the ring, as well as the loads from any bracing.

Whereas skirt-supported vessels are more common in refinery service, leg-supported vessels are more common in the chemical industry. This may be due in part to the ventilation benefits and the toxicity of the stored or processed chemicals. Legs should not be used to support vessels in high-vibration, shock, or cyclic service due to the high localized stresses at the attachments.

Legs are anchored to the foundations by base plates, which are held in place by anchor bolts embedded in the concrete. For large vessels in high seismic areas, a shear bar may be welded to the underside of the base plate which, in turn, fits into a corresponding recessed groove in the concrete.

Saddle Supports

Usually, horizontal pressure vessels and tanks are supported on two vertical cradles called saddles. The use of more than two saddles is unnecessary and should be avoided. Using more than two saddles is normally a stress-related issue, which can be solved in a more conventional manner. The reason for not using more than two saddles is that it creates an indeterminate structure, both theoretically and practically. With two saddles, there is a high tolerance for soil settlement with no change in shell stresses or loading. Even where soil settlement is not an issue, it is difficult to ensure that the load is uniformly distributed. Obviously there are ways to accomplish this, but the additional expense is often unwarranted. Vessels 40-50 ft in diameter and 150 ft long have been supported on two saddles.

A methodology for the determination of the stresses in the shell and heads of a horizontal vessel supported on saddles was first published in 1951 by L. P. Zick. This effort was a continuation of others' work, started as early as the 1930s. This procedure has been used, with certain refinements since that time, and is often called Zick's analysis, or the stresses are referred to as Zick's stresses.

Zick's analysis is based on the assumption that the supports are rigid and are not connected to the vessel shell. In reality, most vessels have flexible supports which are attached to the vessel, usually by welding. Whatever the reason, and there are a myriad of them, Zick's assumptions may yield an analysis that is not 100% accurate. These results should, however, be viewed more in terms of the performance they have demonstrated in the past 45 years, than in the exact analytical numbers they produce. As a strategy, the procedure is successful when utilized properly. There are other issues that also would have an effect on the outcome of the numerical answers such as the relative rigidity of the saddle—from infinitely rigid to flexible. The answers should be viewed in light of the assumptions as well as the necessity for 5-digit accuracy.

The ASME Code, Section VIII, Division 2 contains rules for determining the actual and allowable stresses for a vessel being supported by two saddles, with or without reinforcing plates, and with or without stiffening rings. These rules are based largely on Zick's analysis. However, as with all other types of supports, the ASME Code does not have specific design procedures for the design of saddles. Typically, the allowable stresses utilized are those as outlined in the *AISC Steel Construction Manual.*

The saddle itself has various parts: the web, base plate, ribs, and wear plate. The web can be on the center line of the saddle or offset. The design may have outer ribs only or inner ribs only, but usually it has both. For designs in seismic areas, the ribs perform the function of absorbing the longitudinal, horizontal loads. The saddle itself is normally bolted to a foundation via anchor bolts. The ASME Code does specify the minimum included arc angle (contact angle) of 120°. The maximum efficient saddle angle is 180°, since the weight and saddle splitting force go to zero above the belt line. In effect, taking into account the 6° allowed for reduction of stresses at the horn for wear plates, the maximum angle becomes 168°.

Saddles may be steel or concrete. They may be bolted, welded, or loose. For the loose type, some form of liner should be used between the vessel and the saddle. The typical loose saddle is the concrete type. Usually one end of the vessel is anchored and the other end sliding. The sliding end may have bronze, oiled, or Teflon slide plates to reduce the friction caused by the thermal expansion or contraction of the vessel.

Longitudinal location of the saddles also has a large effect on the magnitude of the stresses in the vessel shell as well as a bearing on the design of the saddle parts themselves. For large diameter, thin-walled vessels, the saddles are best placed within 0.5R of the tangent line to take advantage of the stiffening effect of the heads. Other vessels are best supported where the longitudinal bending at the midspan is approximately equal to the longitudinal bending at the saddles. However, the maximum distance is 0.2 L.

Lugs and Ring Supports

Lugs. Lugs offer one of the least expensive and most direct ways of supporting pressure vessels. They can readily absorb diametral expansion by sliding over greased or bronzed plates, are easily attached to the vessel by minimum amounts of welding, and are easily leveled in the field.

Since lugs are eccentric supports they induce compressive, tensile, and shear forces in the shell wall. The forces from the eccentric moments may cause high localized stresses that are combined with stresses from internal or external pressure. In thin-walled vessels, these high local loads have been known to physically deform the vessel wall considerably. Such deformations can cause angular rotation of the lugs, which in turn can cause angular rotations of the supporting steel.

Two or four lug systems are normally used; however, more may be used if the situation warrants it. There is a wide variety of types of lugs, and each one will cause different stress distributions in the shell. Either one or two gussets can be used, with or without a compression plate. If a compression plate is used, it should be designed to be stiff enough to transmit the load uniformly along the shell. The base plate of the lug can be attached to the shell wall or unattached. Reinforcing pads can be used to reduce the shell stresses. In some cases, the shell course to which the lugs are attached can be made thicker to reduce the local stress.

The method shown utilizes the local load analysis developed by Bijlaard in the 1950s, which was further refined and described in the WRC Bulletin 107. This procedure uses the principles of flexible load surfaces.

When making decisions regarding the design of lugs, a certain sequence of options should be followed. The following represents a ranking of these options based on the cost to fabricate the equipment:

1. 2 lugs, single gusset
2. 2 lugs, double gussets
3. 2 lugs with compression plate
4. Add reinforcing pads under (2) lugs
5. Increase size of (2) lugs
6. 4 lugs, single gusset
7. 4 lugs, double gussets
8. 4 lugs with compression plates
9. Add reinforcing pads under (4) lugs
10. Increase size of (4) lugs
11. Add ring supports

Ring Supports. In reality, ring supports are used when the local stresses at the lugs become excessively high. As can be seen from the previous list, the option to go to complete, 360-degree stiffening rings would, in most cases, be the most expensive option. Typically, vessels supported by rings or lugs are contained within a structure rather than supported at grade and as such would be subject to the seismic movement of which they are a part.

Vessels supported on rings should only be considered for lower or intermediate temperatures, say below 400 or 500 degrees. Using ring supports at higher temperatures could cause extremely large discontinuity stresses in the shell immediately adjacent to the ring due to the differences in expansion between the ring and the shell. For elevated temperature design, rings may still be used, but should not be directly attached to the shell wall. A totally loose ring system can be fabricated and held in place with shear bars. With this system there is no interaction between the shell and the support rings.

The analysis for the design of the rings and the stresses induced in the shell employs the same principles as Lug Method 1, ring analysis. The eccentric load points are translated into radial loads in the rings by the gussets. The composite ring section comprised of the shell and ring is then analyzed for the various loads.

Procedure 4-1: Wind Design Per ASCE [1]

Notation

A_f = projected area, ft^2 (m^2)
$\bar{b}$ = mean hourly wind speed factor
C_f = force coefficient, shape factor 0.7, 0.8, and 0.9 for h/D_e of 1, 7, and 25, respectively (linear interpolation is permitted). See ASCE/SEI 7-10.
c = turbulence intensity factor
D_e = vessel effective diameter, from Table 4-4
F = design wind force
$q_z G C_f A_f (lb)(N)$
F_i = design wind force of section under consideration, i = 1 to n, lb (N)
g_Q = peak factor for background response, use 3.4
g_R = peak factor for resonant response
g_v = peak factor for wind response, use 3.4
G = gust effect factor
G_f = gust response factor for flexible vessels
H_i = height from base of vessel to center of section under consideration, i = 1 to n, ft (m)
h = height of vessel, ft (m)
h_i = length of section under consideration, i = 1 to n, ft (m)
$I_{\bar{z}}$ = the intensity of turbulence at height z
K_d = wind directionality factor, use 0.95 for vessels when using ASCE/SEI 7-10 load combinations
K_z = velocity pressure exposure coefficient from Table 4-3a
K_{zt} = topographic factor, use 1.0 unless vessel is located near or on isolated hills. See ASCE/SEI 7-10 for specific requirements
L_z = integral length scale of turbulence, ft (m)
ℓ = integral length scale factor, ft (m)
M = overturning moment at base, ft-lb (N-m)
M_i = moment at base of section under consideration, i = 1 to n, ft-lb (N-m)
N_1 = reduced frequency
n_1 = fundamental natural frequency, Hz
Q = background response factor
q_z = velocity pressure at height z above the ground
$0.00256\ K_z K_{zt} K_d V^2 (lb/ft^2)$ or
$0.613\ K_z K_{zt} K_d V^2 (N/m^2)$
R = resonant response factor

R_B, R_h, R_L, R_n = calculation factors
T = period of vibration, sec
V = basic wind speed from map, Figures 4-1a, 4-1b, and 4-1c, mph (m/s)
V_i = shear force at base of section under consideration, i = 1 to n, lb (N)
$\overline{V}_{\overline{z}}$ = mean hourly wind speed at height $\overline{z}$, ft/sec (m/s)
W = weight of vessel, lb (N)
z = height above ground level, ft (m)
$\overline{z}$ = equivalent height of vessel, ft (m)
z_{min} = minimum design height, ft (m), from Table 4-3
$\overline{\alpha}$ = mean hourly wind-speed power law exponent
β = damping ratio (structural), percent of critical from Table 4-3
bedrock, endbearing piles, or other rigid bases 0.2%
friction piles or mat foundations on soil 0.4%
$\overline{\epsilon}$ = integral length scale power law exponent
η_B, η_h, η_L = calculation factors

The ASME Code does not give specific procedures for designing vessels for wind. However, Para. UG-22, "Loadings," does list wind as one of the loadings that must be considered. In addition, local, state, or other governmental jurisdictions will require some form of analysis to account for wind loadings. Client specifications and standards also frequently require consideration of wind. There is one nationally recognized standard that is most frequently used for wind design.

ASCE/SEI 7-10

This section outlines the wind design procedures for this standard. Wind design is used to determine the forces and moments at each elevation to check if the calculated shell thicknesses are adequate. The overturning moment at the base is used to determine all of the anchorage and support details. These details include the number and size of anchor bolts, thickness of skirt, size of legs, and thickness of base plates.

As a loading, wind differs from seismic in that it is more or less constant; whereas, seismic is of relatively short duration. In addition, the wind pressure varies with the height of the vessel. A vessel must be designed for the worst case of wind or seismic, but need not be designed for both simultaneously. While typically the worst case for seismic design is with the vessel full (maximum weight), the worst design case for wind is with the vessel empty. This will produce the maximum uplift due to the minimum restraining weight.

The wind forces are obtained by multiplying the projected area of each element, within each height zone by the basic wind pressure for that height zone and by the shape factor for that element. The total force on the vessel is the sum of the forces on all of the elements. The forces are applied at the centroid of the projected area.

Tall towers or columns should be checked for dynamic response. If the vessel is above the critical line in Figure 4-7, R_m/t ratio is above 200 or the h/D ratio is above 15, then dynamic stability should be investigated. This section does not consider aerodynamic damping effects, however it is possible that the aerodynamic damping contribution is negative under certain conditions. If this is the case, the overall effect of the structural damping would be reduced. See Procedure 6-5, "Vibration of Tall Towers and Stacks," for additional information.

Design Procedure

Risk category	=	______
Basic wind speed, V	=	______
Exposure category	=	______
Effective diameter, D_e	=	______
Height of vessel, h	=	______
Shape factor, C_f	=	______
Fundamental frequency, f n_1	=	______
Damping ratio, structural, β	=	______

Step 1: Give or determine the following:

Step 2: Determine if vessel is rigid or flexible.

a. If $n_1 \geq 1$ Hz, then vessel is considered rigid and:

$$F = q_z G_f C_f A_f$$

b. If $n_1 < 1$ Hz, then vessel is considered flexible and:

$$F = q_z G_f C_f A_f$$

Step 3: Calculate shear and moments at each elevation by multiplying force, F, and elevation, H_n, the distance to the center of the projected area.

Step 4: Sum the forces and moments at each elevation down to the base.

Determination of Gust Factor, G, for Vessels Where $n_1 \geq 1$ Hz

For rigid structures, the gust factor may be taken as 0.85 or as calculated below:

Given: D_e = ____________ (effective diameter)
h = ____________ (overall height)
g_Q, g_v = 3.4

Determine the following values from Table 4-3:
Calculate:

$\bar{\alpha}$ =	____	ℓ =	____
$\bar{b}$ =	____	$\bar{\epsilon}$ =	____
c =	____	z_{min} =	____

$$\bar{z} = \max(0.6h, z_{min})$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}, \quad I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6} \text{(SI)}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}, \quad L_{\bar{z}} = \ell\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} \text{(SI)}$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{D_e + h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

Determination of Gust Factor, G_f, for Vessels Where $n_1 < 1$ Hz

In addition to the tabular data above, the following must be given to determine the gust factor, G_f:

Given: n_1 = ____________ (fundamental natural frequency)
V = ____________ (basic wind speed)
β = ____________ (damping ratio, structural)

Calculate ($\bar{z}, I_{\bar{z}}, L_{\bar{z}}, Q$ are as determined above):

$$g_R = \sqrt{2\ln(3,600 n_1)} + \frac{0.577}{\sqrt{2\ln(3,600 n_1)}}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V, \quad \bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V \text{(SI)}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$\eta_h = 4.6 n_1 h / \bar{V}_{\bar{z}}$$

$$\eta_B = 4.6 n_1 D_e / \bar{V}_{\bar{z}}$$

$$\eta_L = 15.4 n_1 D_e / \bar{V}_{\bar{z}}$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}\left(1 - e^{-2\eta_h}\right) \text{ for } \eta > 0,$$

$$R_h = 1 \text{ for } \eta = 0$$

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2}\left(1 - e^{-2\eta_B}\right) \text{ for } \eta > 0,$$

$$R_B = 1 \text{ for } \eta = 0$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}\left(1 - e^{-2\eta_L}\right) \text{ for } \eta > 0,$$

$$R_L = 1 \text{ for } \eta = 0$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

Sample Problem

Vertical vessel on skirt:

Risk category	=	III
Basic wind speed, V	=	115 mph
Exposure category	=	C
Effective diameter, D_e	=	8 ft
Height of vessel, h	=	200 ft
Shape factor, C_f	=	0.9
Fundamental frequency	=	0.57 Hz
Damping ratio (structural)	=	0.01
Empty weight, W	=	100 kips

Values from Table 4-3:

$\bar{\alpha}$	=	1/6.5	ℓ	=	500
$\bar{b}$	=	0.65	$\bar{\epsilon}$	=	1/5.0
c	=	0.20	z_{min}	=	15

Calculate:

$$\bar{z} = \max(0.6h, z_{min}) = \max(0.6\,(200), 15) = 120 \text{ ft}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.20\left(\frac{33}{120}\right)^{1/6} = 0.161$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{120}{33}\right)^{1/5.0} = 647 \text{ ft}$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{D_e + h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1 + 0.63\left(\frac{8 + 200}{647}\right)^{0.63}}} = 0.874$$

$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\,\ln(3,600n_1)} + \frac{0.577}{\sqrt{2\,\ln(3,600n_1)}}$$

$$= \sqrt{2\,\ln(3,600(0.57))} + \frac{0.577}{\sqrt{2\,\ln(3,600(0.57))}}$$

$$= 4.05$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = (0.65)\left(\frac{120}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)(115)$$

$$= 134 \text{ ft/sec}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.57)(647)}{(134)} = 2.76$$

$$\eta_h = 4.6n_1h/\bar{V}_{\bar{z}} = 4.6(0.57)(200)/(134) = 3.92$$

$$\eta_B = 4.6n_1D_e/\bar{V}_{\bar{z}} = 4.6(0.57)(8)/(134) = 0.157$$

$$\eta_L = 15.4n_1D_e/\bar{V}_{\bar{z}} = 15.4(0.57)(8)/(134) = 0.525$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}\left(1 - e^{-2\eta_h}\right)$$

$$= \frac{1}{(3.92)} - \frac{1}{2(3.92)^2}\left(1 - e^{-2\,(3.92)}\right) = 0.222$$

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2}\left(1 - e^{-2\eta_B}\right)$$

$$= \frac{1}{(0.157)} - \frac{1}{2(0.157)^2}\left(1 - e^{-2\,(0.157)}\right) = 0.903$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}\left(1 - e^{-2\eta_L}\right)$$

$$= \frac{1}{(0.525)} - \frac{1}{2(0.525)^2}\left(1 - e^{-2\,(0.525)}\right) = 0.725$$

$$R_n = \frac{7.47\,N_1}{(1 + 10.3N_1)^{5/3}} = \frac{7.47\,(2.76)}{(1 + 10.3(2.76))^{5/3}} = 0.074$$

$$R = \sqrt{\frac{1}{\beta}R_nR_hR_B(0.53 + 0.47R_L)}$$

$$= \sqrt{\frac{1}{(0.01)}(0.074)(0.222)(0.903)(0.53 + 0.47(0.725))}$$

$$= 1.13$$

$$G_f = 0.925\left(\frac{1 + 1.7I_{\bar{z}}\sqrt{g_Q^2Q^2 + g_R^2R^2}}{1 + 1.7g_vI_{\bar{z}}}\right)$$

$$= 0.925\left(\frac{1 + 1.7(0.161)\sqrt{(3.4)^2(0.874)^2 + (4.05)^2(1.134)^2}}{1 + 1.7(3.4)(0.161)}\right)$$

$$= 1.20$$

$$F = q_zG_fC_fA_f = 32.163K_z(0.9)A_f = 28.947A_fK_z$$

where $q_z = 0.00256K_zK_{zt}K_dV^2 = 32.163K_z$, A_f is calculated using the section length and the effective diameter, and K_z is determined using the elevations at the top of each section.

Determine Wind Force on Vessel

Elevation	q_z	G_f	C_f	h_i	A_f	Force on A_f, F_i	ΣShear, V_i	ΣMoment, M_i
190–200 ft	47.1 psf	1.20	0.90	10 ft	80 ft^2	4,061 lb	4,061 lb	20,303 ft-lb
170–190 ft	46.6 psf	1.20	0.90	20 ft	160 ft^2	8,034 lb	12,095 lb	181,855 ft-lb
150–170 ft	45.5 psf	1.20	0.90	20 ft	160 ft^2	7,848 lb	19,943 lb	502,228 ft-lb
130–150 ft	44.3 psf	1.20	0.90	20 ft	160 ft^2	7,644 lb	27,587 lb	977,520 ft-lb
110–130 ft	43.0 psf	1.20	0.90	20 ft	160 ft^2	7,417 lb	35,004 lb	1,603,423 ft-lb
95–110 ft	41.5 psf	1.20	0.90	15 ft	120 ft^2	5,371 lb	40,374 lb	2,168,759 ft-lb
85–95 ft	40.3 psf	1.20	0.90	10 ft	80 ft^2	3,472 lb	43,846 lb	2,589,860 ft-lb
75–85 ft	39.3 psf	1.20	0.90	10 ft	80 ft^2	3,391 lb	47,237 lb	3,045,274 ft-lb
65–75 ft	38.3 psf	1.20	0.90	10 ft	80 ft^2	3,303 lb	50,540 lb	3,534,161 ft-lb
55–65 ft	37.2 psf	1.20	0.90	10 ft	80 ft^2	3,205 lb	53,745 lb	4,055,587 ft-lb
45–55 ft	35.9 psf	1.20	0.90	10 ft	80 ft^2	3,094 lb	56,839 lb	4,608,510 ft-lb
35–45 ft	34.4 psf	1.20	0.90	10 ft	80 ft^2	2,966 lb	59,806 lb	5,191,735 ft-lb
27.5–35 ft	32.6 psf	1.20	0.90	7.5 ft	60 ft^2	2,110 lb	61,916 lb	5,648,191 ft-lb
22.5–27.5 ft	31.0 psf	1.20	0.90	5 ft	40 ft^2	1,337 lb	63,253 lb	5,961,112 ft-lb
17.5–22.5 ft	29.7 psf	1.20	0.90	5 ft	40 ft^2	1,282 lb	64,535 lb	6,280,580 ft-lb
0–17.5 ft	28.2 psf	1.20	0.90	17.5 ft	140 ft^2	4,255 lb	68,789 lb	7,447,165 ft-lb

Exposure Categories

The following ground roughness exposure categories are considered and defined in ASCE/SEI 7-10 Section 26.7.3:

- *Exposure B:* For buildings with a mean roof height of less than or equal to 30 ft (9.1 m). Urban and suburban areas, towns, city out skirts, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single family dwellings or larger.
- *Exposure C:* For cases where Exposures B and D do not apply. Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m).
- *Exposure D:* Flat, unobstructed coastal areas directly exposed to wind blowing over open water.

Notes

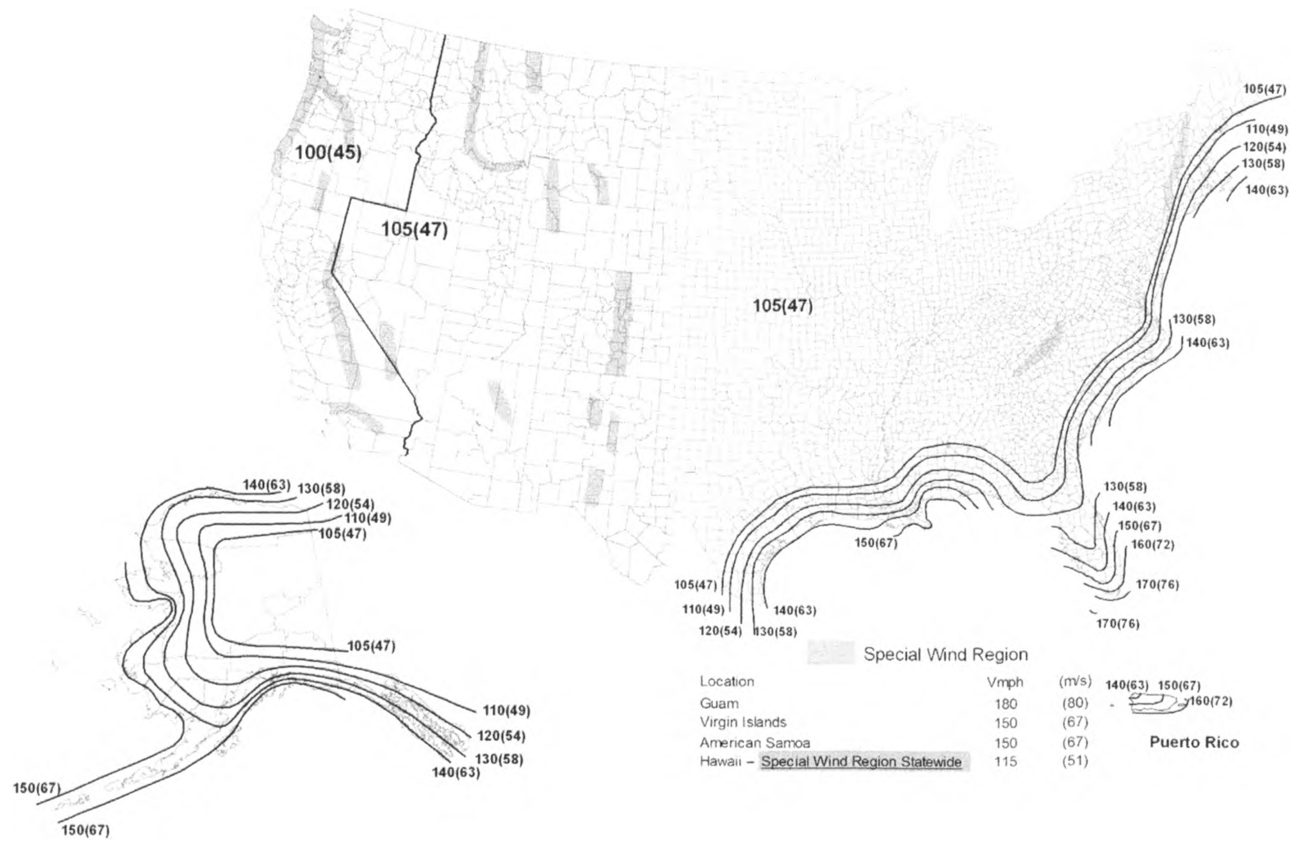
1. Most vessels will be classified as Category III.
2. The basic wind speeds on the map, Figures 4-1a, 4-1b, and 4-1c, correspond to a 3-sec. gust speed at 33 ft above the ground, in Exposure Category C with a 7% / 3% / 15% probability of exceedance in 50 years.
3. The constant, 0.00256 (0.613), reflects the mass density of air for the standard atmosphere (59°F (15 °C) at sea level pressure, 29.92 in. of mercury (101.325 kPa)). The constant is calculated by ½ ρ_{air}/g, where ρ_{air} is the density of air and g is the acceleration due to gravity. The mass density of the air will vary as function of altitude, latitude, temperature, weather, or season. This constant may be varied to suit the actual conditions if they are known with certainty. See ASCE/SEI 7-10.
4. Short, vertical vessels, vessels in structures, or horizontal vessels where the height is divided between two pressure zones may be more conveniently designed by applying the higher pressure uniformly over the entire vessel.
5. Deflection due to wind should be limited to 6 in. per 100 ft of elevation.

For 15 ft. $\le z \le z_g$

$$K_z = 2.01\left(z/z_g\right)^{2/\alpha}$$

For z < 15 ft.

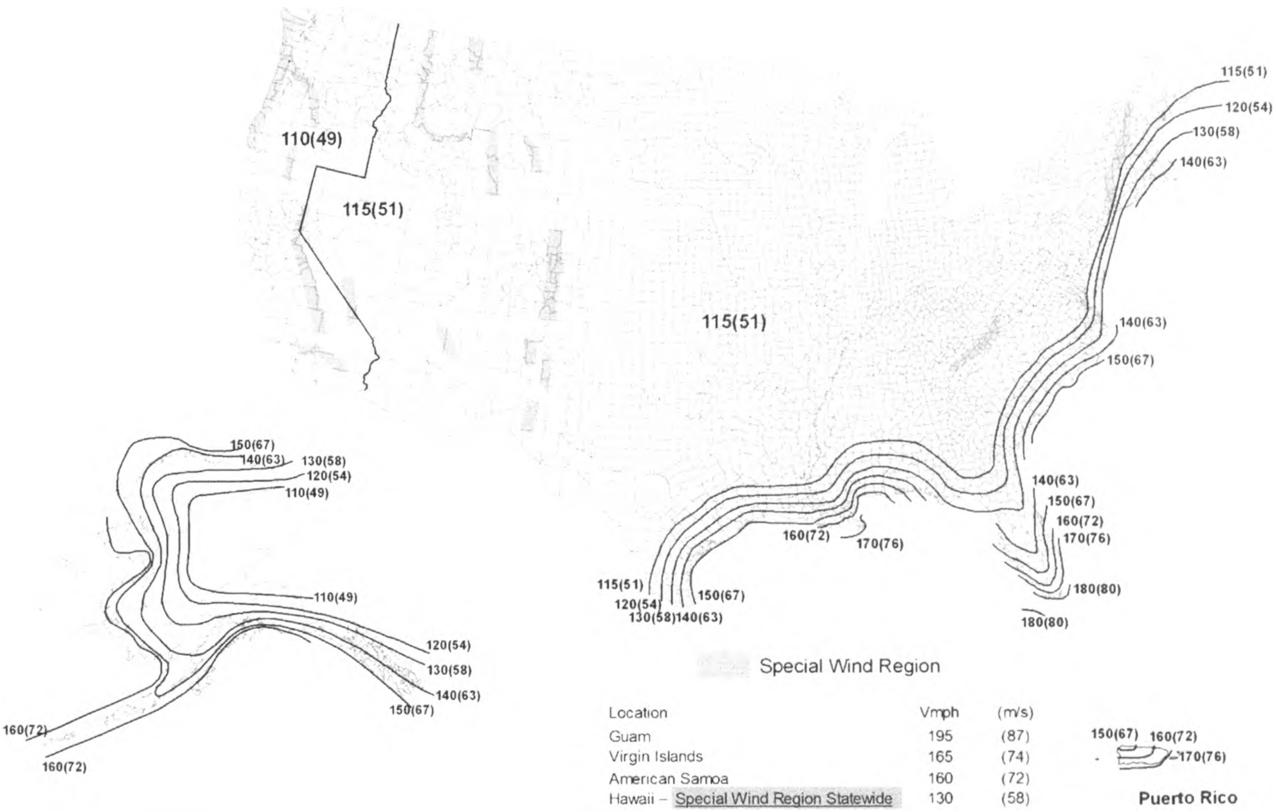
$$K_z = 2.01\left(15/Z_g\right)^{2/\alpha}$$



Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

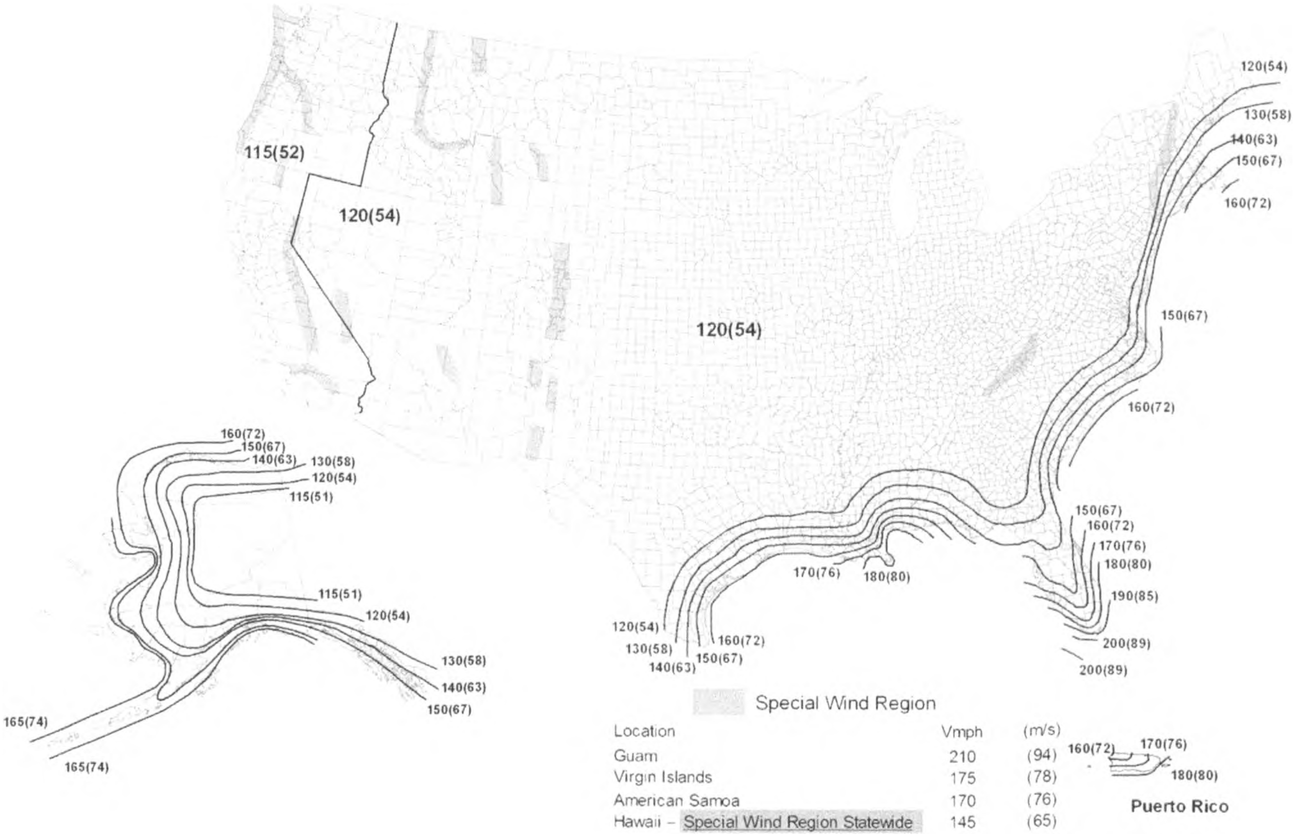
Figure 4-1a. Basic Wind Speeds for Occupancy Category I Buildings and Other Structures. *With permission from ASCE.*



Notes:

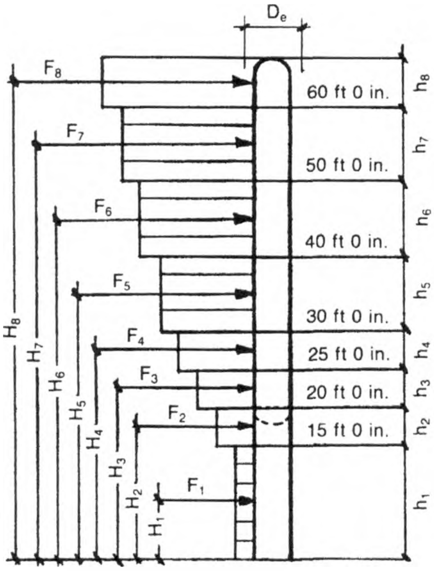
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

Figure 4-1b. Basic Wind Speeds for Occupancy Category II Buildings and Other Structures. *With permission from ASCE.*



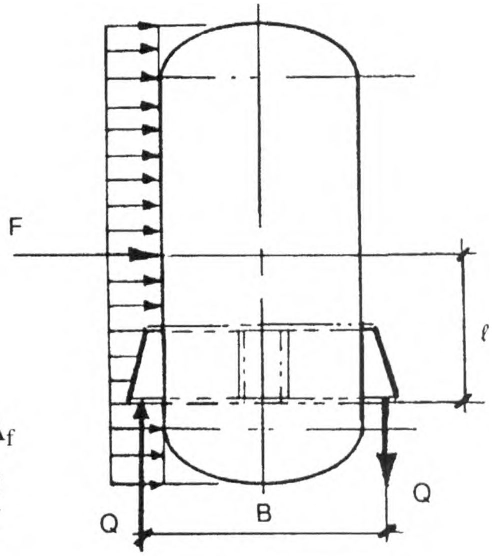
Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).

Figure 4-1c. Basic Wind Speeds for Occupancy Category III and IV Buildings and Other Structures. *With permission from ASCE.*



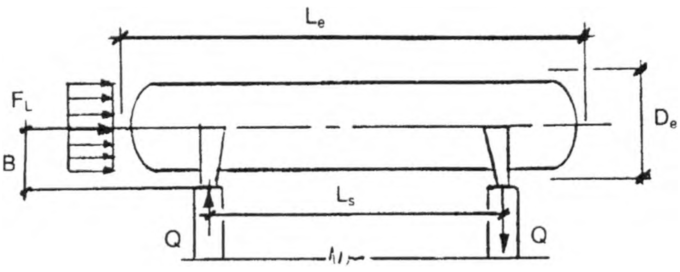
$$A_f = h_n D_e$$
$$F_i = q_z G C_f A_f$$
$$M_i = F_i(h_i/2) + V_{i+1}(h_i) + M_{i+1}$$
$$M = \Sigma F_i H_i$$

Figure 4-2. Vertical vessels.



$$A_f = L_e D_e$$
$$F = q_z G C_f A_f$$
$$Q = \frac{W}{N} \pm \frac{F\ell}{B}$$

Figure 4-4. Vessels on lugs or rings.



Longitudinal

$$A_L = \frac{\pi D_e^2}{4}$$
$$F_L = q_z G C_f A_L$$
$$Q = \frac{W}{2} \pm \frac{F_L B}{L_s}$$

Transverse

$$A_t = L_e D_e$$
$$F_t = 0.5(q_z G C_f A_t)\text{(one support)}$$
$$M_t = F_t B\text{(one support)}$$

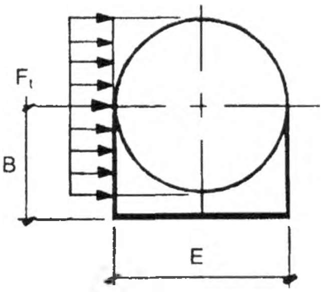
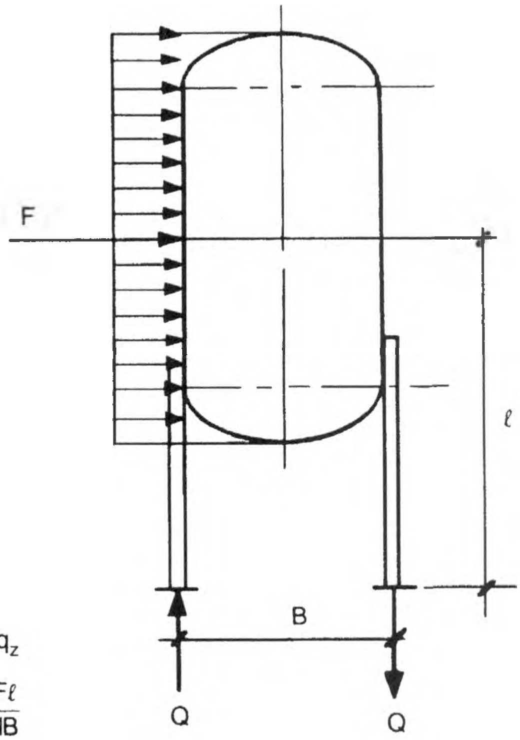


Figure 4-3. Horizontal vessels.



$$A_f = L_e D_e$$
$$F = A_f C_f G q_z$$
$$Q = \frac{W}{N} \pm \frac{4F\ell}{NB}$$

Figure 4-5. Vessels on legs.

Table 4-1
Importance Factor (Wind Loads)

Structure Category	I
I	0.87
II	1.00
III	1.15
IV	1.15

Table 4-2
Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Buildings and structures that represent a low risk to human life in the event of failure.	Category I
All buildings and other structures not covered by Risk Categories I, III, and IV.	Category II
Buildings and other structures containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Typically, the equipment inside of a refinery falls under this category.	Category III
Schools, non-emergency health care facilities, jails, non-essential power stations	Category III
Essential facilities	Category IV
Buildings and other structures containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. (Buildings and other structures containing these substances may be eligible to be classified in a lower category if it can be demonstrated to the jurisdictional authority through a special assessment that the lower risk category is acceptable).	Category IV

With permission from ASCE

Table 4-3
Miscellaneous coefficients

Expos.	α	z_g (ft/m)	$\bar{\alpha}$	$\bar{b}$	$\bar{c}$	ℓ (ft/m)	$\bar{\epsilon}$	*z_{min} (ft/m)
B	7.0	1200/365.76	1/4.0	0.45	0.30	320/97.54	1/3.0	30/9.14
C	9.5	900/274.32	1/6.5	0.65	0.20	500/152.4	1/5.0	15/4.57
D	11.5	700/213.36	1/9.0	0.80	0.15	650/198.12	1/8.0	7/2.13

z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of 0.6 h or z_{min}.

Table 4-3a*
Velocity pressure exposure coefficients, K_z

Height above ground level, z		Exposure Categories		
ft	(m)	B	C	D
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18.0)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Note: Linear interpolation for intermediate values of height z is acceptable.
K_z may be determined from the following formula:

Table 4-4
Effective diameter, D_e*

D (Vessel Diameter + 2 x Insulation Thickness)	Piping with or Without Ladders	Attached Piping, Ladders, and Platforms
≤ 4ft – 0 in.	$D_e = 1.6D$	$D_e = 2.0D$
4ft - 0 in. – 8 ft – 0 in.	$D_e = 1.4D$	$D_e = 1.6D$
> 8 ft – 0 in.	$D_e = 1.2D$	$D_e = 1.4D$

**Suggested only; not from ASCE.*

Procedure 4-2: Seismic Design – General

Pressure vessels and their supports must be designed to resist the forces and loadings anticipated during a seismic event ... an earthquake. The seismic design is not defined by the ASME Code but by building codes (previously NBC, SBC, and UBC, but now IBC) that reference technical standards such as ASCE/SEI 7, ACI 318, and AISC 360. Many countries have their own seismic standards and there are international standards as well. The ASME Code states in UG-22 that the vessel and support structure must be designed to withstand the forces from a seismic event.

A seismic event causes the vessel to sway as a result of the ground motion. How much loading the vessel experiences is dependent on the type of foundation and supports, the size and proportions of the vessel, the geographic location of the vessel, and the type of soil. A tall, thin, slender cylindrical tower mounted at grade, is relatively flexible and will therefore have a long period and low frequency. By contrast a short, squat vessel will have a short period and higher frequency. Vessels mounted in or on structures will be influenced by the relative stiffness of the structure.

Seismic standards are all based on the geographical and statistical data for a given region. The standards use various criteria to estimate the loads on the vessel or structure and probability of occurrence. Some regions have high probability for very strong earthquakes to occur. Other regions are almost negligible in terms of seismic events. Seismic codes and standards date back to the 1920's. Modern, industrial societies have rigorous building codes that account for earthquakes. The building codes may not define all types of procedures but they allow for the various design procedures.

Additionally, the site class has an effect on the design loadings. In general, site classes comprised of hard rock will have less intense shaking than site classes composed of soft soils.

Seismic design procedures can be accomplished for most vessels by one of the two methods as follows;

a. Equivalent Lateral Force (ELF) – Static Analysis
b. Modal Response Spectrum Analysis – Dynamic Analysis

Seismic design criteria may provide linear and nonlinear seismic response history procedures, but these are not as commonly used for vessels.

Equivalent Lateral Force

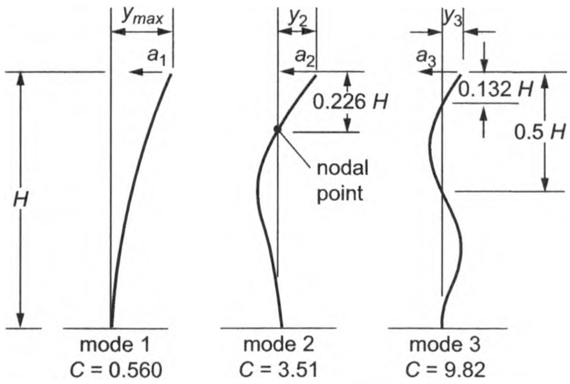
The ELF approximates the effect that the ground displacements would have on the structure by applying an equivalent force to the structure itself. A seismic event is a time-dependent phenomena whereby the loading is not applied instantaneously, but over a period of time. However, the ELF assumes that the entire earthquake force is applied instantaneously. The ELF is conservative and has served industry and society well for many years.

The procedure takes the total base shear and distributes it along the length of the column. Once the vertical distribution of the lateral seismic force is determined, the shear force and bending moment at each plane and the sum at the base of the column can be determined. The vertical component of the seismic design loads can be either added or subtracted to create the most stringent condition. These loads used with the corresponding load combinations are used to design all support components.

Modal Response Spectrum Analysis

The Modal Response Spectrum Analysis (also known as a dynamic analysis) more accurately depicts the response of the structure to the earthquake. This is done by considering the response of multiple modes instead of just the first one (as is done in the ELF). Whereas the use of a static analysis assumes that a load is applied relatively slowly, a dynamic analysis should be used if the application of the load is faster than the response of the structure. For this reason a dynamic analysis is mainly used for vertical vessels which are basically a cantilevered cylinder. A dynamic analysis frequently results in lower overturning moments than the ELF. Lower moments in turn translate into reduced thickness for skirt and base plate and fewer anchor bolts. For this reason the question is asked whether a dynamic analysis is less conservative than a static analysis, however the dynamic analysis is a more accurate representation of the way the structure responds to the earthquake-induced ground motion.

For rigid vessels, the first few modes may represent the majority of the modal mass participation, whereas for flexible vessels, the number of modes may be 20. It is for this reason that dynamic analyses lend themselves to computerized models. Many seismic design standards indicate that the number of modes to be included must have a combined mass participation of at least 90%.



Natural Frequency $f = \frac{1}{T} = C\left[\frac{gEI}{wH^4}\right]^x$

where w is the uniform weight of beam per unit length

Allowable Stresses and Load Combinations

Unlike wind, seismic events are short term loading conditions. As a result, the ASME Code Section VIII, Division 1 allows for an increase in the allowable stress of 1.2. Section VIII, Division 2, building codes, and design standards (such as ASCE/SEI 7) use load combinations and typically do not allow for an increase in allowable stress, however the seismic load is usually reduced when combined with other types of loads and so the effect is similar. The vessel may only experience an earthquake several times during the life of the equipment, though the vessel must be designed to withstand any seismic event.

Designing a vessel to be invulnerable to any earthquake would be both impractical and uneconomical. Building codes and design standards use the ability of the structure to yield and absorb energy in a ductile manner during a seismic event for design. This is part of the basis of the 'R' factor, which is used to reduce the design strength for a structure. As a result of the designed structure undergoing permanent deformation during an earthquake, some of the structure may be lightly or severely damaged. In the case of vessel support design, it should be understood that the anchor bolts provide a benefit to the vessel by yielding and absorbing energy that could otherwise have a greater impact on the support members.

Period of Vibration

Vessels will vibrate based on an exciting force such as wind or earthquake. There are two distinct types of loadings as a result of wind. The first is the static force from wind loading pressure against the vessel shell. The second is a dynamic effect from vortex shedding due to wind flow around the vessel. Tall, slender, vertical vessels are more susceptible to the effects of vortex shedding.

Vessels subject to an external force or ground motion will deflect to a specific shape and then return to its original position once the applied force is dissipated or removed. The extent of deflection is proportional to the applied force. The vessel, or its support, will act as a spring. In the passage to equilibrium, the vessel will vibrate freely, through its various modes. The period of vibration (POV) is the time it takes the vessel to deflect through one mode and return to its original position and is measured in seconds. The frequency, which is the inverse of POV, is the number of cycles per second.

The POV of a pressure vessel is a function of the vessel weight, diameter, height, weight distribution, temperature, flexibility, type of support, damping mechanisms and location if supported in a structure. Typically when we are discussing the period of vibration for a vessel we are talking about the "first" period of vibration, or the first "natural" or "fundamental" period of vibration.

All vibrating systems, of which vessels are included, have multiple modes of vibration, known as the first mode, the second mode, etc. Each individual mode will have its own unique characteristics for that particular system. The deflected shape of a vessel for any single mode of vibration is always the same for that vessel, regardless of the magnitude. In other words, though the amplitude of displacement changes with time, the relation between displacements throughout the height remains constant.

The mode with the lowest frequency (longest period) is called the first, or fundamental mode. The mode with the higher frequencies (shorter periods) are called the higher modes. Each mode would have a different POV and frequency.

The period of vibration is the inverse of the frequency of vibration. Typically the symbol for POV is T and is given in seconds. The symbol for frequency is f, and is given in hertz, which is cycles per second. $T = 1/f$ and $f = 1/T$.

Generally, vessels with a POV less than 0.30 seconds ($f \geq 3.33$ Hz) are considered rigid. Vessels with a POV between 0.30 and 0.75 seconds (1.33 Hz $< f <$ 3.33 Hz) are semirigid. Between 0.75 and 1.25 seconds (0.8 Hz $<$ $f <$ 1.33 Hz) are semiflexible and vessels with a POV greater than 1.25 seconds (0.80 Hz) are flexible.

A vessel will have a different POV in the empty and full condition. It will have a different POV for the new and corroded condition. It will have a different POV for hot and cold conditions due to the modulus of elasticity of the steel at temperature. Vertical vessels on legs and skirts are the most flexible. Vessels on lugs and rings are normally supported in structures and therefore would be subject to the harmonics of the structure itself. Horizontal vessels vibrate with their supports as well and are dependent on pier deflection.

A vertical vessel is modeled as a cantilever beam whereas a horizontal vessel is modeled as a simply supported beam. A cantilever is a much more prone to vibration and deflection then a simply supported beam, therefore the POV is typically much higher. Guiding a vessel supported in a structure will greatly alter its POV because it changes the mode of vibration.

