9
Related Equipment

Procedure 9-1: Design of Davits [1,2]

Notation

C_v = vertical impact factor, 1.5–1.8
C_h = horizontal impact factor, 0.2–0.5
f_a = axial stress, psi
f_b = bending stress, psi
f_h = horizontal force, lb
f_v = vertical force, lb
F_a = allowable axial stress, psi
F_b = allowable bending stress, psi
F_r = radial load, lb
F_{re} = equivalent radial load, lb
F_y = minimum specified yield stress, psi
M_1 = bending moment in mast at top guide or support, in.-lb
M_2 = maximum bending moment in curved davit, in.-lb
M_3 = bending moment in boom, in.-lb
M_x = longitudinal moment, in.-lb
M_ϕ = circumferential moment, in.-lb
W_1 = weight of boom and brace, lb
W_D = total weight of davit, lb
W_L = maximum rated capacity, lb
α, β, K = stress coefficients
P = axial load, lb
I = moment of inertia, $in.^4$
A = cross-sectional area, $in.^2$
Z = section modulus, $in.^3$
r = least radius of gyration, in.
t_p = wall thickness of pipe davit, in.
a = outside radius of pipe, in.

Moments and Forces in Davit and Vessel

- *Loads on davit.*

$f_v = C_V W_L$

$f_h = C_h W_L$

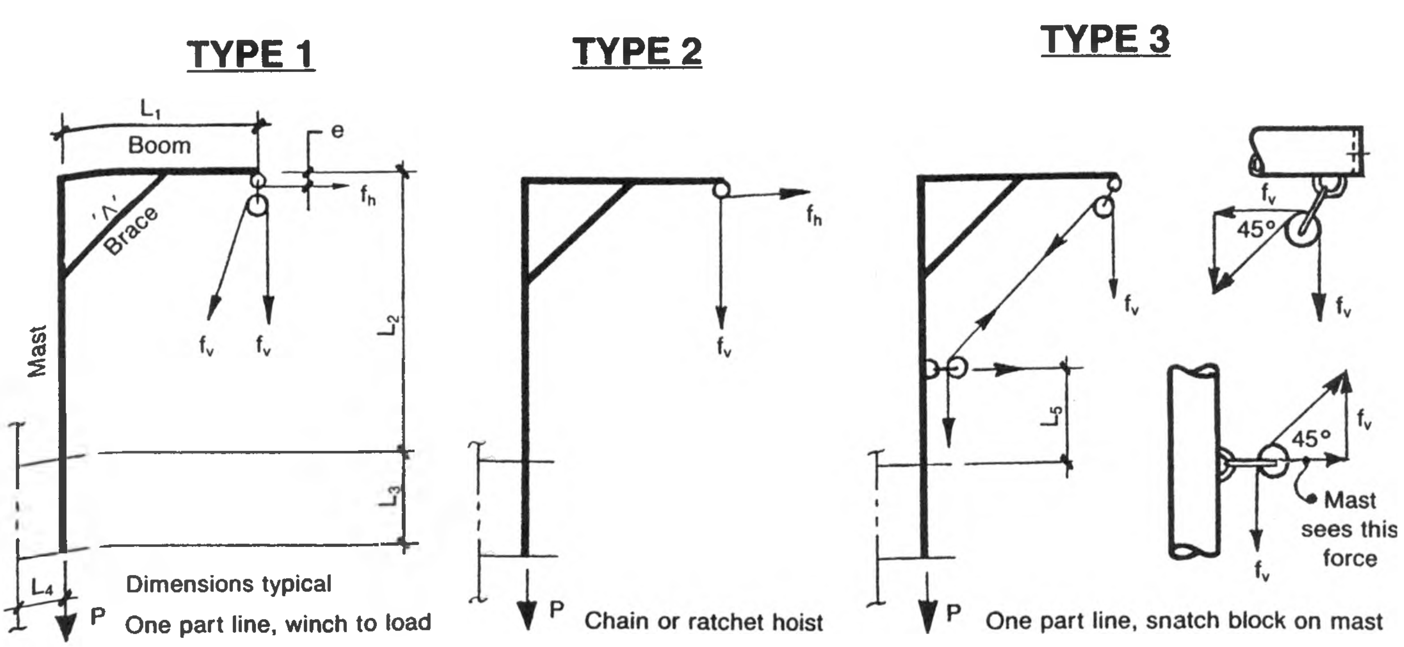


Figure 9-1. Types of rigging.

- *Bending moment in davit mast, M_1.*

Type 1: $M_1 = 2f_vL_1 + 0.5W_1L_1 + f_hL_2$

Type 2: $M_1 = f_vL_1 + 0.5W_1L_1 + f_hL_2$

Type 3: $M_1 = f_v(2L_1 + L_5 - L_2) + 0.5W_1L_1 + f_hL_2$

- *Radial force at guide and support, F_r.*

$$F_r = \frac{M_1}{L_3}$$

F_r is maximum when davit rotation ϕ is at 0°, for other rotations:

$F_r = F_r \cos\phi$

- *Circumferential moment at guide and support, M_ϕ.*

$M_\phi = F_rL_4$

M_ϕ is maximum when davit rotation ϕ is at 90°, for other rotations:

$M_\phi = F_rL_4 \sin\phi$

- *Axial load on davit mast, P.*

Type 1 or 3: $P = 2f_v + W_D$

Type 2: $P = f_v + W_D$

- *Longitudinal moment at support, M_x.*

$M_x = PL_4$

Stresses in Davit

Mast Properties

I =

A =

Z =

r =

t_p =

a =

Slenderness ratio:

$\frac{2.1L_2}{r}$ =

F_a =

$F_b = 0.6F_y$

Type A Davit

- *Axial stress—mast.*

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

- *Bending stress—mast.*

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

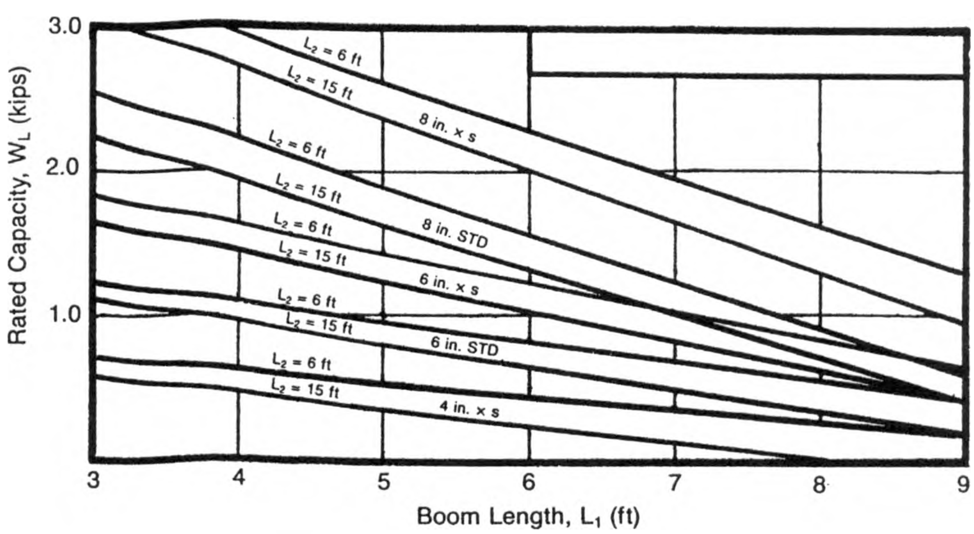


Figure 9-2. Davit selection guide.

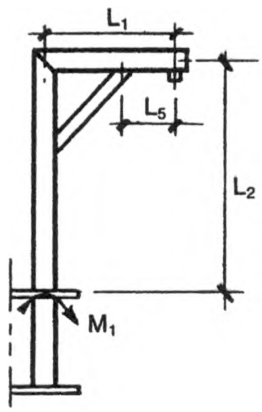
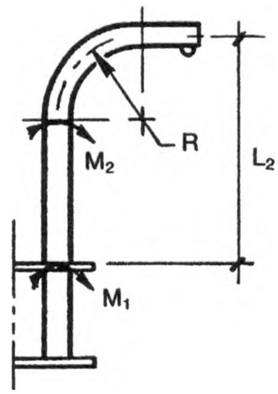


Figure 9-3. Type A davit.



Coefficients

$$\alpha = \frac{t_p R}{a^2}$$

$$\beta = \frac{6}{5 + 6\alpha^2}$$

$$K = 1 - \frac{9}{10 + 12\alpha^2}$$

Figure 9-4. Type B davit.

- *Combined stress—mast.*

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

- *Bending stress—boom.*

$$\text{Type 1: } f_b = \frac{2 f_v L_5}{Z}$$

$$\text{Type 2 or 3: } f_b = \frac{f_v L_5}{Z}$$

Type B Davit

- *Axial stress.*

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

- *Bending moment, M_2.*

$$M_2 = \frac{M_1 (L_2 - R)}{L_2}$$

- *Bending stress.*

$$\text{At } M_1, f_b = \frac{M_1}{Z}$$

$$\text{At } M_2, f_b = \frac{M_2 a}{I} \left(\frac{2}{3K\sqrt{3\beta}} \right)$$

- *Combined stress.*

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

Finding Equivalent Radial Load, F_{re}

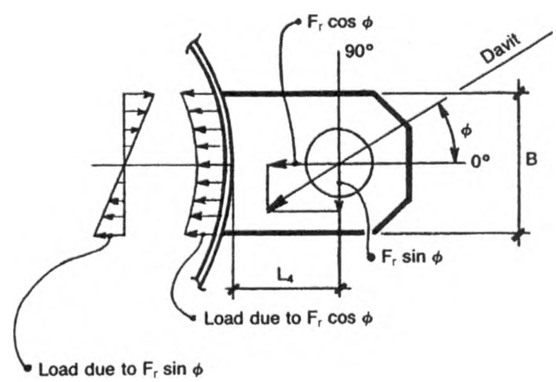


Figure 9-5. Forces in davit guide.