Wind and seismic design standards such as ASCE have base shear factors that are a function of the POV. This makes sense because the response of the vessel is

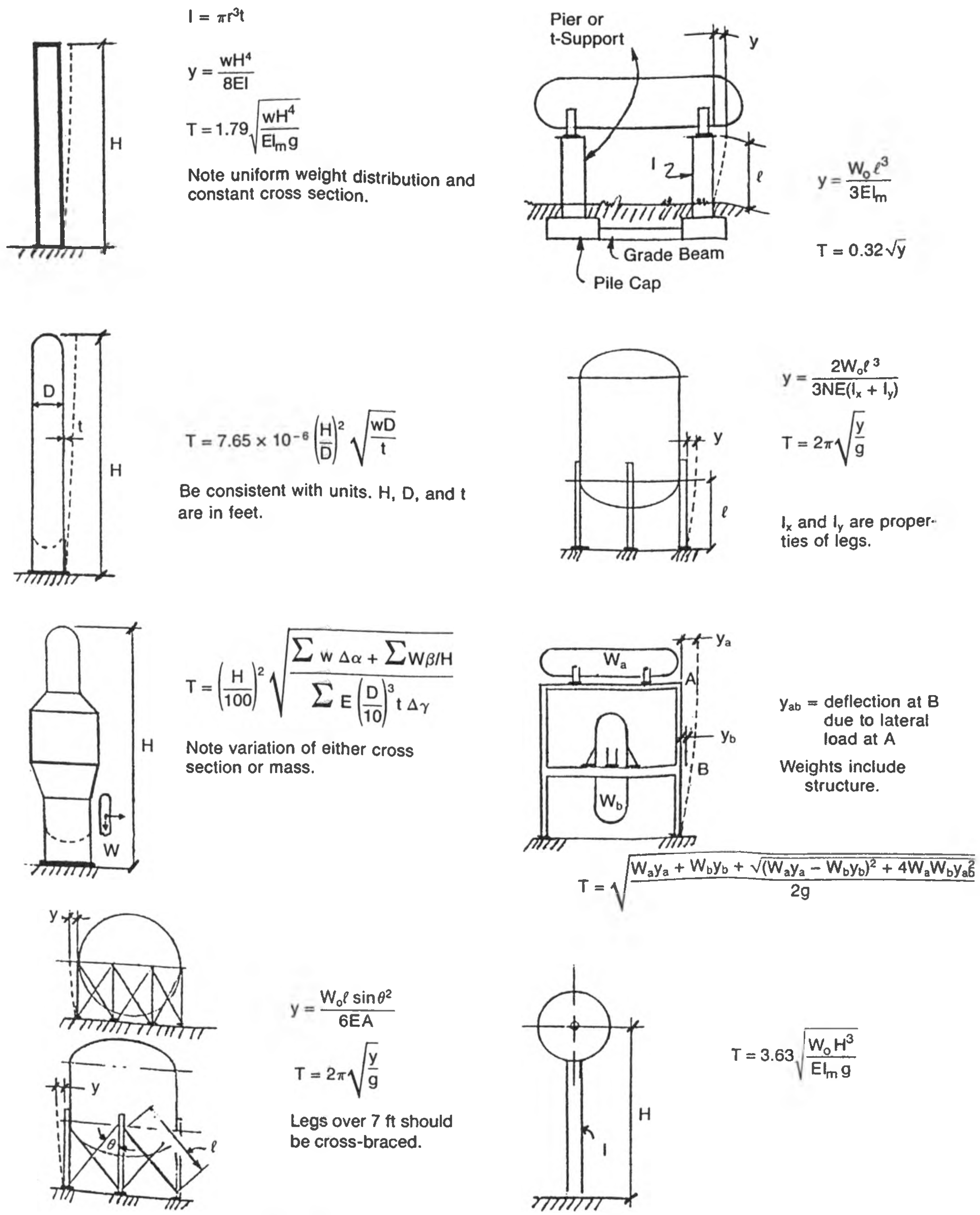
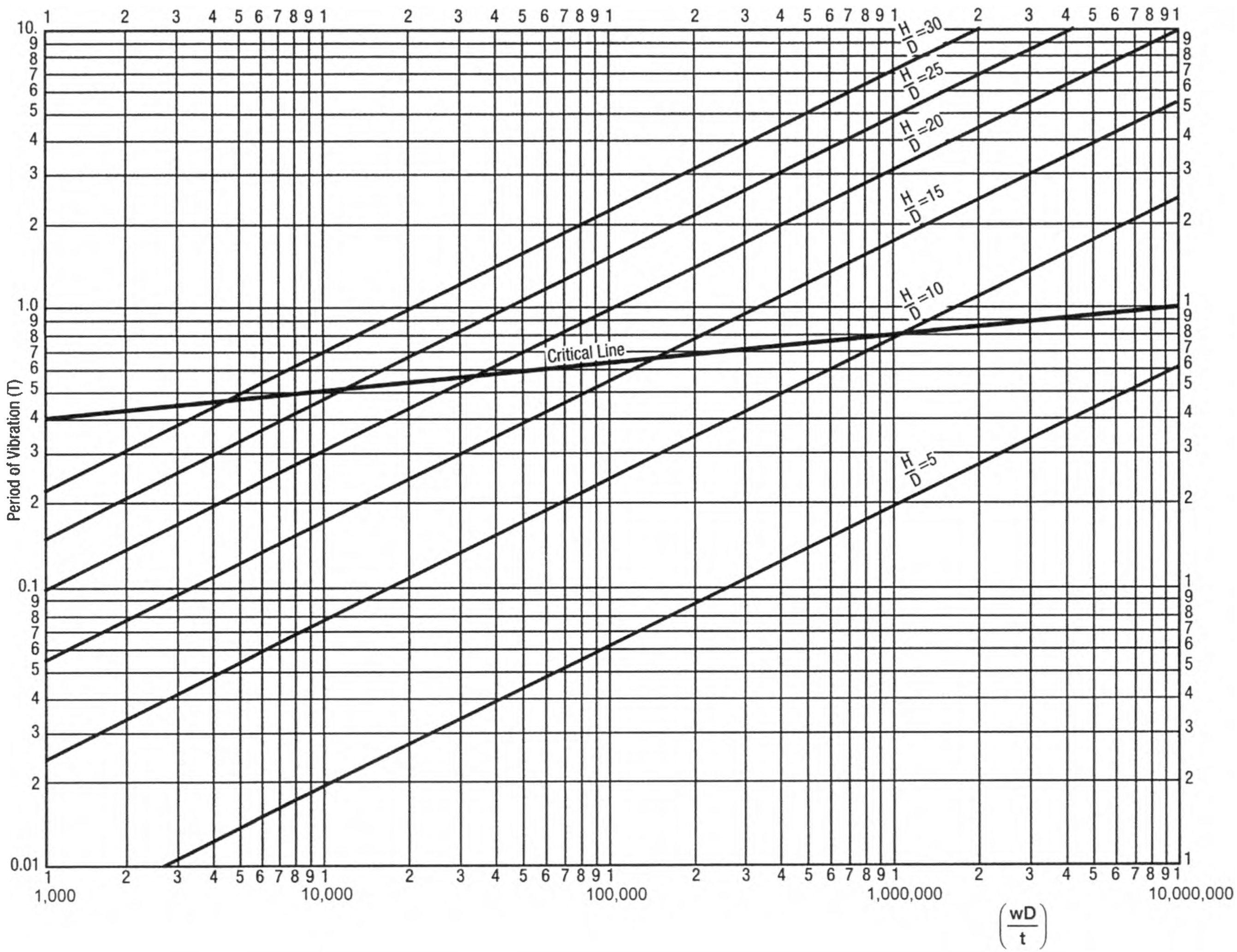


Figure 4-6. Formulas for period of vibration, T, and deflection, y.



General formula for cantilever

$$T = K\sqrt{\frac{wH^4}{EIg}}$$

which for steel cylidrical shell reduces to

$$T = 0.00000765\left(\frac{H}{D}\right)^2\sqrt{\frac{wD}{t}}$$

where T = period, sec
w = weight, lb per ft
H = height, ft
D = diameter of shell, ft
t = thickness of shell, ft

Constant 0.00000765 is based upon:

E = modulus of elasticity of steel 30,000,000 lb per sq in

I = moment of inertia of shell area

$$= 3.142\left(\frac{D}{2}\right)^3 t$$

K = 1.79 for fundamental period of vibration

$g = 32.2$ ft per sec^2

Figure 4-7. Period of vibration for cylindrical steel shells. *Reprinted by permission of Fluor Daniel. Inc., Irvine CA.*

dependent on the relative rigidity of the vessel. The more rigid the vessel (lower POV, high frequency) the higher the base shear will be. The more flexible (higher POV, lower frequency) vessels would have a lower base shear.

Notes

1. Vessels mounted in structures at some elevation other than grade generally will experience amplified base motion near and above the natural frequencies of the support structure.
   - *Light vessels* (less than 1% of structure weight):
     a. If vessel frequency > structure frequency, then vessel is subjected to maximum acceleration of the structure.
     b. If vessel frequency < structure frequency, then vessel will not be affected by structure. It will respond as if it were mounted at grade.
   - *Medium vessels* (less than 20% of structure weight): Approximate methods may be used to develop the in-structure response spectra. The method used should account for interaction between vessel and structure (energy feedback). Consideration should be given to account for ductility of the vessel.
   - *Heavy vessels* (single large vessel or multiple large vessels): The vessel(s) is the principal vibrating element. It requires a combined seismic model, which simulates the mass and stiffness properties of vessel and structure.
2. For tall slender vessels, the main concern is bending. For short, squat vessels the main concern is base shear.
3. The procedures outlined in this chapter are static-force procedures, which assume that the entire seismic force due to ground motion is applied instantaneously. This assumption is conservative but greatly simplifies the calculation procedure. In reality earthquakes are time-dependent events and the full force is not realized instantaneously. ASCE/SEI 7 allows, and in some cases requires, that a dynamic analysis be performed in lieu of the static force method. Although much more sophisticated, often the seismic loadings are reduced significantly.

Procedure 4-3: Seismic Design for Vessels [2]

Notation

C_s = seismic response coefficient
E_h = effect of horizontal earthquake-induced forces
E_v = effect of vertical earthquake-induced forces
F_a = short-period site coefficient (at 0.2 second period)
F_v = long-period site coefficient (at 1.0 second period)
g = acceleration due to gravity, ft/sec^2
I_e = the importance factor
R = response modification coefficient
S_a = design spectral acceleration
S_S = mapped maximum considered earthquake (risk-targeted), 5 percent damped, spectral response acceleration parameter at short periods
S_1 = mapped maximum considered earthquake (risk-targeted), 5 percent damped, spectral response acceleration parameter at a period of 1 second
S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods
S_{D1} = design, 5 percent damped, spectral response acceleration parameter at a period of 1 second
S_{MS} = the maximum considered earthquake (risk-targeted), 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects
S_{M1} = the maximum considered earthquake (risk-targeted), 5 percent damped, spectral response acceleration parameter at a period of 1 second adjusted for site class effects
T = the fundamental period of the structure, seconds
T_L = long-period transition period, seconds (see Figure 4-8)
V = total design lateral force or shear at the base, lbs
W = effective seismic weight of the structure, lbs

Design Procedure

Step 1: Determine the following.
- Risk Category from Table 4-5
- Importance factor as follows:
  - 1.00 for Risk Category I and II
  - 1.25 for Risk Category III
  - 1.50 for Risk Category IV
- Site Class as determined by local soil conditions, see Table 4-6
  - Site Class D may be used unless a governing authority indicates E or F shall be used.
- S_S and S_1 parameters
  - See http://earthquake.usgs.gov/designmaps
- F_a and F_v parameters
  - See Tables 4-7 and 4-8

Step 2: Calculate S_{MS} and S_{M1}

$$S_{MS} = F_a S_s$$
$$S_{M1} = F_v S_1$$

Step 3: Calculate S_{DS} and S_{D1}

$$S_{DS} = (2/3)^* S_{MS}$$
$$S_{D1} = (2/3)^* S_{M1}$$

Step 4: Calculate S_a to develop a response spectrum

If $T < T_o$, ($T_o = 0.2\, S_{D1}/S_{DS}$),

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_o}\right),$$

If $T_o \le T \le T_S$, ($T_S = S_{D1}/S_{DS}$), $S_a = S_{DS}$,

If $T_S < T \le T_L$, $S_a = \dfrac{S_{D1}}{T}$,

If $T > T_L$, $S_a = \dfrac{S_{D1} T_L}{T^2}$

Step 5: Determine Seismic Design Category (SDC) from S_{DS} and S_{D1} and use most severe*
- SDC A-D are determined from Tables 4-9 and 4-10
- SDC E is used where S_1 is greater than or equal to 0.75 for Risk Categories I, II, and III
- SDC F is used where S_1 is greater than or equal to 0.75 for Risk Category IV

Step 6: Calculate the vertical seismic load, E_V
- $E_v = 0.2 S_{DS} D$, where D is the effect of the dead load

Step 7: Determine the response modification factor*

R = 3	for elevated tanks, vessels, bins or hoppers on symmetrically braced legs (height limits may apply)
R = 2	for elevated tanks, vessels, bins or hoppers on unbraced or asymmetrically braced legs (height limits may apply)
R = 3	for horizontal vessels on welded steel saddle supports
R = 2 (3**)	steel stacks, chimneys, silos, skirt-supported vertical vessels

Step 8: Calculate the seismic response coefficient, C_s

$$C_s = \frac{S_{DS}}{(R/I_e)},$$ but need not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I_e)}\ for\ T \le T_L,$$

$$C_s = \frac{S_{D1}T_L}{T^2(R/I_e)}\ for\ T > T_L,$$

and shall not be less than

$$C_s = 0.044 S_{DS} I_e \le 0.01$$

and additionally where $S_1 \ge 0.6$ g

$$C_s = 0.5 S_1/(R/I_e)$$

Step 9: Calculate the seismic base shear, V
- $V = C_s W$ and $E_h = V$

* Additional detailing should be addressed for the use of these R factors per ASCE/SEI 7 which include the effects of the solids or fluids stored and their interaction with the structure, p-delta effects, and other requirements within other design standards.

** For these structures, R may be taken as a value of 3, however, additional instructions apply in these cases. If buckling of the support is determined to be the governing mode of failure, or if the structure is in Risk Category IV, then the seismic response coefficient must be determined using a value of $I_e/R = 1.0$ and checked against the critical buckling resistance (safety factor equal to 1.0).

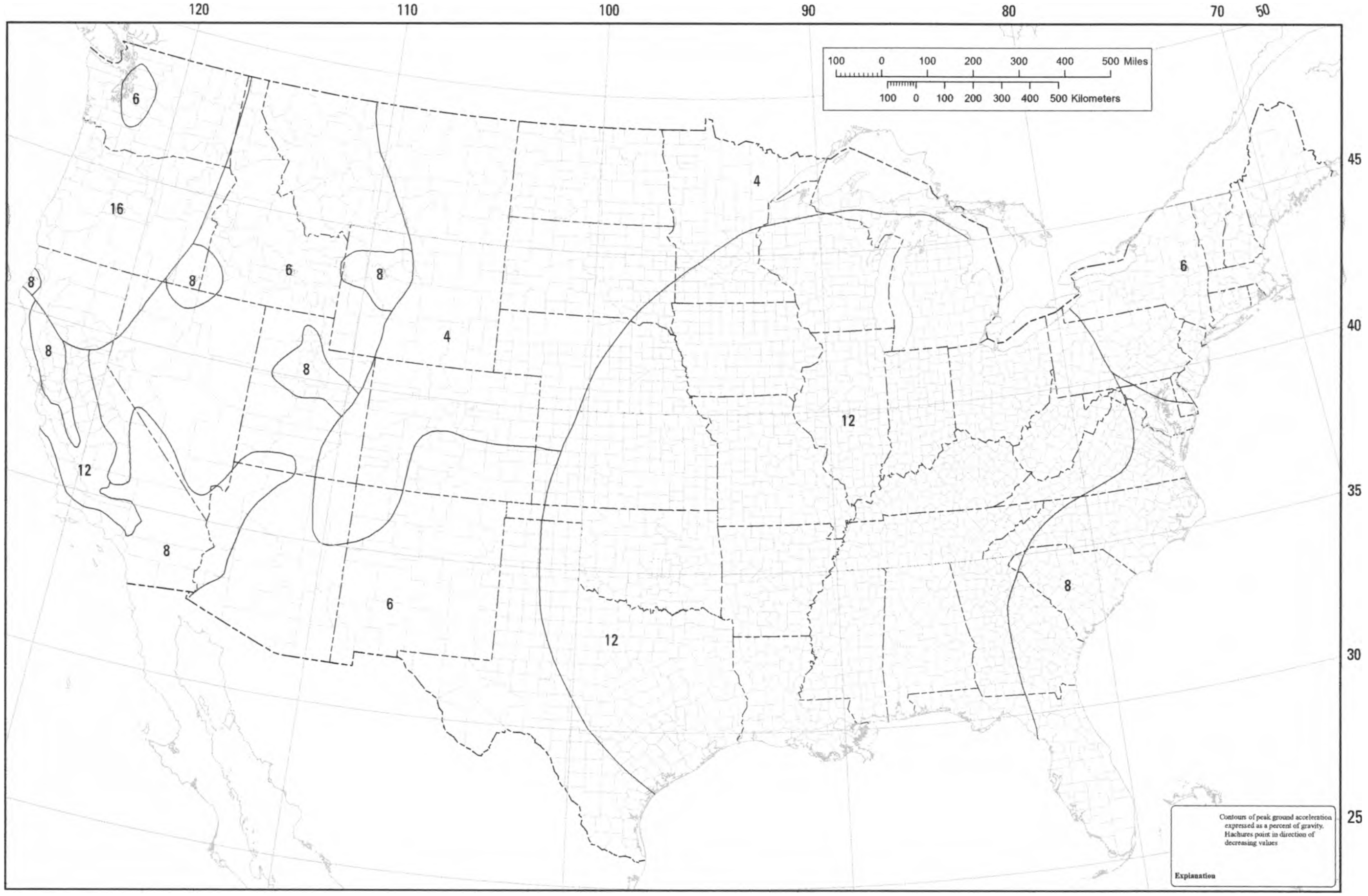


Figure 4-8. Mapped long-period transition period, T_L(s),for the conterminous United States. *With permission from ASCE.*

Table 4-5
Risk category of buildings and other structures for wind and earthquake loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community.	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is suffi cient to pose a threat to the public if released.[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

[a] Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.
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Table 4-6
Site classification

Site Class	v_5	$\bar{N}$ or $\bar{N}_{cA}$	$\bar{S}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{S}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1 of ASCE/SEI 7.		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft² = 0.0479 kN/m².
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Table 4-7
Site coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCER) Parameter at Short Period Spectral Response Acceleration $S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.
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Table 4-8
Site coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCER) Parameter at 1-s Period Spectral Response Acceleration $S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.
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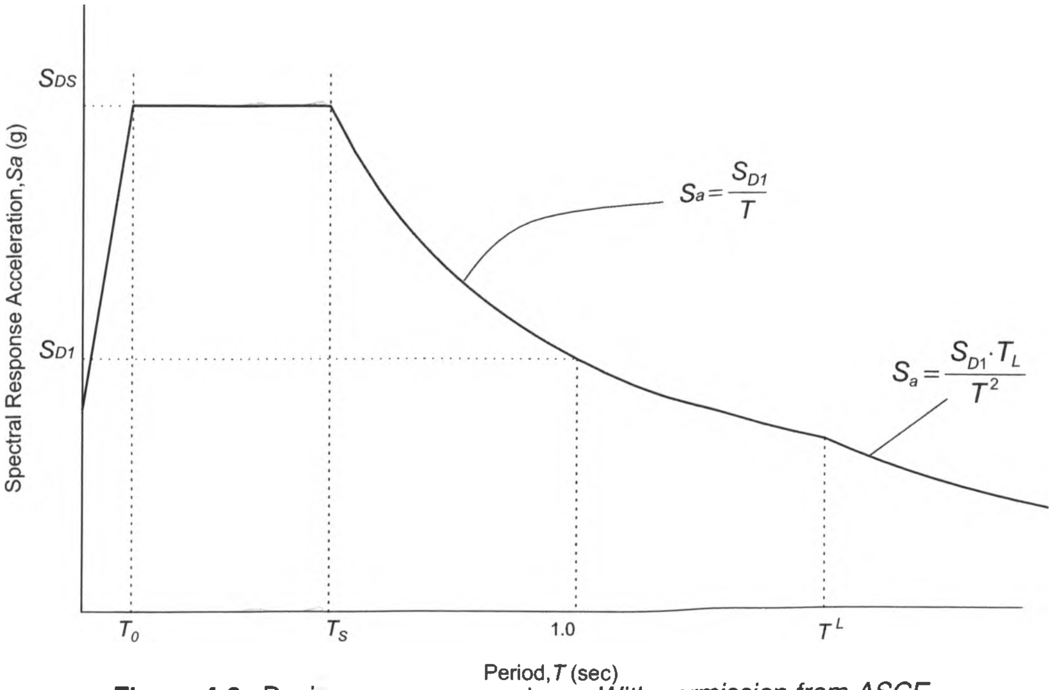

Figure 4-9. Design response spectrum. *With permission from ASCE.*

Table 4-9
SDC Based on S_{DS}

Value of S_{DS}	Risk Category: I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

With permission from ASCE

Table 4-10
SDC Based on S_{D1}

Value of S_{D1}	Risk Category: I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

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Procedure 4-4: Seismic Design – Vessel on Unbraced Legs [4–7]

Notation

A = cross-sectional area, leg, in.^{2}
V = base shear, lb
W = operating weight, lb
n = number of legs
C_v = vertical seismic factor
C_h = horizontal seismic factor
y = static deflection, in.
F_v = vertical seismic force, lb
F_h = horizontal seismic force, lb
F_a = allowable axial stress, psi
F_D = axial load due to dead weight, lb
F_L = axial load due to seismic or wind, lb
F_b = allowable bending stress, psi
F'_e = Euler stress divided by safety factor, psi
f_1 = maximum eccentric load, lb
V_n = horizontal load on leg, lb
F_n = maximum axial load, lb
f_a = axial stress, psi
f_b = bending stress, psi
E = modulus of elasticity, psi
g = acceleration due to gravity, 386 in./sec^2
e = eccentricity of legs, in.

M_b = overturning moment at base, in.-lb
M_t = overturning moment at tangent line, in.-lb
M = bending moment in leg, in.-lb
ΣI_1 = summation of moments of inertias of all legs perpendicular to F_h, $in.^4$
ΣI_2 = summation of moments of inertia of one leg perpendicular to F_h, $in.^4$
I = moment of inertia of one leg perpendicular to F_h, $in.^4$
C_1 = distance from centroid to extreme fiber, in.
C_m = coefficient, 0.85 for compact members
K_1 = end connection coefficient, 1.5-2.0
T = period of vibration, sec
r = least radius of gyration, in.

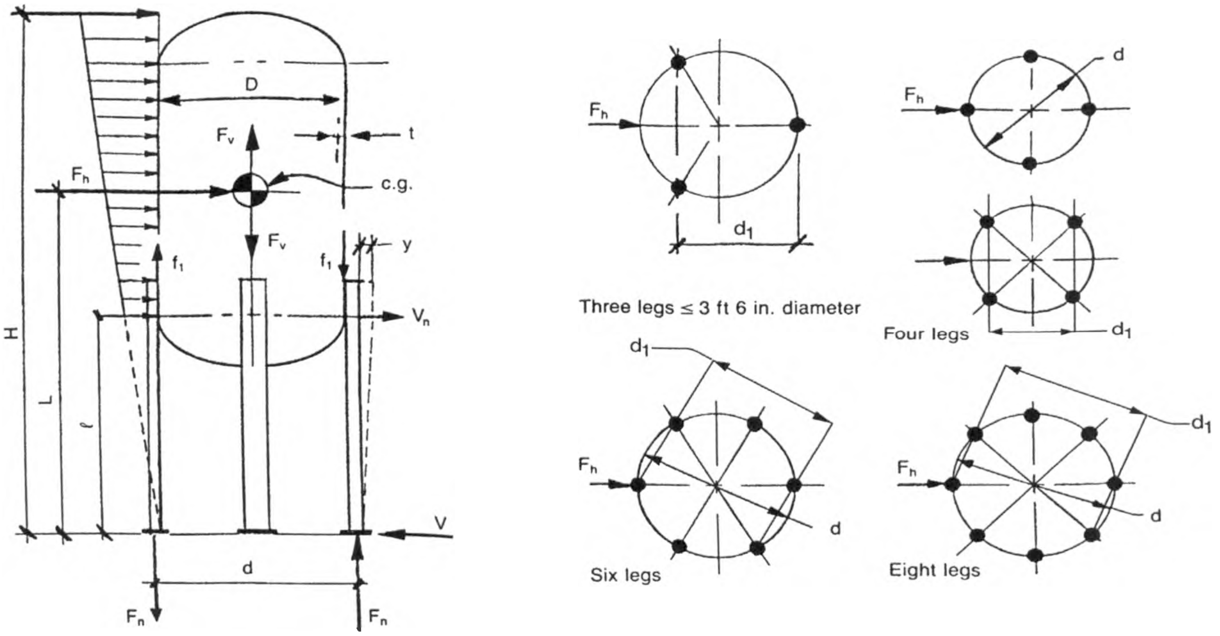


Figure 4-10. Typical dimensional data and forces for a vessel supported on unbraced legs.

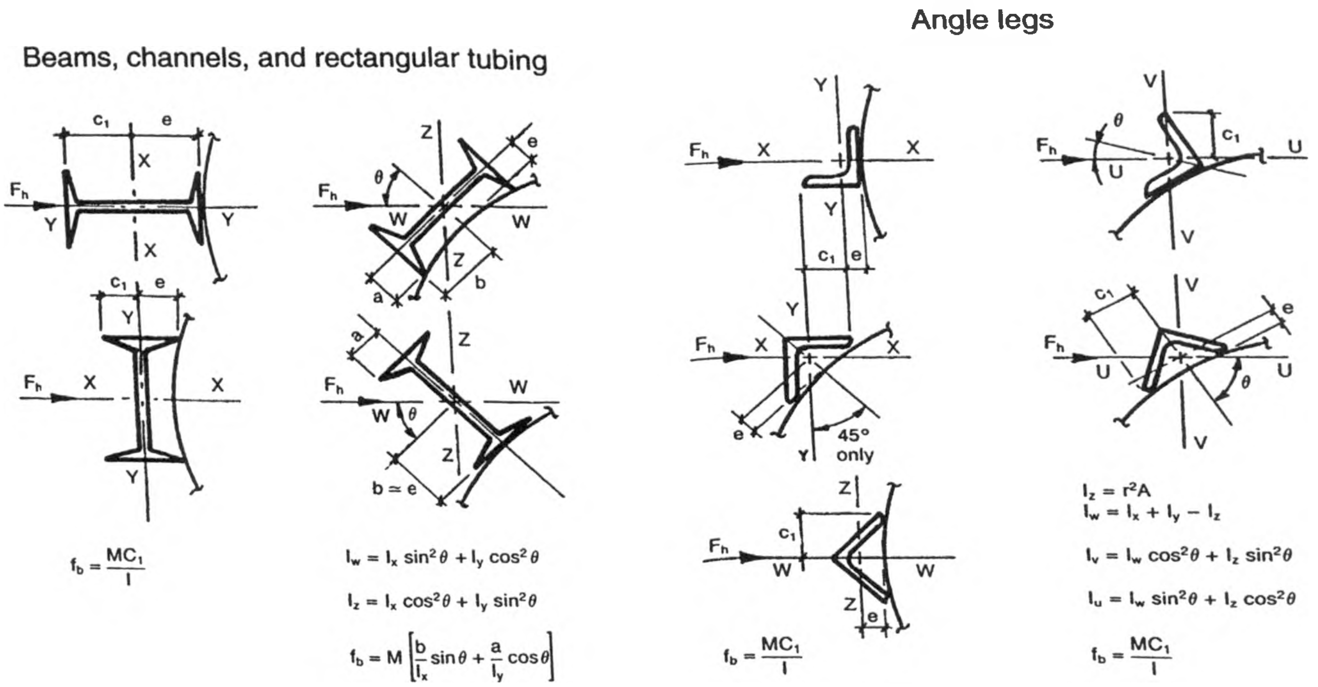


Figure 4-11. Various leg configurations.

Calculations

The following information is needed to complete the leg calculations:

No. ________	I_u = ________
Size ________	I_v = ________
A = ________	ΣI_1 = ________
r = ________	ΣI_2 = ________
I_x = ________	$K_1\ell/r$ = ________
I_y = ________	F_a = ________
I_z = ________	
I_w = ________	

- *Deflection, y, in.*

$$y = \frac{2\,W\,\ell^3}{3nE\sum I_2}$$

Note: Limit deflection to 6 in. per 100 ft or equivalent proportion. This calculation is based on the assumption that the support legs are pinned at the base and fixed at the vessel, and that the vessel is significantly more rigid than the supported legs.

- *Period of vibration, T, sec.*

$$T = 2\pi\sqrt{\frac{y}{g}}$$

- *Base shear, V, lb.*
  See Procedure 4-3.
- *Horizontal force at c.g. of vessel, F_h, lb.*

$$F_h = C_h W$$

- *Vertical force at c.g. of vessel, F_v, lb.*

Downward: $(-)F_v = W$

or $(1 + C_v)W$

Upward: $(+)F_v = (C_v - 1)W$

if vertical seismic is greater than 1.0

- *Overturning moment at base, in.-lb.*

$$M_b = LF_h$$

Note: Include piping moments if applicable.

- *Overturning moment at bottom tangent line, in.-lb.*

$$M_t = (L - \ell)F_h$$

- *Maximum eccentric load, lb.*

$$f_1 = \frac{-F_v}{n} - \frac{4M_t}{nD}$$

Note: f_1 is not considered in leg bending stress if legs are not eccentrically loaded.

- *Horizontal load distribution, V_n* (See Figure 4-12). The horizontal load on any one given leg, V_n, is proportional to the stiffness of that one leg perpendicular to the applied force relative to the stiffness of the other legs. The greater loads will go to the stiffer legs. Thus, the general equation:

$$V_n = \frac{VI}{\sum I_1} \text{ and } \sum V_n = V$$

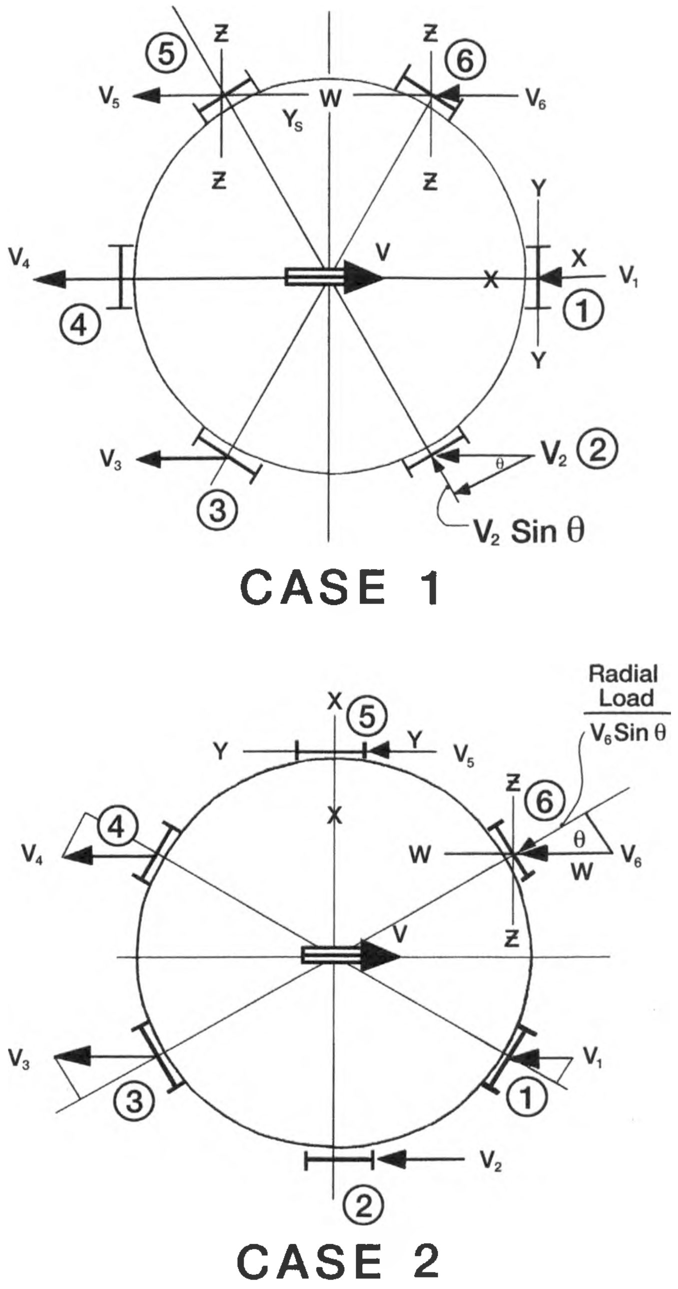


Figure 4-12. Load diagrams for horizontal load distribution.

- *Vertical load distribution, F_n* (See Figure 4-13).

  The vertical load distribution on braced and unbraced legs is identical. The force on any one leg is equal to the dead load plus the greater of seismic or wind and the angle of that leg to the direction of force, V.

- *Bending moment in leg, M, in.-lb.*

  $$M = f_1 e \pm V_n \ell$$

- *Axial stress in leg, f_a, psi.*

  $$f_a = \frac{F_n}{A}$$

- *Bending stress in leg, f_b, psi.*

  $f_b =$

  Select appropriate formula from Figure 4-11.

For Case 1	For Case 2
$F_D = \frac{F_v}{n}$	$F_D = \frac{F_v}{n}$
$F_L = \frac{4M}{nd}$	$F_L = \frac{4Md_1}{nd^2}$
$F_n = F_D \pm F_L \cos \phi_n$	
$F_n = F_D \pm F_L \cos \phi_n$	

- *Combined stress.*

  If $\frac{f_a}{F_a} \leq 0.15$, then $\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1$

  If $\frac{f_a}{F_a} > 0.15$, then $\frac{f_a}{F_a} + \frac{C_m f_b}{\left[1 - \frac{f_a}{F'_e}\right] F_b} < 1$

  where $C_m = 0.85$

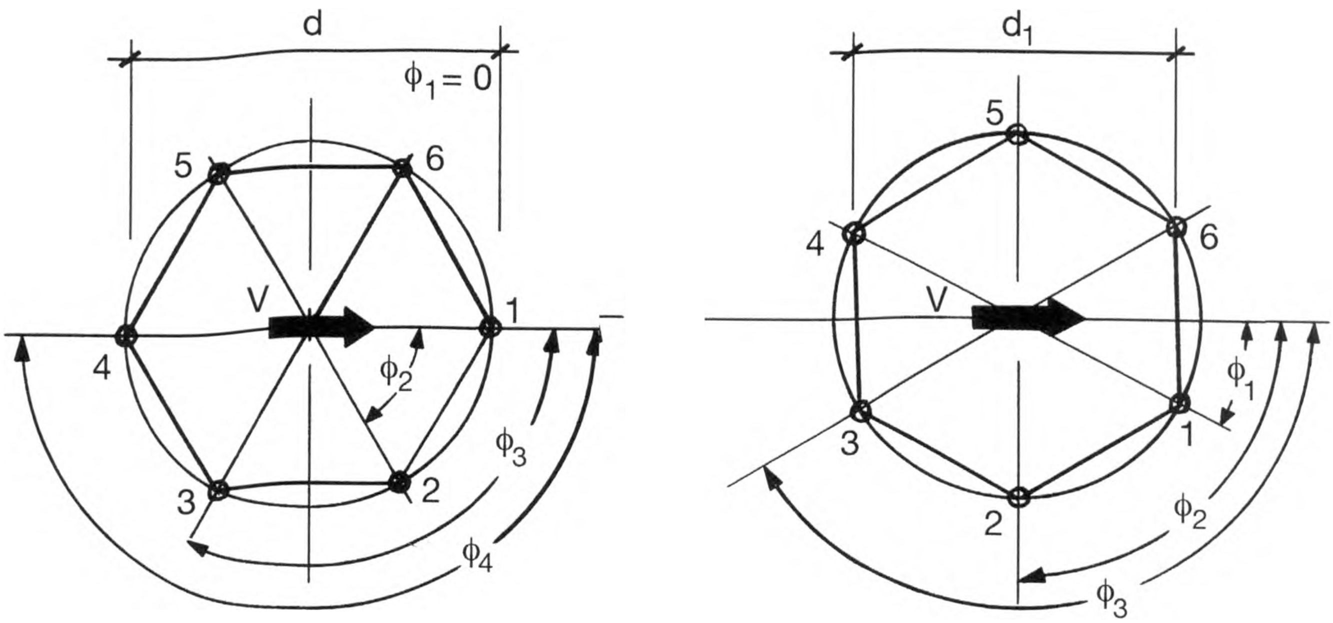
  $$F_e' = \frac{12\pi^2 E}{23\left(\frac{k_1 \ell}{r}\right)^2}$$

- *Maximum compressive stress in shell, f_c, psi (See Figure 4-16).*

  $$L_1 = W + 2\sqrt{(Rt)}$$

  Above leg:

  $$f_c = \frac{f_1}{L_1 t}$$



CASE 1 CASE 2

Figure 4-13. Load diagrams for vertical load distribution.

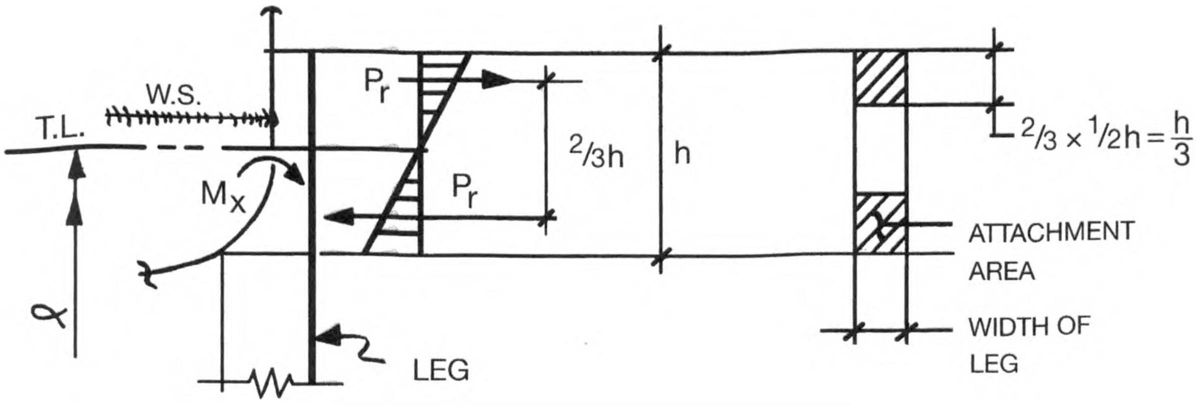


Figure 4-14. Application of local loads in head and shell.

General:

$$f_c = (-)\frac{F_v}{\pi Dt} - \frac{4M_t}{\pi D^2 t}$$

F_c = allowable compressive stress is factor "B" from ASME Code.

$$\text{Factor "A"} = \frac{0.125t}{R}$$

"B" = from applicable material chart of ASME Code, Section II, Part D, Subpart 3.

- *Shear load in welds attaching legs.*

$$\frac{f_1}{2h} = \frac{\text{lb}}{\text{in. of weld}}$$

See Table 4-14 for allowable loads on fillet welds in shear.

- *Local load in shell (See Figure 4-14).*

For unbraced designs, the shell or shell/head section to which the leg is attached shall be analyzed for local loading due to bending moment on leg.

$$M_x = V_n \ell \sin\theta$$

- *Anchor bolts.* If the weight $W > 4\ M_b/d$, then no uplift occurs and anchor bolts should be made a minimum of ¾ in. in diameter. If uplift occurs, then the cross-sectional area of the bolt required for tension alone would be:

$$A_b = \frac{f_2}{S_t}\ \text{in.}^2$$

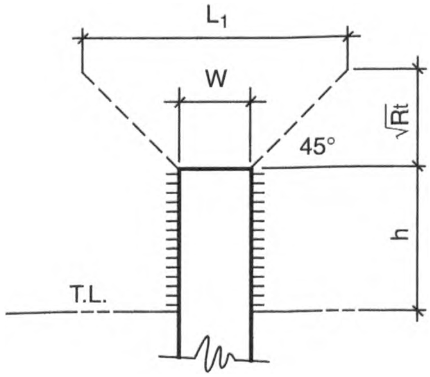


Figure 4-15. Dimensions of leg attachment.

where A_b = area of bolt required
f_2 = axial tension load
S_t = allowable stress in tension

Notes

1. Legs longer than 7 ft should be cross-braced.
2. Do not use legs to support vessels where high vibration, shock, or cyclic service is anticipated.
3. Select legs that give maximum strength for minimum weight for most efficient design. These sections will also distribute local loads over a larger portion of the shell.
4. Legs may be made of pipe, channel, angle, rectangular tubing, or beam sections.
5. This procedure assumes a one-mass bending structure which is not technically correct for tall vessels. Tall towers would have distributed masses and should be designed independently of support structure, i.e., legs.

Table 4-11
Vertical loads on legs, F_n

Quantity of Legs	Leg No.	Case 1	Case 2
6	1	$F_D + 1.000\ F_L$	$F_D + 0.866\ F_L$
	2	$F_D + 0.500\ F_L$	F_D
	3	$F_D - 0.500\ F_L$	$F_D - 0.866\ F_L$
	4	$F_D - 1.000\ F_L$	$F_D - 0.866\ F_L$
	5	$F_D - 0.500\ F_L$	F_D
	6	$F_D + 0.500\ F_L$	$F_D + 0.866\ F_L$
8	1	$F_D + 1.000\ F_L$	$F_D + 0.923\ F_L$
	2	$F_D + 0.707\ F_L$	$F_D + 0.382\ F_L$
	3	F_D	$F_D - 0.382$
	4	$F_D - 0.707\ F_L$	$F_D - 0.923\ F_L$
	5	$F_D - 1.000\ F_L$	$F_D - 0.923\ F_L$
	6	$F_D - 0.707\ F_L$	$F_D - 0.382\ F_L$
	7	F_D	$F_D + 0.382$
	8	$F_D + 0.707\ F_L$	$F_D + 0.923\ F_L$
10	1	$F_D + 1.000\ F_L$	$F_D + 0.951\ F_L$
	2	$F_D + 0.809\ F_L$	$F_D + 0.587\ F_L$
	3	$F_D + 0.309\ F_L$	F_D
	4	$F_D - 0.309\ F_L$	$F_D - 0.809\ F_L$
	5	$F_D - 0.809\ F_L$	$F_D - 0.951\ F_L$
	6	$F_D - 1.000\ F_L$	$F_D - 0.951\ F_L$
	7	$F_D - 0.809\ F_L$	$F_D - 0.587\ F_L$
	8	$F_D - 0.309\ F_L$	F_D
	9	$F_D + 0.309\ F_L$	$F_D + 0.587\ F_L$
	10	$F_D + 0.809\ F_L$	$F_D +0.951\ F_L$
12	1	$F_D + 1.000\ F_L$	$F_D + 0.965\ F_L$
	2	$F_D + 0.866\ F_L$	$F_D + 0.707\ F_L$
	3	$F_D + 0.500\ F_L$	$F_D + 0.258\ F_L$
	4	F_D	$F_D - 0.258\ F_L$
	5	$F_D - 0.500\ F_L$	$F_D - 0.707\ F_L$
	6	$F_D - 0.866\ F_L$	$F_D - 0.965\ F_L$
	7	$F_D - 1.000\ F_L$	$F_D - 0.965\ F_L$
	8	$F_D - 0.866\ F_L$	$F_D - 0.707\ F_L$
	9	$F_D - 0.500\ F_L$	$F_D - 0.258\ F_L$
	10	F_D	$F_D + 0.258\ F_L$
	11	$F_D + 0.500\ F_L$	$F_D + 0.707\ F_L$
	12	$F_D + 0.866\ F_L$	$F_D + 0.965\ F_L$
16	1	$F_D + 1.000\ F_L$	$F_D + 0.980\ F_L$
	2	$F_D + 0.923\ F_L$	$F_D + 0.831\ F_L$
	3	$F_D + 0.707\ F_L$	$F_D + 0.555\ F_L$
	4	$F_D + 0.382\ F_L$	$F_D + 0.195\ F_L$
	5	F_D	$F_D - 0.195\ F_L$
	6	$F_D - 0.382\ F_L$	$F_D - 0.555\ F_L$
	7	$F_D - 0.707\ F_L$	$F_D - 0.831\ F_L$
	8	$F_D - 0.923\ F_L$	$F_D - 0.980\ F_L$
	9	$F_D - 1.000\ F_L$	$F_D - 0.980\ F_L$
	10	$F_D - 0.923\ F_L$	$F_D - 0.831\ F_L$
	11	$F_D - 0.707\ F_L$	$F_D - 0.555\ F_L$
	12	$F_D - 0.382\ F_L$	$F_D - 0.195\ F_L$
	13	F_D	$F_D + 0.195\ F_L$
	14	$F_D + 0.382\ F_L$	$F_D + 0.555\ F_L$
	15	$F_D + 0.707\ F_L$	$F_D + 0.831\ F_L$
	16	$F_D+ 0.923\ F_L$	$F_D + 0.980\ F_L$

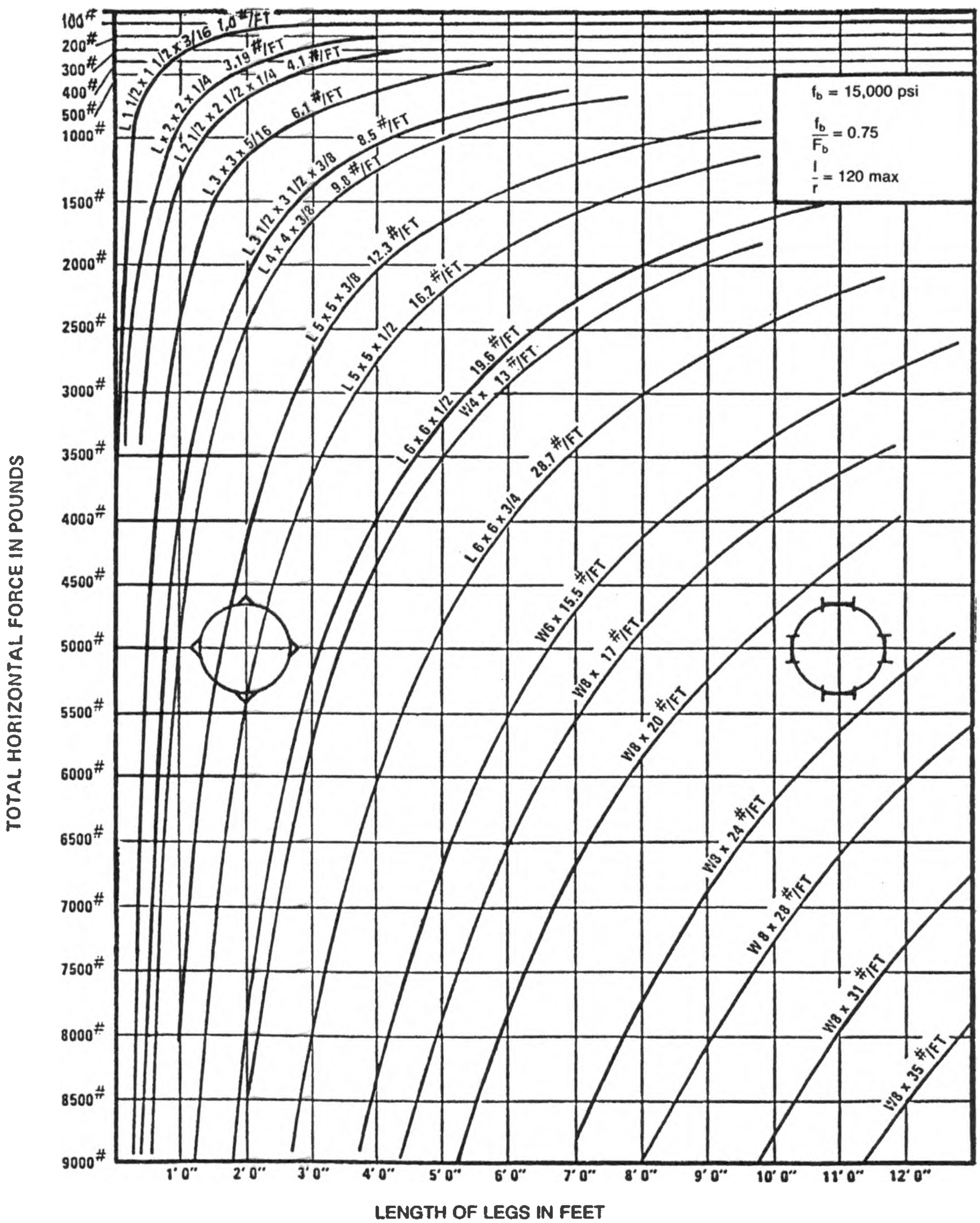
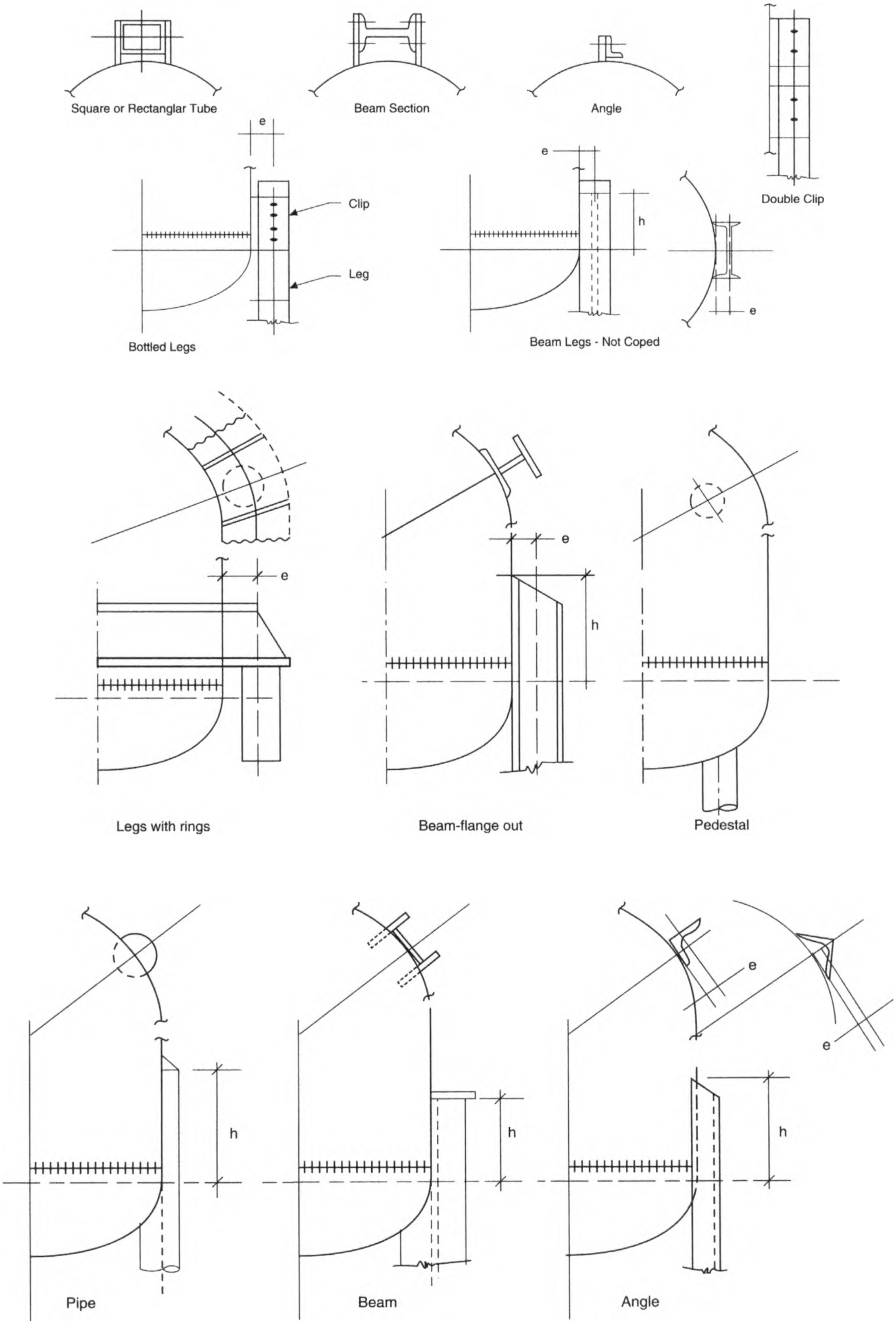


Figure 4-16. Leg sizing chart for vessel supported on four legs.