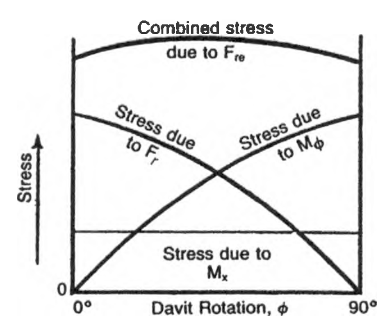


Figure 9-6. Graph of combined stress for various davit rotations.

- *Equivalent unit load, w, lb/in.*

$$w = \frac{F_r \cos\phi}{B} + \frac{6F_r \sin\phi \; L_4}{B^2}$$

- *Equivalent radial load, F_{re}, lb.*

$$F_{re} = \frac{wB}{2}$$

- *Calculate F_{re} for various angles of davit rotation.*

ϕ	W	F_{re}

Shell Stresses (See Note 1)

At Support: Utilizing the area of loading as illustrated in Figure 9-8, find shell stresses due to loads M_x, M_ϕ, and F_r by an appropriate local load procedure.

At Guide: Utilizing the area of loading as illustrated in Figure 9-9, find shell stresses due to loads M_ϕ and F_r by an appropriate local load procedure.

Note: F_{re} may be substituted for M_ϕ and F_r as an equivalent radial load for any rotation of davit other than 0° or 90°.

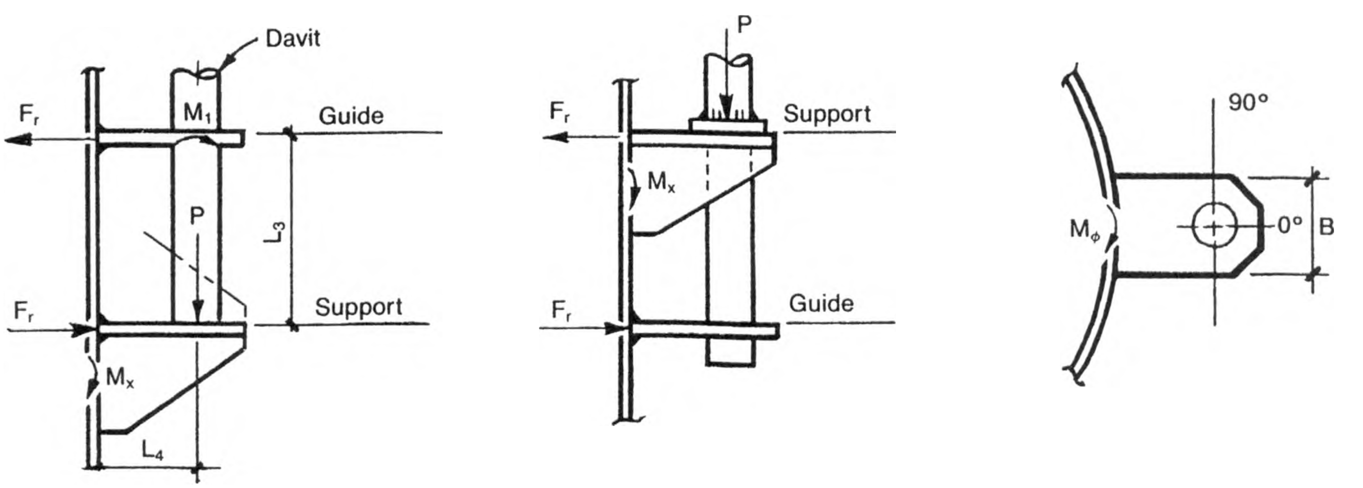


Figure 9-7. Dimensions of forces at davit support and guide.

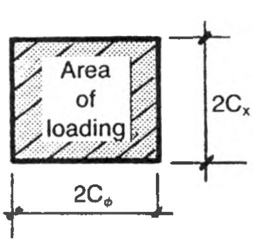


Figure 9-8. Area of loading at davit support.

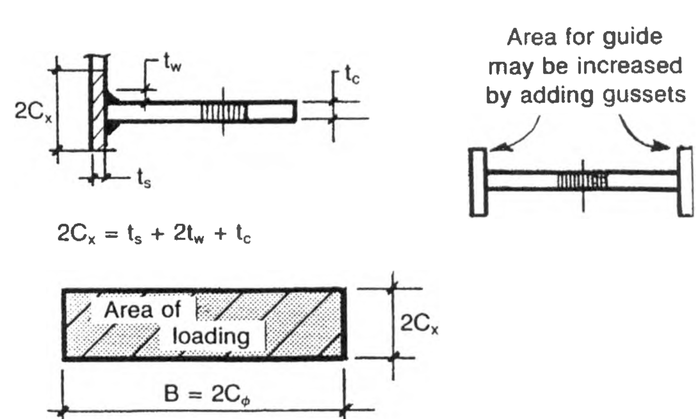
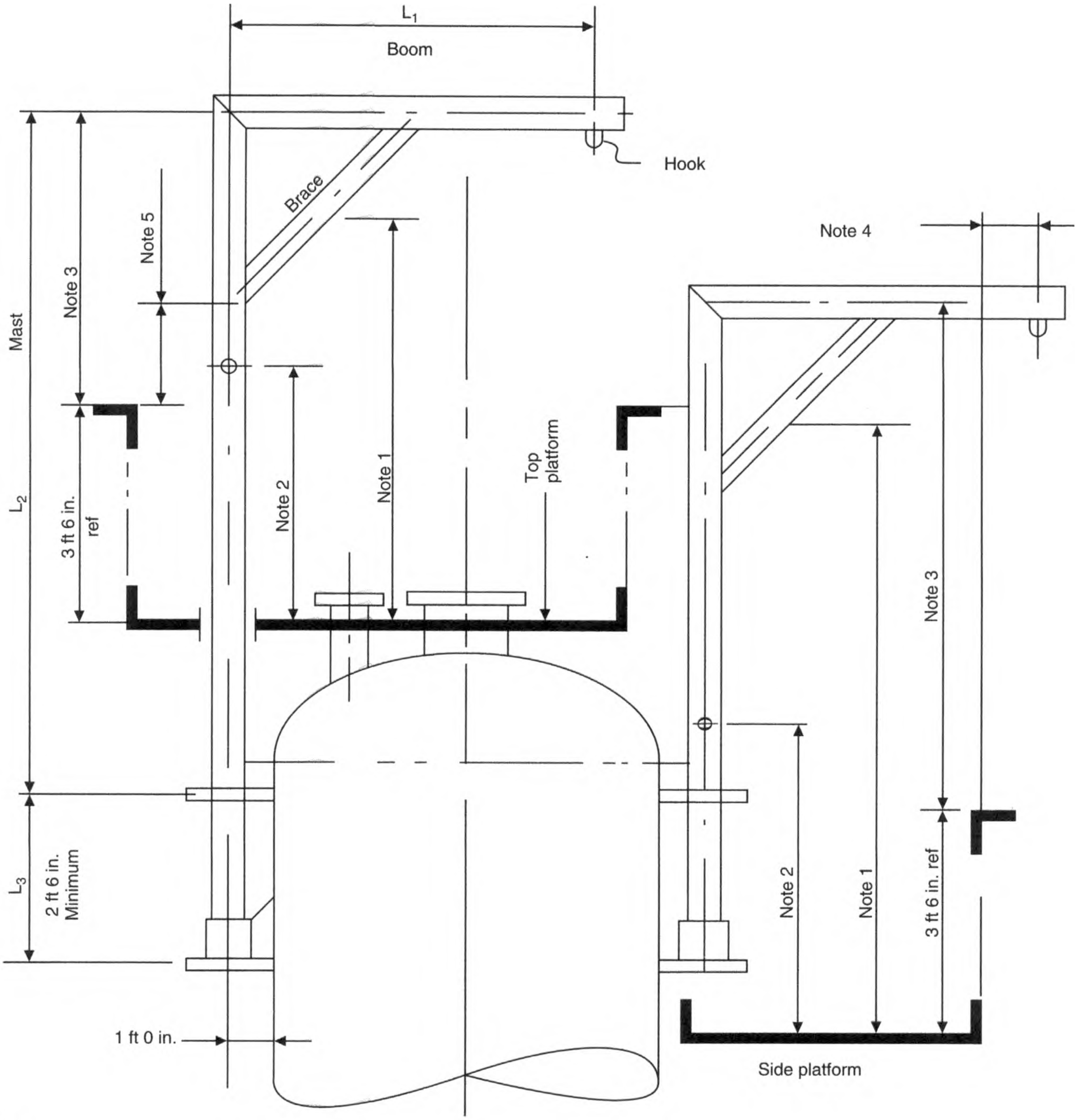


Figure 9-9. Area of loading at davit guide.

Davit Arrangement



Notes:

1. Check head clearance to middle of brace, 7 ft 0 in. minimum.
2. Set location of turning handle, 4 ft 0 in. minimum.
3. Check that equipment handled plus any rigging gear will clear handrail, 3 ft 0 in. minimum. As an alternative, the handrail may be made removable.
4. Check hook clearance to outside of platform, 9 in. minimum.
5. Check clearance between bottom of brace and handrail, 6 in. minimum.

Notes

1. Figure 9-6 illustrates the change in the total combined stress as the davit is rotated between 0° and 90°. As can be seen from the graph the stress due to M_x is constant for any degree of davit rotation. This stress occurs only at the support. The stress due to F_r varies from a maximum at 0° to 0 at 90°. The stress due to M_ϕ is 0 at 0° and increases to a maximum at 90°. To find the worst combination of stress, the equivalent radial load, F_{re} must be calculated for various degrees of davit rotation, ϕ. At the guide shell stresses should be checked by an appropriate local load procedure for the maximum equivalent radial load. At the support shell stresses should be checked for both F_{re} and M_x. Stresses from applicable external loads shall be combined. Remember the force F_{re} is a combination of loads F_r and M_ϕ at a given davit orientation. F_r and M_ϕ are maximum values that do not occur simultaneously.
2. Impact factors account for bouncing, jerking, and swinging of loads.