Types of Leg Attachment



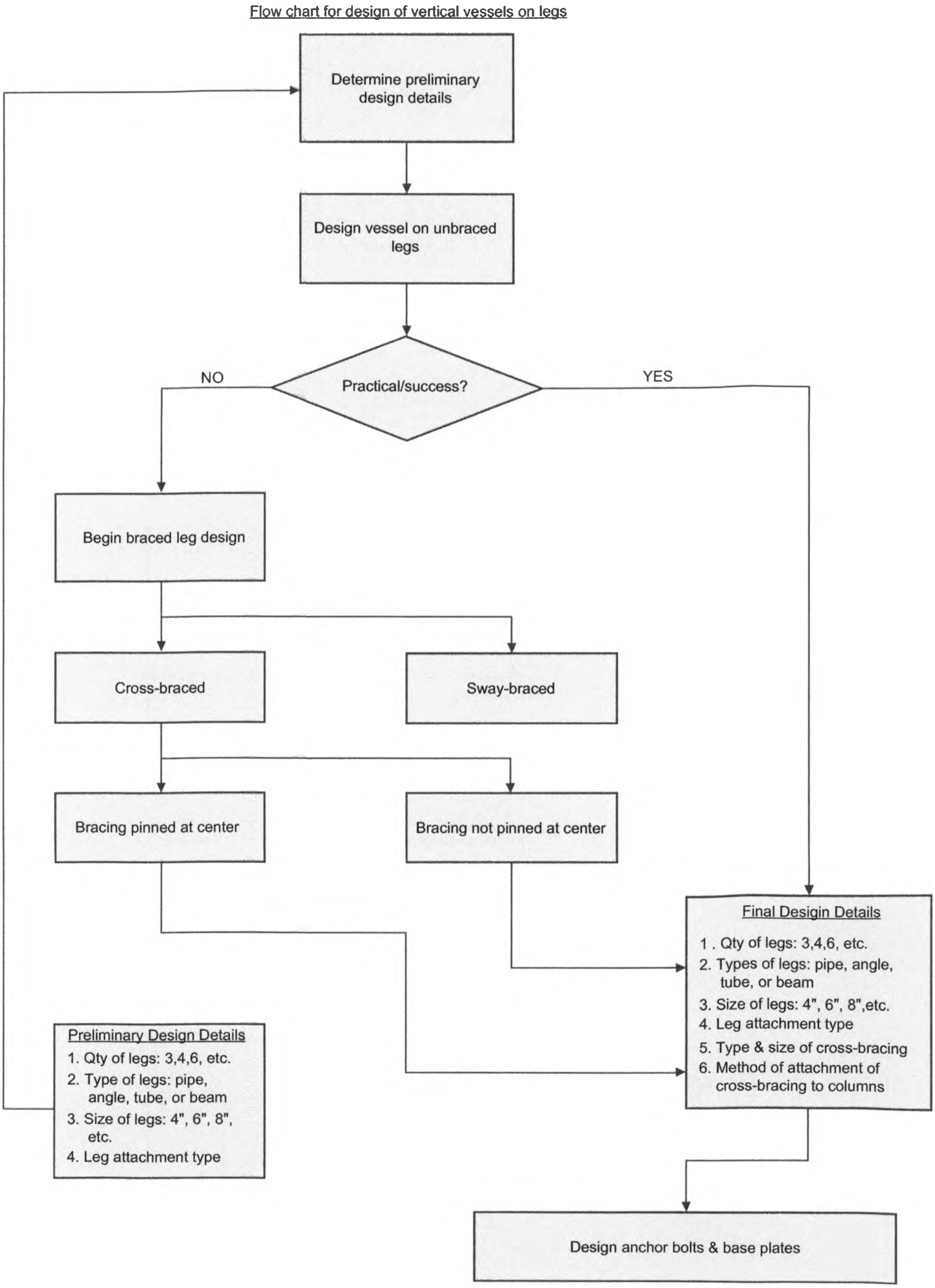


Figure 4-17. Flow chart for design of vertical vessels on legs.

Procedure 4-5: Seismic Design – Vessel on Braced Legs

Notation

A_b = Area, brace, in^2
A_c = Area, column, in^2
A_{br} = Area, brace, required, in^2
C_a = Corrosion allowance, in
D_c = Centerline diameter of columns, in
E = Modulus of elasticity, psi
f = Maximum force in brace, Lbs
f_a = Axial stress, compression, psi
f_t = Tension stress, psi
F_a = Allowable axial stress, psi
F_b = Allowable stress, bending, psi
F_c = Allowable stress, compression, psi
F_D = Axial load on column due to dead weight, lbs
F_h = Horizontal seismic force, Lbs
F_L = Axial load on column due to seismic or wind, lbs
F_t = Allowable stress, tension, psi
F_V = Vertical seismic force, Lbs
F_y = Yield strength of material at temperature, psi
g = Acceleration due to gravity, 386 in/sec^2
I_b = Moment of inertia, bracing, in^4
I_r = Required moment of inertia, in^4
I_c = Moment of inertia, column, in^4
k = End connection coefficient, columns
M_o = Overturning moment, in-Lbs
N = Number of columns
n = Number of active rods per panel use 1 for sway bracing; 2 for cross bracing
n′ = Factor for cross bracing, use 1 for unpinned and 2 for pinned at center
Q = Maximum axial force in column, Lbs
r_b = Radius of gyration, brace, in
r_c = Radius of gyration, column, in
S_r = Slenderness ratio
T = Period of vibration, seconds
V = Base shear, Lbs
V_n = Horizontal force per column, Lbs
W_o = Weight, operating, Lbs
w = Unit weight of liquid, pcf
ΔL = Change in length of brace, in
δ = Lateral deflection of vessel, in

Horizontal Load Distribution, V_n

The horizontal load on any one leg is dependent on the direction of the leg bracing. The horizontal force, V, is transmitted to the legs through the bracing. Thus, the general equation:

Table 4-12
Dimensions for d_1

No. of Legs	d_1
3	.750 Dc
4	.707 Dc
6	.866 Dc
8	.924 Dc
10	.951 Dc
12	.966 Dc
16	.981 Dc

$$V_n = \frac{V(\sin^2 \alpha_{n-1} + \sin^2 \alpha_n)}{N} \quad \text{and} \quad \Sigma V_n = V$$

Vertical Load Distribution, F_n

The vertical load distribution on braced and unbraced legs is identical. The force on any one leg is equal to the dead load plus the greater of seismic or wind and the angle of that leg to the direction of force, V. The general equation for each case is as follows:

For Case 1:

$$F_D = \frac{F_v}{N}$$

$$F_L = \frac{4M}{Nd}$$

$$F_n = F_D \pm F_L \cos \phi_n$$

For Case 2:

$$F_D = \frac{F_v}{N}$$

$$F_L = \frac{4Md_1}{Nd^2}$$

$$F_n = F_D \pm F_L \cos \phi_n$$

Design of Columns

- Base Shear, V
  Use worst case of wind or seismic
  V = ________
- Overturning Moment, M_o

  $M_o = L\,V$
- Maximum Dead load, F_D

  $F_D = (-)W_o/N$
- Maximum Earthquake/Wind Load, $F_{E/W}$

  $F_L = +/-4\,M_o/N\,D_c$
- Maximum Column Load, Q
  Select worst case from Table or use;

$Q = F_D +/- F_L$

Q max compression = Q_C =

Q max tension = Q_T =

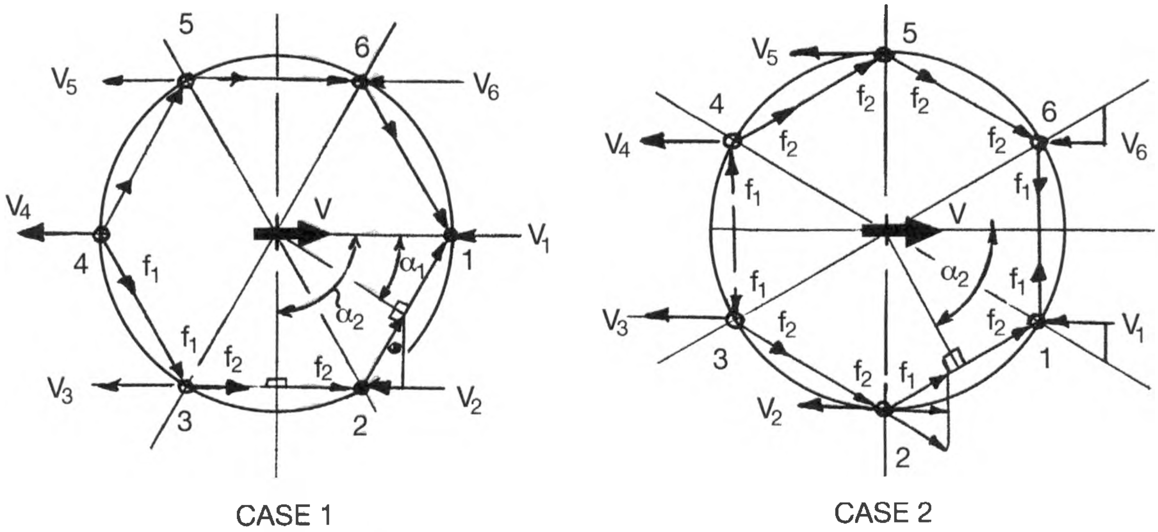
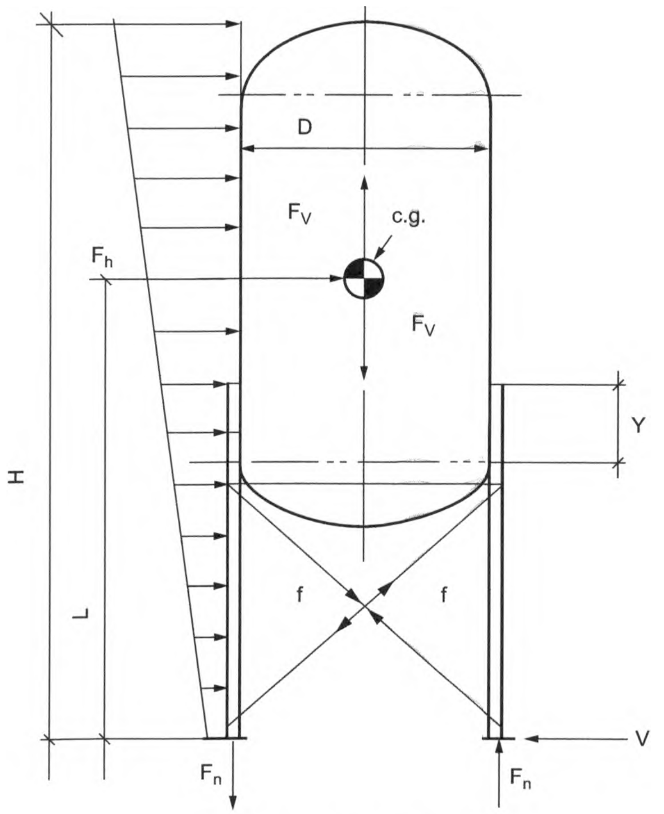


Figure 4-18. Load diagrams for horizontal load distribution.



Four legs (for illustration only)

Note: If there is no uplift then there is no tension force.

- Leg selection;

 Use: = __________

 A_C = __________

Compression Case

- Compressive stress, f_a

$$f_a = Q_C/A_C \leq F_a$$

- Slenderness ratio, $S_r = k_h/r_c$
 $F_a =$

Tension Case

- Tension stress, f_t

$$f_t = Q_T/A_C \leq F_t$$

- Allowable tension stress, F_t,

$$F_T = 1.2(0.6)F_y$$

Cross Bracing

Note: Loads in cross bracing are tension and compression.

Compression Case

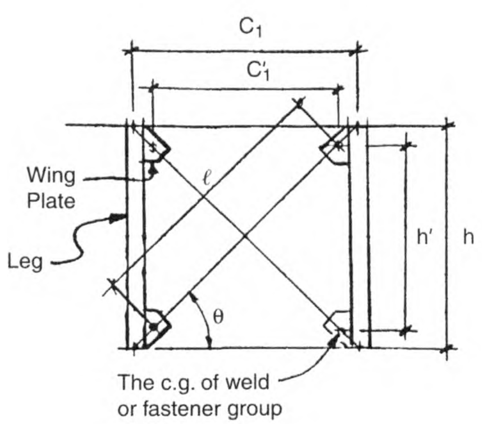
- Case 1: Pinned at center

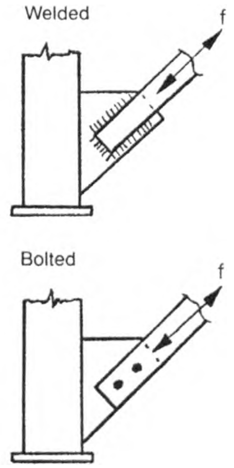
$$I_r = FL_1{}^2/4\pi^2E$$

Case 2: Not pinned at center

$$I_r = FL_1{}^2/\pi^2E$$

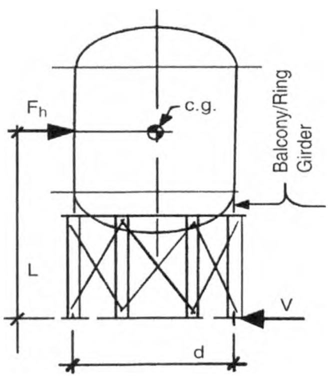
ESTABLISH LEG & BRACE DIMENSIONS



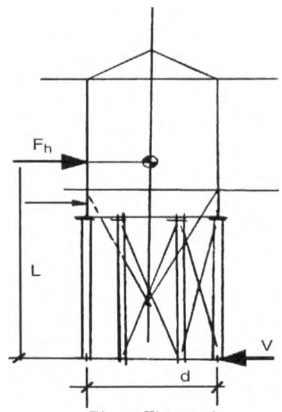


DIMENSIONS:

$\phi = 360 / N =$	$h =$
$R_c =$	$h' = h - 2\,a_2 =$
$E = 2\,[\,\text{Sin}\,.5\,\phi\,(R_c)\,] =$	Find Angle θ ;
$E =$	$\text{Tan}\,\theta = E' / h' =$
$a_1 =$	$\theta =$
$a_2 =$	Find Length of Brace, L_1
$E' = E - 2\,a_1 =$	$L_1 = E' / \text{Sin}\,\theta =$



Large Pressure Vessel

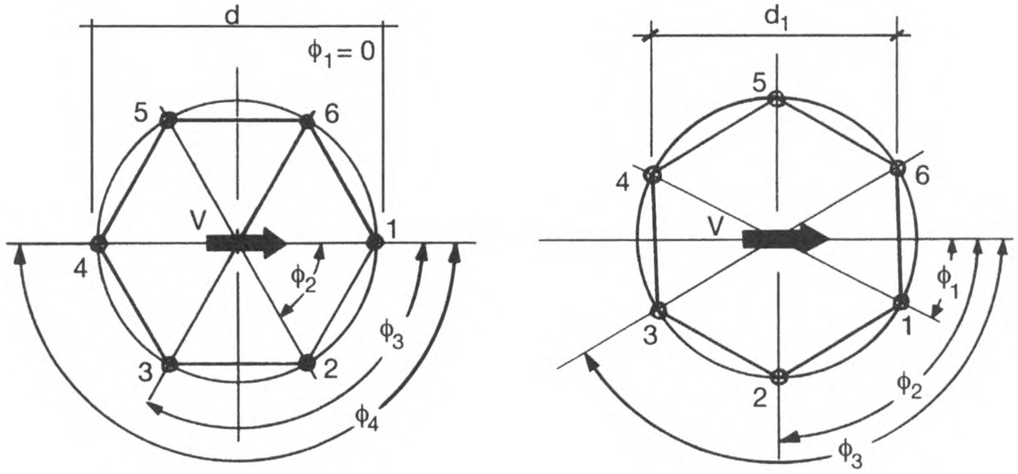


Bin or Elevated Tank

VESSEL ON BRACED LEGS - SEISMIC DESIGN			
	METHOD 1	**METHOD 2**	**METHOD 3**
V_n = Horiz shear per lug	Worst case from Table dependent on number of legs and direction of seismic force (between legs or through legs).	NA	$V_n = V/N$
f = Max force in brace	$f = V_n / n \sin\theta$	$f = 2 W_o / 2 N \sin\theta$	$f = V_n / \sin\theta$
ΔL = Change in length of brace	$\Delta L = (f L_1) / (E A_b)$	$\Delta L = (f L_1) / (E A_b)$	$\Delta L = (2 W_o L_1) / (2 N E A_b \sin\theta)$
δ = Lateral deflection of Vessel	$\delta = \Delta L / \sin\theta$	$\delta = \Delta L / \sin\theta$	$\delta = \Delta L / \sin\theta$
T = Period of vibration	$T = 2\pi(\delta / g)^{1/2}$	$T = 2\pi(\delta / g)^{1/2}$	$T = 2\pi(\delta / g)^{1/2}$

NOTES:

1. Approx POV per ASCE 7-05 ; $T_a = C_t h_n^x$



CASE 1 CASE 2

Figure 4-19. Load diagrams for vertical load distribution.

Table 4-13
Summary of loads, forces & moments at support locations

Qty of Columns	Leg No.	Case 1: At Columns		Case 2: Between Columns	
		Horiz (Vn)	Vertical (Q)	Horiz (Vn)	Vertical (Q)
6	1	+ 0.083 V	$F_D + 1.000 F_L$	+ 0.125 V	$F_D + 0.866 F_L$
	2	+ 0.208 V	$F_D + 0.500 F_L$	+ 0.250 V	F_D
	3	+ 0.208 V	$F_D - 0.500 F_L$	+ 0.125 V	$F_D - 0.866 F_L$
	4	+ 0.083 V	$F_D - 1.000 F_L$	+ 0.125 V	$F_D - 0.866 F_L$
	5	+ 0.208 V	$F_D - 0.500 F_L$	+ 0.250 V	F_D
	6	+ 0.208 V	$F_D + 0.500 F_L$	+ 0.125 V	$F_D + 0.866 F_L$
8	1	+ 0.036 V	$F_D + 1.000 F_L$	+ 0.062 V	$F_D + 0.923 F_L$
	2	+ 0.125 V	$F_D + 0.707 F_L$	+ 0.187 V	$F_D + 0.382 F_L$
	3	+ 0.213 V	F_D	+ 0.187 V	$F_D - 0.382 F_L$
	4	+ 0.125 V	$F_D - 0.707 F_L$	+ 0.062 V	$F_D - 0.923 F_L$
	5	+ 0.036 V	$F_D - 1.000 F_L$	+ 0.062 V	$F_D - 0.923 F_L$
	6	+ 0.125 V	$F_D - 0.707 F_L$	+ 0.187 V	$F_D - 0.382 F_L$
	7	+ 0.213 V	F_D	+ 0.187 V	$F_D + 0.382 F_L$
	8	+ 0.125 V	$F_D + 0.707 F_L$	+ 0.062 V	$F_D + 0.923 F_L$
10	1	+ 0.019 V	$F_D + 1.000 F_L$	+ 0.034 V	$F_D + 0.951 F_L$
	2	+ 0.075 V	$F_D + 0.809 F_L$	+ 0.125 V	$F_D + 0.587 F_L$
	3	+ 0.165 V	$F_D + 0.309 F_L$	+ 0.180 V	F_D
	4	+ 0.165 V	$F_D - 0.309 F_L$	+ 0.125 V	$F_D - 0.587 F_L$
	5	+ 0.075 V	$F_D - 0.809 F_L$	+ 0.034 V	$F_D - 0.951 F_L$
	6	+ 0.019 V	$F_D - 1.000 F_L$	+ 0.034 V	$F_D - 0.951 F_L$
	7	+ 0.075 V	$F_D - 0.809 F_L$	+ 0.125 V	$F_D - 0.587 F_L$
	8	+ 0.165 V	$F_D - 0.309 F_L$	+ 0.180 V	F_D
	9	+ 0.165 V	$F_D + 0.309 F_L$	+ 0.125 V	$F_D + 0.587 F_L$
	10	+ 0.075 V	$F_D + 0.809 F_L$	+ 0.034 V	$F_D + 0.951 F_L$
12	1	+ 0.011 V	$F_D + 1.000 F_L$	+ 0.020 V	$F_D + 0.965 F_L$
	2	+ 0.047 V	$F_D + 0.866 F_L$	+ 0.083 V	$F_D + 0.707 F_L$
	3	+ 0.119 V	$F_D + 0.500 F_L$	+ 0.145 V	$F_D + 0.258 F_L$
	4	+ 0.155 V	F_D	+ 0.145 V	$F_D - 0.258 F_L$
	5	+ 0.119 V	$F_D - 0.500 F_L$	+ 0.083 V	$F_D - 0.707 F_L$
	6	+ 0.047 V	$F_D - 0.866 F_L$	+ 0.020 V	$F_D - 0.965 F_L$
	7	+ 0.011 V	$F_D - 1.000 F_L$	+ 0.020 V	$F_D - 0.965 F_L$
	8	+ 0.047 V	$F_D - 0.866 F_L$	+ 0.083 V	$F_D - 0.707 F_L$

(*Continued*)

Table 4-13
Summary of loads, forces & moments at support locations—cont'd

Qty of Columns	Leg No.	Case 1: At Columns		Case 2: Between Columns	
		Horiz (Vn)	Vertical (Q)	Horiz (Vn)	Vertical (Q)
	9	+ 0.119 V	$F_D - 0.500\ F_L$	+ 0.145 V	$F_D - 0.258\ F_L$
	10	+ 0.155 V	F_D	+ 0.145 V	$F_D + 0.258\ F_L$
	11	+ 0.119 V	$F_D + 0.500\ F_L$	+ 0.083 V	$F_D + 0.707\ F_L$
	12	+ 0.047 V	$F_D + 0.866\ F_L$	+ 0.020 V	$F_D + 0.965\ F_L$
16	1	+ 0.004 V	$F_D + 1.000\ F_L$	+ 0.009 V	$F_D + 0.980\ F_L$
	2	+ 0.021 V	$F_D + 0.923\ F_L$	+ 0.040 V	$F_D + 0.831\ F_L$
	3	+ 0.062 V	$F_D + 0.707\ F_L$	+ 0.084 V	$F_D + 0.555\ F_L$
	4	+ 0.103 V	$F_D + 0.382\ F_L$	+ 0.115 V	$F_D + 0.195\ F_L$
	5	+ 0.120 V	F_D	+ 0.115 V	$F_D - 0.195\ F_L$
	6	+ 0.103 V	$F_D - 0.382\ F_L$	+ 0.084 V	$F_D - 0.555\ F_L$
	7	+ 0.062 V	$F_D - 0.707\ F_L$	+ 0.040 V	$F_D - 0.831\ F_L$
	8	+ 0.021 V	$F_D - 0.923\ F_L$	+ 0.009 V	$F_D - 0.980\ F_L$
	9	+ 0.004 V	$F_D - 1.000\ F_L$	+ 0.009 V	$F_D - 0.980\ F_L$
	10	+ 0.021 V	$F_D - 0.923\ F_L$	+ 0.040 V	$F_D - 0.831\ F_L$
	11	+ 0.062 V	$F_D - 0.707\ F_L$	+ 0.084 V	$F_D - 0.555\ F_L$
	12	+ 0.103 V	$F_D - 0.382\ F_L$	+ 0.115 V	$F_D - 0.195\ F_L$
	13	+ 0.120 V	F_D	+ 0.115 V	$F_D + 0.195\ F_L$
	14	+ 0.103 V	$F_D + 0.382\ F_L$	+ 0.084 V	$F_D + 0.555\ F_L$
	15	+ 0.062 V	$F_D + 0.707\ F_L$	+ 0.040 V	$F_D + 0.831\ F_L$
	16	+ 0.021 V	$F_D + 0.923\ F_L$	+ 0.009 V	$F_D + 0.980\ F_L$

Notes:

1. Radius, R_n in equations will be R_1 if a girder is used, R_c if no girder is used.

Use: ___________

I_b = ___________

r_b = ___________

A_b = ___________

- Compressive Stress, f_a

$$f_a = Q_c/A_c \le F_a$$

- Slenderness ratio, $S_r = KL_1/n'r_b$ $n' = 1$ for not pinned, 2 for pinned

$F_a =$

Tension Case

- Tension stress, f_t

$$f_t = f/A_b \le F_t$$

- Allowable tension stress, F_t,

$$F_T = 1.2(0.6)F_y$$

Sway Bracing

Note: Loads in sway bracing are tension only.

- Area of bracing required, A_{br}

$$A_{br} = f/F_t$$

- Allowable tensile stress, F_t,

$$F_T = 1.2(0.6)F_y$$

End Connections

- Shear per bolt = .5 f / number of bolts
- Shear per inch of weld = .5 f / inch of weld

Table 4-14
Allowable shear load in kips (bolts and welds per AISC steel construction manual, ASD method)

Bolt Size	A-307	A-325
.625"	3.68	7.36
.75"	5.30	10.6
.875"	7.21	14.4
1"	9.42	18.8
1.125"	11.9	23.8
WELD SIZE	E60XX	E70XX
.1875	2.39	2.78
.25	3.18	3.71
.3125	3.98	4.64
.375	4.77	5.57
.4375	5.56	6.50

Table 4-15
Suggested sizes of legs and cross-bracing

Vessel O.D. (in.)	Tan to Tan Length (in.)	Support Leg Angle Sizes (in.)	Base Plate Size (in.)	Bracing Angle Size (in.)	Bolt Size (in.)	Y (in.)
Up to 30	Up to 240	(3) 3 × 3 × ¼	6 × 6 × ⅜	2 × 2 × ¼	¾	12
	Up to 120	(4) 3 × 3 × ¼	6 × 6 × ⅜		¾	8
30 to 42	121 to 169	(4) 3 × 3 × ¼	6 × 6 × ⅜	2 × 2 × ¼	¾	10
	170 to 240	(4) 3 × 3 × ⅜	6 × 6 × ½		¾	12
	Up to 120	(4) 3 × 3 × ⅜	6 × 6 × ½	2½ × 2½ × ¼	¾	8
43 to 54	121 to 169	(4) 3 × 3 × ⅜	6 × 6 × ½		¾	10
	170 to 240	(4) 4 × 4 × ⅜	8 × 8 × ⅜		¾	12
	Up to 120	(4) 4 × 4 × ⅜	8 × 8 × ⅜	2½ × 2½ × ¼	1	8
55 to 56	121 to 169	(4) 4 × 4 × ½	8 × 8 × ½		1	10
	170 to 240	(4) 4 × 4 × ½	8 × 8 × ½		1	12
	Up to 120	(4) 5 × 5 × ⅜	9 × 9 × ½	3 × 3 × ¼	1⅛	8
67 to 78	121 to 169	(4) 5 × 5 × ⅜	9 × 9 × ½		1⅛	10
	170 to 240	(4) 6 × 6 × ½	10 × 10 × ½		1⅛	12
	Up to 120	(4) 6 × 6 × ½	10 × 10 × ½	3 × 3 × ¼	1⅛	10
79 to 80	121 to 169	(4) 6 × 6 × ½	10 × 10 × ½		1⅛	12
	170 to 240	(4) 6 × 6 × ½	10 × 10 × ½		1⅜	12
	Up to 120	(4) 6 × 6 × ½	10 × 10 × ½	3 × 3 × ⅜	1⅜	12
91 to 102	121 to 169	(6) 6 × 6 × ½	10 × 10 × ½		1⅜	12
	170 to 240	(6) 6 × 6 × ⅝	10 × 10 × ¾		1⅜	12

Post Connection Plate

See "Design of Ring Girders"

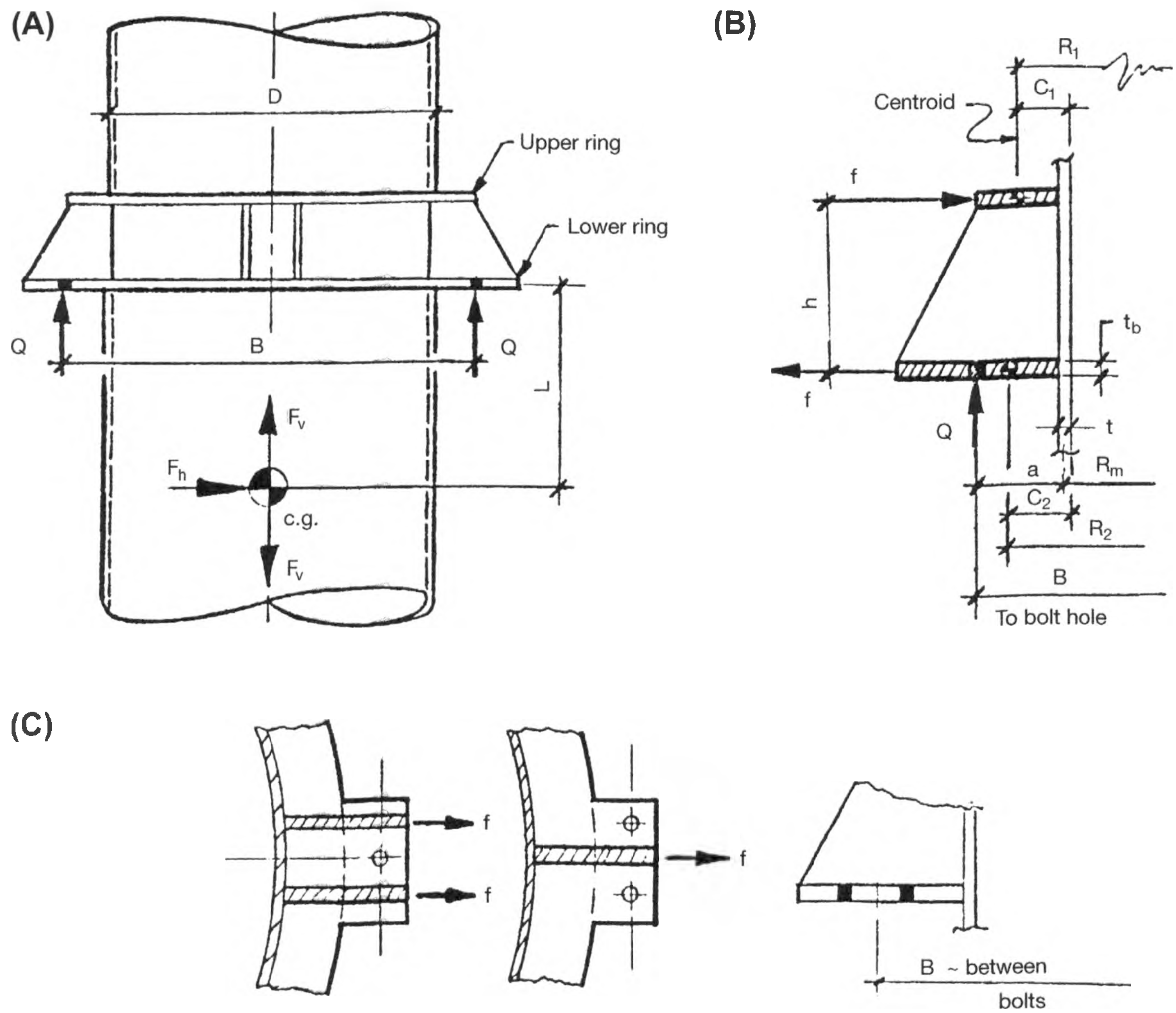
Notes

1. Cross-bracing the legs will conveniently reduce bending in legs due to overturning moments ("wind and earthquake") normally associated with unbraced legs. The lateral bracing of the legs must be sized to take lateral loads induced in the frame that would otherwise cause the legs to bend.
2. Legs may be made from angles, pipes, channels, beam sections, or rectangular tubing.
3. Legs longer than about 7 ft should be cross-braced.
4. Check to see if the cross-bracing interferes with piping from bottom head.
5. Shell stresses at the leg attachment should be investigated for local loads. For thin shells, extend "Y." Legs should be avoided as a support method for vessels with high shock loads or vibration service.

Procedure 4-6: Seismic Design – Vessel on Rings [4,5,8]

Notation

C_v, C_h = vertical/horizontal seismic factors
A_b = bearing area, in.^{2}
F_v, F_h, = vertical/horizontal seismic force, lb
N = number of support points
n = number of gussets at supports
P, P_e = internal/external pressure, psi
W = vessel weight under consideration, lb
σ_b = bending stress, psi
σ_ϕ = circumferential stress, psi
K_r = internal moment coefficient
C_r = internal tension/compression coefficient
Z = required section modulus, ring, in.^{3}
I_{1-2} = moment of inertia of rings, in.^{4}
S = code allowable stress, tension, psi
A_{1-2} = cross-sectional area, ring, in.^{2}
T_C, T_T = compression/tension loads in rings, lb
M = internal moment in rings, in.-lb



Alternate constructions

Figure 4-20. Typical dimensional data and forces for a vessel supported on rings.

M_b = bending moment in base ring, in.-lb, greater of M_x or M_y
B_p = bearing pressure, psi
Q = maximum vertical load at supports, lb
f = radial loads on rings, lb

- *Internal moment in rings, M_1 and M_2.*

Upper ring:

$$M_1 = k_r f R_1 \cos\theta$$

Lower ring:

$$M_2 = k_r f R_2 \cos\theta$$

Note: $\cos\theta$ is to be used for nonradial loads. Disregard if load f is radial.

- *Required section modulus of upper ring, Z.*

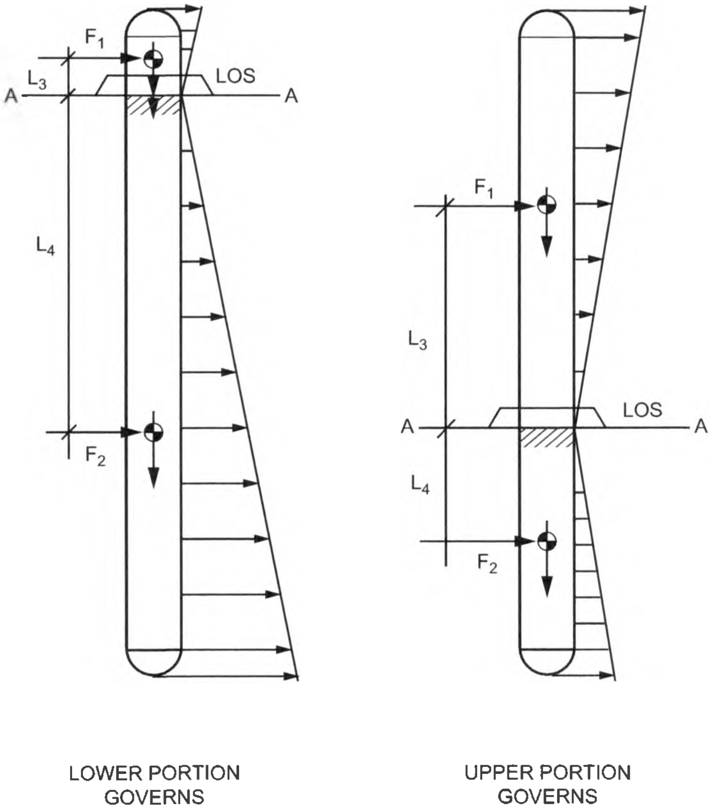
$$Z = \frac{M_1}{S}$$

Note: It is assumed the lower ring is always larger or of equal size to the upper ring.

- *Tension/compression loads in rings. Note*: In general the upper ring is in compression at the application of the loads and in tension between the loads. The lower ring is in tension at the loads and in compression between the loads. Since the governing stress is normally at the loads, the governing stresses would be:

Upper ring:

$$T_c = C_r f \cos\theta$$



INFLUENCE OF RING SUPPORT POSITIONING

M_{max} = GREATER OF ...

$M_{AA} = F_1 L_3$

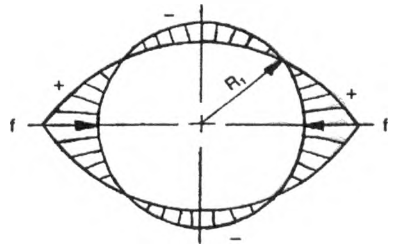
Or $F_2 L_4$

V_{max} = Greater of F_1 or F_2

Figure 4-21. Vessel supported on rings (Influence of support positioning).

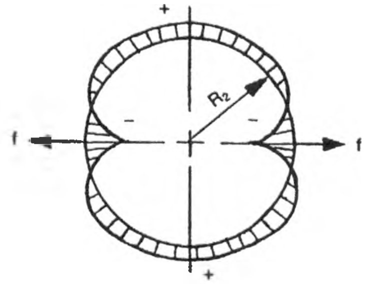
Upper Ring

Moment diagrams shown (typical)



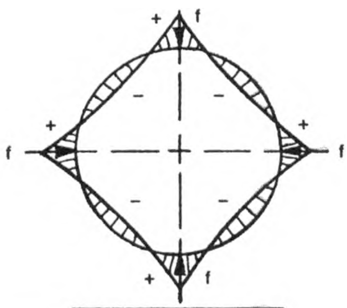
	K_r	C_r
At loads	+0.3183	0
Between loads	−0.1817	−0.5

Lower Ring

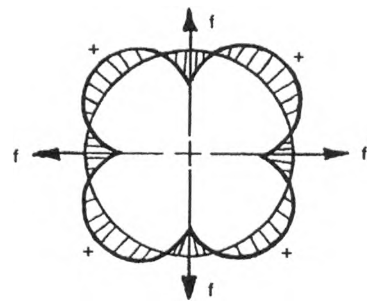


	K_r	C_r
At loads	−.3183	0
Between loads	+.1817	+.5

Two loads

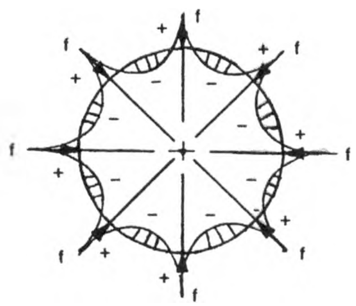


	K_r	C_r
At loads	+0.1366	−0.5
Between loads	−0.0705	−0.7071

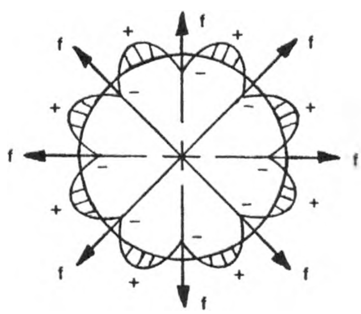


	K_r	C_r
At loads	−0.1366	+0.5
Between loads	+0.0705	+0.7071

Four loads



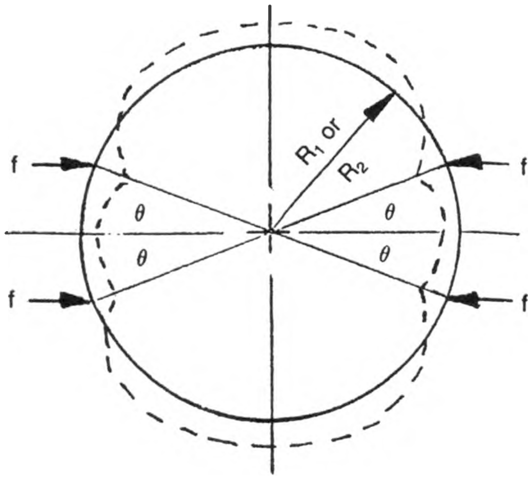
	K_r	C_r
At loads	+0.0661	−1.2071
Between loads	−0.034	−1.306



	K_r	C_r
At loads	−0.0661	+1.2071
Between loads	+0.034	+1.306

Eight Loads

Figure 4-22. Coefficients for rings.



θ	At Loads		Between Loads	
	K_r	C_r	K_r	C_r
1°	+0.619	−0.017	−0.365	−1.00
2°	+0.601	−0.041	−0.366	−0.999
3°	+0.584	−0.052	−0.363	−0.998
4°	+0.566	−0.071	−0.362	−0.997
5°	+0.550	−0.087	−0.360	−0.996
6°	+0.532	−0.105	−0.359	−0.995
7°	+0.515	−0.122	−0.357	−0.992
8°	+0.498	−0.138	−0.355	−0.990
9°	+0.481	−0.155	−0.352	−0.986
10°	+0.466	−0.171	−0.348	−0.985
15°	+0.387	−0.250	−0.329	−0.966
20°	+0.315	−0.321	−0.303	−0.940
25°	+0.254	−0.383	−0.270	−0.906
30°	+0.204	−0.433	−0.229	−0.866
35°	+0.167	−0.469	−0.183	−0.819
40°	+0.144	−0.492	−0.129	−0.766
45°	+0.137	−0.500	−0.070	−0.707

Figure 4-23. Coefficients for rings. (Signs in the table are for loads as shown. Reverse signs for loads are in the opposite direction.).

Lower ring:

$$T_T = C_r f \cos\theta$$

where C_r is the maximum positive value for T_T and the maximum negative value for T_c.