Procedure 9-2: Design of Circular Platforms

Notation

$$\text{Area} = \frac{(R^2 - r^2)\pi\phi}{360}$$

$$\text{Arc length, } \ell = \frac{\pi R\theta}{180}$$

$$\text{Angle, } \theta = \frac{180\,\ell}{\pi R}$$

$$X = \sqrt{R^2 - A^2} - \sqrt{r^2 - A^2}$$

$$Y = L - \sqrt{r^2 - A^2}$$

f = dead load + live load, psf
f_b = bending stress, beam, psi
f_a = axial stress, psi
$f_{x,y,r}$ = bolt loads, lb
F = total load on bracket, lb
A = load area, ft^2
A' = cross-sectional area of kneebrace, $in.^2$
M_1 = moment at shell, ft-lb
M_2 = moment at bolts, ft-lb or in.-lb
C = distance to C.G. of area, ft
K = end connection coefficient, use 1.0
r' = radius of gyration, in.
P = axial load on kneebrace, lb
Z = section modulus of beam, $in.^3$

Table 9-1
Values of ladder spacing and for given diameters

Diameter (ft)	α
2	23°
4	17°
6	14°
8	11.5°
10	10°
12	9°
14	8°
16	7°
18	6°
20	5.5°

Note: Values in table are approximate only for estimating use.

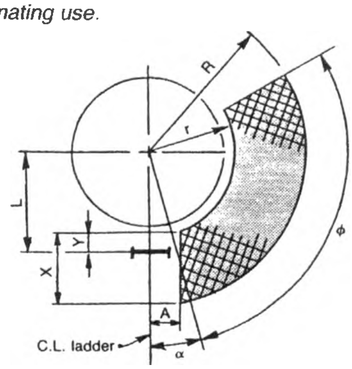


Figure 9-10. Dimensions of a typical circular platform.

AREA OF PLATFORMS				
Platform	ϕ	R	r	Area

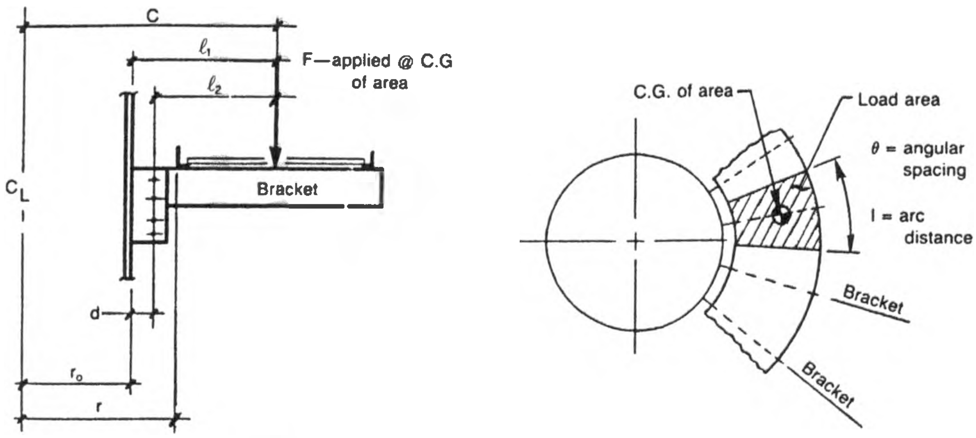


Figure 9-11. Dimensions, force, and local area for circular platforms.

COMPUTING MOMENTS IN SHELL AND BOLT LOADS														
Moments in shell:										Type Clip	Bolt Loads			
Platform	θ	R	r	A	F	C	l_1	l_2	M_1		M_2	f_x	f_y	f_r

Formulas for Chart

$$A = \frac{(R^2 - r^2)\pi\theta}{360}$$

$$F = fA$$

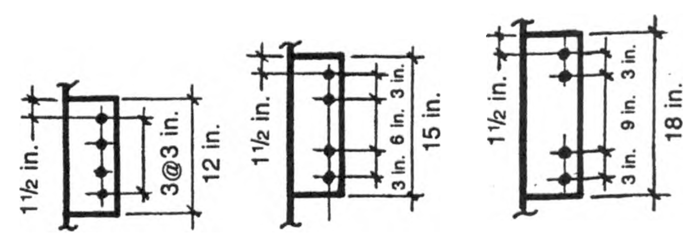
$$C = \frac{38.197(R^3 - r^3)\sin\theta/2}{(R^2 - r^2)\theta/2}$$

$$l_1 = C - r_o$$

$$l_2 = l_1 - d$$

$$M_1 = l_1 F$$

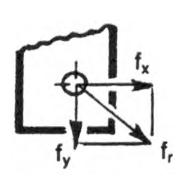
$$M_2 = l_2 F$$



Type 1
$f_x = 0.1\ M_2$

Type 2
$f_x = 0.066\ M_2$

Type 3
$f_x = 0.049\ M_2$



$$f_y = \frac{F}{4}$$

Worst case is corner bolt

$$f_r = \sqrt{f_x^2 + f_y^2}$$

Figure 9-12. Bolt load formulas for various platform support clips. (See Figure 9-16 for additional data.)

Table 9-2
Allowable Shear loads in bolts (kips)

Material	Size (in.)				
	⅝	¾	⅞	1	1–⅛
A-307	3.68	5.30	7.21	9.42	11.9
A-325	7.36	10.6	14.4	18.8	23.8

Design of Kneebrace

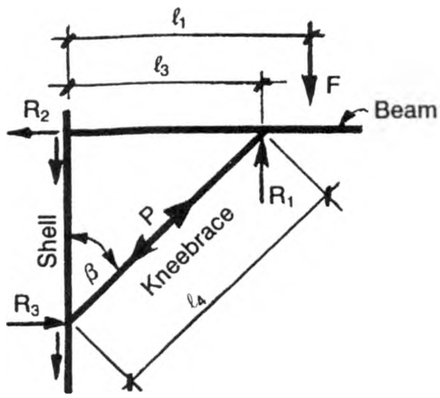


Figure 9-13. Dimensions, forces, and reactions of kneebrace support.

- *Reaction, R_1.*

$$\sum M_{R_2} = l_1 F - l_3 R_1 = 0 \qquad \therefore \; R_1 = \frac{l_1 F}{l_3}$$

- *Shear load on bolts/radial load on shell.*

$$R_2 = R_3 = R_1 \tan \beta$$

- *Bending stress in beam.*

$$f_b = \frac{|l_1 - l_3| F}{Z}$$

- *Axial load in kneebrace.*

$$P = \frac{R_1}{\cos \beta}$$

- *Axial stress.*

$$f_a = \frac{P}{A'}$$

- *Slenderness ratio/allowable stress.*

$$\frac{K l_4}{r'} = F_a$$

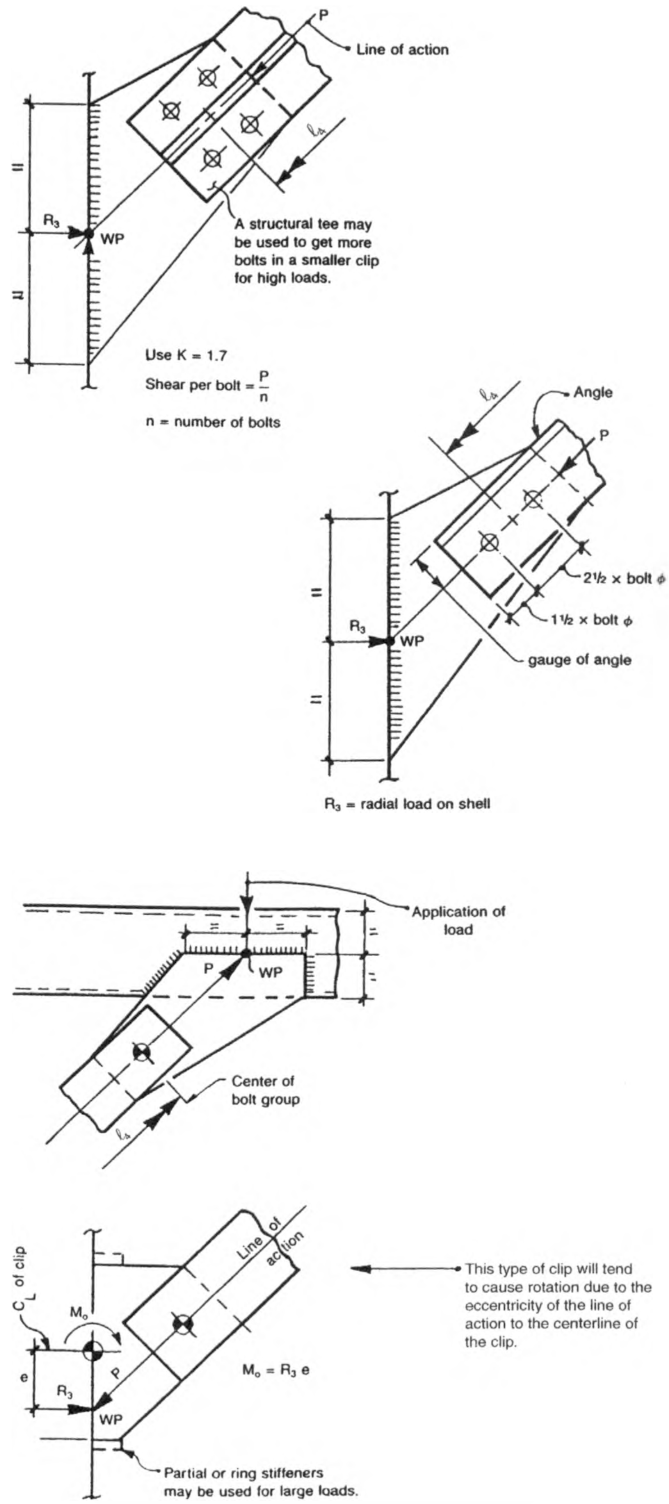


Figure 9-14. Typical bolted connections for kneebrace supports.

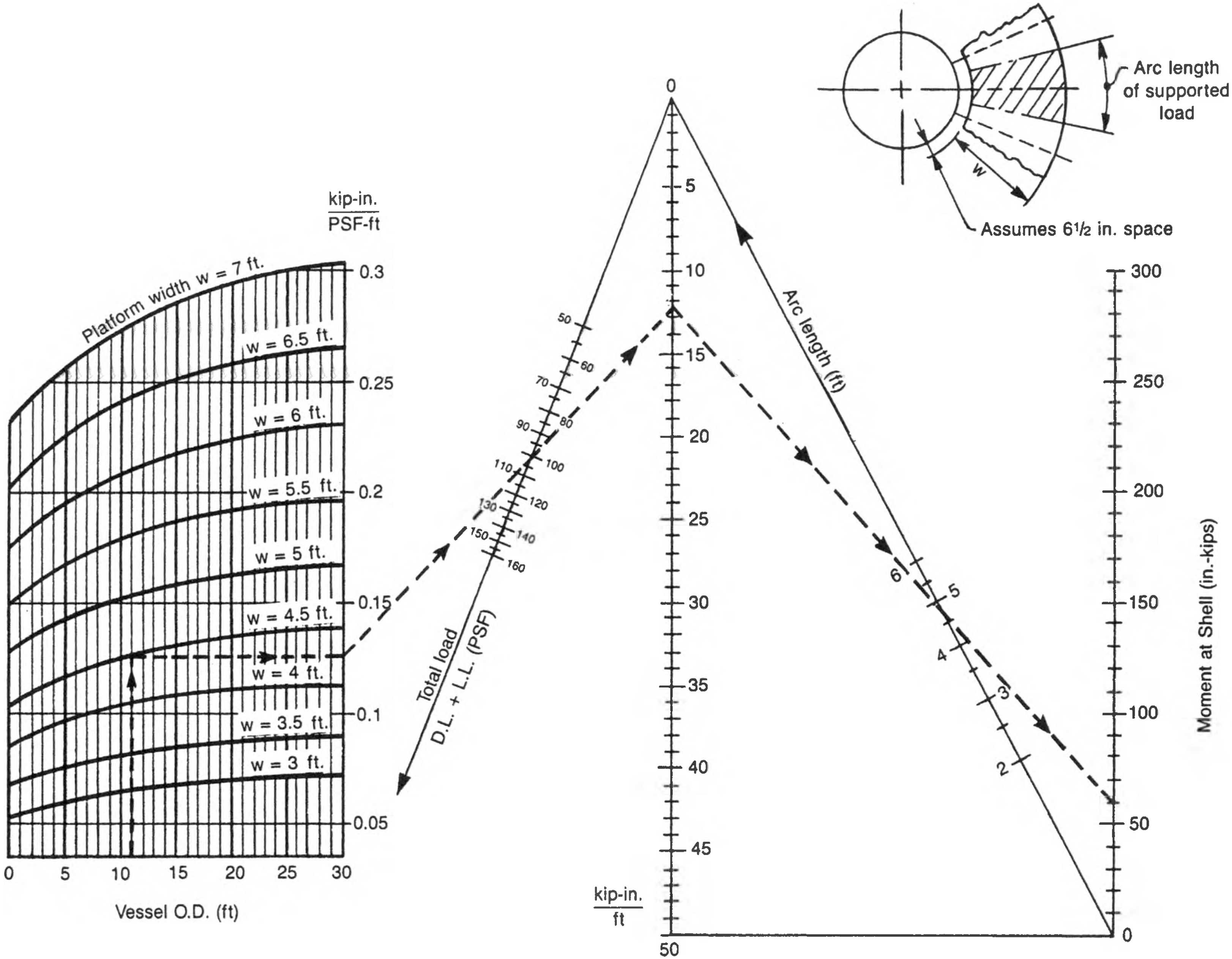


Figure 9-15. Nomograph to find moment at shell due to platform loads.

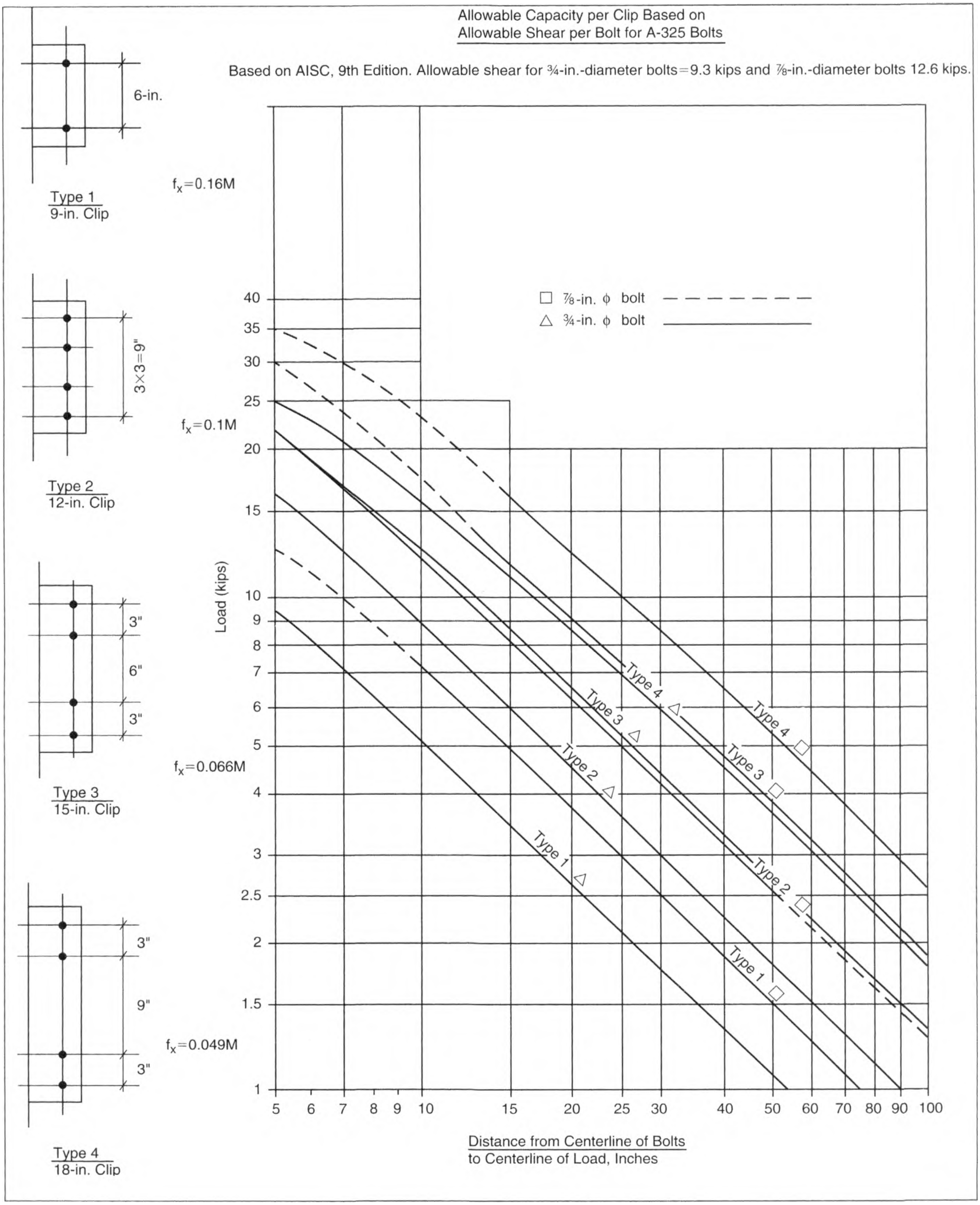
Table 9-3
Grating: Allowable live load based on fiber stress of 18,000 psi

Main Bar Size	Sec. Mod./ft width	Weight lb/sq ft	Bearing Bars at 1 ⅜ Center to Center—Cross Bars at 4 in. Span (ft-in.)											
			Type*	1-0	1-6	2-0	2-6	3-0	3-6	4-0	4-6	5-0	5-6	6-0
1 × ¼	0.380	9.0	U	4562	2029	1142	731	506	372	286	224			
			C	2283	1522	1142	912	762	653	571	506			
1 × 5/16	0.474	11.9	U	5687	2529	1423	910	633	465	355	282			
			C	2845	1898	1423	1139	947	812	712	633			
1¼ × ¼	0.594	10.9	U	7126	3169	1782	1141	793	583	446	353	286	236	196
			C	3564	2376	1782	1426	1186	1019	892	792	713	648	595
1¼ × 5/16	0.741	14.3	U	8888	3948	2221	1423	986	726	555	438	355	295	246
			C	4445	2963	2221	1778	1482	1268	1112	986	889	808	742
1½ × ¼	0.856	12.9	U	10265	4564	2567	1641	1142	836	641	506	412	339	286
			C	5132	3423	2567	2052	1712	1468	1282	1140	1027	932	856
1½ × 5/16	1.066	16.7	U	12796	5689	3198	2048	1423	1045	798	632	512	422	355
			C	6396	4266	3198	2558	2133	1826	1599	1422	1279	1163	1066
1½ × ⅜	1.276	19.6	U	15312	6806	3829	2451	1702	1251	958	758	613	506	425
			C	7654	5105	3829	3063	2553	2188	1914	1702	1532	1393	1276
1¾ × ¼	1.164	14.8	U	13963	6206	3492	2233	1553	1140	875	691	559	463	386
			C	6981	4656	3492	2792	2326	1996	1745	1552	1396	1270	1165
1¾ × 5/16	1.451	19.1	U	17411	7738	4352	2788	1936	1422	1087	861	696	576	484
			C	8708	5805	4352	3483	2903	2488	2176	1935	1742	1583	1452
1¾ × ⅜	1.737	22.5	U	20842	9262	5210	3336	2315	1702	1302	1029	834	688	579
			C	10420	6949	5210	4169	3473	2978	2604	2315	2085	1895	1738
2 × ¼	1.520	16.7	U	18242	8107	4562	2918	2026	1489	1141	902	730	604	507
			C	9121	6082	4562	3648	3040	2608	2281	2027	1825	1858	1521
2 × 5/16	1.895	21.5	U	22740	10102	5686	3637	2526	1858	1422	1123	910	753	633
			C	11371	7581	5686	4547	3791	3248	2842	2529	2275	2067	1895
2 × ⅜	2.269	25.4	U	27224	12098	6808	4356	3026	2223	1702	1344	1088	900	758
			C	13613	9073	6808	5446	4536	3888	3401	3026	2723	2476	2269