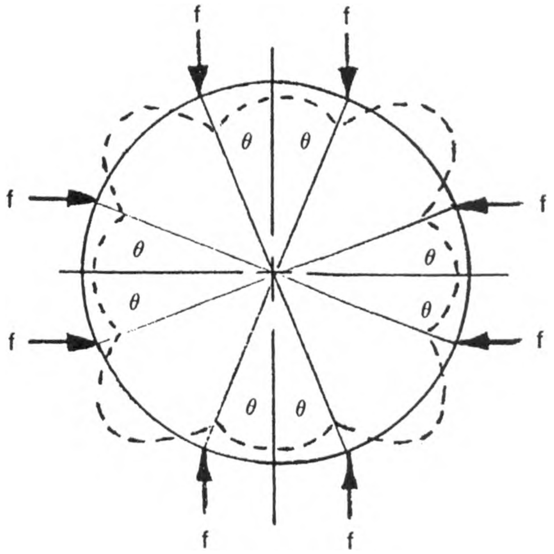
- *Maximum circumferential stress in shell,* σ_ϕ.
  Compression: in upper ring

$$\sigma_\phi = (-)\frac{P_e R_m}{t} - \frac{T_c}{A_1}$$

Tension: in lower ring

$$\sigma_\phi = \frac{PR_m}{t} + \frac{T_T}{A_2}$$

- *Maximum bending stress in shell.*



θ	At Loads		Between Loads	
	K_r	C_r	K_r	C_r
1°	+0.254	−1.018	−0.143	−1.411
2°	+0.238	−1.040	−0.143	−1.410
3°	+0.221	−1.050	−0.142	−1.409
4°	+0.206	−1.066	−0.140	−1.408
5°	+0.194	−1.079	−0.136	−1.407
6°	+0.178	−1.095	−0.135	−1.406
7°	+0.165	−1.108	−0.133	−1.405
8°	+0.153	−1.117	−0.130	−1.404
9°	+0.141	−1.130	−0.124	−1.397
10°	+0.130	−1.141	−0.119	−1.393
15°	+0.090	−1.183	−0.093	−1.366
20°	+0.069	−1.204	−0.056	−1.329
25°	+0.069	−1.204	−0.008	−1.282
30°	+0.090	−1.183	+0.049	−1.225
35°	+0.132	−1.141	+0.115	−1.158
40°	+0.194	−1.079	+0.190	−1.083
45°	+0.273	−1.000	+0.273	−1.000

Figure 4-24. Coefficients for rings. (Signs in the table are for loads as shown. Reverse signs for loads are in the opposite direction.).

Upper ring:

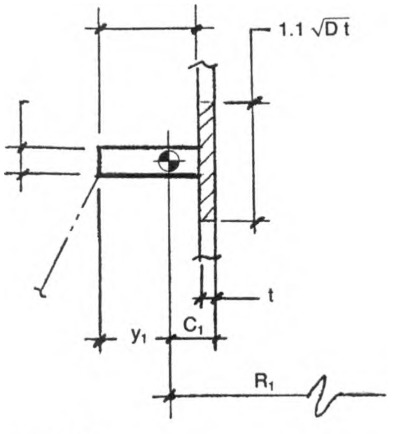
$$\sigma_b = \frac{M_1 C_1}{I_1}$$

Lower ring:

$$\sigma_b = \frac{M_2 C_2}{I_2}$$

- *Maximum bending stress in ring.*
  Upper ring:

$$\sigma_b = \frac{M_1 y_1}{I_1}$$

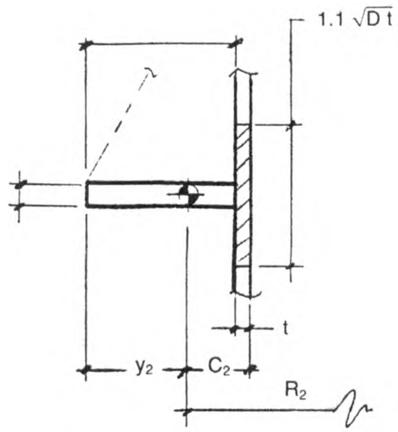


$C_1 = \frac{\Sigma AY}{\Sigma A} =$ $y_1 =$

$I_1 = \Sigma AY^2 + \Sigma I - C_1 \Sigma AY =$

Item	A	Y	Y^2	AY	AY^2	I
Shell						
Ring						
Σ						

Figure 4-25. Properties of upper ring.



$C_2 = \frac{\Sigma AY}{\Sigma A} =$ $y_2 =$

$I_2 = \Sigma AY^2 + \Sigma I - C_2 \Sigma AY =$

Item	A	Y	Y^2	AY	AY^2	I
Shell						
Ring						
Σ						

Figure 4-26. Properties of lower ring.

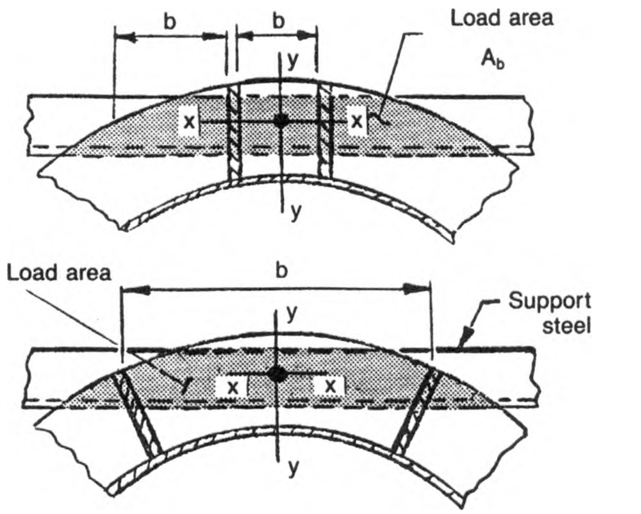


Figure 4-27. Determining the thickness of the lower ring to resist bending.

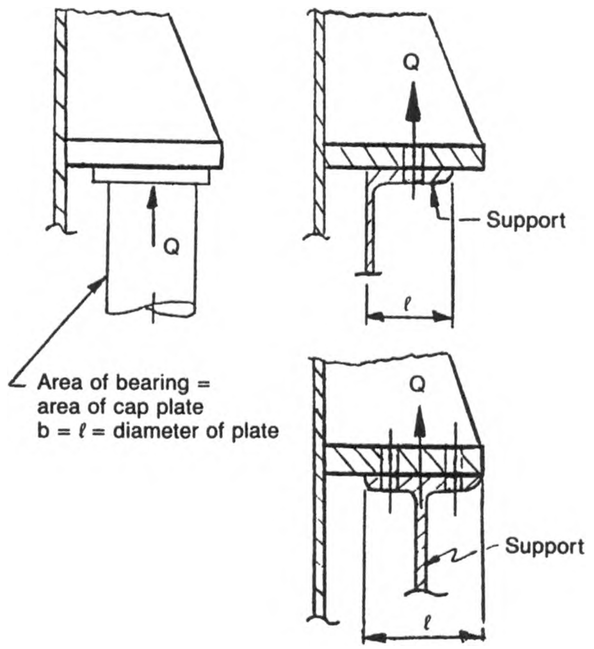


Table 4-16
Maximum bending moments in a bearing plate with gussets

$\frac{\ell}{b}$	$M_x \begin{bmatrix} x = 0.5b \\ y = \ell \end{bmatrix}$	$M_x \begin{bmatrix} x = 0.5b \\ y = 0 \end{bmatrix}$
0	0	$(-)0.500\ B_p\ell^2$
0.333	$0.0078\ B_p\ b^2$	$(-)0.428\ B_p\ell^2$
0.5	$0.0293\ B_p\ b^2$	$(-)0.319\ B_p\ell^2$
0.666	$0.0558\ B_p\ b^2$	$(-)0.227\ B_p\ell^2$
1.0	$0.0972\ B_p\ b^2$	$(-)0.119\ B_p\ell^2$
1.5	$0.1230\ B_p\ b^2$	$(-)0.124\ B_p\ell^2$
2.0	$0.1310\ B_p\ b^2$	$(-)0.125\ B_p\ell^2$
3.0–∞	$0.1330\ B_p\ b^2$	$(-)0.125\ B_p\ell^2$

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From Process Equipment Design, *Table 10.3. (See Note 2.)*

- *Properties of upper ring.*
- *Properties of lower ring.*

Lower ring:

$$\sigma_b = \frac{M_2 y_2}{I_2}$$

- *Thickness of lower ring to resist bending.*

Bearing area, A_b:

$A_b =$

Bearing pressure, B_p:

$$B_p = \frac{Q}{A_b}$$

From Table 4-16, select the equation for the maximum bending moment in the bearing plate. Use the greater of M_x or M_y.

$\frac{\ell}{b} =$

$M_b =$

Minimum thickness of lower ring, t_b:

$$t_b = \sqrt{\frac{6M_b}{S}}$$

Notes

1. Rings may induce high localized stresses in shell immediately adjacent to rings.
2. When $l/b \leq 1.5$, the maximum bending moment occurs at the junction of the ring and shell. When $l/b > 1.5$, the maximum bending moment occurs at the middle of the free edge.
3. Since the mean radius of the rings may be unknown at the beginning of computations, yet is required for determining maximum bending moment, substitute R_m as a satisfactory approximation at that stage.
4. The following values may be estimated:
   - *Ring thickness:* The thickness of each ring is arbitrary and can be selected by the designer. A suggested value is

$$t_b = 0.3\sqrt[3]{\frac{M_{max}}{S}}$$

   - *Ring spacing:* Ring spacing is arbitrary and can be selected by the designer. A suggested minimum value is

$$h = B - D$$

   - *Ring depth:* The depth of ring cannot be computed directly, but must be computed by successive approximations. As a first trial,

$$d = 2.1\sqrt{\frac{M_{max}}{t_r S}}$$

Procedure 4-7: Seismic Design – Vessel on Lugs [5,8–13]

Notation

R_m = center line radius of shell, in.
N = number of equally spaced lugs
W = weight of vessel plus contents, lb
f = radial load, lb
F_h = horizontal seismic force, lb
F_v = vertical seismic force, lb
V_h = horizontal shear per lug, lb
V_v = vertical shear per lug, lb
Q = vertical load on lugs, lb
γ, β = coefficients
M_c = external circumferential moment, in.–lb
M_L = external longitudinal moment, in.-lb
M_ϕ = internal bending moment, circumferential, in.-lb/in.
M_x = internal bending moment longitudinal, in. -lb/in.
N_ϕ = membrane force in shell, circumferential, lb/in.
N_x = membrane force in shell, longitudinal, lb/in.
P = internal pressure, psi
C_h = horizontal seismic factor
C_v = vertical seismic factor
C_c, C_i = multiplication factors for N_ϕ and N_x for rectangular attachments
K_C, K_1 = coefficients for determining β for moment loads on rectangular areas

K_1, K_2 = coefficients for determining β for radial loads on rectangular areas
K_n, K_b, = stress concentration factors (see Note 5)
σ_ϕ = circumferential stress, psi
σ_x = longitudinal stress, psi
t_s = thickness of shell, in.
t_p = thickness of reinforcing pad, in.
α = coefficient of thermal expansion, in/in/°F
ζ = radial deflection, in

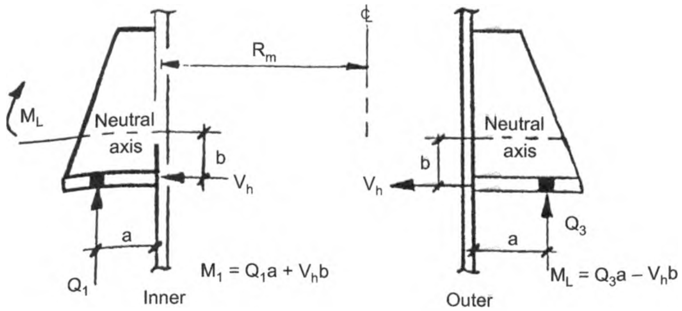


Figure 4-28. Dimensions and forces for support lug.

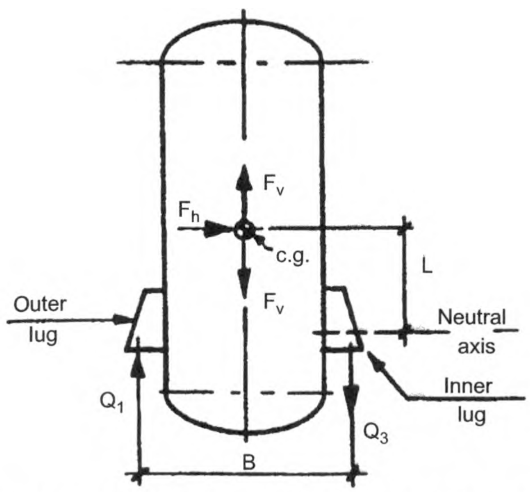


Figure 4-29. Case 1: Lugs below the center of gravity.

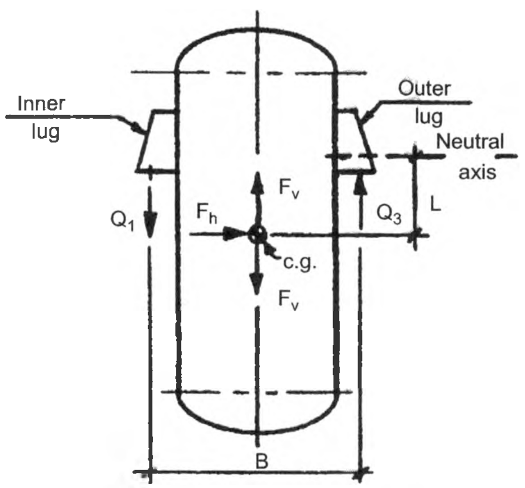


Figure 4-30. Case 2: Lugs above the center of gravity.

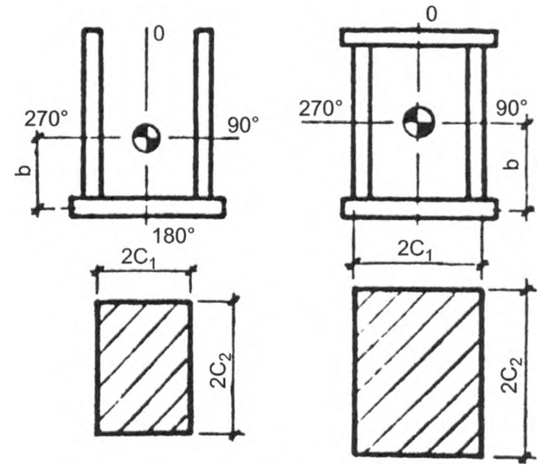
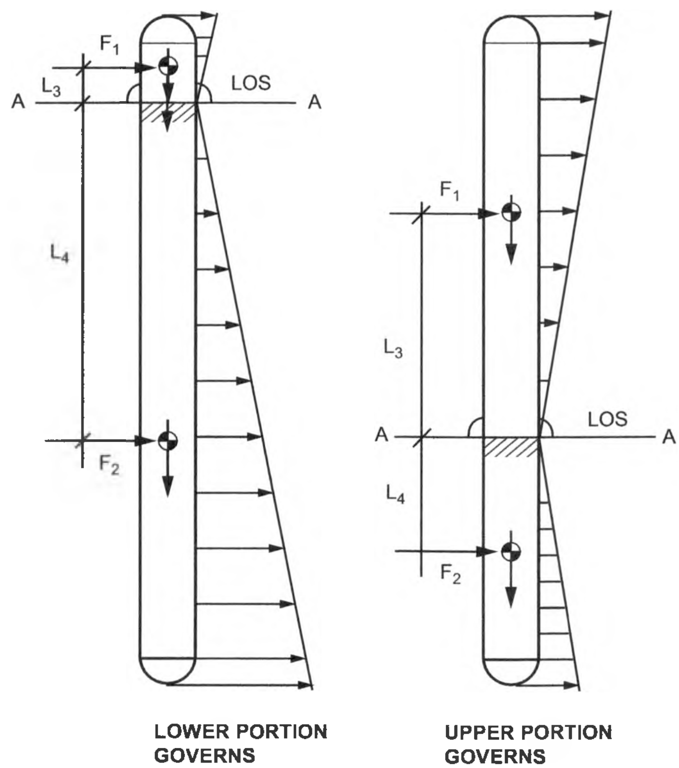


Figure 4-31. Area of loading.



M_{max} = GREATER OF ...

$M_{AA} = F_1 L_3$

Or $F_2 L_4$

V_{max} = Greater of F_1 or F_2

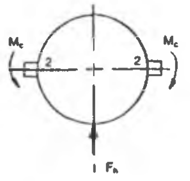
Figure 4-32. Vessel supported on lugs (Influence of support positioning).

Step 1: Compute forces and moments

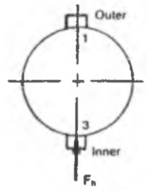
FORCES		
Lateral force	$F_h = C_h W$	
Horizontal shear per lug	$V_h = F_h/N$	
Vertical force	$F_v = (1 + C_v)W$	
Vertical shear per lug	$V_v = F_v/N$	

LOAD DIAGRAMS

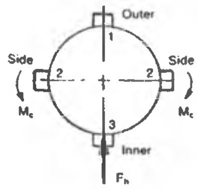
Case 1: Two Lugs



Case 2: Two Lugs



Case 3: Four Lugs



	Case 1: Two Lugs	Case 2: Two Lugs	Case 3: Four Lugs
VERTICAL LOADS AT LUGS, Q			
Outer		$Q_1 = V_v - \frac{F_h L}{B}$	$Q_1 = V_v - \frac{F_h L}{B}$
Sides	$Q_2 = V_V$		$Q_2 = V_V$
Inner		$Q_3 = V_v + \frac{F_h L}{B}$	$Q_3 = V_v + \frac{F_h L}{B}$
LONGITUDINAL MOMENT, M_L			
Outer		$M_{L1} = Q_1 a - V_h b$	$M_{L1} = Q_1 a - V_h b$
Sides	$M_{L2} = Q_2 a$		$M_{L2} = Q_2 a$
Inner		$M_{L3} = Q_3 a + V_h b$	$M_{L3} = Q_3 a + V_h b$
CIRCUMFERENTIAL MOMENT, M_c			
Sides	$M_c = V_h a$		$M_c = V_h a$

Step 2: Compute geometric parameters

$\gamma = R_m/t$	$\beta_1 = C_1/R_m$	$\beta_2 = C_2/R_m$	β_1/β_2

Step 3: Compute equivalent β values (values of C_L, C_C, K_L, and K_C from Tables 4-17 and 4-18)

β Values for Longitudinal Moment

Values of β		C_L	K_L	β
$\beta_a = \sqrt[3]{\beta_1 \beta_2^2}$	N_ϕ			
$\beta_b = \sqrt[3]{\beta_1 \beta_2^2}$	N_x			
$\beta_c = K_L \sqrt[3]{\beta_1 \beta_2^2}$	M_ϕ			
$\beta_d = K_L \sqrt[3]{\beta_1 \beta_2^2}$	M_x			

β Values for Circumferential Moment

Values of β		C_c	K_c	β
$\beta_e = \sqrt[3]{\beta_1^2 \beta_2}$	N_ϕ			
$\beta_f = \sqrt[3]{\beta_1^2 \beta_2}$	N_x			
$\beta_g = K_c \sqrt[3]{\beta_1^2 \beta_2}$	M_ϕ			
$\beta_h = K_c \sqrt[3]{\beta_1^2 \beta_2}$	M_x			

Step 4: Compute stresses

Forces	Figure	β	Values from Figure	Forces and Moments	Stress
			Longitudinal Moment		
Membrane	7.23A	$\beta_a =$	$\frac{N_\phi R_m^2 \beta}{M_L} = (\ )$	$N_\phi = \frac{(\ )C_L M_L}{R_m^2 \beta} =$	$\sigma_\phi = \frac{K_n N_\phi}{t_s} =$
	7.23B	$\beta_b =$	$\frac{N_x R_m^2 \beta}{M_L} = (\ )$	$N_x = \frac{(\ )C_L M_L}{R_m^2 \beta} =$	$\sigma_x = \frac{K_n N_x}{t_s} =$
Bending	7.24A	$\beta_c =$	$\frac{M_\phi R_m \beta}{M_L} = (\ )$	$M_\phi = \frac{(\ )M_L}{R_m \beta} =$	$\sigma_\phi = \frac{6K_b M_\phi}{t_s^2} =$
	7.24B	$\beta_d =$	$\frac{M_x R_m \beta}{M_L} = (\ )$	$M_x = \frac{(\ )M_L}{R_m \beta} =$	$\sigma_x = \frac{6K_b M_x}{t_s^2} =$
			Circumferential Moment		
Membrane	7.25A	$\beta_e =$	$\frac{N_\phi R_m^2 \beta}{M_c} = (\ )$	$N_\phi = \frac{(\ )C_c M_c}{R_m^2 \beta} =$	$\sigma_\phi = \frac{K_n N_\phi}{t_s} =$
	7.25B	$\beta_f =$	$\frac{N_x R_m^2 \beta}{M_c} = (\ )$	$N_x = \frac{(\ )C_c M_c}{R_m^2 \beta} =$	$\sigma_x = \frac{K_n N_x}{t_s} =$
Bending	7.26A	$\beta_g =$	$\frac{M_\phi R_m \beta}{M_c} = (\ )$	$M_\phi = \frac{(\ )M_c}{R_m \beta} =$	$\sigma_\phi = \frac{6K_b M_\phi}{t_s^2} =$
	7.26B	$\beta_h =$	$\frac{M_x R_m \beta}{M_c} = (\ )$	$M_x = \frac{(\ )M_c}{R_m \beta} =$	$\sigma_x = \frac{6K_b M_x}{t_s^2} =$

Table 4-17
Coefficients for circumferential moment, M_c

β_1/β_2	γ	C_c for N_ϕ	C_c for N_x	K_c for M_ϕ	K_c for M_x
	15	0.31	0.49	1.31	1.84
	50	0.21	0.46	1.24	1.62
0.25	100	0.15	0.44	1.16	1.45
	200	0.12	0.45	1.09	1.31
	300	0.09	0.46	1.02	1.17
	15	0.64	0.75	1.09	1.36
	50	0.57	0.75	1.08	1.31
0.5	100	0.51	0.76	1.04	1.16
	200	0.45	0.76	1.02	1.20
	300	0.39	0.77	0.99	1.13
	15	1.17	1.08	1.15	1.17
	50	1.09	1.03	1.12	1.14
1	100	0.97	0.94	1.07	1.10
	200	0.91	0.91	1.04	1.06
	300	0.85	0.89	0.99	1.02
	15	1.70	1.30	1.20	0.97
	50	1.59	1.23	1.16	0.96
2	100	1.43	1.12	1.10	0.95
	200	1.37	1.06	1.05	0.93
	300	1.30	1.00	1.00	0.90
	15	1.75	1.31	1.47	1.08
	50	1.64	1.11	1.43	1.07
4	100	1.49	0.81	1.38	1.06
	200	1.42	0.78	1.33	1.02
	300	1.36	0.74	1.27	0.98

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Table 4-18
Coefficients for longitudinal moment, M_L

β_1/β_2	γ	C_L for N_ϕ	C_L for N_x	K_L for M_ϕ	K_L for M_x
	15	0.75	0.43	1.80	1.24
	50	0.77	0.33	1.65	1.16
0.25	100	0.80	0.24	1.59	1.11
	200	0.85	0.10	1.58	1.11
	300	0.90	0.07	1.56	1.11
	15	0.90	0.76	1.08	1.04
	50	0.93	0.73	1.07	1.03
0.5	100	0.97	0.68	1.06	1.02
	200	0.99	0.64	1.05	1.02
	300	1.10	0.60	1.05	1.02
	15	0.89	1.00	1.01	1.08
	50	0.89	0.96	1.00	1.07
1	100	0.89	0.92	0.98	1.05
	200	0.89	0.99	0.95	1.01
	300	0.95	1.05	0.92	0.96
	15	0.87	1.30	0.94	1.12
	50	0.84	1.23	0.92	1.10
2	100	0.81	1.15	0.89	1.07
	200	0.80	1.33	0.84	0.99
	300	0.80	1.50	0.79	0.91
	15	0.68	1.20	0.90	1.24
	50	0.61	1.13	0.86	1.19
4	100	0.51	1.03	0.81	1.12
	200	0.50	1.18	0.73	0.98
	300	0.50	1.33	0.64	0.83

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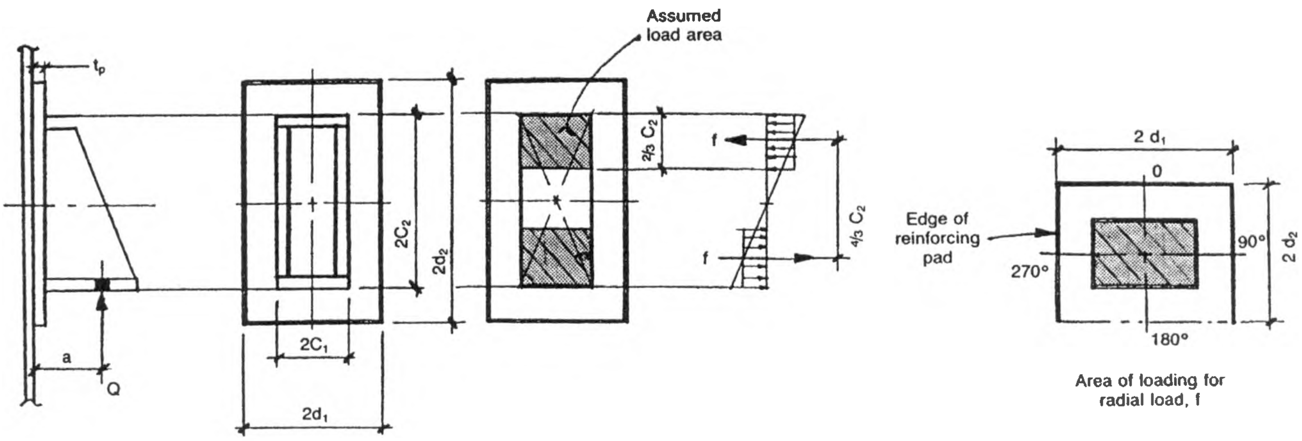
Analysis when Reinforcing Pads are Used



$$2d_{11}\ \text{max} = \frac{4d_1C_2}{3C_1} =$$

Figure 4-33. Dimensions of load areas for radial loads.

Step 1: Compute radial loads f

	Case 1	Case 2	Case 3
Outer		$f_1 = \frac{3M_{L1}}{4C_2}$	$f_1 = \frac{3M_{L1}}{4C_2}$
Sides	$f_2 = \frac{3M_{L2}}{4C_2}$		$f_2 = \frac{3M_{L2}}{4C_2}$
Inner		$f_3 = \frac{3M_{L3}}{4C_2}$	$f_3 = \frac{3M_{L3}}{4C_2}$

Step 2: Compute geometric parameters.

At Edge of Attachment	At Edge of Pad
$R_m = \frac{\text{I.D.} + t_s + t_p}{2}$	$R_m = \frac{\text{I.D.} + t_s}{2}$
$t = \sqrt{t_s^2 + t_p^2}$	$t = t_s$
$\gamma = R_m/t$	$\gamma = R_m/t$
$\beta_1 = C_1/R_m$	$\beta_1 = d_1/R_m$
$\beta_2 = 4C_2/3R_m$	$\beta_2 = d_2/R_m$
β_1/β_2	β_1/β_2

Step 3: Compute equivalent β values.

Table 4-19

Four values of β are computed for use in determining N_ϕ, N_x, M_ϕ, and M_x as follows. The values of K_1 and K_2 are taken from Table 4-19.		Values of coefficient K_1 and K_2		
$\beta_1/\beta_2 \geq 1$	β		K_1	K_2
$\beta = [1 - \frac{1}{3}(\frac{\beta_1}{\beta_2} - 1)(1 - k_1)]\sqrt{\beta_1\beta_2}$	β_a for $N_\phi =$	$N\phi$	0.91	1.48
	β_b for $N_x =$	N_x	1.68	1.2
$\beta_1/\beta_2 < 1$				
$\beta = [1 - \frac{4}{3}(1 - \frac{\beta_1}{\beta_2})(1 - k_2)]\sqrt{\beta_1/\beta_2}$	β_c for $M_\phi =$	$M\phi$	1.76	0.88
	β_d for $M_x =$	M_x	1.2	1.25

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Step 4: Compute stresses for a radial load.

Radial Load	Figure	β	Values from Figure	Forces and Moments	Stress
Membrane	7-21A	$\beta_a =$	$\frac{N_\phi R_m}{f} = (\ )$	$N_\phi = \frac{(\ )f}{R_m} =$	$\sigma_\phi = \frac{K_n N_\phi}{t} =$
	7-21B	$\beta_b =$	$\frac{N_x R_m}{f} = (\ )$	$N_x = \frac{(\ )f}{R_m} =$	$\sigma_x = \frac{K_n N_x}{t} =$
Bending	7-22A	$\beta_c =$	$\frac{M_\phi}{f} = (\ )$	$M_\phi = (\ )f =$	$\sigma_\phi = \frac{6K_b M_\phi}{t^2} =$
	7-22B	$\beta_d =$	$\frac{M_x}{f} = (\ )$	$M_x = (\ )f =$	$\sigma_x = \frac{6K_b M_x}{t^2} =$

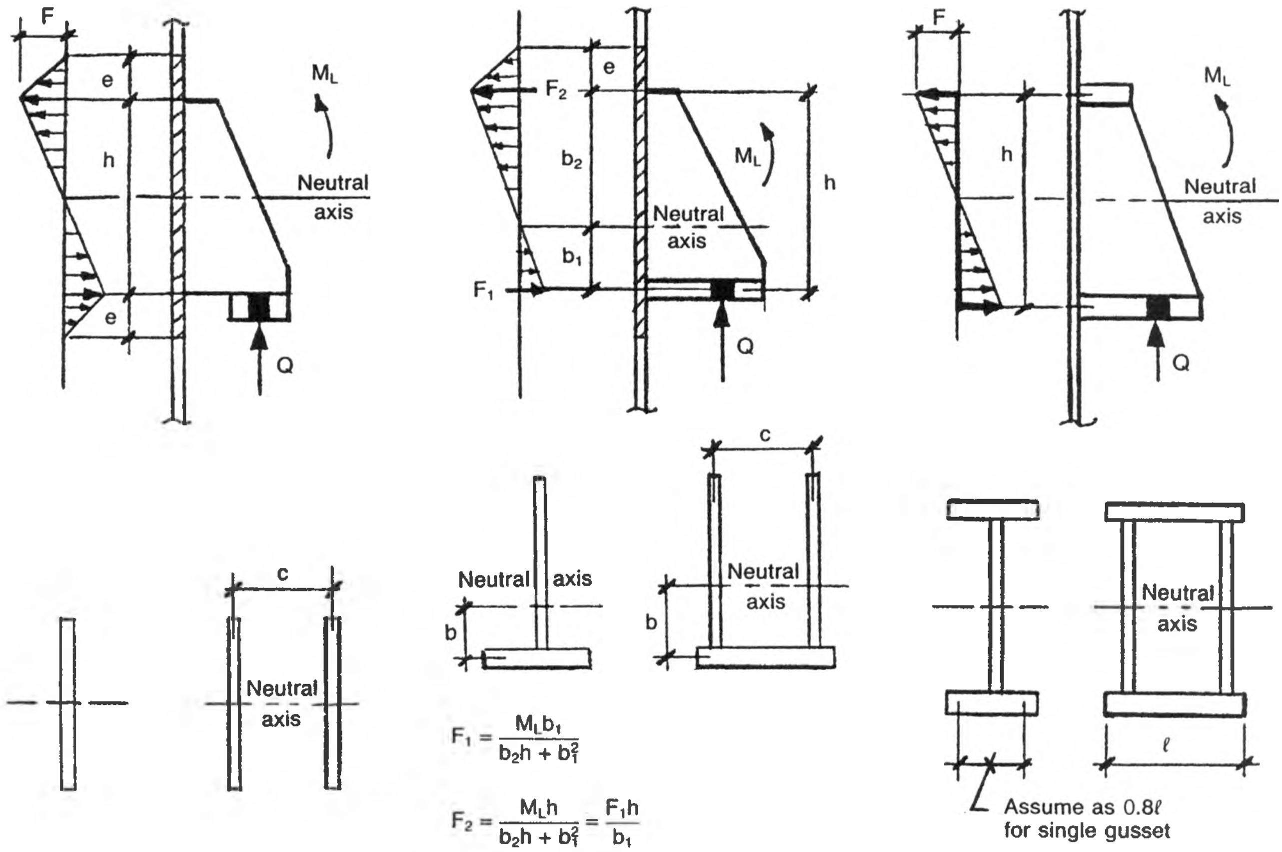
COMBINING STRESSES

WITHOUT REINFORCING PAD										
Stress Due To			σ_x				σ_ϕ			
			0°	90°	180°	270°	0°	90°	180°	270°
Longitudinal moment, M_L	Membrane	N_ϕ					+		−	
		N_x	+		−					
	Bending	M_ϕ					+		−	
		M_x	+		−					
Circumferential moment, M_c	Membrane	N_ϕ						+		−
		N_x		+		−				
	Bending	M_ϕ						+		−
		M_x		+		−				
Internal pressure, P	$\sigma_\phi = \frac{PR_m}{t_s}$						+	+	+	+
	$\sigma_x = \frac{PR_m}{2t_s}$		+	+	+	+				
Total	$\sum$									

WITH REINFORCING PAD										
Stress Due To			σ_x				σ_ϕ			
			0°	90°	180°	270°	0°	90°	180°	270°
Radial load, f	Membrane	N_ϕ					+	+	+	+
		N_x	+	+	+	+				
	Bending	M_ϕ					+	+	+	+
		M_x	+	+	+	+				
Internal pressure, P	$\sigma_\phi = \frac{PR_m}{t_s}$						+	+	+	+
	$\sigma_x = \frac{PR_m}{2t_s}$		+	+	+	+				
Total	$\sum$		+	+	+	+	+	+	+	+

NOTES

1. Make sure to remain consistent by lug, that is, that all loadings are from the same lug. This may require several trials to determine the worst case.
2. The calculations for combining stresses with a reinforcing pad should be completed for stresses at the edge of attachment as well as at the edge of the pad. For thinner shells the stress at the edge of the pad will usually govern.



$e = .78\sqrt{R_m}\,t$ but $< 12t$

$$F = \frac{6M_L}{(h + e)(h + 2e)}\left[\frac{\cos\theta}{n}\right]$$

Type 1

$$F_1 = \frac{M_L b_1}{b_2 h + b_1^2}$$

$$F_2 = \frac{M_L h}{b_2 h + b_1^2} = \frac{F_1 h}{b_1}$$

At top $F = \frac{F_2}{2e}\left[\frac{\cos\theta}{n}\right]$.

At bottom $f = \frac{F_1}{\ell}$

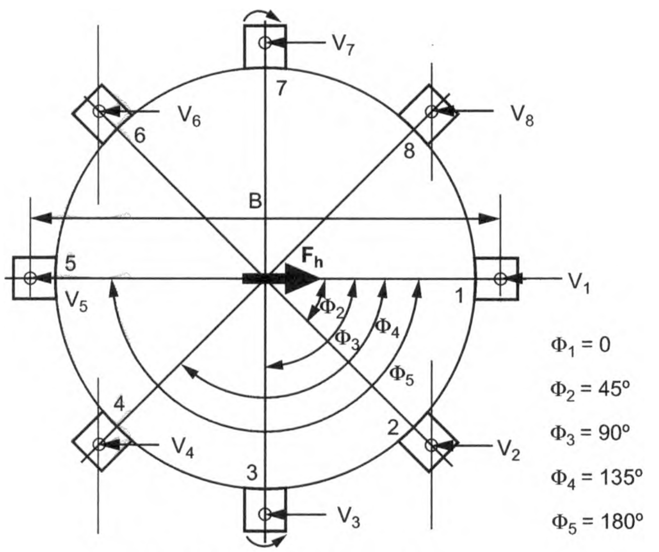
Type 2

$$F = \frac{M_L}{h}$$

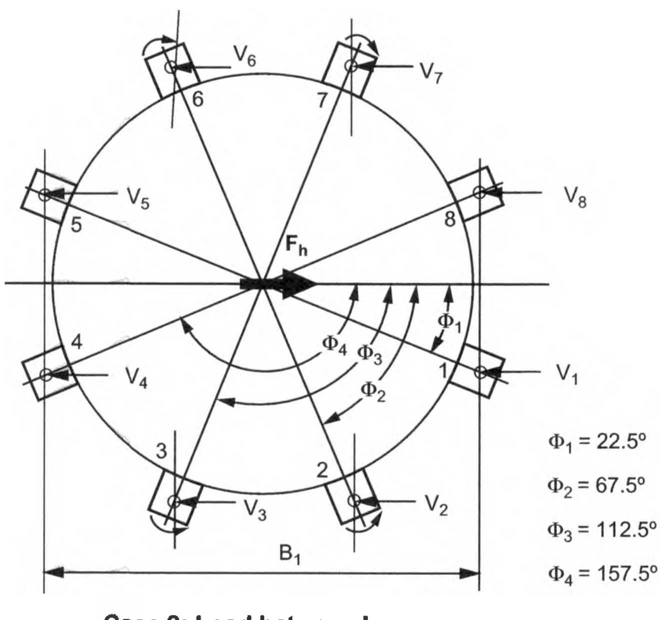
$$f = \frac{F}{\ell}$$

Type 3

Figure 4-34. Radial loads F and f.



Case 1: Load through lugs



Case 2: Load between lugs

Figure 4-35. Vessel supported on (8) lugs.

Design of Vessel Supported on (8) Lugs

Item	Formula	Calculation
Lateral Force	$F_h = C_h W$	
Horizontal Shear/Lug	$V_h = F_h / N$	
Vertical Force	$F_V = (1 + C_V) W$	
Overturning Moment	$M = F_h L$	
Dead Load/Lug	$V_V = F_V / N$	
Worst Case Vertical Live Load per Lug	$F_L = 4 M / N B$	
VERTICAL LIVE LOAD ON EACH LUG, F_{Ln}		
LUG	**CASE 1**	**CASE 2**
1	$- F_L$	$-.924 F_L$
2	$-.707 F_L$	$-.383 F_L$
3	0	$+.383 F_L$
4	$+.707 F_L$	$+.924 F_L$
5	$+ F_L$	$+.924 F_L$
6	$-.707 F_L$	$+.383 F_L$
7	0	$-.383 F_L$
8	$+.707 F_L$	$-.924 F_L$
Formulas in Table are based on the following equations;		
CASE 1	$F_L = 4 M \cos\phi_n / N B$	
CASE 2	$F_L = 4 M \cos\phi_n / N B_1$	
Vertical Load on any Lug	$Q_n = V_V + F_{Ln}$	
Worst Case Load per Lug	$Q = V_V + F_L$	
Longitudinal Moment for any Given Lug	$M_{Ln} = Q_n a +/- V_h b$	
Longitudinal Moment for any Worst Case	$M_L = Q a +/- V_h b$	

Design of Vessel Supported on (8) Lugs (Example)

ITEM	FORMULA	CALCULATION	Data
Lateral Force	$F_h = C_h W$	$F_h = .1 (141^K) = 14.1^K$	$C_h = .1$
Horizontal Shear/Lug	$V_h = F_h / N$	$V_h = 14.1^K / 8 = 1.76^K$	$C_V = .2$
Vertical Force	$F_V = (1 + C_V) W$	$F_V = (1 + .2) 141^K = 169.2^K$	$W = 141^K$
Overturning Moment	$M = F_h L$	$M = 14.1^K (84") = 1184$ In-Kips	$L = 84"$
Dead Load/Lug	$V_V = F_V / N$	$V_V = 169.2^K / 8 = 21.15^K$	$a = 12"$
Worst Case Vertical Live Load per Lug	$F_L = 4 M / N B$	$F_L = 4(1184"^K) / (8) 160.25" = 3.69^K$	$b = 6.24"$
			$B = 160.25"$
			$B_1 = 113.31$
VERTICAL LIVE LOAD ON EACH LUG, F_{Ln}			
LUG	**CASE 1**	**CASE 2**	
1	$- F_L$	$-.924 F_L$	
2	$-.707 F_L$	$-.383 F_L$	
3	0	$+.383 F_L$	
4	$+.707 F_L$	$+.924 F_L$	
5	$+ F_L$	$+.924 F_L$	
6	$-.707 F_L$	$+.383 F_L$	
7	0	$-.383 F_L$	
8	$+.707 F_L$	$-.924 F_L$	
Formulas in Table are based on the following equations;			
CASE 1	$F_L = 4 M \cos\phi_n / N B$		
CASE 2	$F_L = 4 M \cos\phi_n / N B_1$		
Vertical Load on any Lug	$Q_n = V_V + F_{Ln}$		
Worst Case Load per Lug	$Q = V_V + F_L$	$Q = 21.15 + 3.69 = 24.84^K$	
Longitudinal Moment for any Given Lug	$M_{Ln} = Q_n a +/- V_h b$		
Longitudinal Moment for any Worst Case	$M_L = Q a +/- V_h b$	$M_L = 24.84^K (12") + 1.76^K (6.24") = 309$ in-Kips	

Check lug for radial thermal expansion, ζ_r

DT = Design temperature, °F
R = Radius = B / 2
α = Coefficient of thermal expansion, in/in/ °F
ΔT = Change in temperature from 70 °F
$\zeta_r = \alpha \, \Delta T \, R =$

Example;

$R = 80.125$ in
$DT = 925°F$
$\alpha = 7.9(10^{-6})$ in/in/°F
$\Delta T = 925 - 70 = 855°F$
$\zeta_r = 7.9(10^{-6})\ 855\ (80.125) = .541$ in

Use slotted holes!

Size of anchor bolts Required, A_r

Due to Overturning Moment

$$A_r = [(4\,M/B) - W][1/(N_b\,S_b)]$$

N_b = Number of anchor bolts
If A_r is negative, there is no uplift.

Due to Shear

Shear / lug, f_S

$$f_S = F_h/N_b$$

$$A_r = f_S/F_S$$

Use minimum size of anchor bolts of 0.75 in diameter.

Notes

1. A change in location of the c.g. for various operating levels can greatly affect the moment at lugs by increasing or decreasing the "L" dimension. Different levels and weights should be investigated for determining worst case (i.e., full, half-full, empty, etc.)
2. This procedure ignores effects of sliding friction between lugs and beams during heating/cooling cycles. These effects will be negligible for small-diameter vessels, relatively low operating temperatures, or where slide plates are used to reduce friction forces. Other cases should be investigated.
3. Since vessels supported on lugs are commonly located in structures, the earthquake effects will be dependent on the structure as well as on the vessel. Thus horizontal and vertical seismic factors must be provided.
4. If reinforcing pads are used to reduce stresses in the shell or a design that uses them is being checked, then Bijlaard recommends an analysis that converts moment loadings into equivalent radial loads. The attachment area is reduced about two-thirds. Stresses at the edge of load area and stresses at the edge of the pad must be checked. See "Analysis When Reinforcing Pads are Used."
5. Stress concentration factors are found in the procedure on local stresses.
6. To determine the area of attachment, see "Attachment Parameters." Please note that if a top (compression) plate is not used, then an equivalent rectangle that is equal to the moment of inertia of the attachment and whose width-to-height ratio is the same must be determined. The neutral axis is the rotating axis of the lug passing through the centroid.
7. Stiffening effects due to proximity to major stiffening elements, though desirable, have been neglected in this procedure.
8. Assume effects of radial loads as additive to those due to internal pressure, even though the loadings may be in the opposite directions. Although conservative, they will account for the high discontinuity stresses immediately adjacent to the lugs.
9. In general, the smaller the diameter of the vessel, the further the distribution of stresses in the circumferential direction. In small diameter vessels, the longitudinal stresses are confined to a narrow band. The opposite becomes true for larger-diameter vessels or larger R_m/t ratios.
10. If shell stresses are excessive, the following methods may be utilized to reduce the stresses:
    a. Add more lugs.
    b. Add more gussets.
    c. Increase angle θ between gussets.
    d. Increase height of lugs, h
    e. Add reinforcing pads under lugs.

f. Increase thickness of shell course to which lugs are attached.
g. Add top and bottom plates to lugs or increase width of plates.
h. Add circumferential ring stiffeners at top and bottom of lugs.

Procedure 4-8: Seismic Design – Vessel on Skirt [1,2,3]

Notation

T = period of vibration, sec
S_l = code allowable stress, tension, psi
H = overall height of vessel from bottom of base plate, ft
h_x = height from base to center of section or e.g. of a concentrated load, ft
h_i = height from base to plane under consideration, ft
α, β, γ = coefficients from Table 4-20 for given plane based on h_x/H
W_x = total weight of section, kips
W = weight of concentrated load or mass, kips
W_o = total weight of vessel, operating, kips
W_h = total weight of vessel above the plane under consideration, kips
w_x = uniformly distributed load for each section, kips/ft
F_x = lateral force applied at each section, kips
V = base shear, kips
V_x = shear at plane x, kips
M_x = moment at plane x, ft-kips
M_b = overturning moment at base, ft-kips
D = mean shell diameter of each section, ft or in.
E = modulus of elasticity at design temperature, 10^6 psi
E_l = joint efficiency
t = thickness of vessel section, in.
P_i = internal design pressure, psi
P_e = external design pressure, psi
$\Delta\alpha, \Delta\gamma$ = difference in values of α and γ from top to bottom of any given section
l_x = length of section, ft
σ_{xt} = longitudinal stress, tension, psi
σ_{xc} = longitudinal stress, compression, psi
R_o = outside radius of vessel at plane under consideration, in.
A = code factor for determining allowable compressive stress, B
B = code allowable compressive stress, psi
F = lateral seismic force for uniform vessel, kips
C_h = horizontal seismic factor

Cases

Case 1: Uniform Vessels. For vessels of uniform cross section without concentrated loads (i.e., reboilers, packing, large liquid sections, etc.) weight can be assumed to be uniformly distributed over the entire height.

W_o =

H =

D =

t =

$$T = 0.0000265\left(\frac{H}{D}\right)^2\sqrt{\frac{W_oD}{Ht}}$$

Note: P.O.V. may be determined from chart in Figure 4-6 H and D are in feet; t is in inches.

$V = C_hW_o$

$F = V$

$M_b = 2/3(FH)$

Moment at any height h_j

$$M_a = F\left(\frac{2H}{3} - h_i\right)$$

Case 2: Nonuniform Vessels

Procedure for finding period of vibration, moments, and forces at various planes for nonuniform vessels.

A "nonuniform" vertical vessel is one that varies in diameter, thickness, or weight at different elevations. This procedure distributes the seismic forces and thus base shear, along the column in proportion to the weights of

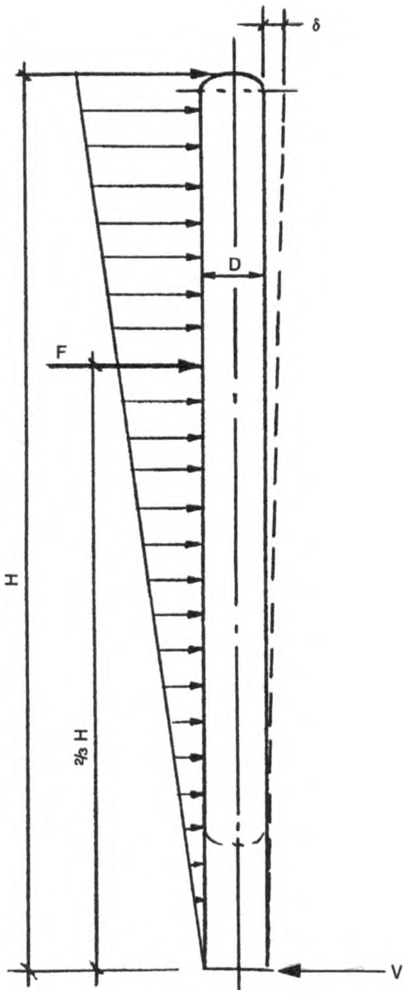


Figure 4-36. Typical dimensional data, forces, and loadings on a uniform vessel supported on a skirt (δ = deflection).

each section. The results are a more accurate and realistic distribution of forces and accordingly a more accurate period of vibration. The procedure consists of two main steps:

Step 1: *Determination of period of vibration (P.O.V.), T.* Divide the column into sections of uniform weight and diameter not to exceed 20% of the overall height. A uniform weight is calculated for each section. Diameter and thicknesses are taken into account through factors α and γ. Concentrated loads are handled as separate sections and not combined with other sections. Factor β will proportion effects of concentrated loads. The calculation form is completed for each section from left to right, then totaled to the bottom. These totals are used to determine T (P.O.V.) and the P.O.V. in turn is used to determine V and F_t.

Step 2: *Determination of forces, shears, and moments.* Again, the vessel is divided into major sections as in Step 1; however, longer sections should be further subdivided into even increments. For these calculations, sections should not exceed 10% of height. Remember, the moments and weights at each plane will be used in determining what thicknesses are required. It is convenient to work in 8 to 10 foot increments to match shell courses. Piping, trays, platforms, insulation, fireproofing, and liquid weights should be added into the weights of each section where they occur. Overall weights of sections are used in determining forces, not uniform weights. Moments due to eccentric loads are added to the overall moment of the column.

Notes for nonuniform vessels

1. Combine moments with corresponding weights at each section and use allowable stresses to determine required shell and skirt thicknesses at the elevation.
2. $\sum \omega \Delta \alpha$ and $W\beta/H$ are separate totals and are combined in computation of P.O.V.
3. $(D/10)^3$ is used in this expression if kips are used. Use $(D)^3$ if lb are used.
4. For vessels having a lower section several times the diameter of the upper portion and where the lower portion is short compared to the overall height, the P.O.V. can more accurately be determined bv finding the P.O.V. of the upper section alone (see Figure 4.38a).
5. For vessels where R/t is large in comparison to the supporting skirt, the P.O.V, calculated by this method may be overly conservative. More accurate methods may be employed (see Figure 4.38b).
6. Make sure to add moment due to any eccentric loads to total moment

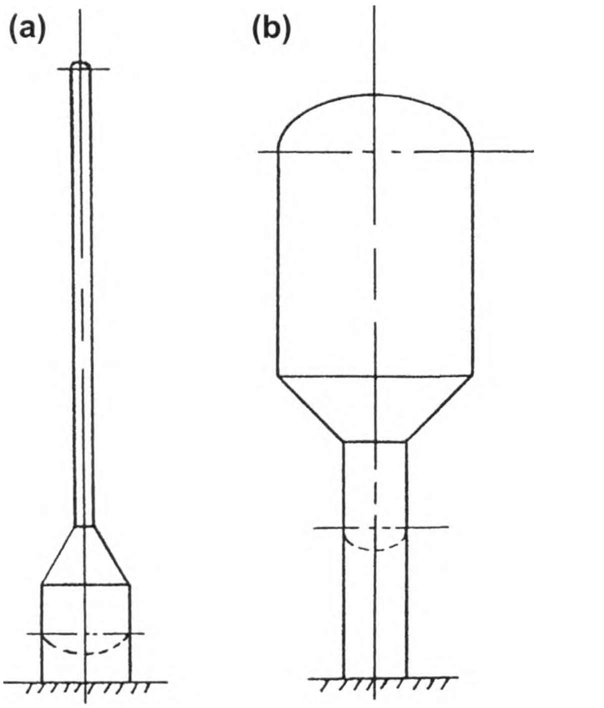
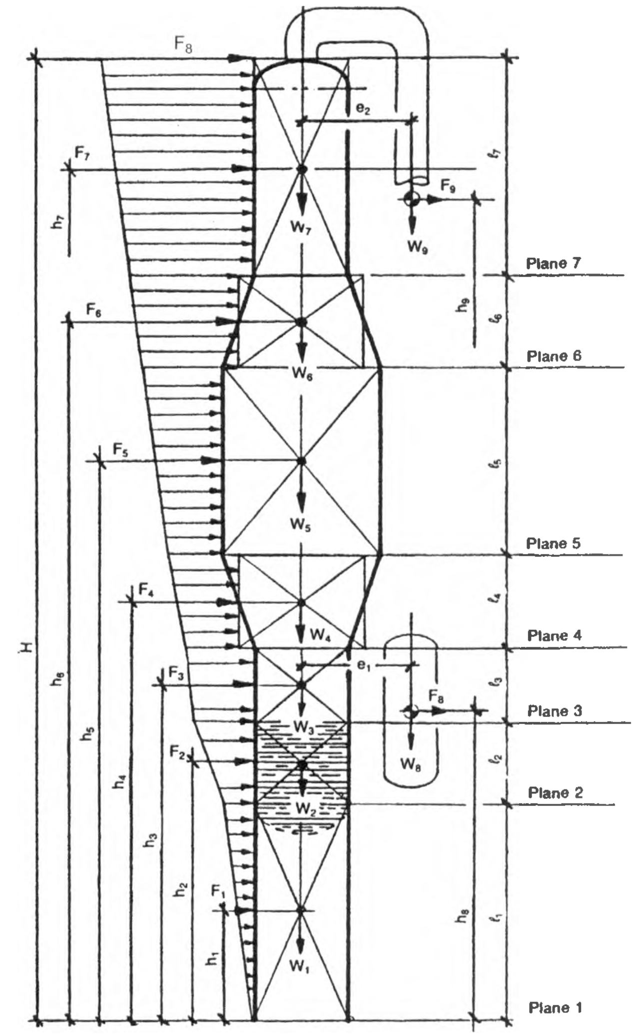


Figure 4-37. Nonuniform vessel illustrating a) Note 4 and b) Note 5.



$$h_x = h_{1,2,3,4,5\cdots}$$

$$W_x = W_{1,2,3,4,5\cdots}$$

$$F_x = F_{1,2,3,4,5\cdots}$$

$$w_x = w_{1,2,3,4,5\cdots}$$

$$M_x = M_{1,2,3,4,5\cdots}$$

$$\ell_x = \ell_{1,2,3,4,5\cdots}$$

$$w_x = \frac{W_x}{\ell_x}$$

$$F_x = \frac{V}{\Sigma W_x h_x}(W_x h_x)$$

$$M_i = \Sigma F_x(h_x - h_i)$$

Figure 4-38. Typical dimensional data, forces, and loadings on a nonuniform vessel supported on a skirt.

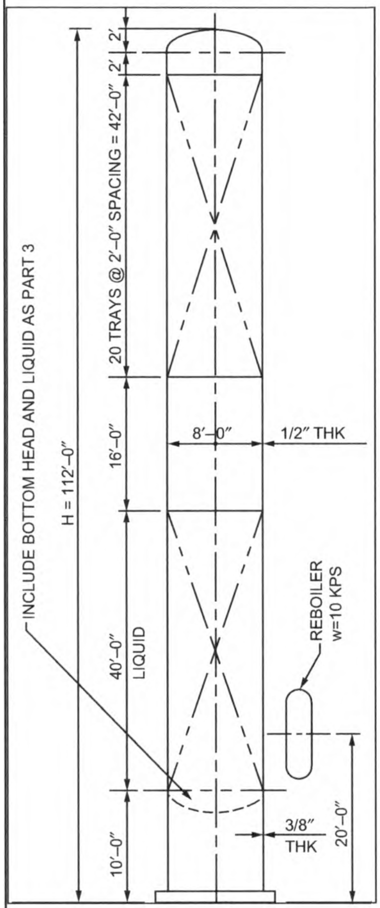
Step 1: PERIOD OF VIBRATION

	Part	ω or W k/ft	hx/H	α	Δα or β	ωΔα or Wβ/H	γ	Δγ	E(D/10)3tΔγ Note 3
			1.0	2.103			1.0		
			0	0			0		
						Σ =			Σ =

$$T = \left(\frac{H}{100}\right)^2 \sqrt{\frac{\Sigma\,\omega\Delta\alpha + \Sigma\,W\beta/H}{\Sigma E(D/10)^3 t\Delta\gamma}}$$

See Notes 2 and 3

Step 1: PERIOD OF VIBRATION EXAMPLE



Part	ω or W k/ft	hx/H 1.0	α 2.103	Δα or β	ωΔα or Wβ/H	γ 1.0	Δγ	E(D/10)3tΔγ Note 3
5	1.18			1.874	2.211		0.017	0.126
		0.59	0.229			0.983		
4	0.59			0.161	0.095		0.052	0.386
		0.45	0.068			0.931		
3	2.78			0.068	0.189		0.598	4.440
2	10 kips			0.024	0.002			
		0.09	0.00003			0.333		
1	0.875			0.00003	2.625E-05		0.333	1.854
		0	0			0		
					Σ = 2.497			Σ = 6.806

$$T=\left(\frac{H}{100}\right)^2\sqrt{\frac{\Sigma\omega\Delta\alpha+\Sigma W\beta/H}{\Sigma E(D/10)^3 t\Delta\gamma}}=\left(\frac{112}{100}\right)^2\sqrt{\frac{2.492\ +.002}{6.807}}=0.759\cdot\text{sec}$$

See Notes 2 and 3

Step 2: SHEAR AND MOMENTS

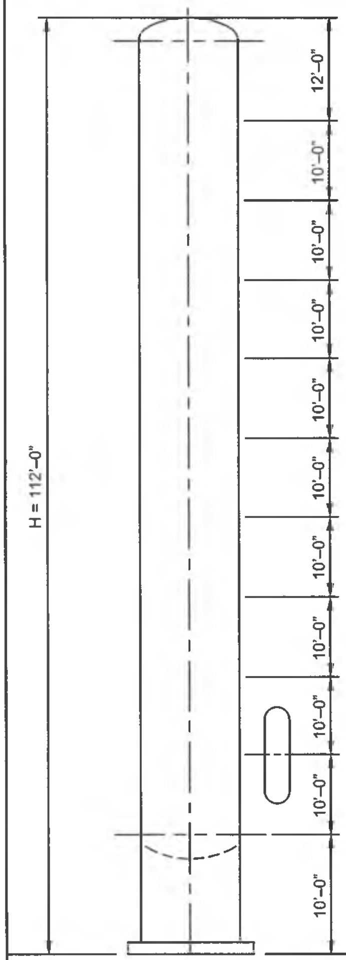
h_i (ft)	Part	W_x (kips)	h_x (ft)	$W_x h_x$ (ft-kips)	F_x (kips)	V_i @ btm (kips)	M_i (kips)
		Σ = 0		Σ = 0			

$$F_x = \frac{V}{\sum W_x h_x^k}\left(W_x h_x^k\right)$$

$$M_i = F_x(h_x - h_i) + V_{i+1}(h_{i+1} - h_i) + M_{i+1}$$

k = 1 for structures with periods of 0.5 seconds or less
k = 2 for structures with periods of 2.5 seconds or more
k shall be linearly interpolated with perdiods between 0.5 and 2.5 seconds

Step 2: SHEAR AND MOMENTS EXAMPLE



h_i (ft)	Part	W_x (kips)	h_x (ft)	W_xh_x (ft-kips)	F_x (kips)	V_i @ btm (kips)	M_i (kips)
112			112.0				
	12	14.16	106	1501	7.32		
100						7.32	43.92
	11	11.8	95	1121	5.47		
90						12.79	144.44
	10	11.8	85	1003	4.89		
80						17.68	296.76
	9	11.8	75	885	4.32		
70						21.99	495.11
	8	8.26	65	537	2.62		
60						24.61	728.13
	7	5.9	55	325	1.58		
50						26.19	982.15
	6	27.8	45	1251	6.10		
40						32.29	1274.58
	5	27.8	35	973	4.74		
30						37.04	1621.25
	4	27.8	25	695	3.39		
20	3	10	20	200	0.98		
						41.40	2008.58
	2	27.8	15	417	2.03		
10						43.44	2432.78
	1	8.75	5	44	0.21		
0						43.65	2868.21
		Σ = 194		Σ = 8951			

$$F_X = \frac{V}{\Sigma W_X h_X^k}\left(W_X h_X^k\right) \qquad F_X = \frac{43.65}{8951}\left(W_X h_X^k\right)$$

$$M_i = F_X(h_X - h_i) + V_{i+1}(h_{i+1} - h_i) + M_{i+1}$$

k = 1 for structures with periods of 0.5 seconds or less
k = 2 for structures with periods of 2.5 seconds or more
k shall be linearly interpolated with perdiods between 0.5 and 2.5 seconds

Longitudinal Stresses

In the following equations, D is in inches. The term "$48M_X$" is used for ft-lb or ft-kips. If in.-lb or in.-kips are used, then the term "$4M_X$" should be substituted where "$48M_X$" is used. The allowable stresses S_1E_1 or B may be substituted in the equations for t to determine or verify thickness at any elevation. Compare the stresses or thicknesses required at each elevation against the thickness required for circumferential stress due to internal pressure to determine which one will govern. If there is no external pressure condition, assume the maximum compression will occur when the vessel is not pressurized and the term $P_eD/4t$ will drop out.

$$\sigma_{xt} = \text{tension side} = \frac{P_iD}{4t} + \frac{48M_x}{\pi D^2 t} - \frac{W_h}{\pi Dt}$$

$$\sigma_{xc} = \text{compression side} = (-)\frac{P_eD}{4t} - \frac{48M_x}{\pi D^2 t} - \frac{W_h}{\pi Dt}$$

- *Allowable longitudinal stresses.*

  tension : S_1E_1 =

Elevation	M_x	W_n	D	t	Tension		Compression	
					S_1E_1	σ_{xt}	B	σ_{xc}

Table 4-20
Coefficients for determining period of vibration of free-standing cylindrical shells having varying cross sections and mass distribution

$\frac{h_x}{H}$	α	β	γ	$\frac{h_x}{H}$	α	β	γ	$\frac{h_x}{H}$	α	β	γ
1.00	2.103	8.347	1.000000	0.65	0.3497	2.3365	0.99183	0.30	0.010293	0.16200	0.7914
0.99	2.021	8.121	1.000000	0.64	0.3269	2.2240	0.99065	0.29	0.008769	0.14308	0.7776
0.98	1.941	7.898	1.000000	0.63	0.3052	2.1148	0.98934	0.28	0.007426	0.12576	0.7632
0.97	1.863	7.678	1.000000	0.62	0.2846	2.0089	0.98789	0.27	0.006249	0.10997	0.7480
0.96	1.787	7.461	1.000000	0.61	0.2650	1.9062	0.98630	0.26	0.005222	0.09564	0.7321
0.95	1.714	7.246	0.999999	0.60	0.2464	1.8068	0.98455	0.25	0.004332	0.08267	0.7155
0.94	1.642	7.037	0.999998	0.59	0.2288	1.7107	0.98262	0.24	0.003564	0.07101	0.6981
0.93	1.573	6.830	0.999997	0.58	0.2122	1.6177	0.98052	0.23	0.002907	0.06056	0.6800
0.92	1.506	6.626	0.999994	0.57	0.1965	1.5279	0.97823	0.22	0.002349	0.05126	0.6610
0.91	1.440	6.425	0.999989	0.56	0.1816	1.4413	0.97573	0.21	0.001878	0.04303	0.6413
0.90	1.377	6.227	0.999982	0.55	0.1676	1.3579	0.97301	0.20	0.001485	0.03579	0.6207
0.89	1.316	6.032	0.999971	0.54	0.1545	1.2775	0.97007	0.19	0.001159	0.02948	0.5992
0.88	1.256	5.840	0.999956	0.53	0.1421	1.2002	0.96688	0.18	0.000893	0.02400	0.5769
0.87	1.199	5.652	0.999934	0.52	0.1305	1.1259	0.96344	0.17	0.000677	0.01931	0.5536
0.86	1.143	5.467	0.999905	0.51	0.1196	1.0547	0.95973	0.16	0.000504	0.01531	0.5295
0.85	1.090	5.285	0.999867	0.50	0.1094	0.9863	0.95573	0.15	0.000368	0.01196	0.5044
0.84	1.038	5.106	0.999817	0.49	0.0998	0.9210	0.95143	0.14	0.000263	0.00917	0.4783
0.83	0.988	4.930	0.999754	0.48	0.0909	0.8584	0.94683	0.13	0.000183	0.00689	0.4512
0.82	0.939	4.758	0.999674	0.47	0.0826	0.7987	0.94189	0.12	0.000124	0.00506	0.4231
0.81	0.892	4.589	0.999576	0.46	0.0749	0.7418	0.93661	0.11	0.000081	0.00361	0.3940
0.80	0.847	4.424	0.999455	0.45	0.0678	0.8876	0.93097	0.10	0.000051	0.00249	0.3639
0.79	0.804	4.261	0.999309	0.44	0.0612	0.6361	0.92495	0.09	0.000030	0.00165	0.3327
0.78	0.762	4.102	0.999133	0.43	0.0551	0.5872	0.91854	0.08	0.000017	0.00104	0.3003
0.77	0.722	3.946	0.998923	0.42	0.0494	0.5409	0.91173	0.07	0.000009	0.00062	0.2669
0.76	0.683	3.794	0.998676	0.41	0.0442	0.4971	0.90448	0.06	0.000004	0.00034	0.2323
0.75	0.646	3.845	0.998385	0.40	0.0395	0.4557	0.89679	0.05	0.000002	0.00016	0.1966
0.74	0.610	3.499	0.998047	0.39	0.0351	0.4167	0.88884	0.04	0.000001	0.00007	0.1597
0.73	0.576	3.356	0.997656	0.38	0.0311	0.3801	0.88001	0.03	0.000000	0.00002	0.1218
0.72	0.543	3.217	0.997205	0.37	0.0275	0.3456	0.87088	0.02	0.000000	0.00000	0.0823
0.71	0.512	3.081	0.996689	0.36	0.0242	0.3134	0.86123	0.01	0.000000	0.00000	0.0418
0.70	0.481	2.949	0.998101	0.35	0.0212	0.2833	0.85105	0.	0.	0.	0.
0.69	0.453	2.820	0.995434	0.34	0.0185	0.2552	0.84032				
0.68	0.425	2.694	0.994681	0.33	0.0161	0.2291	0.82901				
0.67	0.399	2.571	0.993834	0.32	0.0140	0.2050	0.81710				
0.88	0.374	2.452	0.992885	0.31	0.0120	0.1826	0.80459				

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compression:

$$A = \frac{0.125t}{R_o}$$

B = from applicable material chart of ASME Code, Section II, Part D, Subpart 3.

Note: Joint efficiency for longitudinal seams in compression is 1.0.

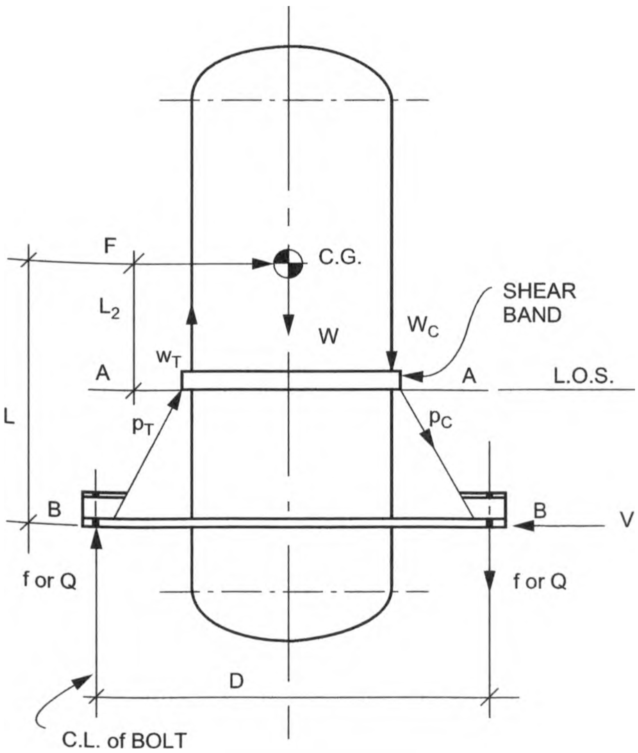
Notes

1. This procedure is for use in determining forces and moments at various planes of uniform and nonuniform vertical pressure vessels.
2. To determine the plate thickness required at any given elevation compare the moments from both wind and seismic at that elevation. The larger of the two should be used. Wind-induced moments may govern the longitudinal loading at one elevation, and seismic-induced moments may govern another.

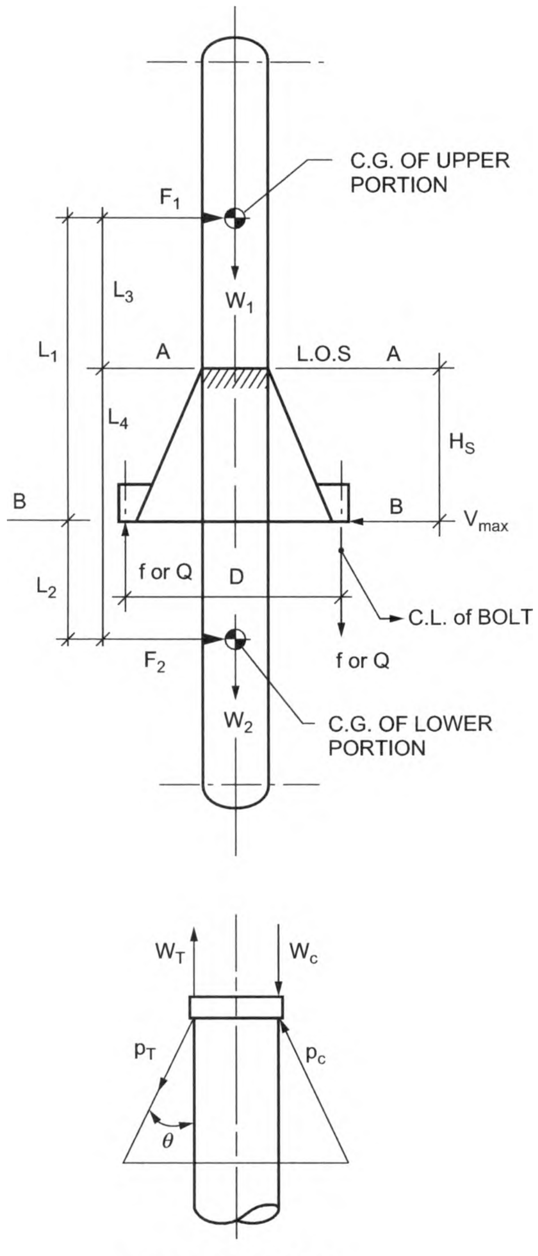
Procedure 4-9: Seismic Design – Vessel on Conical Skirt

Nomenclature

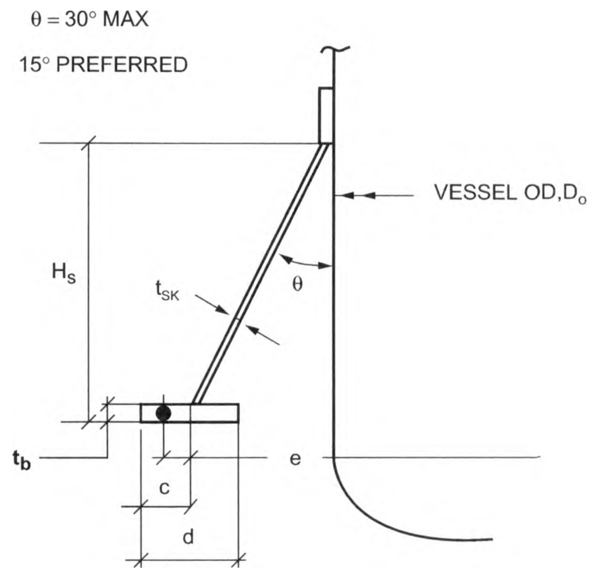
A = ASME Code strain factor, dimensionless
A_b = Area of base plate supported on steel, in^2
A_t = Area required for one anchor bolt, in^2
A_S = Area of shear band, L_S X t_S, in^2
B_P = Allowable bearing pressure, PSI
D_O = OD of vessel shell, in
D_{SK} = OD of skirt at base plate, in
E = Modulus of elasticity, PSI
F_c = Allowable compressive stress, PSI
f = Load at support points, Lbs
f_P = Bearing pressure, PSI
F_T = Allowable stress, tension, PSI
F_y = Minimum specified yield strength of skirt at design temperature, PSI
F_1 or F_2 = Seismic load for upper or lower portion of vessel
$M_{AA \text{ or } BB}$ = Overturning moment due to earthquake, In-Lbs, at elevation A-A or B-B
M_b = Bending moment, In-Lbs
M_X or M_y = Internal bending moment in base plate, in-lbs
N = Number of support points
N_b = Number of anchor bolts
P = Design pressure, PSIG
p_T, p_C = Load at top of skirt, tension or compression, Lbs/in
Q = Load at support points, Lbs
R_m = Mean radius of shell, in
S = Shell allowable stress, tension, PSI
S_b = Allowable stress, anchor bolts, PSI
t = Thickness of shell, in
t_r = Thickness required, skirt, in
V = Base shear, Lbs
V_{max} = Greater of V_1 or V_2, Lbs
W = Weight, operating, Lbs
W_1 = Weight of vessel, insulation, piping, etc above LOS. Include weight of contents if contents are supported above the LOS. Do not include weight of skirt, Lbs
W_2 = Weight of vessel, insulation, piping, etc., below LOS. Include weight of contents if supported below the LOS. Do not include weight of skirt or base.
w_T, w_C = Uniform load in shell, tension or compression, Lbs/in
ΔT = Temperature differential in skirt; DT – 70° F
λ = Damping Factor
σ_{LT} = Longitudinal tension stress, skirt, PSI
σ_{LC} = Longitudinal compressive stress, skirt, PSI
$\sigma_{\Delta T}$ = Stress in skirt due to ΔT loading, PSI
σ_X = Longitudinal bending stress in shell, PSI
τ_r = Allowable shear stress in shear band, PSI
τ_w = Allowable shear stress in weld, PSI



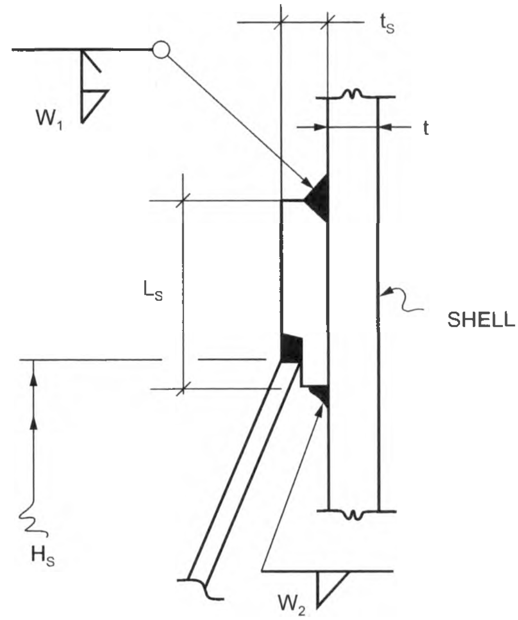
SIMPLE VESSEL DIAGRAM
SEE NOTE 1



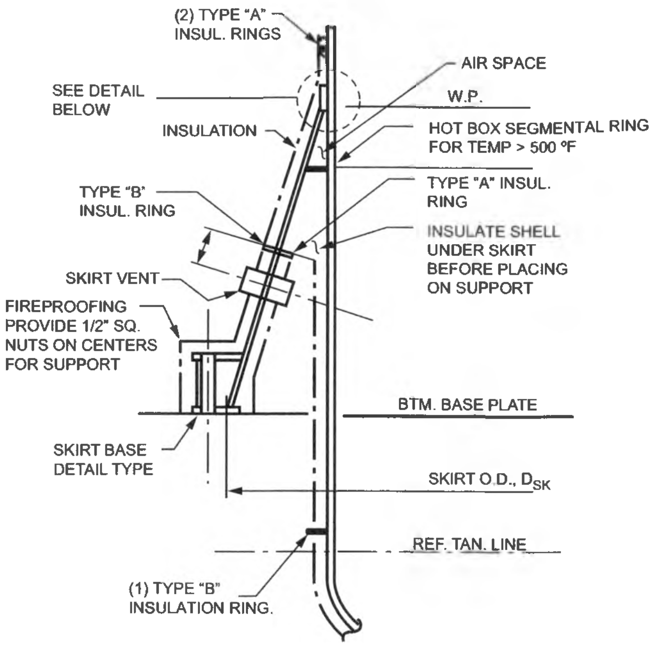
DETAIL OF FORCES



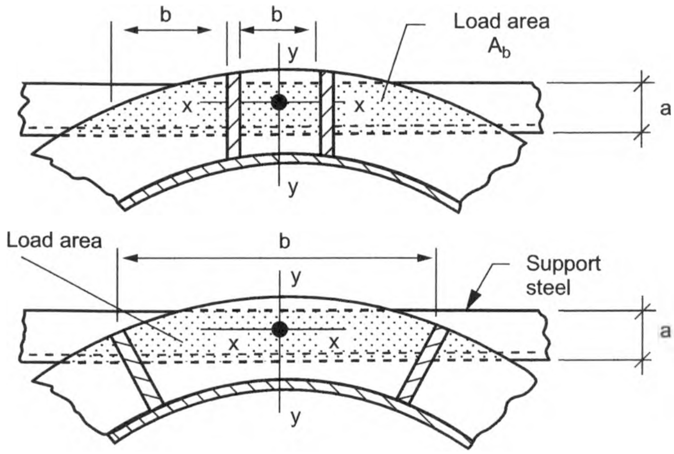
SKIRT DIMENSIONS



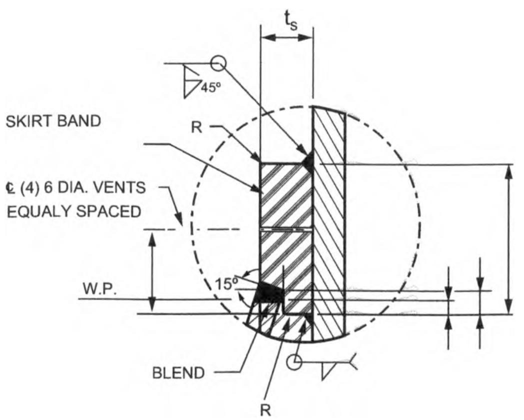
DIMENSIONS OF SHEAR BAND



SKIRT & BASE DETAILS



BASE PLATE LOADING



DETAIL OF SHEAR BAND

Table 4-21
Maximum bending moment in a bearing plate with gussets

a/b	$M_x \begin{bmatrix} x = .5b \\ y = \ell \end{bmatrix}$	$M_y \begin{bmatrix} x = .5b \\ y = 0 \end{bmatrix}$
0	0	$(-).500\ B_p\ l^2$
.333	$.0078\ B_p\ b^2$	$(-).428\ B_p\ \ell^2$
.5	$.0293\ B_p\ b^2$	$(-).319\ B_p\ \ell^2$
.666	$.0558\ B_p\ b^2$	$(-).227\ B_p\ \ell^2$
1.0	$.0972\ B_p\ b^2$	$(-).119\ B_p\ \ell^2$
1.5	$.1230\ B_p\ b^2$	$(-).124\ B_p\ \ell^2$
2.0	$.1310\ B_p\ b^2$	$(-).125\ B_p\ \ell^2$
3.0–∞	$.1330\ B_p\ b^2$	$(-).125\ B_p\ \ell^2$

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From Process Equipment Design, *Table 10.3 (See Note 2.)*

Calculation

Case 1: Simplified Approach (Note 1)

GIVEN:

D = ________ L = ________
F = ________ L_2 = ________
W = ________

Calculate moments;

$M_{AA} = F\,L_2$

$M_{BB} = F\,L$

Case 2: Rigorous Approach (Note 2)

GIVEN:

H_S = ________ L_3 = ________
L_4 = ________ W_1 = ________
W_2 = ________ F_1 = ________
F_2 = ________

V_{max} = Greater of F_1 or F_2

M_{max} = Greater of following;

$M_{AA} = F_1\,L_3$

Or $= F_2\,L_4$

$M_{BB} = (V_{max}\,H_S) + M_{max}$

$W = W_1 + W_2$

Design of Skirt

- Uniform loads in vessel at ELEV A-A;

$$w_T = [(-)W/(\pi\,D_O)] + [(4\,M_{AA}/(\pi\,D_O^2)]$$
$$w_C = [(-)W/(\pi\,D_O)] - [(4\,M_{AA}/(\pi\,D_O^2)]$$

- Find angle, θ, by layout or calculation;

$$X = .5\,[(D - 2\,e) - (D_O + 2\,t_S)]$$

$$\operatorname{Tan}\theta = X/(H_S - t_b)$$

θ = ________

- Uniform load in skirt at ELEV A-A

$p_T = w_T/\operatorname{Cos}\theta$

$p_C = w_C/\operatorname{Cos}\theta$

- Allowable stress, skirt;

1. Compression, F_C
   Assume a thickness of skirt and calculate;

$A = (.125\,t_{SK})/(.5\,D_{SK})$

$F_C = (A\,E/2) < .5\,F_y$

2. Tension, F_T

S = from ASME II(D) $< .66\,F_y$

$F_T = 1.2\,S$

- Thickness required, skirt, t_r

Tension; $t_r = p_T/F_T$

Comp; $t_r = p_C/F_C$

Use t_{SK} = ________

- Stress due to ΔT

$$\sigma_{\Delta T} = [(48\,\Delta T)/(H_S - t_b)][D_O\,t_{SK}]^{1/2}$$

- Longitudinal stress in skirt due to loadings;

Tension; $\sigma_{LT} = (p_T/t_{SK}) + \sigma_{\Delta T}$

Comp; $\sigma_{LC} = (p_C/t_{SK}) + \sigma_{\Delta T}$

Shear Ring

- Allowable shear stresses;

Ring; $\tau_r = .7\,S$

Weld; $\tau_w = .4\,S$

- Minimum length of shear band, L_{min}

$L_{min} = w_C/\tau_r$

- Size fillet welds, w_1 and w_2

$w = w_1 + w_2$

$w = w_C/(.707\,\tau_w)$

Use $w_1 = w_2$ = ________

- Thickness required for shear band, t_S

$t_S = 2\,w_1$

Use t_S = ________

Base Plate

The base plate thickness depends on how the vessel is supported. The vessel can either have continuous support or partial support. Partial support describes a vessel supported on 4 or 8 points on steel in a structure. Continuous support describes a concrete table top where there is full width, 360° contact between the base plate and the support.

Case A: Full Support

- Maximum load, f
  Note: The maximum loading is assumed to occur at the bolt circle.
  $f = [W/\pi D] \pm [4 M_{BB}/\pi D^2]$
- Bearing pressure, f_P
  $f_P = f/d < B_P$
- Base plate thickness, t_b
  $t_b = C\left[(3 f_P)/(.6 F_y)\right]^{1/2}$

Case B: Partial Support

- Load Q
  $Q = W/N \pm M_{BB}/D$
- Bearing pressure, f_P
  $f_P = Q/A_b$
- Maximum bending moment, M_b, from Table 4-21
  $a/b =$
  M_b = greater of M_X or M_y
- Thickness of base plate, t_b
  $t_b = \left[(6 M_b/.6 F_y)\right]^{1/2}$

Anchor Bolts

- Determine if anchor bolts are required due to uplift
  $N_b A_t = [(48 M_{BB}/D) - W][1/S_b]$

If N_b A_t is negative, then anchor bolts are not required. Use minimum size and maximum spacing for this case.
If N_b A_t is positive then anchor bolts are required.

- Area required, A_t
  $A_t = [(48 M_{BB}/D) - W][1/N_b S_b]$

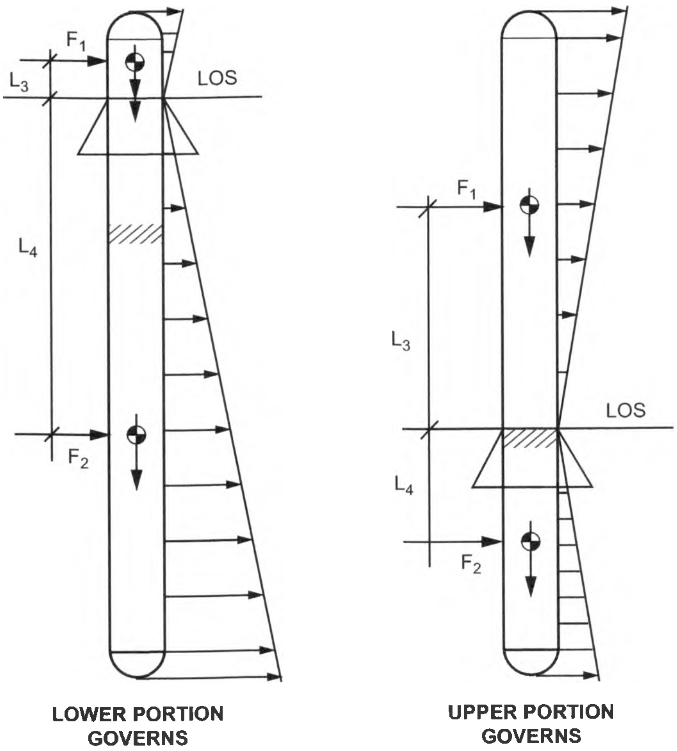
Use N_b = ________
d_b = ________

Longitudinal Stress in Shell due to Shear Band

- Cross sectional area of shear band, A_S
  $A_S = L_S t_S$
- Damping Factor, λ
  $\lambda = 1.285/(R_m t)^{1/2}$
- Bending moment in shell, M
  $M = [P/2 \lambda^2][A_S/(A_S + t L_S + 2 t/\lambda)]$
- Longitudinal bending stress in shell, σ_X
  $\sigma_X = 6 M/t^2$

Notes

1. The "Simplified Approach" is valid for average size vessels where L/D < 5 and the support point is near the C.G. of the vessel. The simplified approach applies the full seismic force at the C.G. of the vessel.
2. The "Rigorous Approach" is for vessel where L/D > 5 or the vessel is supported near the top or bottom of the vessel. In such cases the simplified approach may not be adequate. In this case the vessel is divided into two parts; the upper and lower part. The division between the upper and lower part is the line of support.
3. A third approach, not shown here, would be to determine the loadings by determining the shear and moments at each weld plane for each part of the vessel. This procedure is illustrated in Procedure 4-8.
4. The upper weight, W_1, will produce a compressive force in the shell equal to W_1 / A, where A is the cross sectional area of the vessel.
5. The lower weight, W_2, will produce a tensile force in the vessel shell equal to W_2 / A. This would be additive to effects due to internal pressure.
6. The effects of the unbalanced inward (or outward) load on the shell to cone junction should be evaluated for circumferential membrane and bending stresses, as well as longitudinal bending stresses.



M_{max} = GREATER OF ...

$M_{AA} = F_1 L_3$

Or $F_2 L_4$

$M_{BB} = (V_{max} H_s) + M_{max}$

V_{max} = Greater of F_1 or F_2

Figure 4-39. Vessel supported on conical skirt (Influence of support positioning).

Procedure 4-10: Design of Horizontal Vessel on Saddles [1,3,14,15]

Notation

A_r = cross-sectional area of composite ring stiffener, in.^{2}
E = joint efficiency
E_1 = modulus of elasticity, psi
C_h = seismic factor
I_1 = moment of inertia of ring stiffener, in.^{4}
t_w = thickness of wear plate, in.
t_s = thickness of shell, in.
t_h = thickness of head, in.
Q = total load per saddle (including piping loads, wind or seismic reactions, platforms, operating liquid, etc.) lb
W_o = operating weight of vessel, lb
M_1 = longitudinal bending moment at saddles, in.-lb
M_2 = longitudinal bending moment at midspan, in.-lb
S = allowable stress, tension, psi
S_c = allowable stress, compression, psi
S_{1-14} = shell, head, and ring stresses, psi
K_{1-9} = coefficients
F_L = longitudinal force due to wind, seismic, expansion, contraction, etc., lb
F_T = transverse force, wind or seismic, lb
σ_x = longitudinal stress, internal pressure, psi
σ_ϕ = circumferential stress, internal pressure, psi

σ_e = longitudinal stress, external pressure, psi
σ_s = circumferential stress in stiffening ring, psi
σ_h = latitudinal stress in head due to internal pressure, psi
F_y = minimum yield stress, shell, psi
P = internal pressure, psi
P_e = external pressure, psi
K_s = pier spring rate,
μ = friction coefficient
y = pier deflection, in.

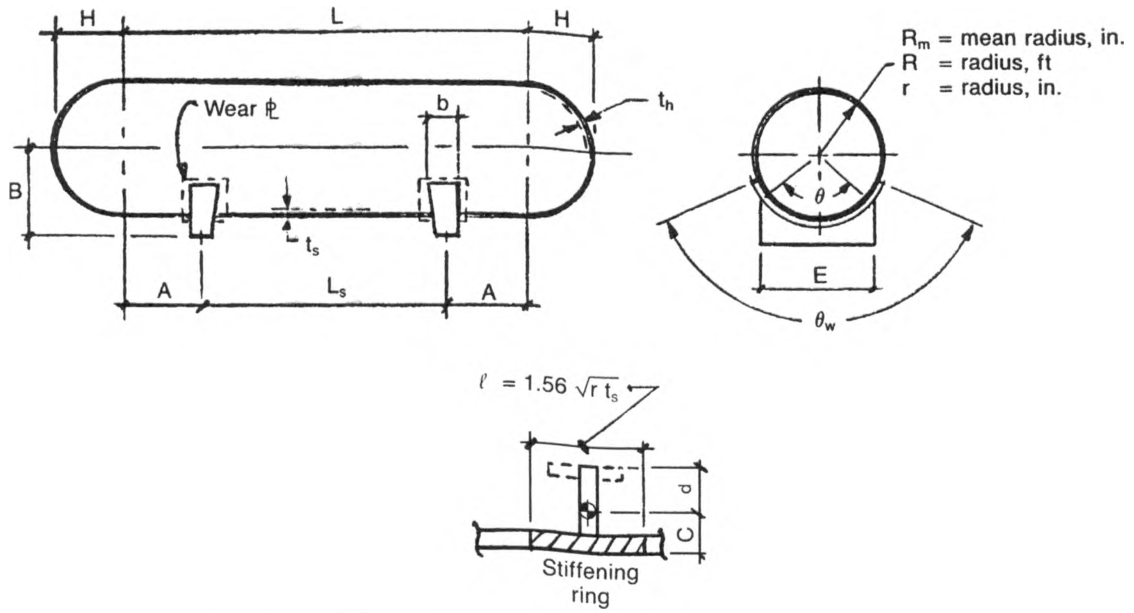


Figure 4-40. Typical dimensions for a horizontal vessel supported on two saddles.

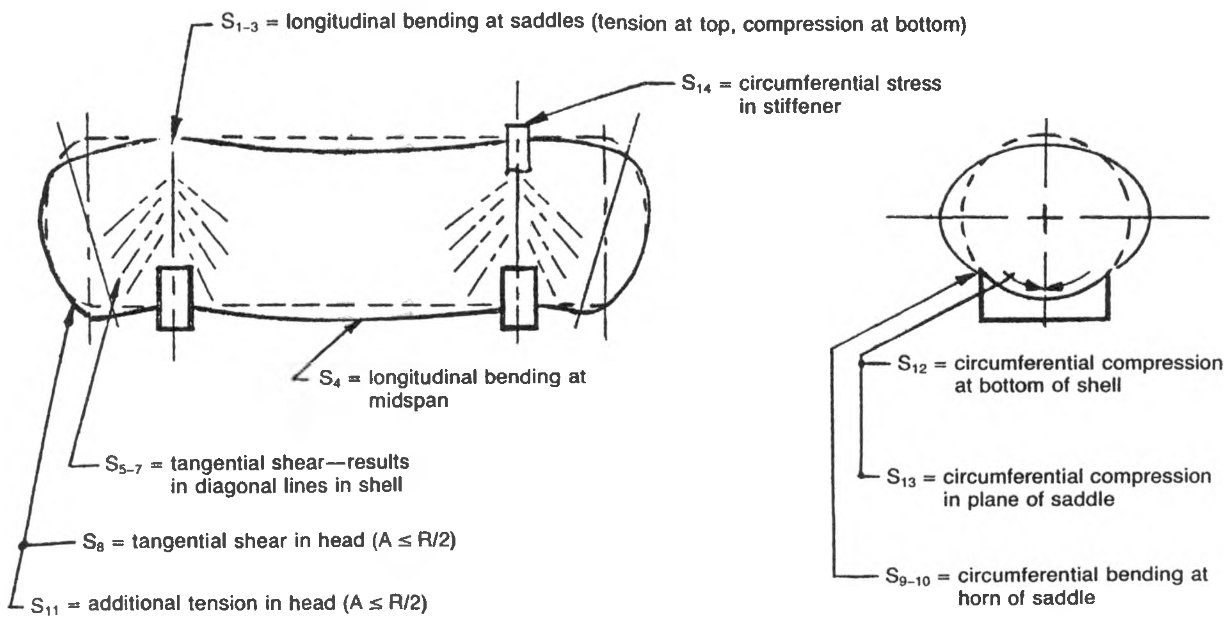
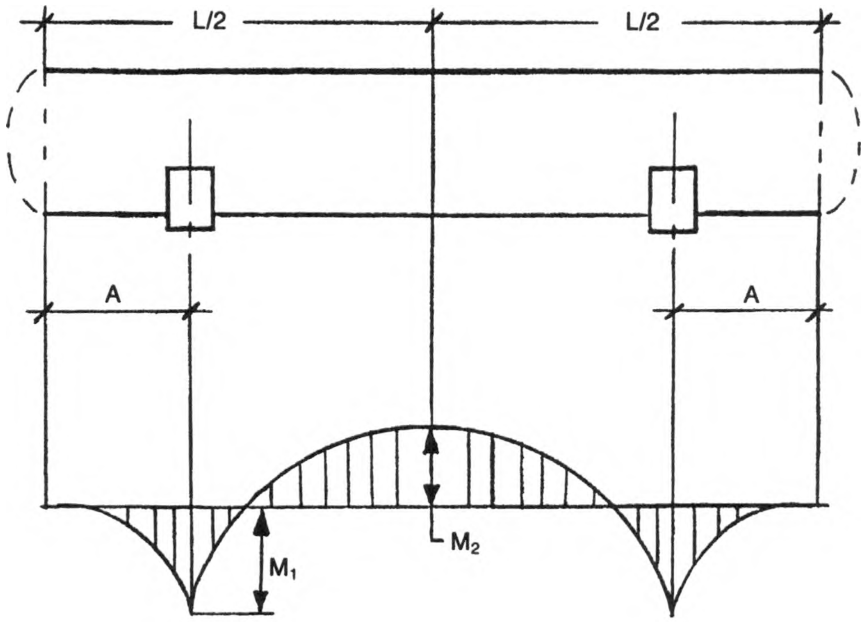


Figure 4-41. Stress diagram.



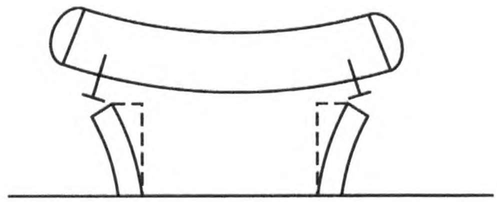
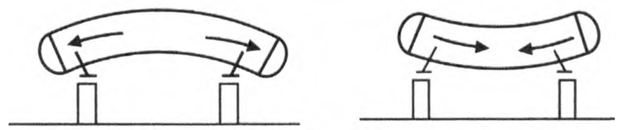
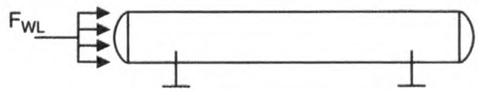
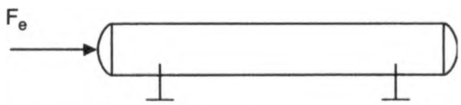
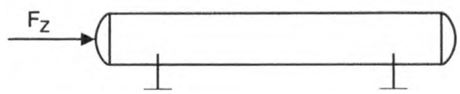
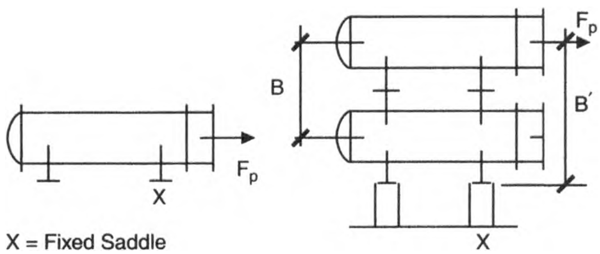
M_2 is negative for

- Hemi-heads.
- If any of the below conditions are exceeded.