**C—concentrated*
U—uniform

Table 9-4
Floor plate: Allowable live load based on fiber stress of 20,000 psi

Long Span (ft-in.)	Nominal Thickness (in.)	Short Span (ft-in.)							
		2-6	3-0	3-6	4-0	4-6	5-0	5-6	6-0
Supports on Four Sides									
2-6	¼	656							
	5/16	1026							
3-0	¼	514	452						
	5/16	806	708						
3-6	¼	441	366	328					
	5/16	691	573	515					
4-0	¼	393	316	274	249				
	5/16	617	496	431	391				
4-6	¼	366	284	239	210	195			
	5/16	575	446	376	331	307			
5-0	¼	350	262	215	185	167	156		
	5/16	550	411	338	291	264	246		
5-6	¼	340	248	198	168	148	135	126	
	5/16	532	391	312	265	234	214	201	
6-0	¼	330	240	187	154	134	120	111	104
	5/16	517	377	293	244	213	191	173	166
6-6	¼			178	145	124	109	96	93
	5/16			281	230	197	174	158	140
7-0	¼			173	138	116	101	91	83
	5/16			273	218	184	162	145	135
7-6	¼			170	133	111	95	84	76
	5/16			268	210	175	152	135	122
8-0	¼					106	90	79	71
	5/16					168	143	127	114
8-6	¼					102	86	75	67
	5/16					163	137	120	106
9-0	¼							72	63
	5/16							114	101
Supports on Two Sides									
∞	¼	255	174	125	93	71	55		
∞	5/16	402	275	198	148	114	90	72	58



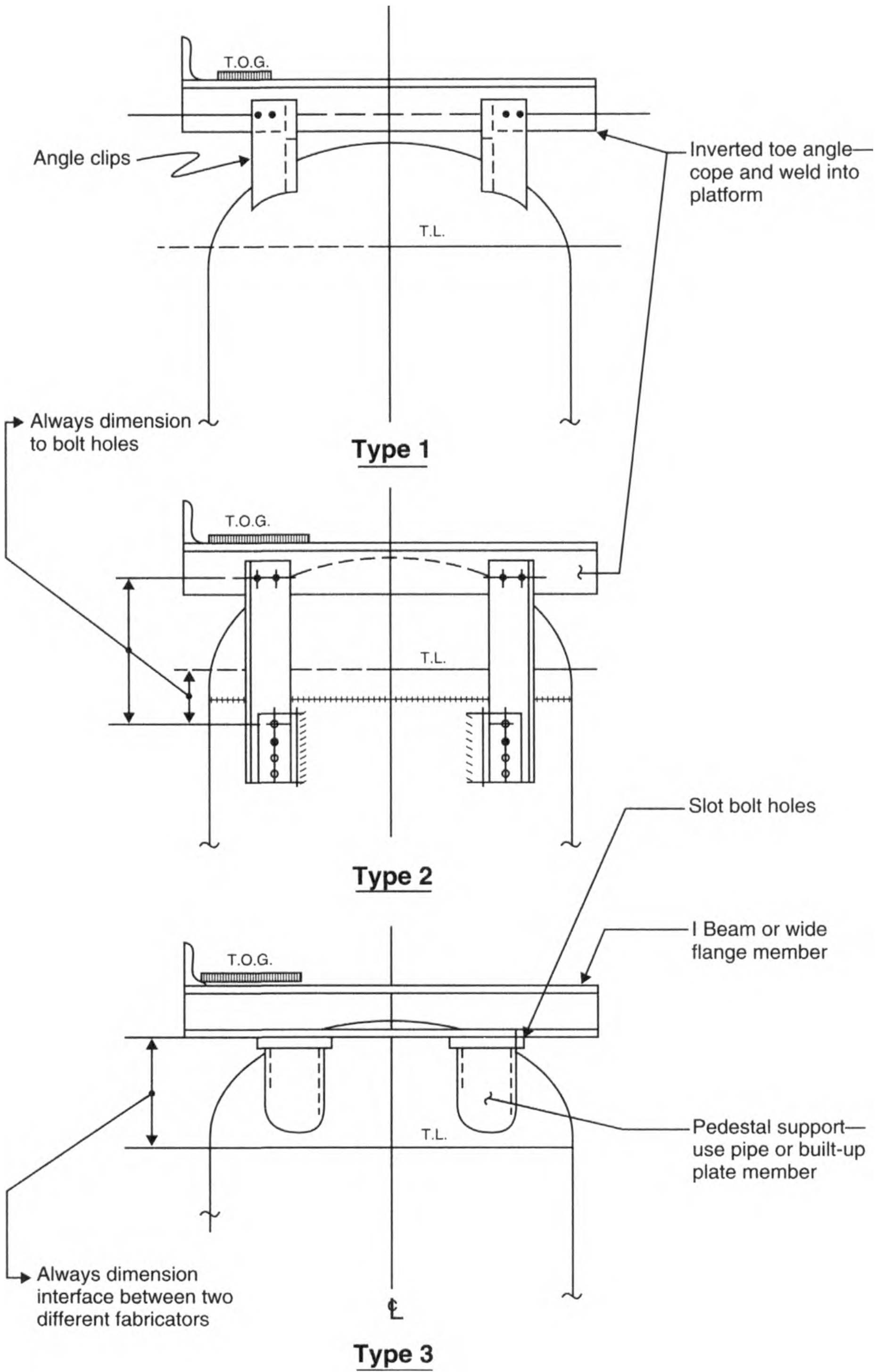
Allowable Capacity per Clip Based on
Allowable Shear per Bolt for A-325 Bolts
Based on AISC, 9th Edition. Allowable shear for ¾-in.-diameter bolts=9.3 kips and ⅞-in.-diameter bolts 12.6 kips.
6-in.
Type 1
9-in. Clip
3×3=9"
Type 2
12-in. Clip
3"
6"
3"
Type 3
15-in. Clip
3"
9"
3"
Type 4
18-in. Clip
fx=0.16M
fx=0.1M
fx=0.066M
fx=0.049M
□ ⅞-in. ϕ bolt
△ ¾-in. ϕ bolt
Load (kips)
40
35
30
25
20
15
10
9
8
7
6
5
4
3
2.5
2
1.5
1
Type 4
Type 3
Type 2
Type 1
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
90
100
Distance from Centerline of Bolts
to Centerline of Load, Inches