M_2 is positive for

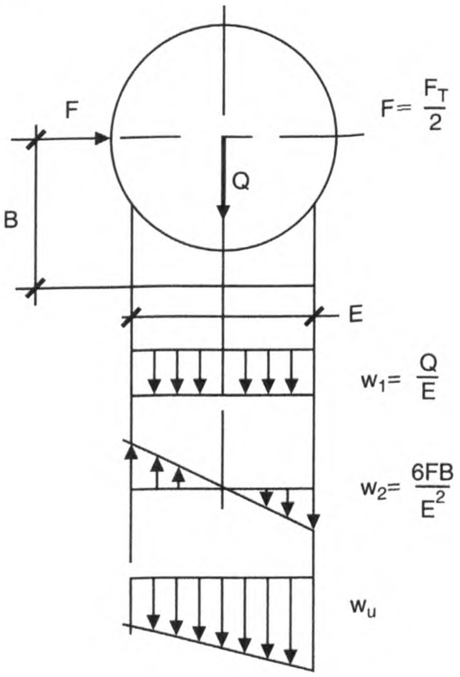
- Flat heads where $A/R < 0.707$.
- 100%–6% F&D heads where $A/R < 0.44$.
- 2:1 S.E. heads where $A/R < 0.363$.

Figure 4-42. Moment diagram.

Longitudinal Forces, F_L	
Case 1: Pier Deflection $F_{L1} = \frac{K_s y}{2}$ $S_a = S$	
Case 2: Expansion/Contraction $F_{L2} = \mu Q_0$ $S_a = S$	
Case 3: Wind $F_{L3} = F_{wL} = A_f C_f G q_z$ $S_a = 1.33S$	
Case 4: Seismic $F_{L4} = F_e = C_h W_o$ $S_a = 1.33S$	
Case 5: Shipping/Transportation F_{L5} (See Chapter 10.) $S_a = 0.9F_y$	
Case 6: Bundle Pulling $F_{L6} = F_p$ $S_a = 0.9F_y$ Full load applies to fixed saddle only!	 X = Fixed Saddle
Note: For Cases 5 and 6, assume the vessel is cold and not pressurized.	

Transverse Load: Basis for Equations

Method 1



- *Unit load at edge of base plate, w_u.*

  $W_u = W_1 + W_2$

- *Derivation of equation for w_2.*

  $$\sigma = \frac{M}{Z} \qquad M = FB \qquad Z = \frac{E^2}{6}$$

  Therefore

  $$\frac{M}{Z} = \frac{6FB}{E^2}$$

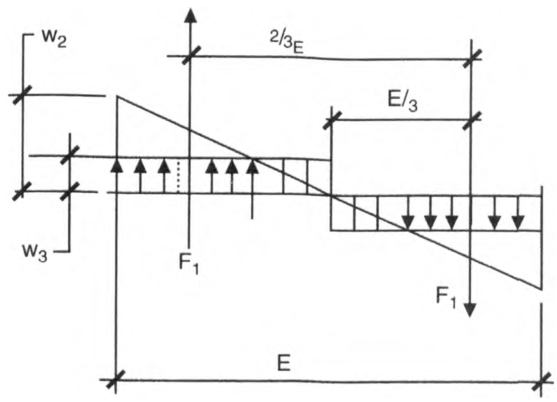
- *Equivalent total load Q_2.*

  $Q_2 = w_u E$

This assumes that the maximum load at the edge of the baseplate is uniform across the entire baseplate. This is very conservative, so the equation is modified as follows:

- *Using a triangular loading and 2/3 rule to develop a more realistic "uniform load"*

  $$F_1 = \frac{FB}{(2/3)E} = \frac{3FB}{2E}$$

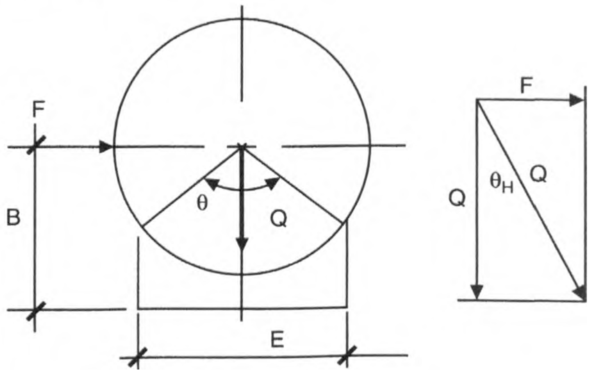


$$w_3 = \frac{3FB}{2E} \div \frac{E}{2} = \frac{6FB}{2E^2} = \frac{3FB}{E^2}$$

Therefore the total load, Q_F, due to force F is

$$Q_F = w_3 E = \frac{3FB}{E^2} E = \frac{3FB}{E}$$

Method 2



This method is based on the rationale that the load is no longer spread over the entire saddle but is shifted to one side.

- *Combined force, Q_2.*

  $$Q_2 = \sqrt{F^2 + Q^2}$$

- *Angle, θ_H.*

  $$\theta_H = \left(\arctan\right)\frac{F}{Q}$$

- *Modified saddle angle, θ_1.*

  $$\theta_1 = 2\left[\frac{\theta}{2}\right] - \theta_H$$

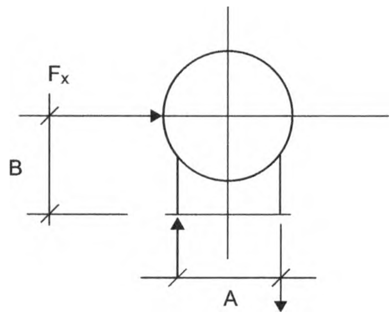
Saddle Reactions and Moments for Exchangers or Vessels with Offset Saddles

Due to... | **Load per Saddle** | **Diagram**

F_x

$$Q_1 = \frac{W_s L_2}{L_1} + \frac{F_x B}{2A}$$

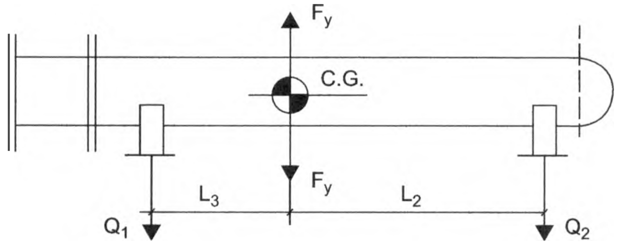
$$Q_2 = \frac{W_s L_3}{L_1} + \frac{F_x B}{2A}$$



F_y

$$Q_1 = \frac{(W_s + F_y) L_2}{L_1}$$

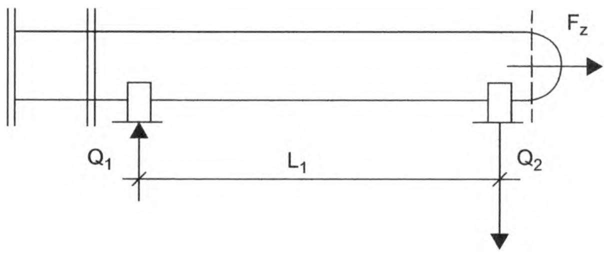
$$Q_2 = \frac{(W_s + F_y) L_3}{L_1}$$

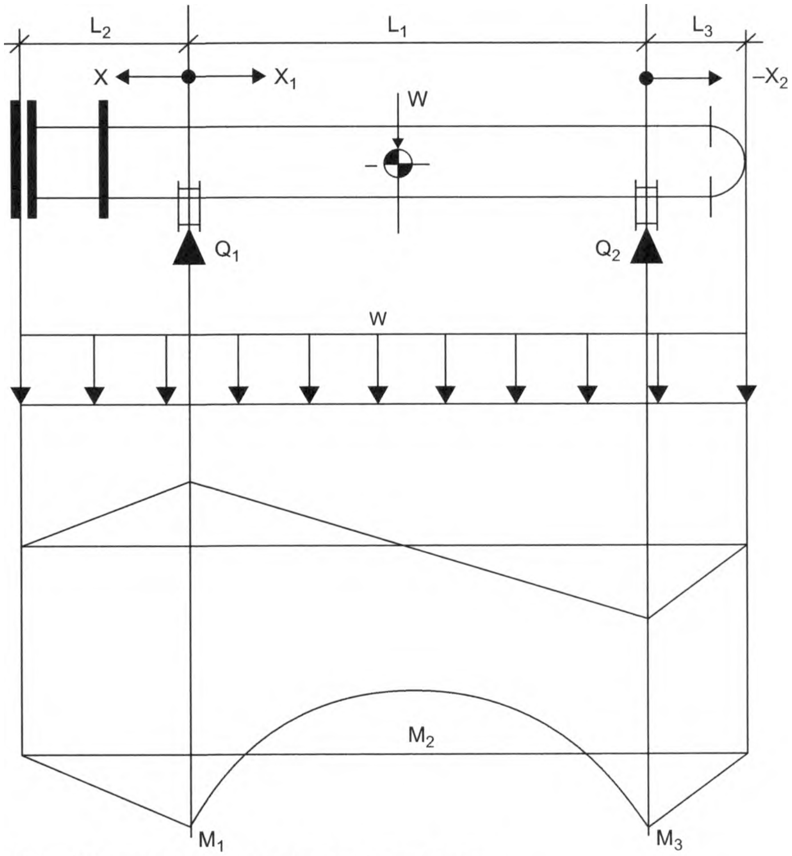


F_z

$$Q_1 = \frac{W_S L_2}{L_1} + \frac{F_z B}{L_1}$$

$$Q_2 = \frac{W_S L_3}{L_1} + \frac{F_z B}{L_1}$$





Note: W = weight of vessel plus any impact factors

$$OAL = L_1 + L_2 + L_3 \quad w = \frac{W}{OAL}$$

$$Q_1 = \frac{w\left[(L_1 + L_2)^2 - L_3^2\right]}{2L_1}$$

$$Q_2 = W - Q_1$$

$$M_1 = \frac{wL_2^2}{2}$$

$$M_2 = Q_1\left(\frac{Q_1}{2w} - L_2\right)$$

$$M_3 = \frac{wL_3^2}{2}$$

$$M_x = \frac{w(L_2 - X)^2}{2}$$

$$M_{x1} = \frac{w(L_2 + X_1)^2}{2} - Q_1X_1$$

$$M_{x2} = \frac{w(L_3 - X_2)^2}{2}$$

Types of Stresses and Allowables

- *S_1 to S_4: longitudinal bending.*

  Tension: S_1, S_3, or $S_4 + \sigma_x < SE$

  Compression: S_2, S_3, or $S_4 - \sigma_e < S_c$

  where S_c = factor "B" or S or $t_sE_1/16r$ whichever is less.

1. Compressive stress is not significant where R_m/t <200 and the vessel is designed for internal pressure only.
2. When longitudinal bending at midspan is excessive, move saddles away from heads; however, do not exceed $A \geq 0.2$ L.
3. When longitudinal bending at saddles is excessive, move saddles toward heads.

4. If longitudinal bending is excessive at both saddles and midspan, add stiffening rings. If stresses are still excessive, increase shell thickness.

- *S_5 to $S_8 < 0.8S$: tangential shear.*
  1. Tangential shear is not combined with other stresses.
  2. If a wear plate is used, t_s may be taken as $t_s + t_w$, providing the wear plate extends R/10 above the horn of the saddle.
  3. If the shell is unstiffened, the maximum tangential shear stress occurs at the horn of the saddle.
  4. If the shell is stiffened, the maximum tangential shear occurs at the equator.
  5. When tangential shear stress is excessive, move saddles toward heads, $A \leq 0.5$ R, add rings, or increase shell thickness.
  6. When stiffening rings are used, the shell-to-ring weld must be designed to be adequate to resist the tangential shear as follows:

$$S_t = \frac{Q}{\pi r} : \frac{\text{lb}}{\text{in. circumference}} < \frac{\text{allowable shear}}{\text{in. of weld}}$$

- *$S_{11} + \sigma_h < 1.25$ SE: additional stress in head.*
  1. S_{11} is a shear stress that is additive to the hoop stress in the head and occurs whenever the saddles are located close to the heads, $A \leq 0.5$ R. Due to their close proximity the shear of the saddle extends into the head.
  2. If stress in the head is excessive, move saddles away from heads, increase head thickness, or add stiffening rings.
- *S_9 and $S_{10} < 1.5$ S and $0.9F_y$: circumferential bending at horn of saddle.*
  1. If a wear plate is used, t_s may be taken as $t_s + t_w$ providing the wear plate extends R/10 above the horn of the saddle. Stresses must also be checked at the top of the wear plate.
  2. If stresses at the horn of the saddle are excessive:
     a. Add a wear plate.
     b. Increase contact angle θ.
     c. Move saddles toward heads, $A < R$.
     d. Add stiffening rings.
- *$S_{12} < 0.5F_y$ or 1.5 S: circumferential compressive stress.*
  1. If a wear plate is used, t_s may be taken as $t_s + t_w$, providing the width of the wear plate is at least $b + 1.56\sqrt{rt_s}$.
  2. If the shell is unstiffened the maximum stress occurs at the horn of the saddle.
  3. If the shell is stiffened the maximum hoop compression occurs at the bottom of the shell.
  4. If stresses are excessive add stiffening rings.
- *$(+)S_{13} + \sigma_\phi < 1.5$ S: circumferential tension stress—shell stiffened.*
- *$(-)S_{13} - \sigma_s < 0.5F_y$: circumferential compression stress—shell stiffened.*
- *$(-)S_{14} - \sigma_s < 0.9F_y$: circumferential compression stress in stiffening ring.*

Procedure for Locating Saddles

Trial 1: Set $A = 0.2$ L and $\theta = 120°$ and check stress at the horn of the saddle, S_9 or S_{10}. This stress will govern for most vessels except for those with large L/R ratios.

Trial 2: Increase saddle angle θ to 150° and recheck stresses at horn or saddle, S_9 or S_{10}.

Trial 3: Move saddles near heads ($A = R/2$) and return θ to 120°. This will take advantage of stiffness provided by the heads and will also induce additional stresses in the heads. Compute stresses S_4, S_8, and S_9 or S_{10}. A wear plate may be used to reduce the stresses at the horn or saddle when the saddles are near the heads ($A < R/2$) and the wear plate extends R/10 above the horn of the saddle.

Trial 4: Increase the saddle angle to 150° and recheck stresses S_4, S_8, and S_9 or S_{10}. Increase the saddle angle progressively to a maximum of 168° to reduce stresses.

Trial 5: Move saddles to $A = 0.2L$ and $\theta = 120°$ and design ring stiffeners in the plane of the saddles using the equations for S_{13} and S_{14} (see Note 7).

Total Saddle Reaction Forces, Q.

Q = greater of Q_1 or Q_2

Longitudinal, Q_1

$$Q_1 = \frac{W_o}{2} + \frac{F_L B}{L_s}$$

Transverse, Q_2

$$Q_2 = \frac{W_o}{2} + \frac{3F_t B}{E}$$

Shell Stresses

There are 14 main stresses to be considered in the design of a horizontal vessel on saddle supports:

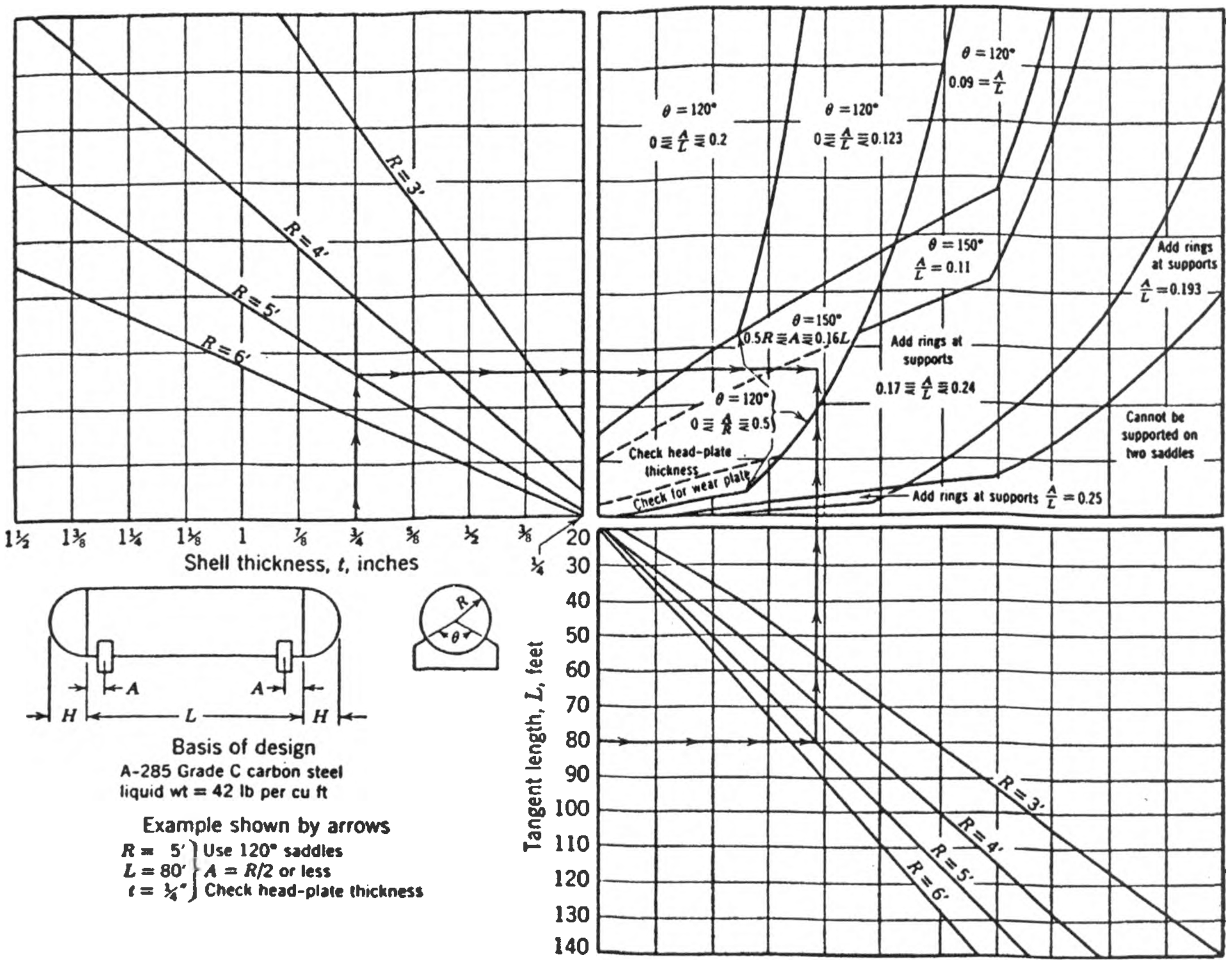


Figure 4-43. Chart for selection of saddles for horizontal vessels. *Reprinted by permission of the American Welding Society.*

S_1 = longitudinal bending at saddles without stiffeners, tension
S_2 = longitudinal bending at saddles without stiffeners, compression
S_3 = longitudinal bending at saddles with stiffeners
S_4 = longitudinal bending at midspan, tension at bottom, compression at top
S_5 = tangential shear—shell stiffened in plane of saddle
S_6 = tangential shear—shell not stiffened, $A > R/2$
S_7 = tangential shear—shell not stiffened except by heads, $A \leq R/2$
S_8 = tangential shear in head—shell not stiffened, $A \leq R/2$
S_9 = circumferential bending at horn of saddle—shell not stiffened, $L \geq 8R$
S_{10} = circumferential bending at horn of saddle—shell not stiffened, $L < 8R$
S_{11} = additional tension stress in head, shell not stiffened, $A \leq R/2$
S_{12} = circumferential compressive stress—stiffened or not stiffened, saddles attached or not
S_{13} = circumferential stress in shell with stiffener in plane of saddle
S_{14} = circumferential stress in ring stiffener

Longitudinal Bending

- *S_1, longitudinal bending at saddles—without stiffeners, tension.*

$$M_1 = 6Q\left[\frac{8AH + 6A^2 - 3R^2 + 3H^2}{3L + 4H}\right]$$

$$S_1 = (+)\frac{M_1}{K_1 r^2 t_s}$$

- *S_2, longitudinal bending at saddles—without stiffeners, compression.*

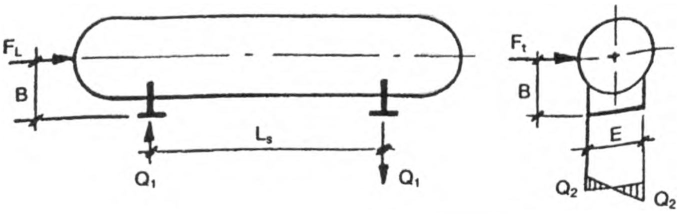


Figure 4-44. Saddle reaction forces.

$$S_2 = (-)\frac{M_1}{K_7 r^2 t_s}$$

- *S_3, longitudinal bending at saddles—with stiffeners.*

$$S_3 = (\pm)\frac{M_1}{\pi r^2 t_s}$$

- *S_4, longitudinal bending at midspan.*

$$M_2 = 3Q\left[\frac{3L^2 + 6R^2 - 6H^2 - 12AL - 16AH}{3L + 4H}\right]$$

$$S_4 = (\pm)\frac{M_2}{\pi r^2 t_s}$$

Tangential Shear

- *S_5, tangential shear—shell stiffened in the plane of the saddle.*

$$S_5 = \frac{Q}{\pi r t_s}\left[\frac{L - 2A}{L + \frac{4}{3}H}\right]$$

- *S_6, tangential shear—shell not stiffened, $A > 0.5R$.*

$$S_6 = \frac{K_2 Q}{r t_s}\left[\frac{L - 2A}{L + \frac{4}{3}H}\right]$$

- *S_7, tangential shear—shell not stiffened, $A \leq 0.5R$.*

$$S_7 = \frac{K_3 Q}{r t_s}$$

- S_8, *tangential shear in head—shell not stiffened, $A \leq 0.5R$.*

$$S_8 = \frac{K_3 Q}{r t_h}$$

Note: If shell is stiffened or $A > 0.5R$, $S_8 = 0$.

Circumferential Bending

- *S_9, circumferential bending at horn of saddle—shell not stiffened ($L \geq 8R$).*

$$S_9 = (-)\frac{Q}{4t_s(b + 1.56\sqrt{rt_s})} - \frac{3K_6 Q}{2t_s^2}$$

Note: $t_s = t_s + t_w$ and $t_s^2 = t_s^2 + t_w^2$ only if $A \leq 0.5R$ and wear plate extends R/10 above horn of saddle.

- *S_{10}, circumferential bending at horn of saddle—shell not stiffened ($L < 8R$).*

$$S_{10} = (-)\frac{Q}{4t_s(b + 1.56\sqrt{rt_s})} - \frac{12K_6 QR}{Lt_s^2}$$

Note: Requirements for t_s are same as for S_9.

Additional Tension Stress in Head

- *S_{11}, additional tension stress in head—shell not stiffened, $A \leq 0.5R$.*

$$S_{11} = \frac{K_4 Q}{r t_h}$$

Note: If shell is stiffened or $A > 0.5R$, $S_{11} = 0$.

Circumferential Tension/Compression

- *S_{12}, circumferential compression.*

$$S_{12} = (-)\frac{K_5 Q}{t_s(b + 1.56\sqrt{rt_s})}$$

Note: ts = ts + tw only if wear plate is attached to shell and width of wear plate is a minimum of $b + 1.56\sqrt{rt_s}$.

- *S_{13}, circumferential stress in shell with stiffener (see Note 8).*

$$S_{13} = (-)\frac{K_8 Q}{A_r} \pm \frac{K_9 QrC}{I_1}$$

Note: Add second expression if vessel has an internal stiffener, subtract if vessel has an external stiffener.

- *S_{14}, circumferential compressive stress in stiffener (see Note 8).*

$$S_{14} = (-)\frac{K_8 Q}{A_r} - \frac{K_9 Qrd}{I_1}$$

Pressure Stresses

$$\sigma_x = \frac{PR_m}{2t_s}$$

$$\sigma_\phi = \frac{PR_m}{t_s}$$

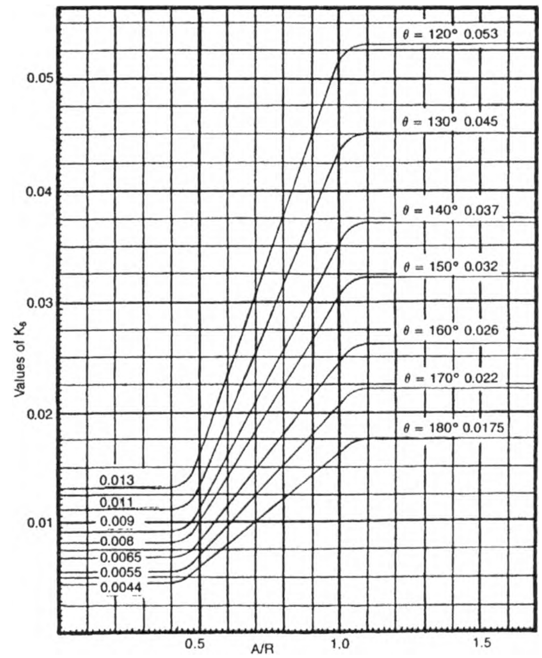
$$\sigma_e = \frac{P_e R_m}{2t_s}$$

$$\sigma_s = \frac{PIR_m}{A_r}$$

$\sigma_h = \sigma_\phi$, maximum circumferential stress in head is equal to hoop stress in shell

Combined Stresses

Tension		Compression	
Stress	**Allowable**	**Stress**	**Allowable**
$S_1 + \sigma_x$	SE =	$-S_2 - \sigma_e$	S_c =
$S_3 + \sigma_x$	SE =	$-S_3 - \sigma_e$	S_c =
$S_4 + \sigma_x$	SE =	$-S_4 - \sigma_e$	S_c =
$S_{11} + \sigma_h$	1.25SE =	$-S_{13} - \sigma_s$	$0.5F_y$ =
$S_{13} + \sigma_\phi$	1.5SE =	$-S_{14} - \sigma_s$	$0.9F_y$ =



Contact Angle θ	K_1*	K_2	K_3	K_4	K_5	K_7	K_8	K_9	Contact Angle θ	K_1*	K_2	K_3	K_4	K_5	K_7	K_8	K_9
120	0.335	1.171	0.880	0.401	0.760	0.603	0.340	0.053	152	0.518	0.781	0.466	0.289	0.669	0.894	0.298	0.031
122	0.345	1.139	0.846	0.393	0.753	0.618	0.338	0.051	154	0.531	0.763	0.448	0.283	0.665	0.913	0.296	0.030
124	0.355	1.108	0.813	0.385	0.746	0.634	0.336	0.050	156	0.544	0.746	0.430	0.278	0.661	0.933	0.294	0.028
126	0.366	1.078	0.781	0.377	0.739	0.651	0.334	0.048	158	0.557	0.729	0.413	0.272	0.657	0.954	0.292	0.027
128	0.376	1.050	0.751	0.369	0.732	0.669	0.332	0.047	160	0.571	0.713	0.396	0.266	0.654	0.976	0.290	0.026
130	0.387	1.022	0.722	0.362	0.726	0.689	0.330	0.045	162	0.585	0.698	0.380	0.261	0.650	0.994	0.286	0.025
132	0.398	0.996	0.694	0.355	0.720	0.705	0.328	0.043	164	0.599	0.683	0.365	0.256	0.647	1.013	0.282	0.024
134	0.409	0.971	0.667	0.347	0.714	0.722	0.326	0.042	166	0.613	0.668	0.350	0.250	0.643	1.033	0.278	0.024
136	0.420	0.946	0.641	0.340	0.708	0.740	0.324	0.040	168	0.627	0.654	0.336	0.245	0.640	1.054	0.274	0.023
138	0.432	0.923	0.616	0.334	0.702	0.759	0.322	0.039	170	0.642	0.640	0.322	0.240	0.637	1.079	0.270	0.022
140	0.443	0.900	0.592	0.327	0.697	0.780	0.320	0.037	172	0.657	0.627	0.309	0.235	0.635	1.097	0.266	0.021
142	0.455	0.879	0.569	0.320	0.692	0.796	0.316	0.036	174	0.672	0.614	0.296	0.230	0.632	1.116	0.262	0.020
144	0.467	0.858	0.547	0.314	0.687	0.813	0.312	0.035	176	0.0687	0.601	0.283	0.225	0.629	1.137	0.258	0.019
146	0.480	0.837	0.526	0.308	0.682	0.831	0.308	0.034	178	0.702	0.589	0.271	0.220	0.627	1.158	0.254	0.018
148	0.492	0.818	0.505	0.301	0.678	0.853	0.304	0.033	180	0.718	0.577	0.260	0.216	0.624	1.183	0.250	0.017
150	0.505	0.799	0.485	0.295	0.673	0.876	0.300	0.032									

*$K_1 = 3.14$ if the shell is stiffened by ring or head ($A < R/2$).

Figure 4-45. Coefficients.

Table 4-22
Coefficients for Zick's analysis (angles 80° to 120°)

						$A/R \le 0.5$	$A/R \ge 1.0$			
SADDLE ANGLE θ	**K_1**	**K_2**	**K_3**	**K_4**	**K_5**	**K_6**	**K_6**	**K_7**	**K_8**	**K_9**
80	0.1711	2.2747	2.0419	0.6238	0.9890	0.0237	0.0947	0.3212	0.3592	−0.0947
81	0.1744	2.2302	1.9956	0.6163	0.9807	0.0234	0.0934	0.3271	0.3592	0.0934
82	0.1777	2.1070	1.9506	0.6090	0.9726	0.0230	0.0922	0.3331	0.3593	0.0922
83	0.1811	2.1451	1.9070	0.6018	0.9646	0.0227	0.0910	0.3391	0.3593	0.0910
84	0.1845	2.1044	1.8645	0.5947	0.9568	0.0224	0.0897	0.3451	0.3593	0.0897
85	0.1879	2.0648	1.8233	0.5877	0.9492	0.0221	0.0885	0.3513	0.3593	0.0885
86	0.1914	2.0264	1.7831	0.5808	0.9417	0.0218	0.0873	0.3575	0.3592	0.0873
87	0.1949	1.9891	1.7441	0.5741	0.9344	0.0215	0.0861	0.3637	0.3591	0.0861
88	0.1985	1.9528	1.7061	0.5675	0.9273	0.0212	0.0849	0.3700	0.3590	0.0849
89	0.2021	1.9175	1.6692	0.5610	0.9203	0.0209	0.0838	0.3764	0.3588	0.0830
90	0.2057	1.8832	1.6332	0.5546	0.9134	0.0207	0.0826	0.3828	0.3586	0.0826
91	0.2094	1.8497	1.5981	0.5483	0.9067	0.0204	0.0815	0.3893	0.3584	0.0815
92	0.2132	1.8172	1.5640	0.5421	0.9001	0.0201	0.0803	0.3959	0.3582	0.0803
93	0.2169	1.7856	1.5308	0.5360	0.8937	0.0198	0.0792	0.4025	0.3579	0.0792
94	0.2207	1.7548	1.4984	0.5300	0.8874	0.0195	0.0781	0.4092	0.3576	0.0781
95	0.2246	1.7247	1.4668	0.5241	0.8812	0.0192	0.0770	0.4160	0.3573	0.0770
96	0.2285	1.6955	1.4360	0.5183	0.8751	0.0190	0.0759	0.4228	0.3569	0.0759
97	0.2324	1.6670	1.4060	0.5125	0.8692	0.0187	0.0748	0.4296	0.3565	0.0748
98	0.2364	1.6392	1.3767	0.5069	0.8634	0.0184	0.0737	0.4366	0.3561	0.0737
99	0.2404	1.6122	1.3482	0.5013	0.8577	0.0182	0.0727	0.4436	0.3557	0.0727
100	0.2445	1.5858	1.3203	0.4959	0.8521	0.0179	0.0716	0.4506	0.3552	0.0716
101	0.2486	1.5600	1.2931	0.4905	0.8466	0.0176	0.0706	0.4577	0.3547	0.0706
102	0.2528	1.5349	1.2666	0.4852	0.8412	0.0174	0.0696	0.4649	0.3542	0.0696
103	0.2570	1.5104	1.2407	0.4799	0.8359	0.0171	0.0686	0.4721	0.3536	0.0686
104	0.2612	1.4865	1.2154	0.4748	0.8308	0.0169	0.0675	0.4794	0.3531	0.0675
105	0.2655	1.4631	1.1907	0.4697	0.8257	0.0166	0.0666	0.4868	0.3525	0.0666
106	0.2698	1.4404	1.1665	0.4647	0.8207	0.0164	0.0656	0.4942	0.3518	0.0656
107	0.2742	1.4181	1.1429	0.4597	0.8159	0.0161	0.0646	0.5017	0.3512	0.0646
108	0.2786	1.3964	1.1199	0.4549	0.8111	0.0159	0.0636	0.5092	0.3505	0.0636
109	0.2830	1.3751	1.0974	0.4500	0.8064	0.0157	0.0627	0.5168	0.3498	0.0627
110	0.2875	1.3544	1.0753	0.4453	0.8018	0.0154	0.0617	0.5245	0.3491	0.0617
111	0.2921	1.3341	1.0538	0.4406	0.7973	0.0152	0.0608	0.5322	0.3483	0.0608
112	0.2966	1.3143	1.0327	0.4360	0.7928	0.0150	0.0599	0.5400	0.3475	0.0599
113	0.3013	1.2949	1.0121	0.4314	0.7885	0.0147	0.0590	0.5478	0.3467	0.0590
114	0.3059	1.2760	0.9920	0.4269	0.7842	0.0145	0.0581	0.5557	0.3459	0.0581
115	0.3107	1.2575	0.9723	0.4225	0.7800	0.0143	0.0572	0.5636	0.3451	0.0572
116	0.3154	1.2394	0.9530	0.4181	0.7759	0.0141	0.0563	0.5717	0.3442	0.0563
117	0.3202	1.2216	0.9341	0.4137	0.7719	0.0139	0.0554	0.5797	0.3433	0.0554
118	0.3251	1.2043	0.9157	0.4095	0.7680	0.0136	0.0546	0.5878	0.3424	0.0546
119	0.3300	1.1873	0.8976	0.4052	0.7641	0.0134	0.0537	0.5960	0.3414	0.0537
120	0.3349	1.1707	0.8799	0.4011	0.7603	0.0132	0.0529	0.6043	0.3405	0.0529
θ	K_1	K_2	K_3	K_4	K_5	K_6	K_6	K_7	K_8	K_9
SADDLE ANGLE						$A/R \le 0.5$	$A/R \ge 1.0$			

Notes:

1. These coefficients are derived from Zick's equations.
2. The ASME Code does not recommend the use of saddles with an included angle, θ, less than 120°. Therefore the values in this table should be used for very small-diameter vessels or to evaluate existing vessels built prior to this ASME recommendation.
3. Values of K_6 for A/R ratios between 0.5 and 1 can be interpolated.

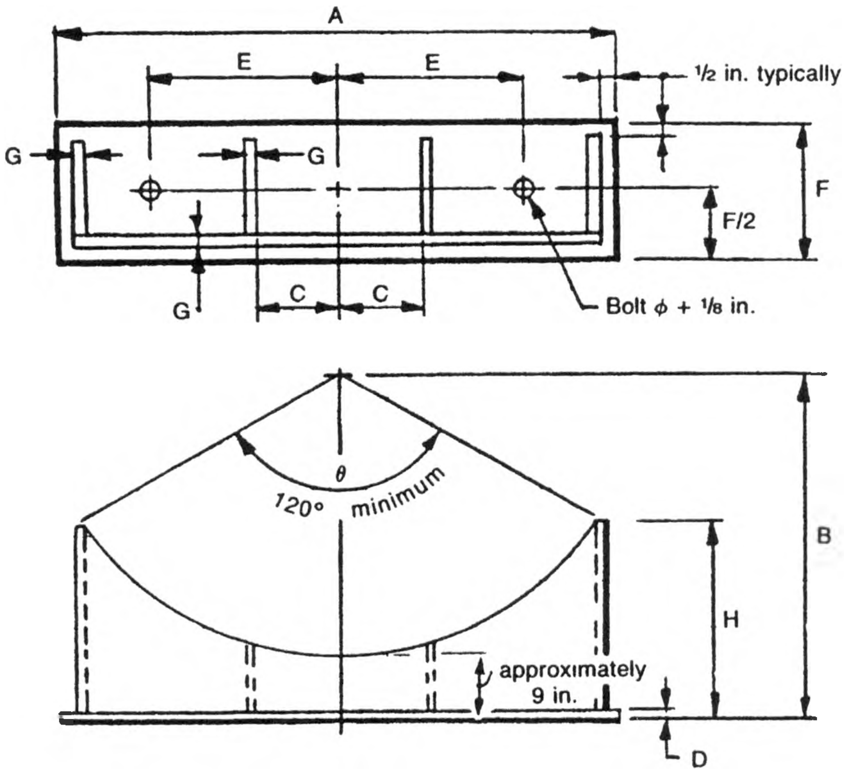


Figure 4-46. Saddle dimensions.

Table 4-23
Slot dimensions

Temperature °F	Distance Between Saddles				
	10ft	20ft	30ft	40ft	50ft
−50	0	0	0.25	0.25	0.375
100	0	0	0.125	0.125	0.250
200	0	0.250	0.375	0.375	0.500
300	0.250	0.375	0.625	0.750	1.00
400	0.375	0.625	0.875	1.125	1.375
500	0.375	0.750	1.125	1.500	1.625
600	0.500	1.00	1.375	1.875	2.250
700	0.625	1.125	1.625	2.125	2.625
800	0.750	1.250	1.625	2.375	3.000
900	0.750	1.375	2.000	2.500	3.375

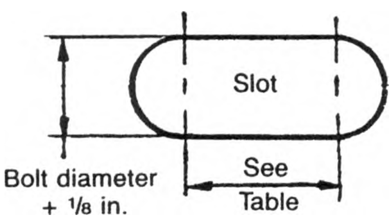


Table 4-24
Typical saddle dimensions*

Vessel O.D.	Maximum Operating Weight	A	B	C	D	E	F	G	H	Bolt Diameter	θ	Approximate Weight/Set
24	15,400	22	21	N.A.	0.5	7	4	0.25	15.2	1	122°	80
30	16,700	27	24			9	4		16.5		120°	100
36	15,700	33	27			12	6		18.8		125°	170
42	15,100	38	30			15			20.0		123°	200
48	25,330	44	33			18			22.3		127°	230
54	26,730	48	36			20			22.7		121°	270
60	38,000	54	39			23			25.0		124°	310
66	38,950	60	42			26			27.2		127°	35D
72	50,700	64	45	10		28		0.375	27.6		122°	420
78	56,500	70	48	11	0.75	31	8		29.8		124°	710
84	57,525	74	51	12		33			30.2		121°	810
90	64,200	80	54	13		36			32.5		123°	880
96	65,400	86	57	14		39			34.7		125°	940
102	94,500	92	60	15		42	10	0.500	37.0	1¼	126°	1,350
108	85,000	96	63	16		44			37.3		123°	1,430
114	164,000	102	66	17		47		0.625	39.6		125°	1,760
120	150,000	106	69	18		49			40.0		122°	1,800
132	127,500	118	75	20		55			44.5		125°	2,180
144	280,000	128	81	22		60			47.0		124°	2,500
156	266,000	140	87	24		66			51.6		126°	2,730

** Table is in inches and pounds and degrees.*

Notes

1. Horizontal vessels act as beams with the following exceptions:
   a. Loading conditions vary for full or partially full vessels.
   b. Stresses vary according to angle θ and distance "A."
   c. Load due to weight is combined with other loads.
2. Large-diameter, thin-walled vessels are best supported near the heads, provided the shell can take the load between the saddles. The resulting stresses in the heads must be checked to ensure the heads are stiff enough to transfer the load back to the saddles.
3. Thick-walled vessels are best supported where the longitudinal bending stresses at the saddles are about equal to the longitudinal bending at midspan. However, "A" should not exceed 0.2 L.
4. Minimum saddle angle $\theta = 120°$, except for small vessels. For vessels designed for external pressure only θ should always $= 120°$. The maximum angle is 168° if a wear plate is used.
5. Except for large L/R ratios or $A > R/2$, the governing stress is circumferential bending at the horn of the saddle. Weld seams should be avoided at the horn of the saddle.
6. A wear plate may be used to reduce stresses at the horn of the saddle *only* if saddles are near heads ($A \leq R/2$), and the wear plate extends R/10 (5.73 deg.) above the horn of the saddle.
7. If it is determined that stiffening rings will be required to reduce shell stresses, move saddles away from the heads (preferable to $A = 0.2$ L). This will prevent designing a vessel with a flexible center and rigid ends. Stiffening ring sizes may be reduced by using a saddle angle of 150°.
8. An internal stiffening ring is the most desirable from a strength standpoint because the maximum stress in the shell is compressive, which is reduced by internal pressure. An internal ring may not be practical from a process or corrosion standpoint, however.
9. Friction factors:

Surfaces	*Friction Factor, μ*
Lubricated steel-to-concrete	0.45
Steel-to-steel	0.4
Lubrite-to-steel	
• Temperature over 500°F	0.15
• Temperature 500°F or less	0.10
• Bearing pressure less than 500 psi	0.15
Teflon-to-Teflon	
• Bearing 800 psi or more	0.06
• Bearing 300 psi or less	0.1

Procedure 4-11: Design of Saddle Supports for Large Vessels [4,15–17,20]

Notation

A_s = cross-sectional area of saddle, in.^{2}
A_b = area of base plate, in.^{2}
A_p = pressure area on ribs, in.^{2}
A_r = cross-sectional area, rib, in.^{2}
Q = maximum load per saddle, lb
Q_1 = $Q_o + Q_R$, lb
Q_2 = $Q_o + Q_L$, lb
Q_o = load per saddle, operating, lb
Q_T = load per saddle, test, lb
Q_L = vertical load per saddle due to longitudinal loads, lb
Q_R = vertical load per saddle due to transverse loads, lb
F_L = maximum longitudinal force due to wind, seismic, pier deflection, etc. (see Procedure 4-10 for detailed description)
F_a = allowable axial stress, psi
F_b = allowable bending stress, psi
F_T = transverse wind or seismic load, lb
N = number of anchor bolts in the fixed saddle
a_t = cross-sectional area of bolts in tension, in.^{2}
Y = effective bearing length, in.
T = tension load in outer bolt, lb
n_1 = modular ratio, steel to concrete, use 10
F_b = allowable bending stress, psi
F_y = yield stress, psi
f_h = saddle splitting force, lb
f_a = axial stress, psi
f_b = bending stress, psi
f_u = unit force, lb/in.
B_p = bearing pressure, psi
M = bending moment, or overturning moment, in.-lb
I = moment of inertia, in.^{4}
Z = section modulus, in.^{3}
r = radius of gyration, in.
K_1 = saddle splitting coefficient
n = number of ribs, including outer ribs, in one saddle
P = equivalent column load, lb
d = distance from base to centroid of saddle arc, in.
W_o = operating weight of vessel plus contents, lb
W_T = vessel weight full of water, lb
σ_T = tension stress, psi
w = uniform load, lb

Forces and Loads

Vertical Load per Saddle

For loads due to the following causes, use the given formulas.

- *Operating weight.*

$$Q_o = \frac{W_o}{2}$$

- *Test weight.*

$$Q_T = \frac{W_T}{2}$$

- *Longitudinal wind or seismic.*

$$Q_L = \frac{F_L B}{L_s}$$

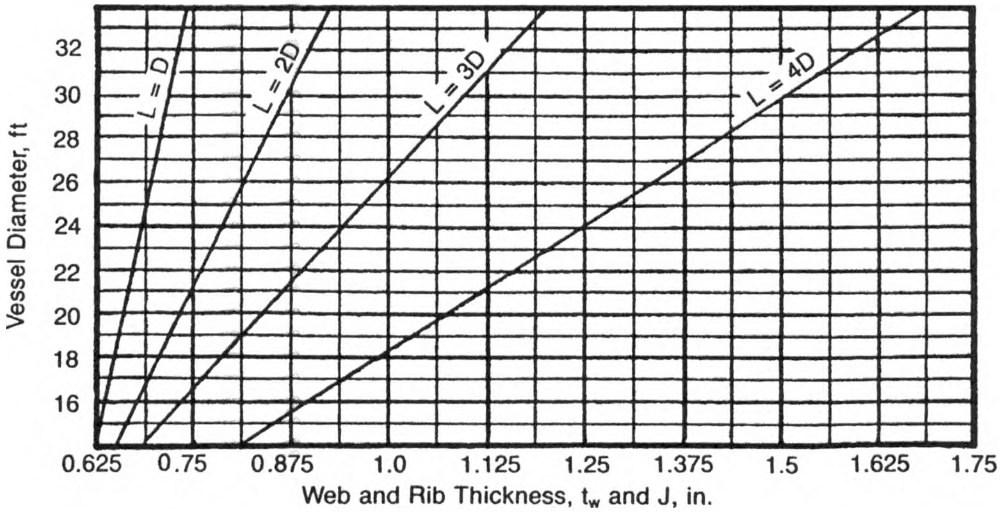


Figure 4-47. Graph for determining web and rib thicknesses.