Notes

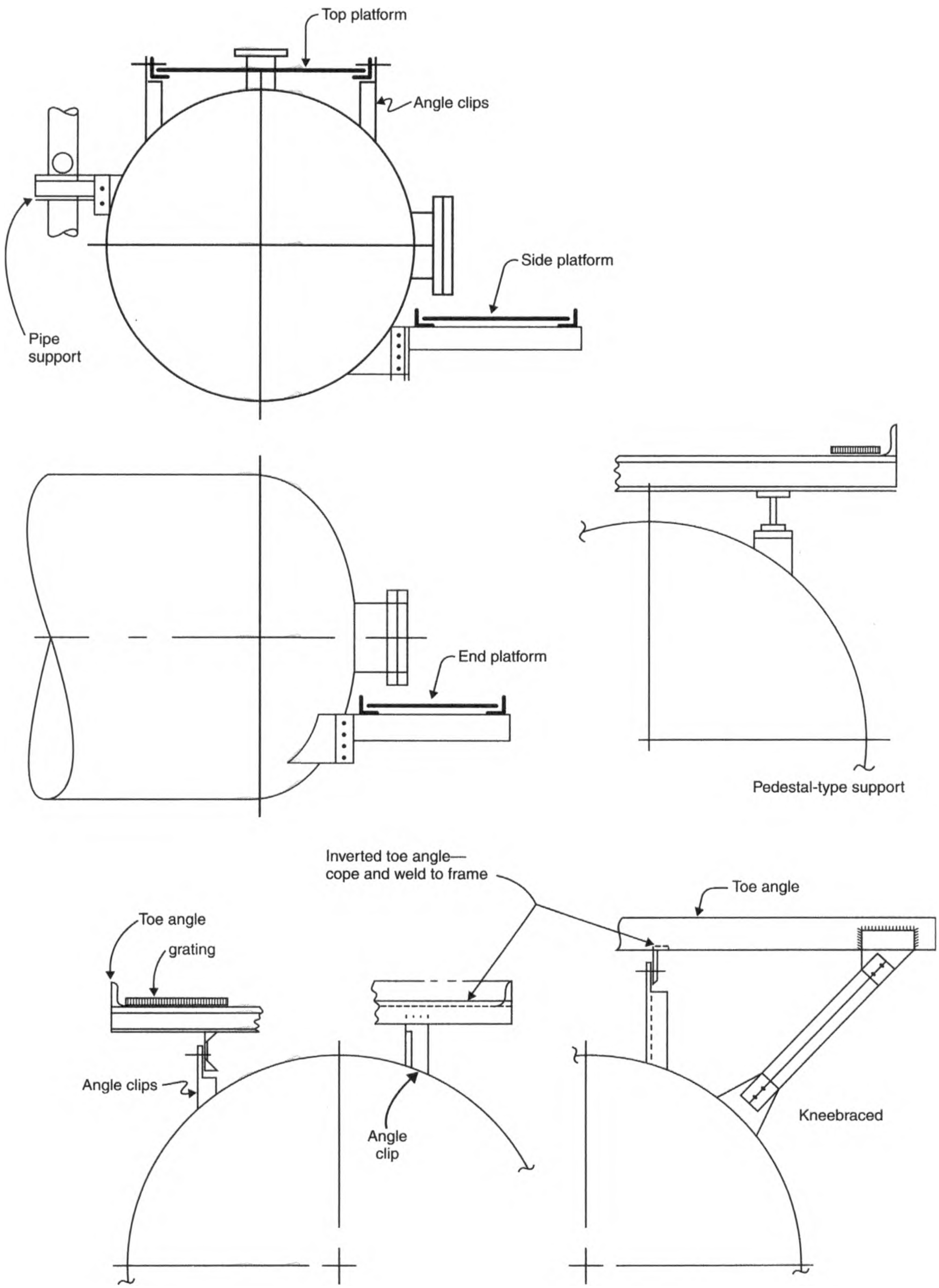
1. Dead loads: 30psf. Platform steel weight. This includes grating or floor plate, structural framing, supports, toe angle or plate, and handrailing. To find weight of steel, multiply area of platforms by 30 psf.
2. Live loads:
   - *Operating*: Approximately 25–30 psf. Live load is small because it is assumed there are not a lot of people or equipment on the platform while vessel is operating. Combine effects with shell stress due to design pressure.
   - *Maintenance/construction*: 50–75 psf. Live load is large because there could be numerous persons, tools, and equipment on platforms; however, there would be no internal pressure on vessel.
3. Assume each bracket shares one-half of the area between each of the adjoining brackets. Limit bracket spacing to 6 ft-0 in. arc distance and overhangs to 2 ft-0 in. For stability, bracket spacing should not exceed 60°.
4. Kneebraces should be 45° wherever possible. Always dimension to bolt holes, not to edge of brackets or top of clips.
5. Bracket spacing is governed by one of the following conditions:
   - *Shell stress*: Based on dead-load and live-load induced stress from platform support brackets. Shell stresses may be reduced by using a longer clip or reducing the angle between brackets.
   - *Bolt shear stress*: A-307 or A-325 in single or double shear. Bolt shear stresses may be reduced by increasing the size or number of bolts or increasing the distance between bolts.
   - *Maximum arc distance*: Measured at the outside of the platform. Based on the ability of the toe angle to transmit loads to brackets. Affects "stability" of platform.
   - *Stress/deflection of floor plate or grating*: Allowable live load affects "springiness of platforms." Use Tables 9-3 and 9-4 and assume "allowable live load" of 150–200 psf.
6. Shell stresses should be checked by an appropriate "local load" procedure.

Procedure 9-3: Design of Square and Rectangular Platforms

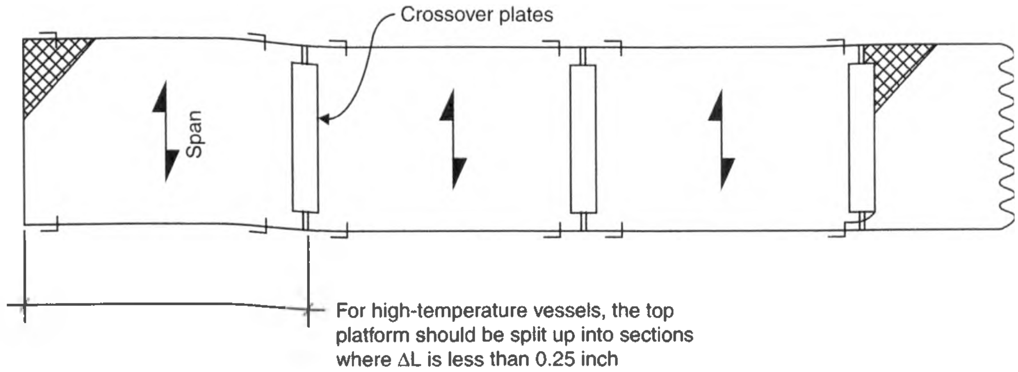
Top Platforms for Vertical Vessels



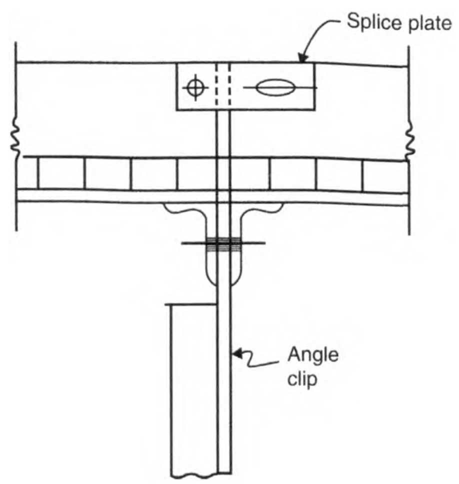
Platforms for Horizontal Vessels



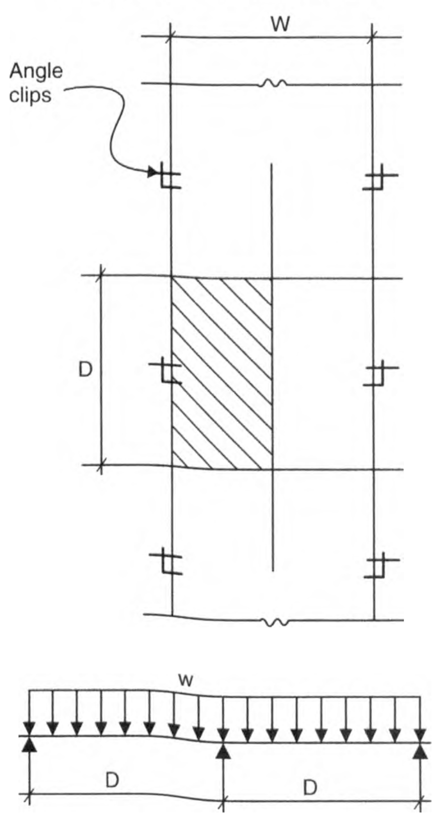
Long Walkways or Continuous Platform on Horizontal Vessel



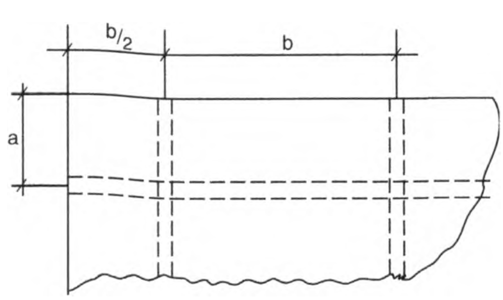
Horizontal Platform Splice (Not for Thermal Expansion)



Check of Toe Angle Frame



Maximum Length of Unsupported Toe Angle (Based on 105-psf Load and L6 × 3½ × 5/16)



a (ft)	b (ft)	
	Grating	Check plate
<1	15	∞
1½	10	12
2	8	9
2½	6	6

Check clip spacing:

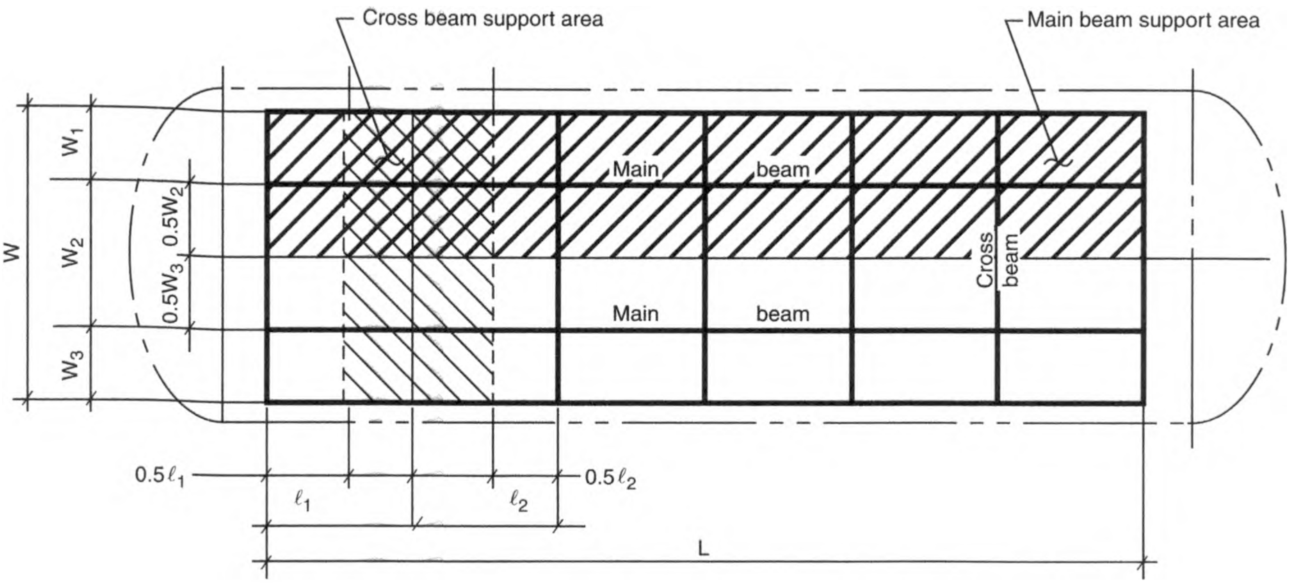
$$P = D.L + L.L. = \text{psf}$$

$$w = \frac{WP}{2} \frac{\text{lb}}{\text{ft}}$$

$$M = \frac{wD^2}{8} \text{ ft-lbs}$$

$$\sigma = \frac{12M}{Z} < 21.6 \text{ ksi}$$

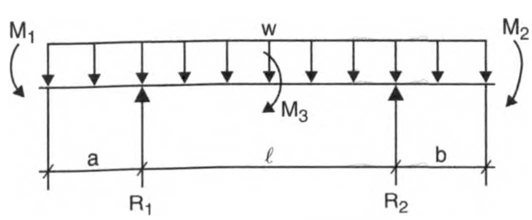
Z = Section modulus of toe angle, in^3



Notation

A = area, sq ft
p = unit load, psf
P = total load, lb
w = unit load on beam, lb/linear foot
R = reaction, lb
M = moment, in.-lb
d = deflection, in.
K = end connection coefficient, use 1.0
r = radius of gyration of column, in.
f_a = axial stress, psi
F_a = allowable compressive stress, psi

Calculations



Main or Cross Beam

Main or Cross Beam

- *Area, A.*

$$A = (0.5w_2 + w_1)L$$

- *Load, P.*

$$P = A_p$$

- *Unit load on beam, w.*

$$w = \frac{P}{L}$$

$$R_1 = \frac{w\left[(a+\ell)^2 - b^2\right]}{2l}$$

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

$$M_2 = \frac{wb^2}{2}$$

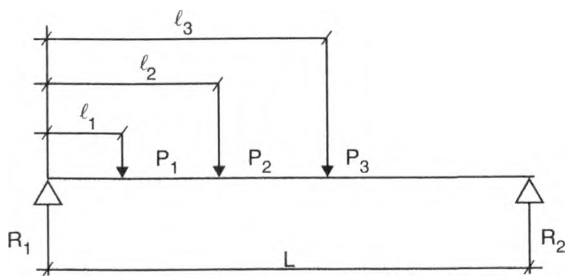
$$M_3 = R_1\left(\frac{R_1}{2w} - a\right)$$

$$\delta_{end} = \frac{wa}{24EI}\left(\ell^3 - 6a^2\ell - 3a^3\right)$$

$$\delta_{center} = \frac{w\ell^2}{384EI}\left(5\ell^2 - 24a^2\right)$$

Notes

1. Maximum distance between cross beams is governed by one of two conditions:
   a. Maximum span of grating or checkered plate.
   b. Deflection/stress of toe angle. Ability of toe angle to carry the load.
2. Each beam supports the load from one-half the area between the adjacent beams.



Beams—Multiple Loads

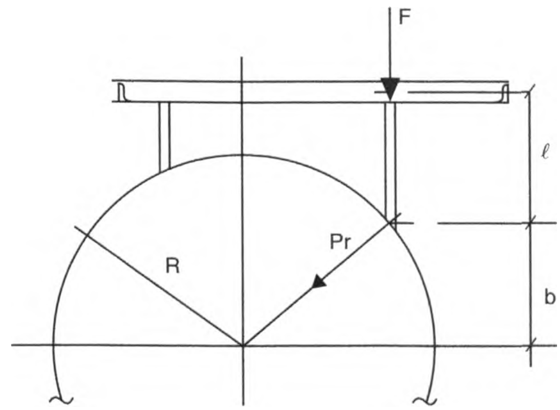
$$R_2 = \frac{l_1P_1 + l_2P_2 \cdots + \cdots l_nP_n}{L}$$

$$R_1 = \sum P_n - R_2$$

To find maximum moment:

1. Select maximum reaction.
2. Total all downward loads, starting from the reaction, until the value of the reaction is exceeded. This is the point where the maximum moment will occur.
3. The moments are equal to the right or left of that point. Sum the moments in either direction.

Design of Vessel Clips



- *Slenderness ratio.*

$$\frac{k\ell}{r}$$

Use k = 1.0.

- *F = reaction from main beam columns.*
- *Allowable compressive stress, F_a, based on slenderness ratio.*
- *Axial stress, f_a.*

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

- *Check stress ratio.*

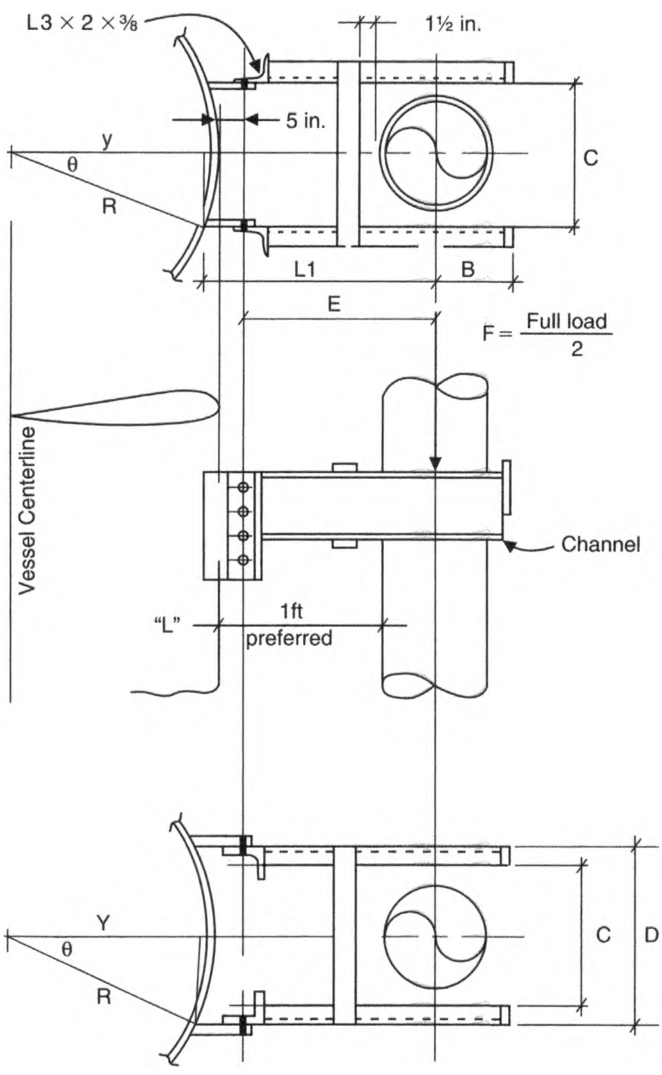
$$\frac{f_a}{F_a} < 0.15$$

- *Radial load in shell, P_r.*

$$P_r = \frac{bF}{R}$$

Procedure 9-4: Design of Pipe Supports

Unbraced Pipe Supports



Alternate Case: Legs Turned In

Types of Brackets

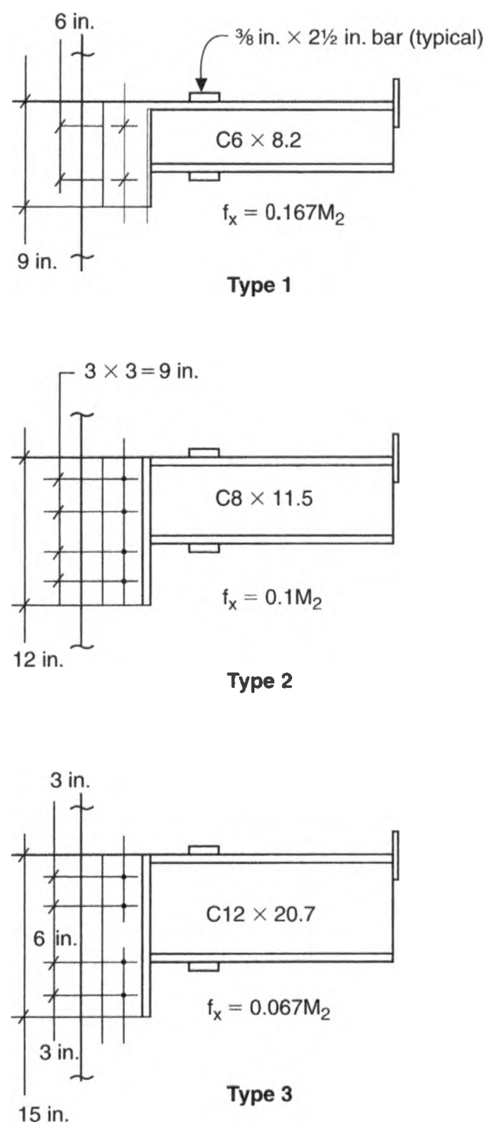


Table 9-5
Pipe support dimensions

Dimension	Pipe Size											
	2 in.	3 in.	4 in.	6 in.	8 in.	10 in.	12 in.	14 in.	16 in.	18 in.	20 in.	24 in.
B	2.75	3.5	4.25	6	7.5	9	10.5	11.25	12.75	14	16	18
C	4	5	6.5	9	12	14	16	17	19	21	23	27
D—Type 1	7.75	8.75	10.25	12.75	15.75	17.75	19.75	20.75				
D—Type 2				13.5	16.5	18.5	20.5	21.5	23.5			
D—Type 3					18	20	22	23	25	27	29	33

Table 9-6
Weight of pipe supports, lb (without clips)