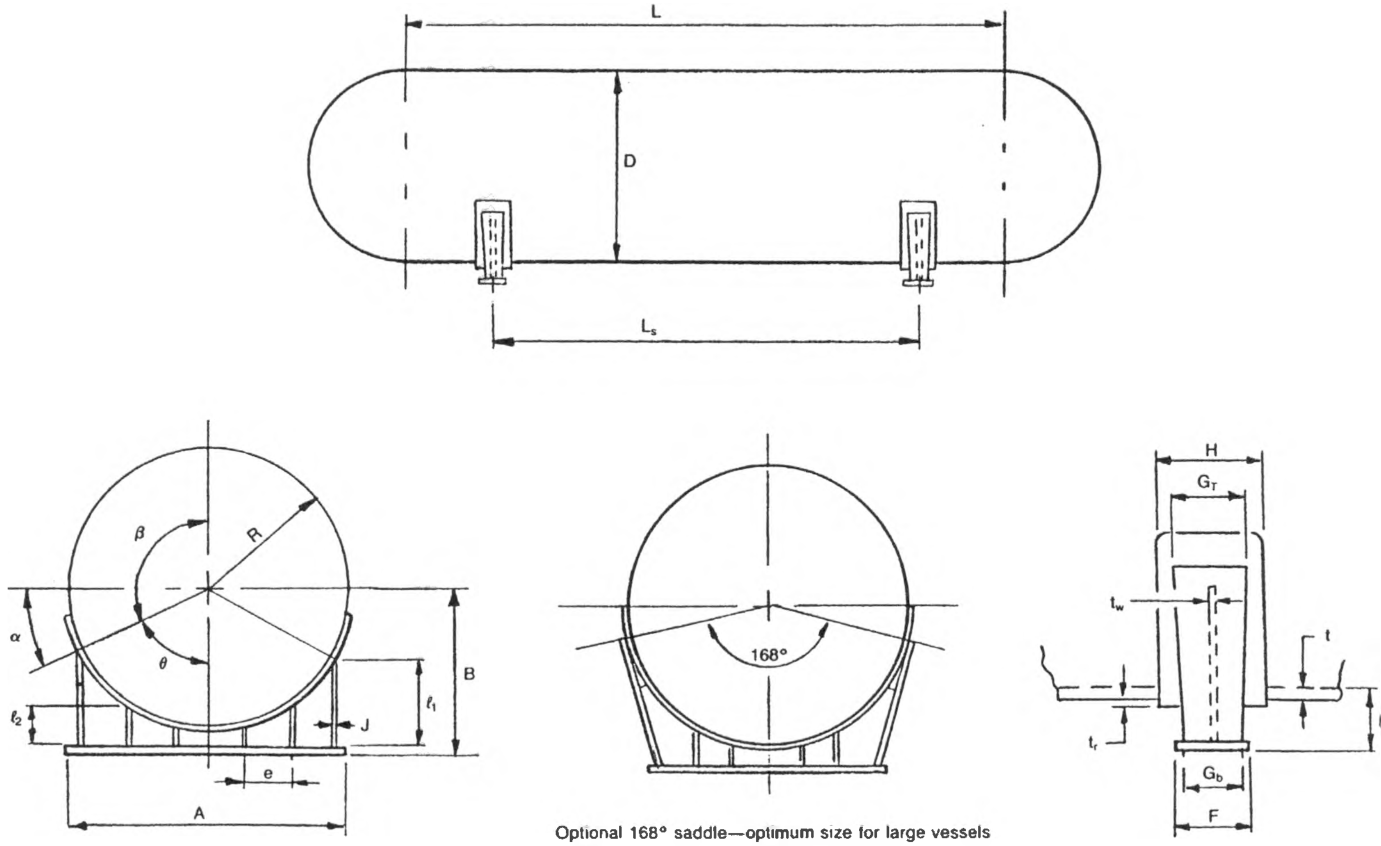


Figure 4-48. Dimensions of horizontal vessels and saddles.

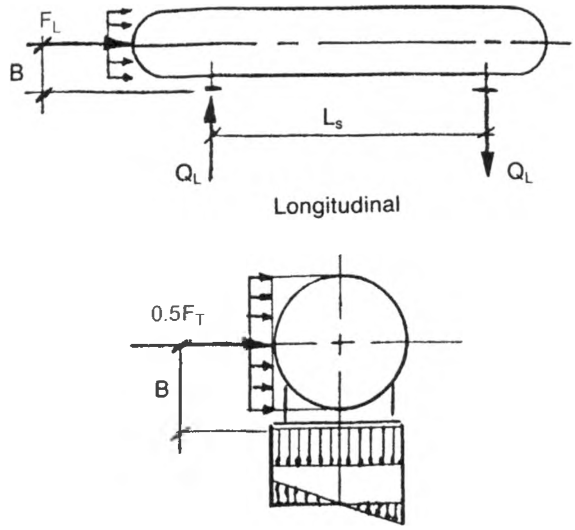


Figure 4-49. Saddle loadings.

- *Transverse wind or seismic.*

$$Q_R = \frac{3F_TB}{A}$$

Maximum Loads

- *Vertical.*
  greater of Q_1, Q_2, or Q_T

$$Q_1 = Q_o + Q_R$$
$$Q_2 = Q_o + Q_L$$

- *Longitudinal.*
  $F_L =$ greater of F_{L1} through F_{L6}

Saddle Properties

- *Preliminary web and rib thicknesses, t_w and J.* From Figure 4-47:

$$J = t_w$$

- *Number of ribs required, n.*

$$n = \frac{A}{24} + 1$$

Round up to the nearest even number.

- *Minimum width of saddle at top, G_T, in.*

$$G_T = \sqrt{\frac{5.012F_L}{J(n-1)F_b}\left[h + \frac{A}{1.96}(1 - \sin\alpha)\right]}$$

where F_L and F_b are in kips and ksi or lb and psi, and J, h, A are in inches.

- *Minimum wear plate dimensions.*

Width:

$$H = G_T + 1.56\sqrt{Rt_s}$$

Thickness:

$$t_r = \frac{(H - G_T)^2}{2.43R}$$

- *Moment of inertia of saddle, I.* See Figure 4-50

$$C_1 = \frac{\sum AY}{\sum A}$$
$$C_2 = h - C_1$$
$$I = \sum AY^2 + \sum I_o - C_1 \sum AY$$

- *Cross-sectional area of saddle (excluding shell).*

$$A_s = \sum A - A_1$$

Design of Saddle Parts

Web

Web is in tension and bending as a result of saddle splitting forces. The saddle splitting forces, f_h, are the sum of all the horizontal reactions on the saddle.

- *Saddle coefficient.* See Table 4-25

$$K_1 = \frac{1 + \cos\beta - 0.5\sin^2\beta}{\pi - \beta + \sin\beta\cos\beta}$$

Note: β is in radians.

- *Saddle splitting force.* See Figure 4-51 and 4-52

$$f_h = K_1(Q \text{ or } Q_T)$$

- *Tension stress.*

$$\sigma_T = \frac{f_h}{A_s} < 0.6F_y$$

Note: For tension assume saddle depth "h" as R/3 maximum.

- *Bending moment.*

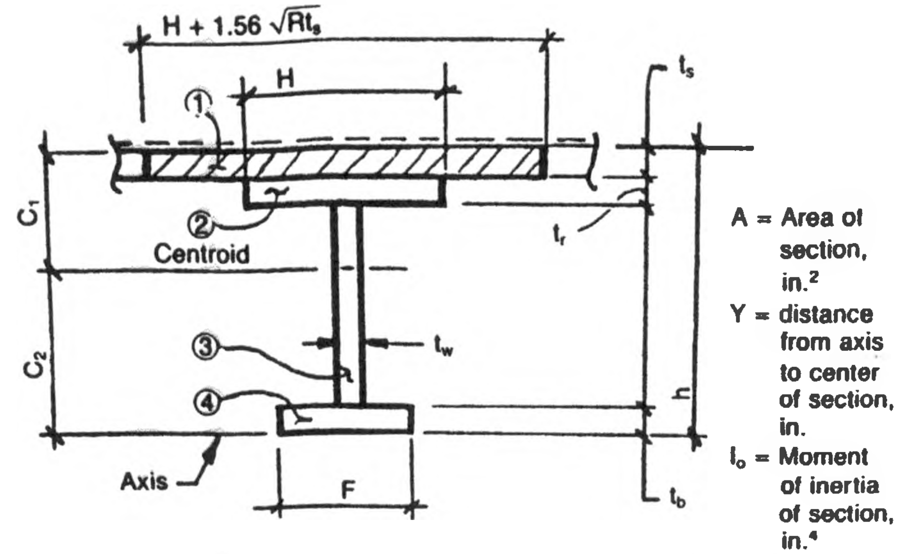
$$d = B - \frac{R\sin\theta}{\theta}$$

θ is in radians.

$$M = f_h d$$

- *Bending stress.*

$$f_b = \frac{MC_1}{I} < 0.66F_y$$



	A	Y	AY	AY²	I_o
①					
②					
③					
④					
Σ					

Note: I_o for rectangles $= \frac{bh^3}{12}$

Figure 4-50. Cross-sectional properties of saddles.

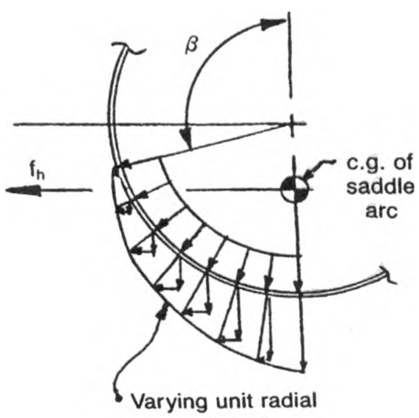
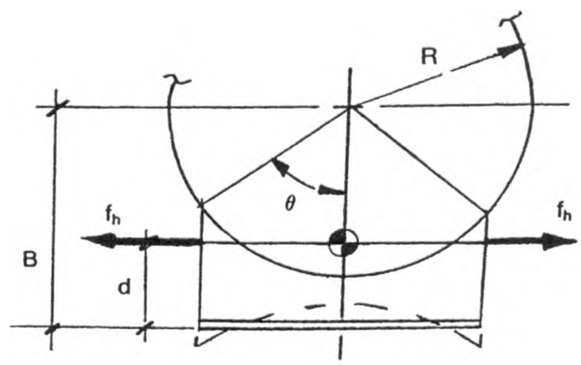


Figure 4-51. Saddle splitting forces.



Note: Circumferential bending at horn is neglected for this calculation.

Figure 4-52. Bending in saddle due to splitting forces.

Base plate with center web see Figure 4-53

- *Area.*

$$A_b = AF$$

- *Bearing pressure.*

$$B_p = \frac{Q}{A_b}$$

- *Base plate thickness.*

$$\text{Now } M = \frac{QF}{8}$$

$$Z = \frac{At_b^2}{6}$$

$$\text{and } f_b = \frac{M}{Z} = \frac{3QF}{4At_b^2}$$

Therefore

Table 4-25
Values of K_1

k_1	2θ
0.204	120°
0.214	126°
0.226	132°
0.237	138°
0.248	144°
0.260	150°
0.271	156°
0.278	162°
0.294	168°

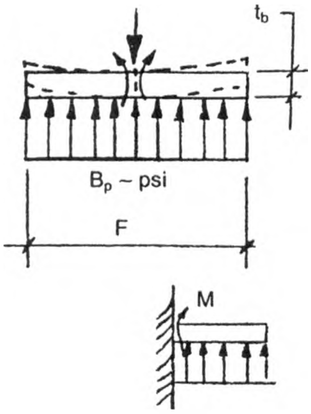


Figure 4-53. Loading diagram of base plate.

$$t_b = \sqrt{\frac{3QF}{4AF_b}}$$

Assumes uniform load fixed in center.

Base plate analysis for offset web (see Figure 4-54)

- *Overall length, $\sum L$.*

  Web $L_w = A - 2d_1 - 2J$

  ribs $L_r = n(G - t_w)$

  $$\sum L = L_w + L_r$$

- *Unit linear load, fu.*

  $$fu = \frac{Q}{\sum L} \text{ lb/linear in.}$$

- *Distances ℓ_1 and ℓ_2.*

  $$\ell_1 = d_2 + t_w + W_w + t_b$$

  $$\ell_2 = F - \ell_1$$

- *Loads / moment.*

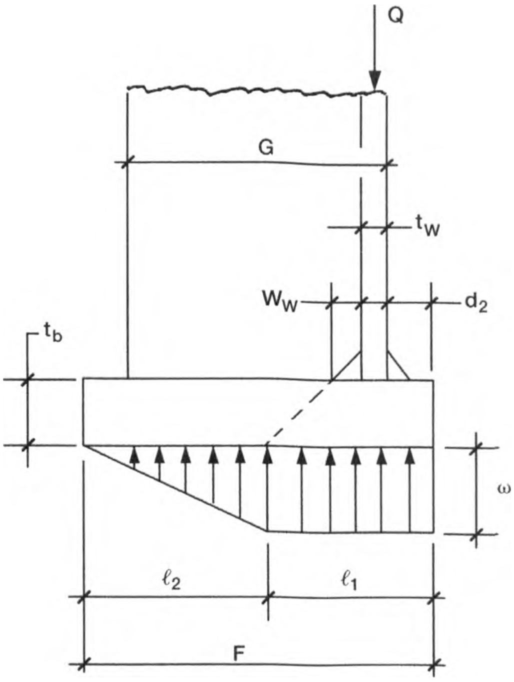


Figure 4-54. Load diagram and dimensions for base plate with an offset web.

$$\omega = \frac{fu}{\ell_1 + 0.5\ell_2}$$

$$M = \frac{\omega \ell_2^2}{6}$$

- *Bending stress, f_b.*

  $$f_b = \frac{6M}{t_b^2}$$

Anchor Bolts

Anchor bolts are governed by one of the three following load cases:

1. *Longitudinal load*: If $Q_o > Q_L$, then no uplift occurs, and the minimum number and size of anchor bolts should be used.
   If $Q_o < Q_L$, then uplift does occur:
   $$\frac{Q_L - Q_o}{N} = \text{load per bolt}$$
2. *Shear*: Assume the fixed saddle takes the entire shear load.
   $$\frac{F_L}{N} = \text{shear per bolt}$$

3. *Transverse load*: This method of determining uplift and overturning is determined from Ref. [20] (see Figure 4-56).

$$M = 0.5\,F_T\,B$$

$$e = \frac{M}{Q_o}$$

If $e < A/6$, then there is no uplift.

If $e \geq A/6$, then proceed with the following steps. This is an iterative procedure for finding the tension force, T, in the outermost bolt.

Step 1: Find the effective bearing length, Y. Start by calculating factors $K_{1\text{-}3}$.

$$K_1 = 3(e - 0.5A)$$

$$K_2 = \frac{6n_1a_t}{F}(f + e)$$

$$K_3 = (-)K_2\left[\frac{A}{2} + f\right]$$

Step 2: Substitute values of K_{1-3} into the following equation and assume a value of $Y = 2/3\,A$ as a first trial.

$$Y^3 + K_1Y^2 + K_2Y + K_3 = 0$$

If not equal to 0, then proceed with Step 3.

Step 3: Assume a new value of Y and recalculate the equation in Step 2 until the equation balances out to approximately 0. Once Y is determined, proceed to Step 4.

Step 4: Calculate the tension force, T, in the outermost bolt or bolts.

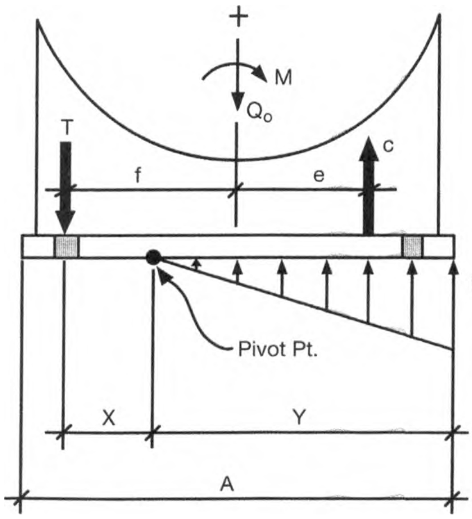


Figure 4-55. Dimensions and loading for base plate and anchor bolt analysis.

$$T = (-)Q_o\left[\frac{\frac{A}{2} - \frac{Y}{3} - e}{\frac{A}{2} - \frac{Y}{3} + f}\right]$$

Step 5: Select an appropriate bolt material and size corresponding to tension force, T.

Step 6: Analyze the bending in the base plate.

Distance, $x = 0.5A + f - Y$

Moment, $M = Tx$

Bending stress, $f_b = \frac{6M}{t_b^2}$

Ribs

Outside Ribs

- *Axial load, P.*

$$P = B_pA_p$$

- *Compressive stress, f_a.*

$$f_a = \frac{P}{A_r}$$

- *Radius of gyration, r.*

$$r = \sqrt{\frac{I_1}{A_r}}$$

- *Slenderness ratio, ℓ_1/r.*

$l_1/r =$

$F_a =$

Outside Ribs

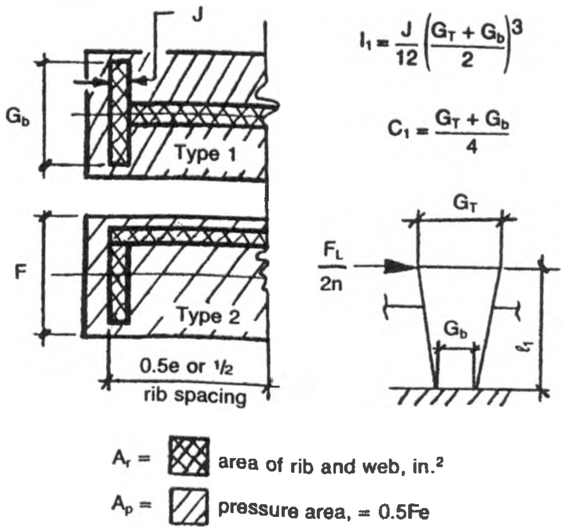


Figure 4-56. Dimensions of outside saddle ribs and webs.

- *Unit force, f_u.*

$$f_u = \frac{F_L}{2A}$$

- *Bending moment, M.*

$$M = 0.5 f_u e l_1$$

- *Bending stress, $F_b = 0.66\ F_y$.*

$$f_b = \frac{MC_1}{I} < F_b$$

- *Combined stress.*

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1$$

Inside Ribs

- *Axial load, P.*

$$P = B_p A_p$$

- *Compressive stress, f_a.*

$$f_a = \frac{P}{A_r}$$

- *Radius of gyration, r.*

$$r = \sqrt{\frac{I_2}{A_r}}$$

- *Slenderness ratio, ℓ_2/r.*

$l_2/r =$

$F_a =$

- *Unit force, f_u.*

$$f_u = \frac{F_L}{2A}$$

- *Bending moment, M.*

$$M = f_u l_2 e$$

- *Bending stress, f_b.*

$$f_b = \frac{MC_2}{I}$$

- *Combined stress.*

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1$$

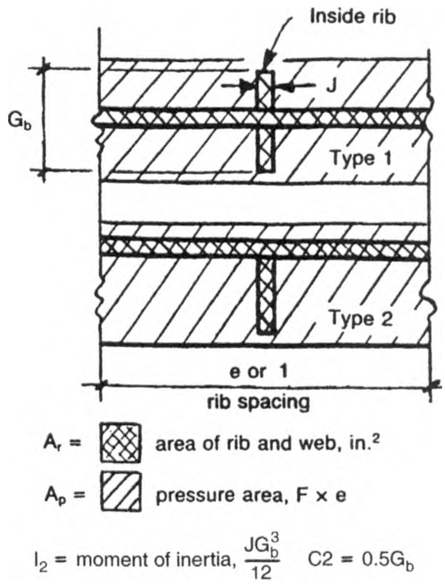


Figure 4-57. Dimensions of inside saddle ribs and webs.

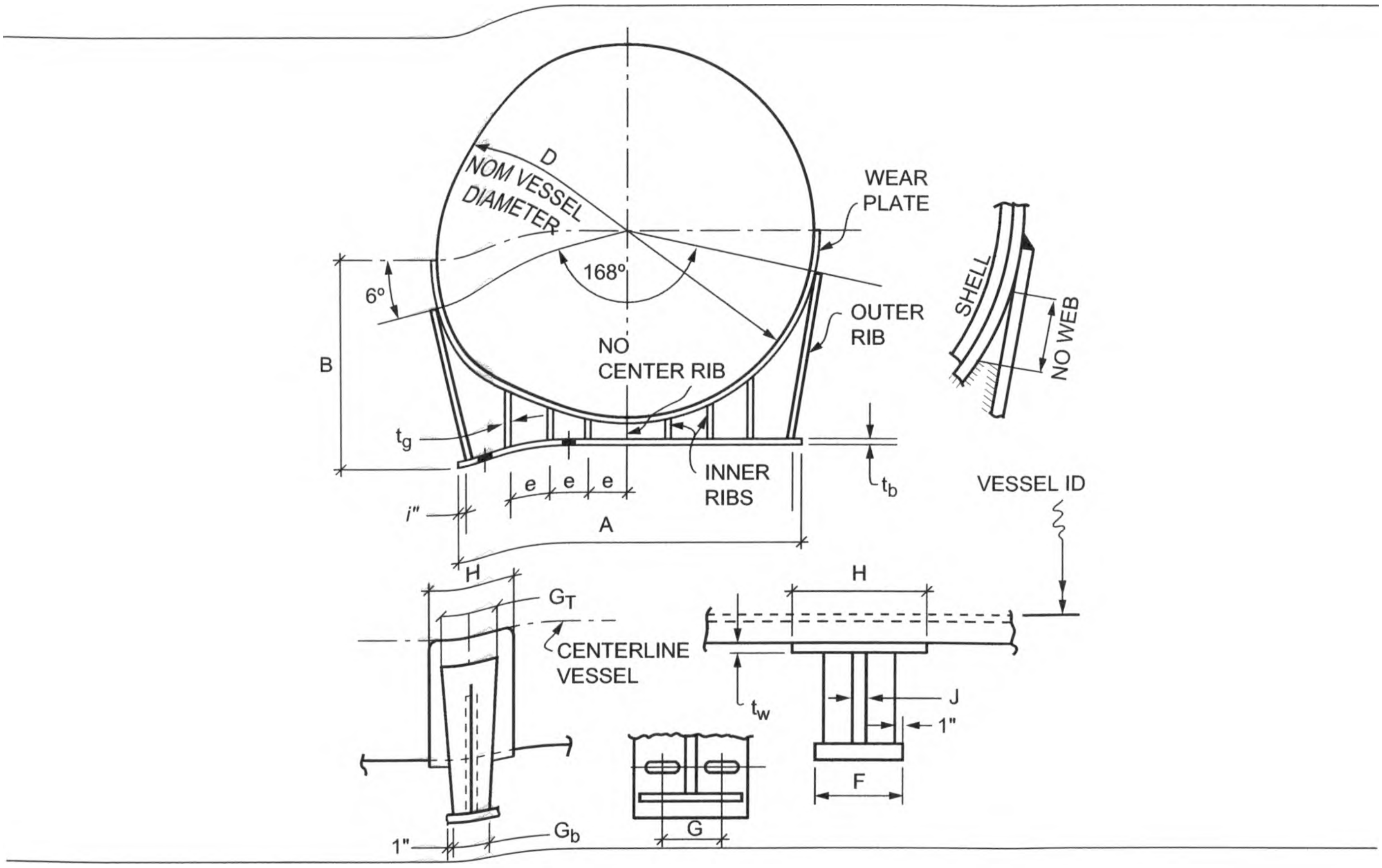
Notes

1. The depth of web is important in developing stiffness to prevent bending about the cross-sectional axis of the saddle. For larger vessels, assume 6 in. as the minimum depth from the bottom of the wear plate to the top of the base plate.
2. The full length of the web may be assumed effective in carrying compressive stresses along with ribs. Ribs are not effective at carrying compressive load if they are spaced greater than 25 times the web thickness apart.
3. Concrete compressive stresses are usually considered to be uniform. This assumes the saddle is rigid enough to distribute the load uniformly.
4. Large-diameter horizontal vessels are best supported with 168° saddles. Larger saddle angles do not effectively contribute to lower shell stresses and are more difficult to fabricate. The wear plate need not extend beyond center lines of vessel in any case or 6° beyond saddles.
5. Assume fixed saddle takes all of the longitudinal loading.

Table 4-26
Allowable tension load on bolts, kips, per AISC

Nominal Bolt Diameter, in		0.625	0.750	0.875	1.000	1.125	1.250	1.375	1.500
Cross-sectional Area, a_b, in^2		0.3068	0.4418	0.6013	0.7854	0.9940	1.2272	1.4849	1.7671
A-307 F_t	22.5	6.9	9.9	13.5	17.7	22.4	27.6	33.4	39.8
A-325 F_t	45.0	13.8	19.9	27.1	35.3	44.7	55.2	66.8	79.5

Table 4-27
Large diameter saddle supports



DIA (Ft)	A	B	D (Note 3)	E	F	G	G_B	G_T	H	J	N	n	t_b	t_g	t_w	Approx wt for 2 Saddles (Kips)
14	155	92	1.375	19	18	9	16	28	34	0.75	8	8	1	0.75	0.75	7
15	171	100	1.375	21	18	9	16	28	34	0.75	8	8	1	0.75	0.75	8
16	183	108	1.375	18	18	9.25	16	28	34	1	10	8	1.125	0.875	0.875	10
17	193	114	1.625	19	21	10.75	19	31	37	1	10	8	1.25	1	1	11
18	207	122	1.625	17	21	11	19	31	37	1.125	12	12	1.375	1.125	1	12
20	207	132	1.625	17	21	11	19	31	37	1.125	12	12	1.375	1.125	1.125	15.5
22	219	144	1.875	18	24	12.5	22	34	40	1.25	12	12	1.5	1.25	1.25	19
24	241	156	1.875	17	24	12.5	22	34	40	1.25	14	12	1.5	1.25	1.25	22
26	255	172	1.875	18	24	12.5	22	34	40	1.375	14	12	1.625	1.25	1.25	26
28	275	184	2.125	17	27	14	25	37	43	1.375	16	16	1.625	1.375	1.25	31
30	308	196	2.125	19	27	14.25	25	37	43	1.5	16	16	1.75	1.5	1.375	37
32	328	208	2.125	18	27	14.25	25	37	43	1.5	18	16	1.75	1.5	1.375	44
34	346	220	2.375	19	31	16	29	41	47	1.75	18	16	2	1.75	1.375	54
36	364	230	2.375	18	31	16	29	41	47	1.75	20	16	2	1.75	1.375	66
38	384	244	2.375	19	32	16.25	30	42	48	2	20	16	2.25	2	1.5	80
40	404	256	2.625	20	34	17.75	32	44	50	2	20	20	2.5	2	1.5	100

Notes:

1. All dimensions are in inches unless noted otherwise
2. All saddles in this size range must be fully designed. The dimensions shown are a starting place or to be used for estimating only!
3. Assume that anchor bolts diameter is d – .125", where d is the diameter of the hole. Assume that slots for sliding saddle are 6 d long.
4. N = Number of ribs
5. n = Number of anchor bolts

Procedure 4-12: Design of Base Plates for Legs [20,21]

Notation

Y = effective bearing length, in.
M = overturning moment, in.-lb
M_b = bending moment, in.-lb
P = axial load, lb
f_t = tension stress in anchor bolt, psi
A = actual area of base plate, in.^{2}
A_r = area required, base plate, in.^{2}
f'_c = ultimate 28-day strength, psi
f_c = bearing pressure, psi
f_1 = equivalent bearing pressure, psi
F_b = allowable bending stress, psi
F_t = allowable tension stress, psi
F_c = allowable compression stress, psi
E_s = modulus of elasticity, steel, psi
E_c = modulus of elasticity, concrete, psi
n = modular ratio, steel-concrete
n' = equivalent cantilever dimension of base plate, in.
B_p = allowable bearing pressure, psi
$K_{1,2,3}$ = factor
T = tension force in outermost bolt, lb
C = compressive load in concrete, lb
V = base shear, lb
N = total number of anchor bolts
N_t = number of anchor bolts in tension
A_b = cross-sectional area of one bolt, in.^{2}
A_s = total cross-sectional area of bolts in tension, in.^{2}
α = coefficient
T_s = shear stress

Calculations

- *Axial loading only, no moment.*

Angle legs:

$$f_c = \frac{P}{BD}$$

L = greater of m, n, or n′

$$t = \sqrt{\frac{3f_cL^2}{F_b}}$$

Beam legs:

$$A_r = \frac{P}{0.7f'_c}$$

$$m = \frac{D - 0.95d}{2}$$

$$n = \frac{B - 0.8d}{2}$$

$$\alpha = \frac{b - t_w}{2(d - 2t_f)}$$

$$n' = \frac{b - t_w}{2}\sqrt{\frac{1}{1 + 3.2\alpha^3}} \text{ (See Table 4-27)}$$

Pipe legs:

$$m = \frac{B - 0.707W}{2}$$

$$f_c = \frac{P}{A}$$

$$t = \sqrt{\frac{3f_cm^2}{F_b}}$$

- *Axial load plus bending, load condition #1, full compression, uplift, $e \leq D/6$.* (See Figure 4-59)

Eccentricity:

$$e = \frac{M}{P} \leq \frac{D}{6}$$

Loadings:

$$f_c = \frac{P}{A}\left[1 + \frac{6e}{D}\right]$$

$$f_1 = \frac{P}{A}\left[1 + \frac{6e(D - 2a)}{D^2}\right]$$

Moment:

$$M_b = \frac{a^2B}{6}(f_1 + 2f_c)$$

Thickness:

$$t = \sqrt{\frac{6M_b}{BF_b}}$$

- *Axial load plus bending, load condition #2, partial compression, uplift, $e > D/6$.* (See Figure 4-59)

Eccentricity:

$$e = \frac{M}{P} > \frac{D}{6}$$

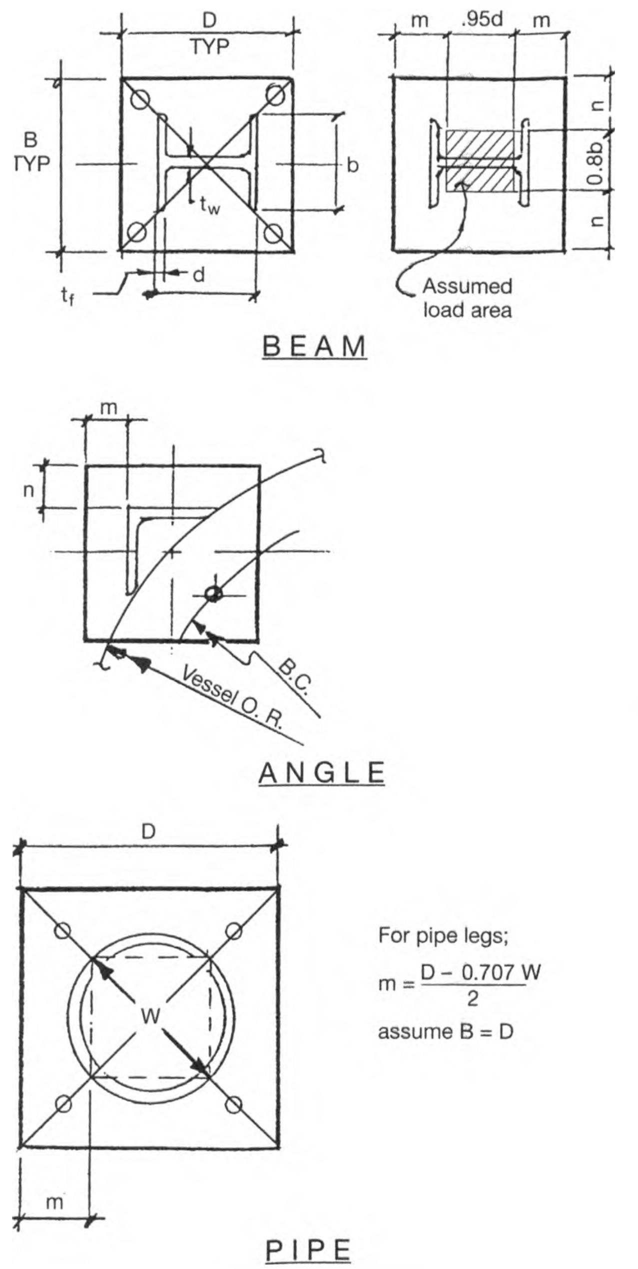


Figure 4-58. Dimensions and loadings of base plates.

Coefficient: (See Table 4-29)

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

Dimension:

$$f = 0.5d + z$$

By trial and error, determine Y, effective bearing length, utilizing factors K_{1-3}.

Factors:

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

$$K_2 = \frac{6n_rA_s}{B}(f + e)$$

$$K_3 = (-)K_2(0.5D + f)$$

By successive approximations, determine distance Y. Substitute $K_{1\text{-}3}$ into the following equation and assume an initial value of $Y = \frac{2}{3}A$ as a first trial.

$$Y^3 + K_1Y^2 + K_2Y + K_3 = 0$$

Tension force:

$$T = (-)P\left[\frac{\frac{D}{2} - \frac{Y}{3} - e}{\frac{D}{2} - \frac{Y}{3} + f}\right]$$

Bearing pressure:

$$f_c = \frac{2(P + T)}{YB} < f'_c$$

Moment:

$$x = 0.5D + f - Y$$

$$M_t = Tx$$

$$f_1 = f_c\left(\frac{Y - a}{Y}\right)$$

$$M_c = \frac{a^2B}{6}(f_1 + 2f_c)$$

Thickness:

$$t = \sqrt{\frac{6M_b}{BF_b}}$$

where M_b is greater of M_T or M_c.

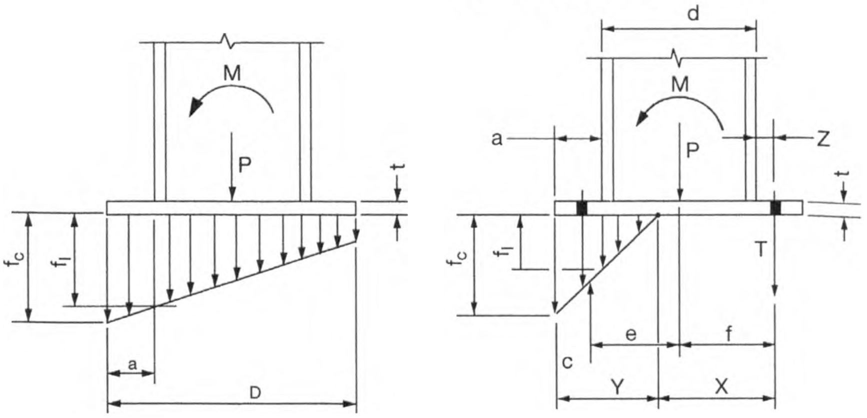
- *Anchor bolts.*
  Without uplift: design anchor bolts for shear only.

$$T_s = \frac{V}{NA_b}$$

With uplift: design anchor bolts for full shear and tension force, T.

$$f_t = \frac{T}{N_TA_b}$$

Load Condition #1 Load Condition #2



Full compression, no uplift, e ≤ D/6 Partial compression, uplift, e > D/6

Figure 4-59. Load conditions on base plates.

Table 4-28
Values of n′ for beams

Column Section	n′	Column Section	n′
W14 × 730 – W14 × 145	5.77	W10 × 45 – W10 × 33	3.42
W14 × 132 – W14 × 90	5.64	W8 × 67 – W8 × 31	3.14
W14 × 82 – W14 × 61	4.43	W8 × 28 – W8 × 24	2.77
W14 × 53 – W14 × 43	3.68	W6 × 25 – W6 × 15	2.38
W12 × 336 – W12 × 65	4.77	W6 × 16 – W6 × 9	1.77
W12 × 58 – W12 × 53	4.27	W5 × 19 – W5 × 16	1.91
W12 × 50 – W12 × 40	3.61	W4 × 13	1.53
W10 × 112 – W10 × 49	3.92		

Table 4-29
Average properties of concrete

Water Content/Bag	Ult f'_c 28 -Day Str (psi)	Allowable Compression, F_c (psi)	Allowable B_p (psi)	Coefficient, n_r
7.5	2000	800	500	15
6.75	2500	1000	625	12
6	3000	1200	750	10
5	3750	1400	938	8

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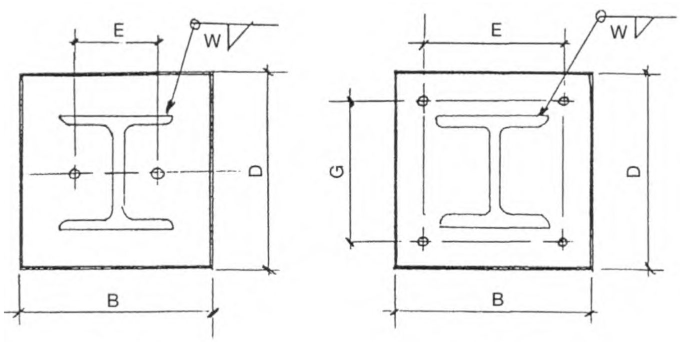
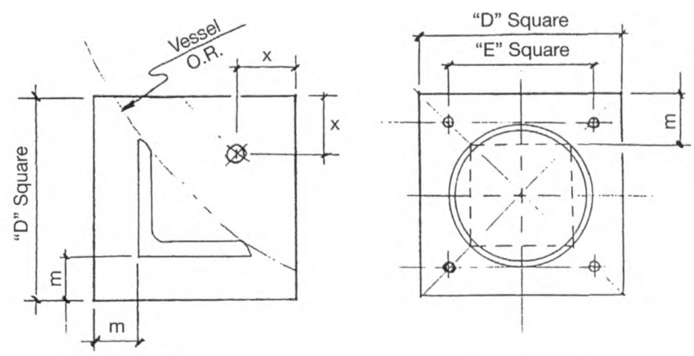


Type 1 Type 2

Figure 4-60. Dimensions for base plates-beams.



Angle Legs Pipe Legs

Figure 4-61. Dimensions for base plates—angle/pipe.

Dimensions for Type 1—(2) Bolt Base Plate

Column Size	D, in.	B, in.	E, in.	W, in.	Min Plate Thk, in.	Max Bolt φ, in.
W4	8	8	4	¼	⅝	¾
W6	8	8	4	¼	¾	¾
W8	10	10	6	¼	¾	¾
W10–33 thru 45	12	12	6	5/16	¾	1
W10–49 thru 112	13	13	6	5/16	¾	1
W12–40 thru 50	14	10	6	5/16	⅞	1
W12–53 thru 58	14	12	6	5/16	⅞	1
W12–65 thru 152	15	15	8	5/16	⅞	1¼

Dimensions for Type 2—(4) Bolt Base Plate

Column Size	D, in.	B, in.	G, in.	E, in.	W, in.	Min Plate Thk, in.	Max Bolt φ, in.
W4	10	10	7	7	¼	⅝	1
W6	12	12	9	9	5/16	¾	1
W8	15	15	11	11	⅜	⅞	1
W10–33 thru 45	17	15	13	11	⅜	⅜	1¼
W10–49 thru 112	17	17	13	13	⅜	⅜	1¼
W12–40 thru 50	19	15	15	11	⅜	1	1½
W12–53 thru 58	19	17	15	13	⅜	1	1½
W12–65 thru 152	19	19	15	15	⅜	1	1½

Dimensions for Angle Legs

Leg Size	D	X	m	Min. Plate Thk
L2 in. × 2 in.	4 in.	1.5	1	½ in.
L2½in. × 2½in.	5 in.	1.5	1.25	½ in.
L3 in. × 3 in.	6 in.	1.75	1.5	½ in.
L4 in. × 4 in.	8 in.	2	2	⅝ in.
L5 in. × 5 in.	9 in.	2.75	2	⅝ in.
L6 in. × 6 in.	10 in.	3.5	2	¾ in.

Dimensions for Pipe Legs

Leg Size	D	E	m	Min. Plate Thk
3 in. NPS	7 ½in.	4 ½ in.	2.5 in.	½ in.
4 in. NPS	8 ½in.	5 ½ in.	2.7 in.	½ in.
6 in. NPS	10 in.	7 in.	2.7 in.	⅝ in.
8 in. NPS	11 ½in.	8 ½ in.	2.7 in.	¾ in.
10 in. NPS	14 in.	10 in.	3.2 in.	⅞ in.
12 in. NPS	16 in.	12 in.	3.5in.	1 in.

Procedure 4-13: Design of Lug Supports

Notation

Q = vertical load per lug, lb
Q_a = axial load on gusset, lb
Q_b = bending load on gusset, lb
n = number of gussets per lug
F_a = allowable axial stress, psi
F_b = allowable bending stress, psi
f_a = axial stress, psi
f_b = bending stress, psi
A = cross-sectional area of assumed column, in.^{2}
Z = section modulus, in.^{3}
w = uniform load on base plate, lb/in.
I = moment of inertia of compression plate, in.^{4}
E_v = modulus of elasticity of vessel shell at design temperature, psi
E_s = modulus of elasticity of compression plate at design temperature, psi
e = log base 2.71
M_b = bending moment, in.-lb
M_x = internal bending moment in compression plate, in.-lb
K = spring constant or foundation modulus
β = damping factor

Design of Gussets

Assume gusset thickness from Table 4-30.

$$Q_a = Q \sin\theta$$

$$Q_b = Q \cos\theta$$

$$C = \frac{b \sin\theta}{2}$$

$$A = t_g C$$

$$F_a = 0.4F_y$$

$$F_b = 0.6F_y$$

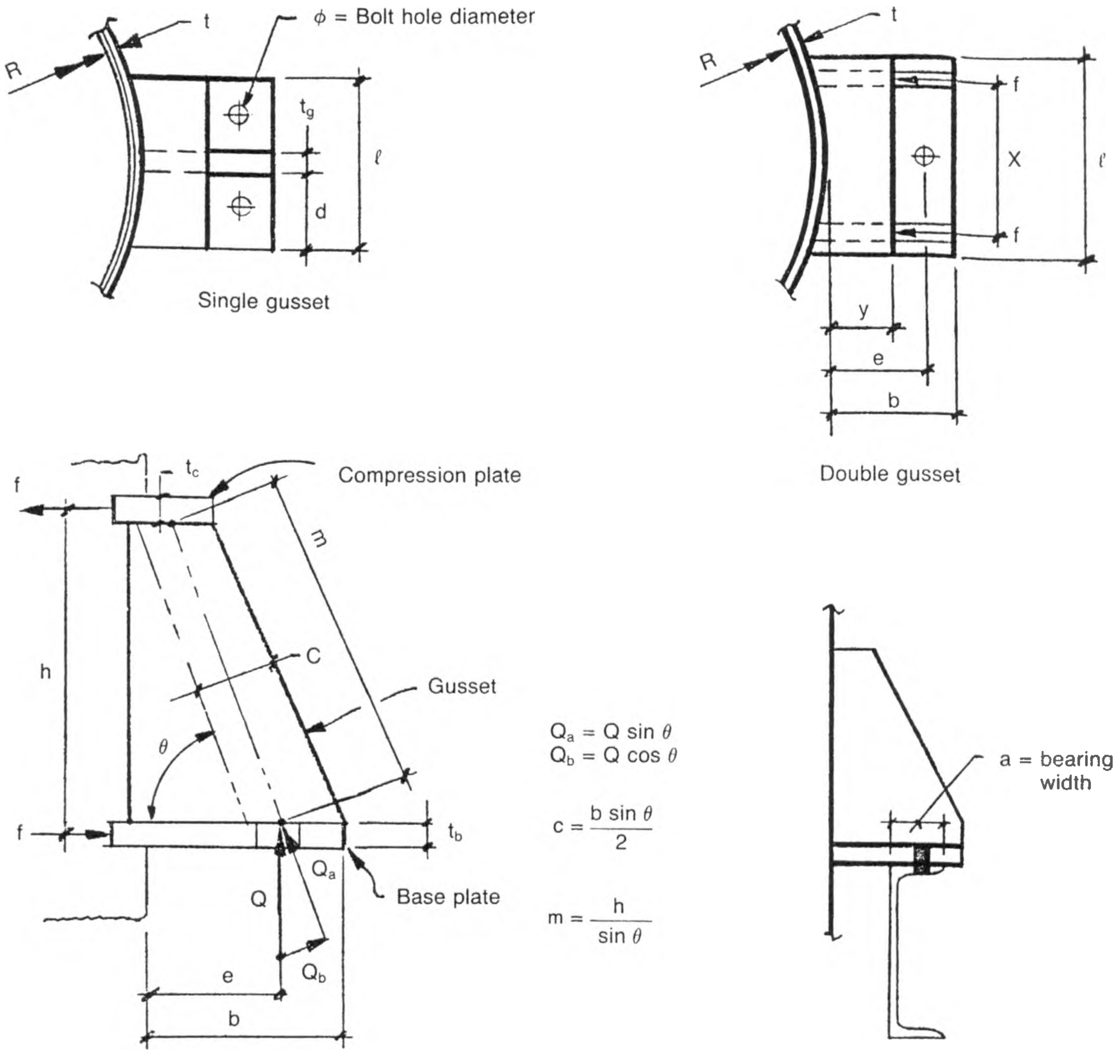


Figure 4-62. Dimensions and forces on a lug support.

$$Z = \frac{t_g C^2}{6}$$

$$M_b = \frac{Q_b m}{n}$$

$$f_a = \frac{Q_a}{nA}$$

$$f_b = \frac{M_b}{Z}$$

Design of Base Plate

Single Gusset

- *Bending.* Assume to be a simply supported beam.

$$M_b = \frac{Ql}{4}$$

- *Bearing.*

$$w = \frac{Q}{al}$$

$$M_b = \frac{wd^2}{2}$$

- *Thickness required base plate.*

$$t_b = \sqrt{\frac{6M_b}{(b - \phi)F_b}}$$

where M_b is greater moment from bending or bearing.

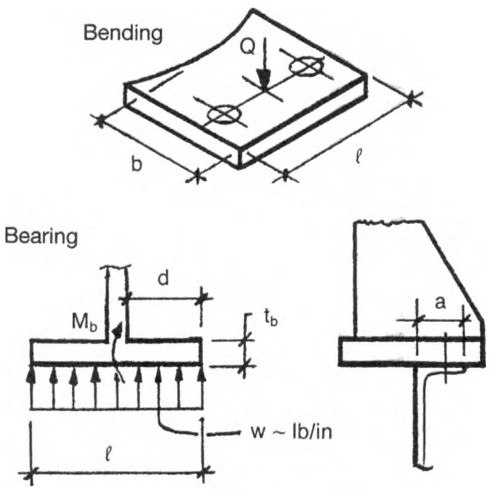


Figure 4-63. Loading diagram of base plate with one gusset.

Double Gusset

- *Bending.* Assume to be between simply supported and fixed.

$$M_b = \frac{Ql}{6}$$

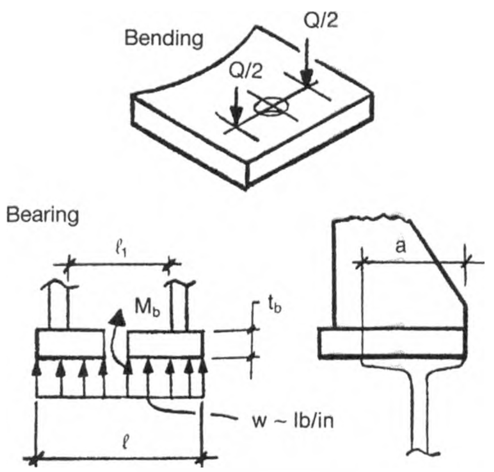


Figure 4-64. Loading diagram of base plate with two gussets.

- *Bearing.*

$$w = \frac{Q}{al}$$

$$M_b = \frac{wl_1^2}{10}$$

- *Thickness required base plate.*

$$t_b = \sqrt{\frac{6M_b}{(b - \phi)F_b}}$$

where M_b is greater moment from bending or bearing.

Compression Plate

Single Gusset

$$f = \frac{Qe}{h}$$

$$K = \frac{E_y t}{R^2}$$

Assume thickness t_c and calculate **I** and **Z**:

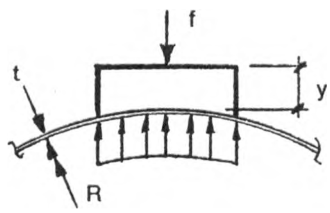


Figure 4-65. Loading diagram of compression plate with one gusset.

$$I = \frac{t_c y^3}{12}$$

$$Z = \frac{t_c y^2}{6}$$

$$\beta = \sqrt[4]{\frac{K}{4E_s I}}$$

$$M_x = \frac{f}{4\beta}$$

$$f_b = \frac{M_x}{Z} < 0.6F_y$$

Note: These calculations are based on a beam on elastic foundation methods.

Double Gusset

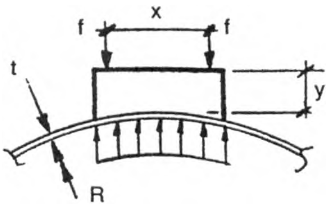


Figure 4-66. Loading diagram of compression plate with two gussets.

$$f = \frac{Qe}{2h}$$

$$K = \frac{E_y t}{R^2}$$

$$I = \frac{t_c y^3}{12}$$

$$Z = \frac{t_c y^2}{6}$$

$$\beta = \sqrt[4]{\frac{K}{4E_s I}}$$

Table 4-30
Standard lug dimensions

Type	e	b	y	x	h	$t_g = t_b$	Capacity (lb)
1	4	6	2	6	6	3/8	23,500
2	4	6	2	6	9	7/16	45,000
3	4	6	2	6	12	½	45,000
4	5	7	2.5	7	15	9/16	70,000
5	5	7	2.5	7	18	5/8	70,000
6	5	7	2.5	7	21	11/16	70,000
7	6	8	3	8	24	3/4	100,000

$$M_x = \frac{f}{4\beta}\left[1 + \left(e^{-\beta x}(\cos \beta x - \sin \beta x)\right)\right]$$

β x is in radians.

$$f_b = \frac{M_x}{Z} < 0.6F_y$$

Procedure 4-14: Design of Base Details for Vertical Vessels – Shifted Neutral Axis Method [4,9,13,17,18]

Notation

A_b = required area of anchor bolts, in.^{2}
B_d = anchor bolt diameter, in.
B_p = allowable bearing pressure, psi
b_p = bearing stress, psi
C = compressive load on concrete, lb
d = diameter of bolt circle, in.
d_b = diameter of hole in base plate of compression plate or ring, in.
F_{LT} = longitudinal tension load, lb/in.
F_{LC} = longitudinal compression load, lb/in.
F_b = allowable bending stress, psi
F_c = allowable compressive stress, concrete, psi
F_s = allowable tension stress, anchor bolts, psi
F_y = minimum specified yield strength, psi
f_b = bending stress, psi
f_c = compressive stress, concrete, psi
f_s = equivalent tension stress in anchor bolts, psi
M_b = overturning moment at base, in.-lb
M_t = overturning moment at tangent line, in.-lb
M_x = unit bending moment in base plate, circumferential, in.-lb/in.
M_y = unit bending moment in base plate, radial, in.-lb/in.
H = overall vessel height, ft
δ = vessel deflection, in.
M_o = bending moment per unit length in.-lb/in.
N = number of anchor bolts
n = ratio of modulus of elasticity of steel to concrete
P = maximum anchor bolt force, lb
P_1 = maximum axial force in gusset, lb

E = joint efficiency of skirt-head attachment weld
R_a = root area of anchor bolt, in.^{2}
r = radius of bolt circle, in.
W_b = weight of vessel at base, lb
W_t = weight of vessel at tangent line, lb
w = width of base plate, in.
Z_1 = section modulus of skirt, in.^{3}
S_t = allowable stress (tension) of skirt, psi
S_c = allowable stress (compression) of skirt, psi
G = width of unreinforced opening in skirt, in.
C_c, C_T, J, Z, K = coefficients
γ_1, γ_2 = coefficients for moment calculation in compression ring
S = code allowable stress, tension, psi
E_1 = modulus of elasticity, psi
t_s = equivalent thickness of steel shell which represents the anchor bolts in tension, in.
T = tensile load in steel, lb
ν = Poisson's ratio, 0.3 for steel
B = code allowable longitudinal compressive stress, psi

Equivalent Area Method

The "Equivalent Area Method" is also known as the "Shifted Neutral Axis Method". This procedure is in contrast with the "Centered Neutral Axis Method" which assumes that the neutral axis is on the centerline. The Centered Neutral Axis Method is easier to apply but also results in a conservative anchorage design. The Equivalent Area method is more accurate and will result in reduced anchorage requirements. Both methods are used to determine the anchorage requirements and the base plate details of a vertical vessel supported on a skirt.

The Equivalent Area Method is based on reinforced concrete beam design that utilizes a balance between the steel in tension and the concrete in compression. Because of the different properties the neutral axis is shifted from the centerline. This procedure enables the designer to find the exact position of the neutral axis and compute the properties required based on this location.

In order to find the minimum anchor bolt area required that is consistent with a given base ring area and a given working stress in the anchor bolts, it is necessary to resort to a trial and error basis, an iterative procedure. To start, the variables are either given or assumed. The variables in this process are as follows;

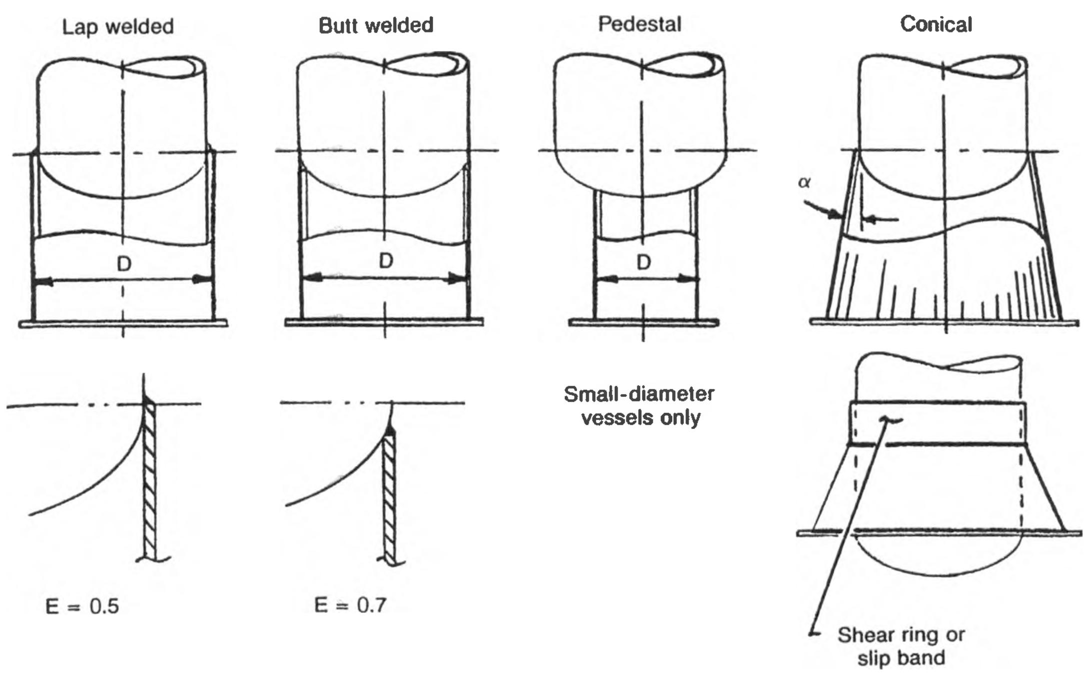


Figure 4-67. Skirt types.

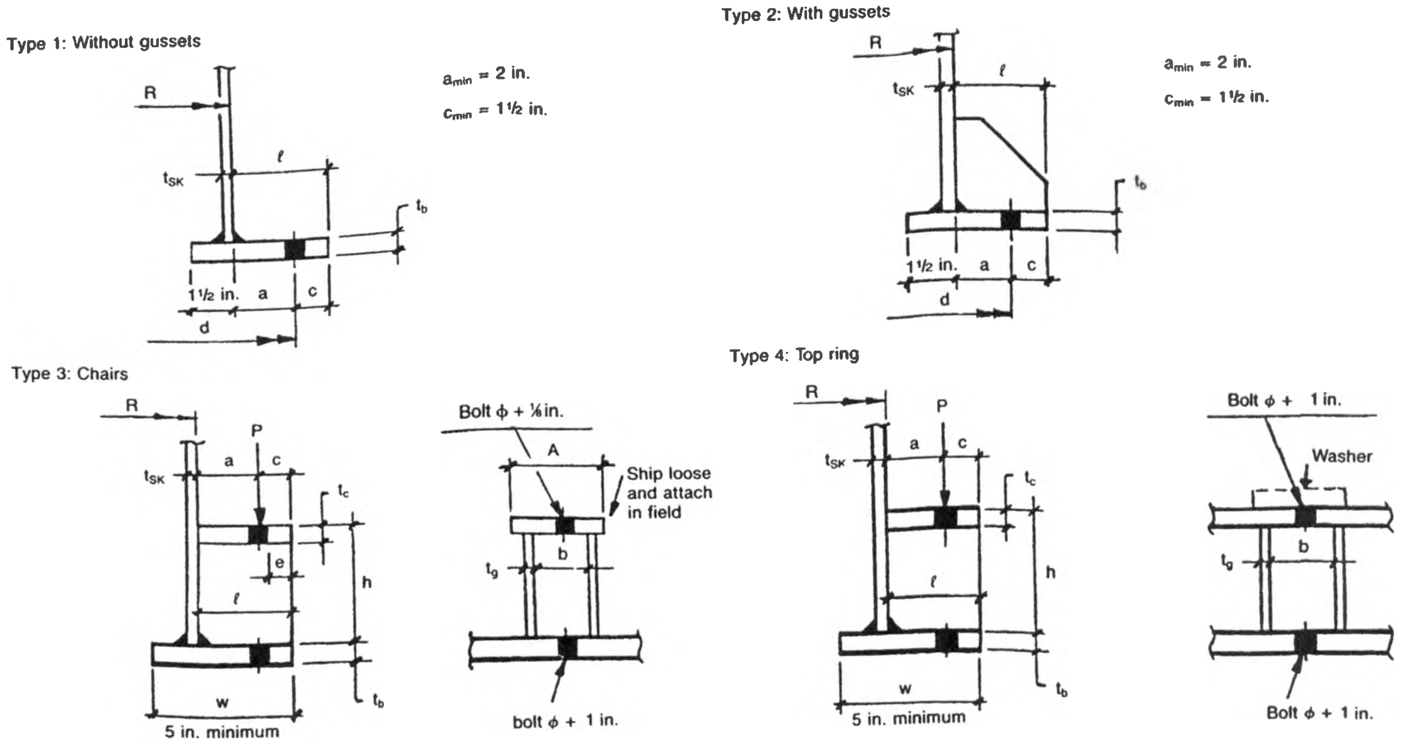


Figure 4-68. Base details of various types of skirt-supported vessels.

Table 4-31
Bolt chair data

Size (in.)	A_{min}	R_a	a_{min}	b	c_{min}
¾–10	5.50	0.302	2	3.50	1.5
⅞–9	5.50	0.419	2	3.50	1.5
1–8	5.50	0.551	2	3.50	1.5
1 ⅛–7	5.50	0.693	2	3.50	1.5
1 ¼–7	5.50	0.890	2	3.50	1.5
1 ⅜–6	5.50	1.054	2.13	3.50	1.75
1 ½–6	5.75	1.294	2.25	3.50	2
1 ⅝–5 ½	5.75	1.515	2.38	4.00	2
1 ¾–5	6.00	1.744	2.5	4.00	2.25
1 7/8–5	6.25	2.049	2.63	4.00	2.5
2–4 ½	6.50	2.300	2.75	4.00	2.5
2¼–4 ½	7.00	3.020	3	4.50	2.75
2½–4	7.25	3.715	3.25	4.50	3
2 ¾–4	7.50	4.618	3.50	4.75	3.25
3–4	8.00	5.621	3.75	5.00	3.50

Table 4-32
Number of anchor bolts, N

Skirt Diameter (in.)	Minimum	Maximum
24–36	4	4
42–54	4	8
60–78	8	12
84–102	12	16
108–126	16	20
132–144	20	24

**See also Table 4-40*

Table 4-33
Allowable stress for bolts, F_s

Spec	Diameter (in.)	Allowable Stress (KSI)
A-307	All	20.0
A-36	All	19.0
A-325	<1-1/2"	44.0
A-449	<1"	39.6
	1-1/8" to 1-1/2"	34.7
	1-5/8" to 3"	29.7

Table 4-34
Average properties of concrete

Water Content/ Bag	Ult 28–Day Str (psi)	Allowable Compression, F_c (psi)	Allowable B_p (psi)	Coefficient, n
7.5	2000	800	500	15
6.75	2500	1000	625	12
6	3000	1200	750	10
5	3750	1400	938	8

**See also Table 4-43*
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Table 4-35
Bending moment unit length

ℓ/b	$M_x\left(\begin{matrix}x = 0.5b\\ y = \ell\end{matrix}\right)$	$M_y\left(\begin{matrix}x = .5b\\ y = 0\end{matrix}\right)$
0	0	$-0.5f_c\,\ell^2$
0.333	$0.0078f_cb^2$	$-0.428f_c\,\ell^2$
0.5	$0.0293f_cb^2$	$-0.319f_c\,\ell^2$
0.667	$0.0558f_cb^2$	$-0.227f_c\,\ell^2$
1.0	$0.0972f_cb^2$	$-0.119f_c\,\ell^2$
1.5	$0.123f_cb^2$	$-0.124f_c\,\ell^2$
2.0	$0.131f_cb^2$	$-0.125f_c\,\ell^2$
3.0	$0.133f_cb^2$	$-0.125f_c\,\ell^2$
∞	$0.133f_cb^2$	$-0.125f_c\,\ell^2$

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Table 4-36
Constant for moment calculation, γ_1, and γ_2

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

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Table 4-37
Values of constants as a function of K

K	C_c	C_t	J	Z	K	C_c	C_t	J	Z
0.1	0.852	2.887	0.766	0.480	0.55	2.113	1.884	0.785	0.381
0.15	1.049	2.772	0.771	0.469	0.6	2.224	1.765	0.784	0.369
0.2	1.218	2.661	0.776	0.459	0.65	2.333	1.640	0.783	0.357
0.25	1.370	2.551	0.779	0.448	0.7	2.442	1.510	0.781	0.344
0.3	1.510	2.442	0.781	0.438	0.75	2.551	1.370	0.779	0.331
0.35	1.640	2.333	0.783	0.427	0.8	2.661	1.218	0.776	0.316
0.4	1.765	2.224	0.784	0.416	0.85	2.772	1.049	0.771	0.302
0.45	1.884	2.113	0.785	0.404	0.9	2.887	0.852	0.766	0.286
0.5	2.000	2.000	0.785	0.393	0.95	3.008	0.600	0.760	0.270

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1. Width of base ring
2. Quantity of anchor bolts
3. Sizes of anchor bolts
4. Strength of anchor bolts
5. Strength of concrete

If the width of the base plate is increased, the neutral axis will be displaced toward the compression side and the stresses in the concrete and steel will be reduced. The maximum compressive stress between base plate and the concrete occurs at the outer periphery of the base plate. When uplift occurs, part of the base plate lifts up, resulting in a shift of the neutral axis toward the compression side.

The value of K represents the location of the neutral axis between the anchor bolts in tension and the concrete in compression. A preliminary value of K is estimated based on a ratio of the "allowable" stresses of the anchor bolts and concrete and a ratio of the modulus of elasticity of the two materials. From this preliminary value, anchor bolt sizes and numbers are determined and actual stresses computed. Using these actual stresses, the location of the neutral axis is found and thus an actual corresponding K value. A comparison of these K values tells the designer whether the location of the neutral axis that was assumed for selection of anchor bolts was accurate. In successive trials, the anchor bolt sizes and quantity and width of base plate can be varied to obtain an optimum design. At each trial a new K is estimated and calculations repeated until the estimated K and actual K are approximately equal. This indicates both a balanced design and accurate calculations.

Rather than apportioning a load to each anchor bolt, the anchor bolt area is assumed as a continuous uniform cylinder whose thickness corresponds to the area of the bolts.

The equations can be manipulated to find the exact width of base plate required, w_r, for the parameters of each case. The equation is;

$$w_r = [W_b + (C_t\,f_S - C_C\,f_C\,n)r\,t_S]/(C_C\,f_C\,r)$$

Example is based on the illustrated case in this procedure;

Trial 1:

$W_b = 194,000$ Lbs

$C_t = 2.113$

$C_C = 1.884$

$n = 10$

$r = 52.5$ in

$t_S = 0.225$ in

$f_S = 13,660$ PSI

$f_C = 449$ PSI

$$w_r = [194,000 + (2.113\ (13,660) - 1.884(449)\ 10)$$

$$\times 52.5(225)/[1.884(449)52.5] = 9.79 \text{ in}$$

Trial 2:

$C_t = 2.355$

$C_C = 1.610$

$t_S = 0.225$ in

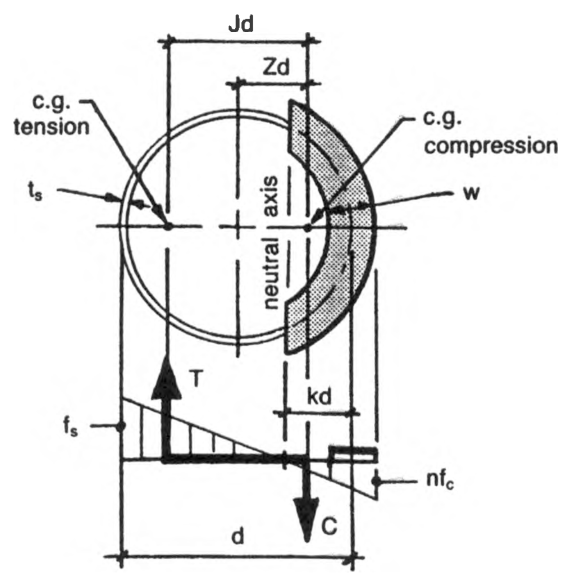
$f_S = 12,100$ PSI

$f_C = 611$ PSI

$$w_r = [194,000 + (2.355(12,100) - 1.61(611)10)$$

$$\times 52.5(.225)/[1.610(611)52.5] = 8.02 \text{ in}$$

ANCHOR BOLTS: EQUIVALENT AREA METHOD



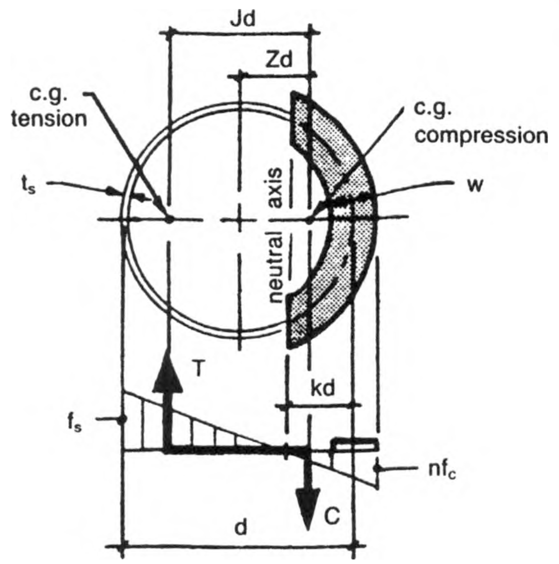
PROCEDURE

1. Calculate preliminary K value based on allowables.
2. Make preliminary selection of anchor bolts and width of base plate.
3. Calculate loads and stresses.
4. Calculate K based on actual stresses and compare with value computed in Step 2.
5. If difference exceeds .01, select a new K between both values and repeat Steps 2–8.

TRIAL 1				TRIAL 2	
1		**Data**		**1 Data**	
F_s (Table 4-33)		M_b			
F_c (Table 4-34)		d			
n (Table 4-34)		r			
W_b					
2 Approximate K Using Allowables		**Coefficients**		**2 Approximate K Using Allowables**	
$K = \dfrac{1}{1 + \dfrac{F_s}{nF_c}}$		C_c			
		C_t			
		J			
		Z			
3 Tensile Load in Steel				**3 Tensile Load in Steel**	
$T = \dfrac{M_b - W_b(Zd)}{Jd}$					
4 Number of Anchor Bolts Required				**4 Number of Anchor Bolts Required**	
$A_b = \dfrac{T\pi d}{F_s r C_t}$		R_a (Table 4-31)	in.^{2}		
A_b/N		Use ( )	bolts		
5 Stress in Equivalent Steel Band				**5 Stress in Equivalent Steel Band**	
$t_s = \dfrac{NR_a}{\pi d}$		$f_s = \dfrac{T}{t_s r C_t}$			
6 Compressive Load in Concrete				**6 Compressive Load in Concrete**	
$C = T + W_b$					
7 Stress in Concrete				**7 Stress in Concrete**	
$f_c = \dfrac{C}{[(w - t_s) + nt_s]rC_c}$					
8 Recheck K Using Actual f_s and f_c				**8 Recheck K Using Actual f_s and f_c**	
$K = \dfrac{1}{1 + \dfrac{f_s}{nf_c}}$					

See example of completed form on next page.

ANCHOR BOLTS: EQUIVALENT AREA METHOD EXAMPLE



PROCEDURE

1. Calculate preliminary K value based on allowables.
2. Make preliminary selection of anchor bolts and width of base plate.
3. Calculate loads and stresses.
4. Calculate K based on actual stresses and compare with value computed in Step 2.
5. If difference exceeds .01, select a new K between both values and repeat Steps 2–8.

TRIAL 1

1 Data

F_s (Table 4-33)	= 15 KSI	M_b = 3034 FT-KIPS	
F_c (Table 4-34)	= 1.2 KSI	d = 8.75' or 105"	
n (Table 4-34)	= 10	r = 4.38' or 52.5"	
W_b	= 194 KIPS		

2 Approximate K Using Allowables

$$K = \frac{1}{1 + \frac{F_s}{nF_c}} = .444$$

Coefficients: C_c = 1.884; C_t = 2.113; J = .785; Z = .404

3 Tensile Load in Steel

$$T = \frac{M_b - W_b(Zd)}{Jd} = \frac{3034 - 194(.404)8.75}{.785(8.75)} = 341^K$$

4 Number of Anchor Bolts Required

$$A_b = \frac{T\pi d}{F_s r C_t} = \frac{341\ \pi\ 8.75}{15(4.38)2.113} = 67.5\ in^2$$

R_a (Table 4-31) = 3.715 in.^{2}

A_b/N = 67.5/20 = 3.37 Use (20) 2½" φ bolts

5 Stress in Equivalent Steel Band

$$t_s = \frac{NR_a}{\pi d} = \frac{20(3.715)}{\pi\ 105} = .225$$

$$f_s = \frac{T}{t_s r C_t} = \frac{341}{.225(52.5)2.113} = 13.66\ KSI < 15\ KSI\ OK$$

6 Compressive Load in Concrete

$C = T + W_b$ = 341 + 194 = 535^K

7 Stress in Concrete

$$f_c = \frac{C}{[(w - t_s) + nt_s]rC_c} = \frac{535}{[(10 - .225) + 2.25]52.5(1.884)} = .449\ KSI$$

8 Recheck K Using Actual f_s and f_c

$$K = \frac{1}{1 + \frac{f_s}{nf_c}} = \frac{1}{1 + \frac{13.66}{10(.449)}} = .247 \neq .444$$

NO GOOD!

TRIAL 2

1 Data

USE W = 8.25" and K = .34

2 Approximate K Using Allowables

K = .34
C_c = 1.610 J = .7826
C_t = 2.355 Z = .429

3 Tensile Load in Steel

336.7^K

4 Number of Anchor Bolts Required

$$\frac{336.7\ \pi\ 8.75}{15(4.38)2.355} = 59.82 \qquad \frac{59.82}{20} = 2.99\ in^2$$

USE (20) 2½" φ BOLTS

5 Stress in Equivalent Steel Band

t_s = .225

$$f_s = \frac{336.7}{.225(52.5)2.355} = 12.10\ OK$$

6 Compressive Load in Concrete

C = 336.7 + 194 = 530.7^K

7 Stress in Concrete

$$f_c = \frac{530.7}{10.275(52.5)1.61} = .611\ KSI$$

8 Recheck K Using Actual f_s and f_c

$$K = \frac{1}{1 + \frac{12.10}{10(.611)}} = .336\ OK \approx .34$$

Base Plate

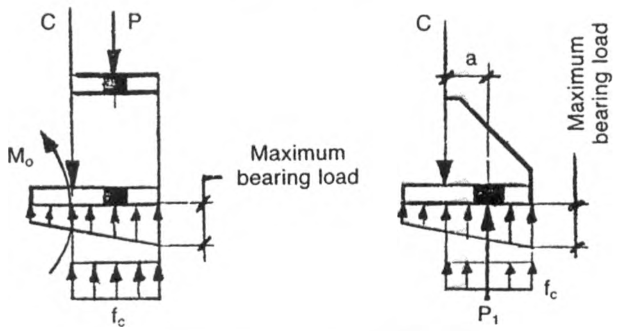


Figure 4-69. Loading diagram of base plate with gussets and chairs.

Type 1: Without Chairs or Gussets

K = from "Anchor Bolts."

l =

f_c = from "Anchor Bolts."

d =

- *Bending moment per unit length.*

$$M_o = 0.5 f_c l^2$$

- *Maximum bearing load.*

$$b_p = f_c\left(\frac{2Kd + w}{2Kd}\right) < B_p \text{ (see Table 4-34)}$$

- *Thickness required.*

$$t_b = \sqrt{\frac{6M_o}{F_b}}$$

Type 2: With Gussets Equally Spaced, Straddling Anchor Bolts

- *With same number as anchor bolts.*

$$b = \frac{\pi d}{N}$$

$$\frac{l}{b}$$

M_o = greater of M_x or M_y from Table 4-35

$$t_b = \sqrt{\frac{6M_o}{F_b}}$$

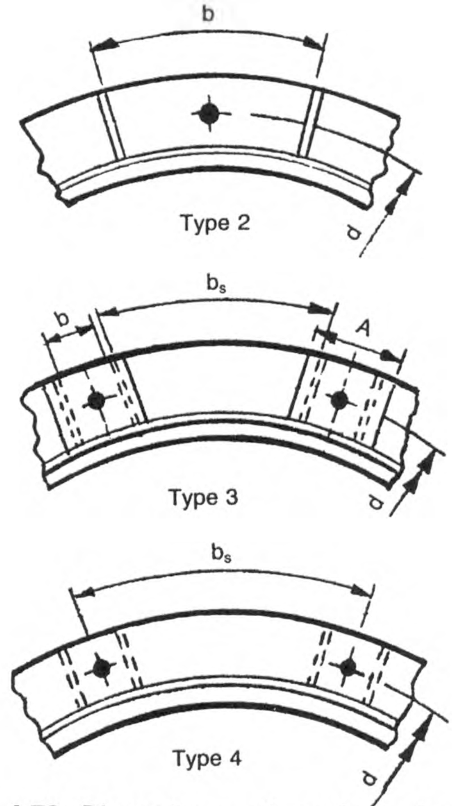


Figure 4-70. Dimensions of various base plate configurations.

- *With twice as many gussets as anchor bolts.*

$$b = \frac{\pi d}{2N}$$

$$\frac{l}{b}$$

M_o = greater of M_x or M_y from Table 4-35

$$t_b = \sqrt{\frac{6M_o}{F_b}}$$

Type 3 or 4: With Anchor Chairs or Full Ring

- *Between gussets.*

$$P = F_s R_a$$

$$M_o = \frac{Pb}{8}$$

$$t_b = \sqrt{\frac{6M_o}{(w - d_b)F_b}}$$

- *Between chairs.*

$$\frac{\ell}{b_s}$$

M_o = greater of M_x or M_y from Table 4-35

$$t_b = \sqrt{\frac{6M_o}{F_b}}$$

Top Plate or Ring (Type 3 or 4)

- *Minimum required height of anchor chair (Type 3 or 4).*

$$h_{min} = \frac{7.29\delta d}{H} < 18 \text{ in.}$$

- *Minimum required thickness of top plate of anchor chair.*

$$t_c = \sqrt{\frac{P}{F_b e}(0.375b - 0.22d_b)}$$

Top plate is assumed as a beam, e x A with partially fixed ends and a portion of the total anchor bolt force P/3, distributed along part of the span. (See Figure 4-71.)

- *Bending moment, M_o, in top ring (Type 4).*

$$\frac{b}{\ell}$$

γ_1 = (see Table 4-36)
γ_2 = (see Table 4-36)

1. If $a = \ell/2$ and $b/\ell > 1$, M_y governs

$$M_o = \frac{P}{4\pi}\left[(1+\nu)\log\left(\frac{2\ell}{\pi g}\right) + (1-\gamma_1)\right]$$

2. If $a \neq \ell/2$ but $b/\ell > 1$, M_y governs

$$M_o = \frac{P}{4\pi}\left[(1+\nu)\log\left(\frac{2\ell \sin\frac{\pi a}{\ell}}{\pi g}\right) + 1\right] - \frac{\gamma_1 P}{4\pi}$$

3. If $b/\ell < 1$, invert b/ℓ and rotate axis X-X and Y-Y 90°

$$M_o = \frac{P}{4\pi}\left[(1+\nu)\log\left(\frac{2\ell \sin\frac{\pi a}{\ell}}{\pi g}\right) + 1\right] - \left[(1-\nu-\gamma_2)\frac{P}{4\pi}\right]$$

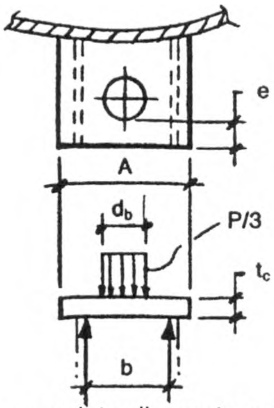


Figure 4-71. Top plate dimensions and loadings.

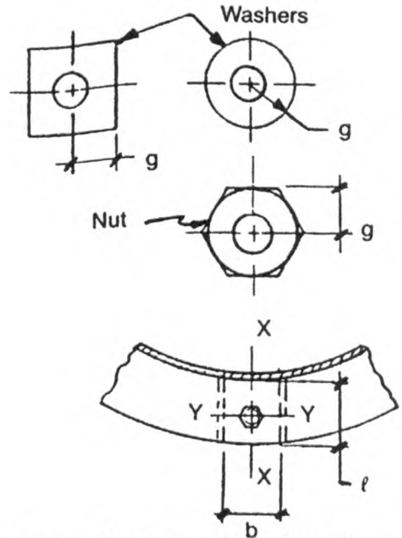


Figure 4-72. Compression plate dimensions.

- *Minimum required thickness of top ring (Type 4).*

$$t_c = \sqrt{\frac{6M_o}{F_b}}$$

Gussets

- *Type 2.* Assume each gusset shares load with each adjoining gusset. The uniform load on the base is f_c, and the area supported by each gusset is $\ell \times b$. Therefore the load on the gusset is

$$P_1 = f_c \ell b$$

Thickness required is

$$t_g = \frac{P_1(6a - 2\ell)}{F_b \ell^2}$$

- Type 3 or 4.

$$t_g = \frac{P}{18,000\,\ell} > \frac{3}{8}\text{ in.}$$

Skirt

- *Thickness required in skirt at compression plate or ring due to maximum bolt load reaction.*
  For Type 3:

$$Z = \frac{1.0}{\dfrac{1.77At_b}{\sqrt{Rt_{sk}}}\left[\dfrac{t_b}{t_{sk}}\right]^2 + 1}$$

$$S = \frac{Pa}{t_{sk}^2}\left[\frac{1.32Z}{\dfrac{1.43Ah^2}{Rt_{sk}} + \left[4Ah^2\right]^{0.333}} + \frac{0.031}{\sqrt{Rt_{sk}}}\right] < 25\text{ ksi}$$

For Type 4:
Consider the top compression ring as a uniform ring with N number of equally spaced loads of magnitude.

$$\frac{Pa}{h}$$

See Procedure 7-1 for details.

The moment of inertia of the ring may include a portion of the skirt equal to 16 t_{sk} on either side of the ring (see Figure 4-74).

- *Thickness required at opening of skirt.*
  Note: If skirt is stiffened locally at the opening to compensate for lost moment of inertia of skirt cross section, this portion may be disregarded.
  G = width of opening, in.

$$f_b = \frac{1}{\pi D - 3G}\left[\frac{48\ M_b}{D} + W_b\right]$$

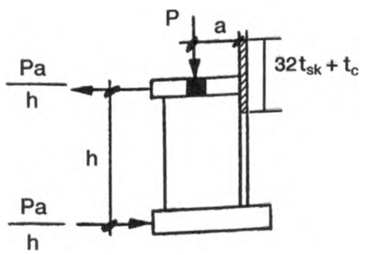


Figure 4-73. Dimensions and loadings on skirt due to load P.

Actual weights and moments at the elevation of the opening may be substituted in the foregoing equation if desired.

Skirt thickness required:

$$t_{sk} = \frac{f_b}{8F_y}\quad\text{or}\quad\sqrt{\frac{f_b}{4,640,000}}$$

whichever is greater

- *Determine allowable longitudinal stresses.*
  Tension

  S_t = lesser of $0.6F_y$ or $1.2\,S$

  Compression

  $$S_c = 0.333\ F_y$$
  $$= 1.2 \times \text{factor "B"}$$
  $$= \frac{t_{sk}E_1}{16\ R}$$
  $$= 1.2\,S$$

  whichever is less.
  Longitudinal forces

  $$F_{LT} = \frac{48\ M_b}{\pi D^2} - \frac{W_b}{\pi D}$$
  $$F_{LC} = (-)\frac{48\ M_b}{\pi D^2} - \frac{W_b}{\pi D}$$

  Skirt thickness required

  $$t_{sk} = \frac{F_{LT}}{S_t}\quad\text{or}\quad\frac{F_{LC}}{S_c}$$

  whichever is greater.
- *Thickness required at skirt-head attachment due to M_t.*
  Longitudinal forces

  $$F_{LT} = \frac{48\ M_t}{\pi D^2} - \frac{W_t}{\pi D}$$
  $$F_{LC} = (-)\frac{48\ M_t}{\pi D^2} - \frac{W_t}{\pi D}$$

  Skirt thickness required

  $$t_{sk} = \frac{F_{LT}}{0.707\ S_tE}\quad\text{or}\quad\frac{F_{LC}}{0.707S_cE}$$

  whichever is greater.

Notes

1. Base plate thickness:
   - If t ≤ ½ in., use Type 1.
   - If ½ in. < t ≤ ¾ in., use Type 2.
   - If t > ¾ in., use Type 3 or 4.
2. To reduce sizes of anchor bolts:
   - Increase number of anchor bolts.
   - Use higher-strength bolts.
   - Increase width of base plate.
3. Number of anchor bolts should always be a multiple of 4. If more anchor bolts are required than spacing allows, the skirt may be angled to provide a larger bolt circle or bolts may be used inside and outside of the skirt. Arc spacing should be kept to a minimum if possible.
4. The base plate is not made thinner by the addition of a compression ring, t_b would be the same as required for chair-type design. Use a compression ring to reduce induced stresses in the skirt or for ease of fabrication when chairs become too close.
5. Dimension "a" should be kept to a minimum to reduce induced stresses in the skirt. This will provide a more economical design for base plate, chairs, and anchor bolts.
6. For heavy-wall vessels, it is advantageous to have the center lines of the skirt and shell coincide if possible. For average applications, the O.D. of the vessel and O.D. of the skirt should be the same.
7. Skirt thickness should be a minimum of R/200.

Procedure 4-15: Design of Base Details for Vertical Vessels – Centered Neutral Axis Method

Notation

E = joint efficiency
E_1 = modulus of elasticity at design temperature, psi
A_b = cross-sectional area of bolts, in.^{2}
d = diameter of bolt circle, in.
W_b = weight of vessel at base, lb
W_T = weight of vessel at tangent line, lb
w = width of base plate, in.
S = code allowable stress, tension, psi
N = number of anchor bolts
F'_c = allowable bearing pressure, concrete, psi
F_y = minimum specified yield stress, skirt, psi
F_s = allowable stress, anchor bolts, psi
f_{LT} = axial load, tension, lb/in.-circumference
f_{LC} = axial load, compression, lb/in.-circumference
F_T = allowable stress, tension, skirt, psi
F_c = allowable stress, compression, skirt, psi
F_b = allowable stress, bending, psi
f_s = tension force per bolt, lb
f_c = bearing pressure on foundation, psi
M_b = overturning moment at base, ft-lb
M_T = overturning moment at tangent line, ft-lb

Allowable Stresses

$$F_T = \text{lesser of} \begin{cases} \bullet 0.6F_y = \\ \bullet 1.2\,S = \end{cases}$$

$$F_c = \text{lesser of} \begin{cases} \bullet 0.333F_y = \\ \bullet 1.2 \text{ Factor } B = \\ \bullet \dfrac{t_{sk}E_1}{16\,R} = \\ \bullet 1.2\,S = \end{cases}$$

$$F_b = 0.6\,F_y$$

$$F'_c = 500 \text{ psi for } 2000 \text{ lb concrete}$$
$$750 \text{ psi for } 3000 \text{ lb concrete}$$

$$\text{Factor A} = \frac{0.125t_{sk}}{R} =$$

Factor B = from applicable material chart of ASME Code, Section II, Part D, Subpart 3

Anchor Bolts

- *Force per bolt due to uplift.*

$$f_s = \frac{48M_b}{dN} - \frac{W_b}{N}$$

- *Required bolt area, A_b.*

$$A_b = \frac{f_s}{F_s} =$$

Use ( ) _____ diameter bolts
Note: Use four ¾-in.-diameter bolts as a minimum.

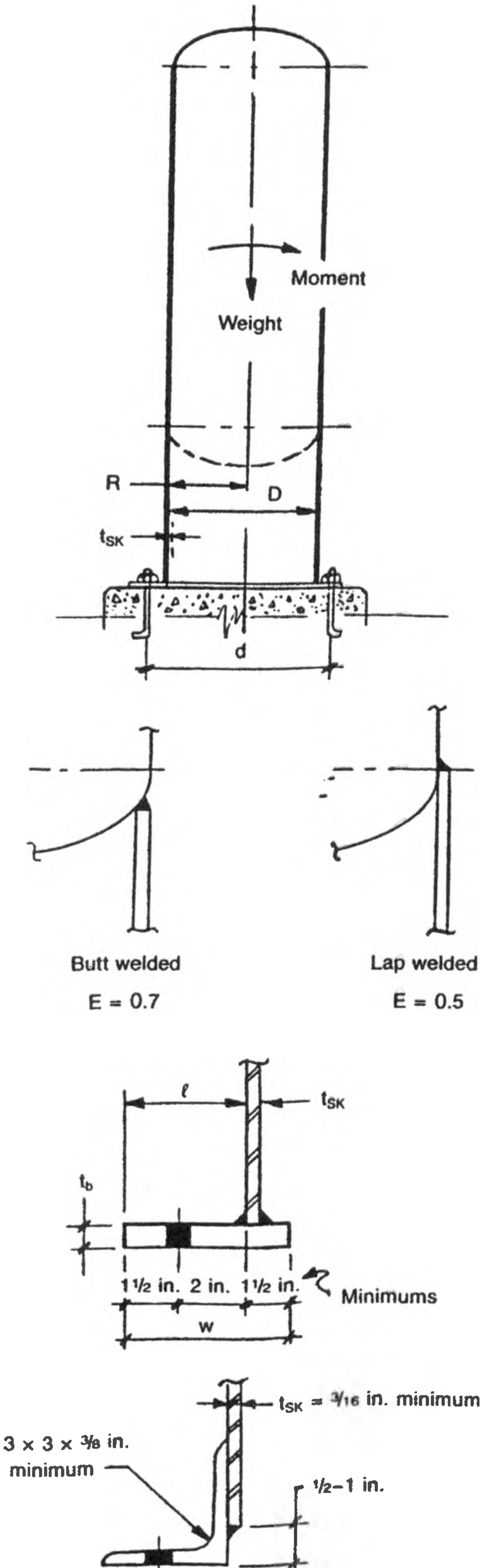


Figure 4-74. Typical dimensional data and forces for a vertical vessel supported on a skirt.

Base Plate

- *Bearing pressure, f_c (average at bolt circle).*

$$f_c = \frac{48M_b}{\pi d^2 w} + \frac{W_b}{\pi dw} =$$

- *Required thickness of base plate, t_b.*

$$t_b = 1\sqrt{\frac{3f_c}{20,000}}$$

Skirt

- *Longitudinal forces, f_{LT} and f_{LC}.*

$$f_{LT} = \frac{48M_b}{\pi D^2} - \frac{W_b}{\pi D}$$

$$f_{LC} = (-)\frac{48M_b}{\pi D^2} - \frac{W_b}{\pi D}$$

- Thickness required of skirt at base plate, t_{sk}.

$$t_{sk} = \text{greater of } \frac{f_{LT}}{F_T} =$$

$$\text{or } \frac{f_{LC}}{F_C} =$$

- Thickness required of skirt at skirt-head attachment.

Longitudinal forces:

$$f_{LT}, f_{LC} = \pm\frac{48M_T}{\pi D^2} - \frac{W_T}{\pi D} =$$

$$f_{LT} =$$

$$f_{LC} =$$

Thickness required:

$$t_{sk} = \text{greater of } \frac{f_{LT}}{0.707\ F_T E} =$$

$$\text{or } \frac{f_{LC}}{0.707\ F_C E} =$$

Notes

1. This procedure is based on the centered neutral axis method and should be used for relatively small or simple vertical vessels supported on skirts.
2. If moment M_b is from seismic, assume W_b as the operating weight at the base. If M_b is due to wind, assume empty weight for computing the maximum value of f_{LT} and operating weight for f_{LC}.

Procedure 4-16: Design of Anchor Bolts for Vertical Vessels

Notation

A_b = Cross sectional area of anchor bolt, in^2
A_r = Area of one anchor bolt required, In^2
D_b = Diameter of bolt circle, Ft
M = Overturning moment due to wind or seismic, Ft-lbs
N = Number of anchor bolts
S_b = Allowable tensile stress, PSI
W = Weight of vessel under consideration. Typically use empty for wind and full for seismic for worst case, Lbs

Formulas

$$N\,A_b = [(48\,M/D_b) - W]\,[1/S_b]$$

- If N A_b is negative, no anchor bolts are required
- If N A_b is positive, than anchor bolts are required
- Size of anchor bolts required is as follows, A_r;

$$A_r = [(48\,M/D_b) - W]\,[1/(N\,S_b)]$$

Notes

1. Values for S_b in table are based on .333 F_U
2. Assumes centered neutral axis method

Table 4-38
Area of anchor bolts, A_b

DIA	A_b	DIA	A_b
¾"–10	.302	1-3/4"–5	1.744
7/8"–9	.419	2"–4-1/2	2.3
1"–8	.551	2-1/2"–4	3.715
1-1/4"–7	.890	2-3/4"–4	4.618
1-1/2"–6	1.294	3"-4	5.621

Table 4-39
Allowable stress, KSI

MATL	DIA	F_y	F_U	S_b
A-36	<4"	36	58	19.14
A-307	<8"	36	60	20
A-193-B7	<2.5"	105	125	41.25
	2.5 –4"	95	115	38
A-449	<1"	92	120	39.6
	1–1.5"	81	105	34.65
	<3"	58	90	29.7

Table 4-40
Recommended quantity and spacing of anchor bolts

Diameter, D		Quantity, N		Spacing, bs (Ft)	
Ft	In	MIN (1)	MAX (2)	MIN (3)	MAX (4)
2	24	4	4	1.75	6
3	36		4	2.35	
4	48		8	1.57	
5	60		12	1.31	
6	72		12	1.57	
7	84		16	1.37	
8	96		16	1.57	
9	108	8	20	1.41	6
10	120		20	1.57	
11	132		24	1.44	
12	144		24	1.57	
13	156		28	1.46	
14	168		28	1.57	
15	180		32	1.47	
16	192	12	32	1.57	6
17	204		36	1.48	
18	216		36	1.57	
19	228		40	1.49	
20	240		40	1.57	

(Continued)

Table 4-40
Recommended quantity and spacing of anchor bolts—cont'd

Diameter, D		Quantity, N		Spacing, b_S (Ft)	
Ft	In	MIN (1)	MAX (2)	MIN (3)	MAX (4)
21	252		44	1.5	
22	264		44	1.57	
23	276	16	48	1.51	6
24	288		48	1.57	
25	300		52	1.51	
26	312		52	1.57	
27	324		56	1.51	
28	336		56	1.57	
29	348		60	1.51	
30	360	20	60	1.57	6
31	372		64	1.52	
32	384		64	1.57	

Notes:

1. Minimum quantity is based on minimum arc spacing of 4' and maximum arc spacing of 6'.
2. Maximum quantity is based on 2D.
3. Minimum spacing of anchor bolts is based on the maximum quantity of anchor bolts, $\pi D_{b/N_{max}}$.
4. Maximum spacing is based on 6' max arc spacing as practical limit.
5. Minimum anchor bolt size is 3/4".

Table 4-41
Anchor bolt torque values

Bolt Dia (in)	Tensile Area, R_a	Design Bolt Tension (KIPS) (1) (2)	Torque Bolt Tension (KIPS) (4)	Torque (Ft-Lbs)
		CASE 2: A-449		
0.75 – 10 UNC	0.302	8.5	9.1	85
0.875 – 9 UNC	0.419	11.9	12.8	140
1-8 UNC	.551	15.1	16.8	210
1.25 – 7 UNC	0.89	23.9	27.5	430
1.5 – 6 UNC	1.294	33.8	40.5	760
1.75 – 5 UNC	1.744	42.5	54.9	1200
2 – 4.5 UNC	2.3	53.5	72	1800
2.25 – 4.5 UNC	3.02	69.2	93.5	2630
2.5 -4 UNC	3.715	85.2	115.2	3600
2.75 -4 UNC	4.618	99.3	142.3	4890
3-4 UNC	5.621	113.9	171.7	6440
		CASE 2: A-449		
0.75 – 10 UNC	0.302	22	23.5	220
0.875 – 9 UNC	0.419	29.6	32	350
1-8 UNC	.551	38.8	43.2	540
1.25 – 7 UNC	0.89	53.9	62.1	970
1.5 – 6 UNC	1.294	76	91.2	1710
1.75 – 5 UNC	1.744	68.5	88.2	1930

Table 4-41
Anchor bolt torque values—cont'd

Bolt Dia (in)	Tensile Area, R_a	Design Bolt Tension (KIPS) (1) (2)	Torque Bolt Tension (KIPS) (4)	Torque (Ft-Lbs)
		CASE 2: A-449		
2 – 4.5 UNC	2.3	86.3	116	2900
2.25 – 4.5 UNC	3.02	111.1	150.8	4230
2.5 -4 UNC	3.715	137.2	185.6	5800
2.75 -4 UNC	4.618	159.9	228.9	7870
3-4 UNC	5.621	183.7	276.8	10380

Notes:

1. Values in Table for A-36 and A-307 bolts are based on approximately 25 KSI tensile stress on the tensile area.
2. Values in Table for A-449 bolts are based on .7 F_y tensile stress on the tensile area.
3. The threads and underside of nuts should be waxed prior to installation to reduce friction.
4. Torque bolt tension allows a % increase over bolt tension to allow for loss of pretension due to creep of concrete and bolt material.
5. All torque values result in a tension stress less than .8 F_y.

Procedure 4-17: Properties of Concrete

Notation

f'_C = Ultimate 28 day Compressive Stress, PSI
F_C = Allowable Compressive Stress, PSI
B_P = Allowable Bearing pressure, PSI
E_S = Modulus of elasticity, steel, PSI
E_C = Modulus of elasticity, concrete, PSI
n = Ratio, E_S / E_C

Table 4-42
Soil bearing pressure

Type of Soil	Bearing Pressure, PSF
Rock	4000
Rocky	3000
Gravel	2000
Sandy	1500
Clay	1000

Table 4-43
Allowable stress, concrete

Ultimate 28 Day Compressive Stress, f'_C (PSI)	Allowable Compressive Stress, F_C (PSI) (1)	Allowable Bearing Pressure, B_P (PSI)(2)	Ratio, n
2000	800	500	15
2500	1000	625	12
3000	1200	750	10
3750	1500	938	8
4000	1600	1000	6

Notes:

1. F_C = 40% of f'_C
2. B_P = 25% of f'_C
3. See ACI 318 or AISC Steel construction Manual for F_c based on either ASD or LRFD methods.

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