L Dimension	Support Type	Pipe Size									
		2 in.	3 in.	4 in.	6 in.	8 in.	10 in.	12 in.	14 in.	16 in.	18 in.
12 in.	1	12	13	15	18	22	25				
	2				22	27	31	35	37	41	45
	3					44	51	57	61	67	72
14 in.	1	15	16	18	21	25	28				
	2				26	31	35	39	41	45	49
	3					52	58	64	68	74	79
16 in.	1	18	19	20	24	28	31				
	2				30	35	39	43	45	49	51
	3					59	65	71	74	81	86
18 in.	1	20	21	23	26	31	34				
	2				34	39	43	46	49	53	56
	3					65	72	77	81	88	93
20 in.	1	23	24	26	29	33	36				
	2				38	42	46	50	53	57	60
	3					72	79	84	88	95	100

Kneebraced Pipe Supports

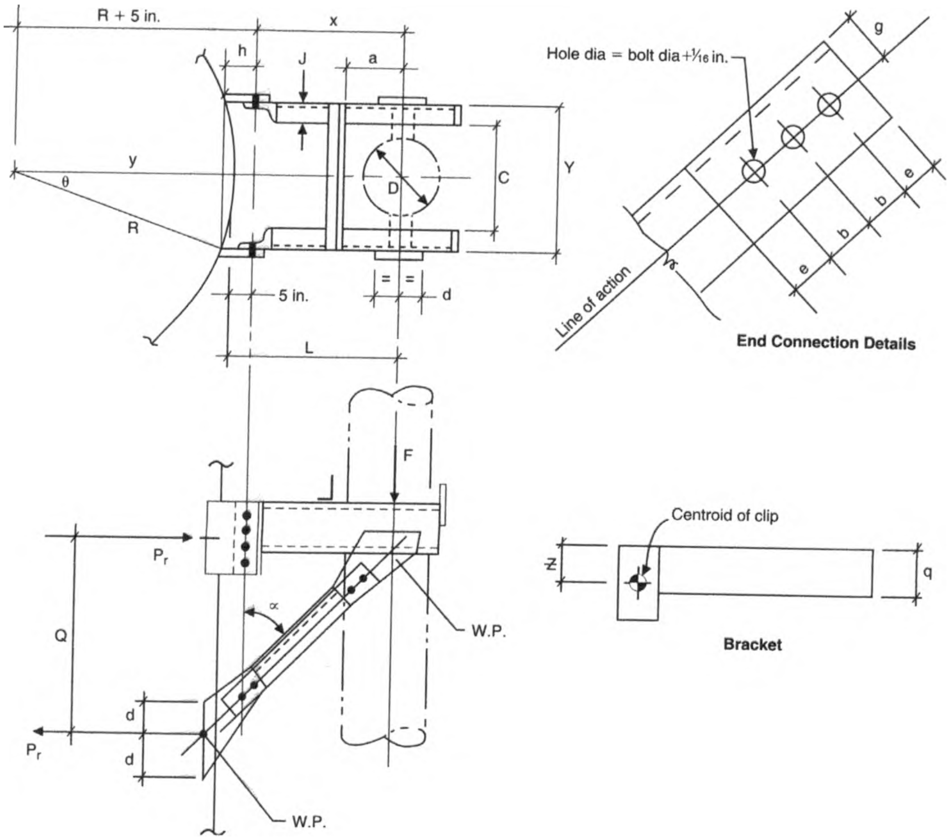
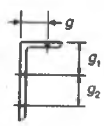


Table 9-7
Usual gauges for angles, in.



Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1
g	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_1	3	2½	2¼	2										
g_2	3	3	2½	1¾										

Dimensions

$$a = \frac{D}{2} + 3 \text{ in.}$$

$$Y = C + 2j + 1 \text{ in.}$$

$$x = L - 5 \text{ in.}$$

$$\theta = \arcsin\left(\frac{Y}{2R}\right)$$

$$y = R \cos\theta$$

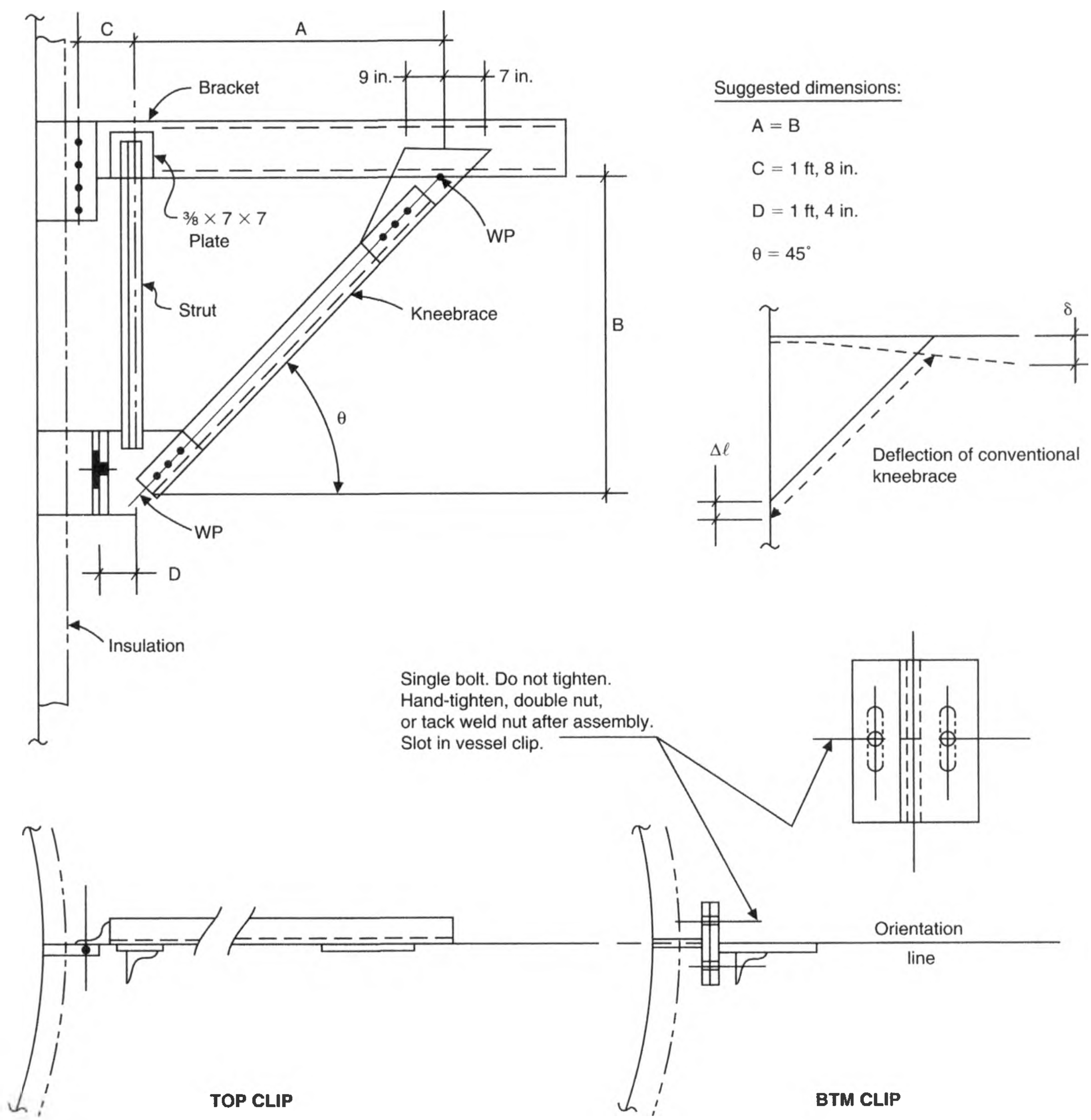
$$h = (R + 5 \text{ in.}) - y$$

$$Q = (x + h) + (q - z)$$

Table 9-8
Kneebraced pipe support dimensions

Allowable Load (kips)	Bracket Type	"L" Max	Angle Size	Bolt Qty & Size	b	e	d	j
12.5	1	36	2½ × 2 × ⅜	(2) ¾	2.5	1.25	2.5	1.92
17.5	1	36	3 × 2 × ⅜	(2) ⅞	2.75	1.5	3	1.92
24	1	36	3 × 2 × ⅜	(3) ⅞	2.75	1.5	3	1.92
21.5	1	54	3½ × 3 × ⅜	(2) 1	3	1.75	3.25	1.92
24	2	54	3½ × 3 × ⅜	(2) 1⅛	3.25	2	4	2.26
26.5	2	54	4 × 3 × ⅜	(2) 1¼	3.5	2.25	4.5	2.26
30	2	54	4 × 3 × ⅜	(3) 1	3	1.75	3.25	2.26
33.5	2	54	5 × 3 × ⅜	(3) 1⅛	3.25	2	4	2.26
37.5	2	54	5 × 3 × ⅜	(3) 1¼	3.5	2.25	4.5	2.26
26.5	3	66	6 × 3½ × ⅜	(2) 1¼	3.5	2.25	5.25	2.942
37.5	3	66	6 × 3½ × ⅜	(3) 1¼	3.5	2.25	5.25	2.942
26.5	3	75	6 × 4 × ⅜	(2) 1¼	3.5	2.25	5.25	2.942
37.5	3	75	6 × 4 × ⅜	(3) 1¼	3.5	2.25	5.25	2.942
50	3	75	6 × 4 × ⅜	(4)1¼	3.5	2.25	5.25	2.942

High-Temperature Brackets

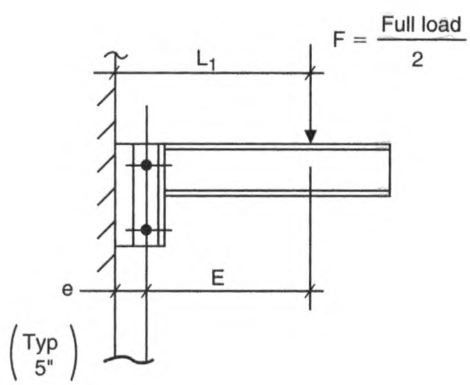


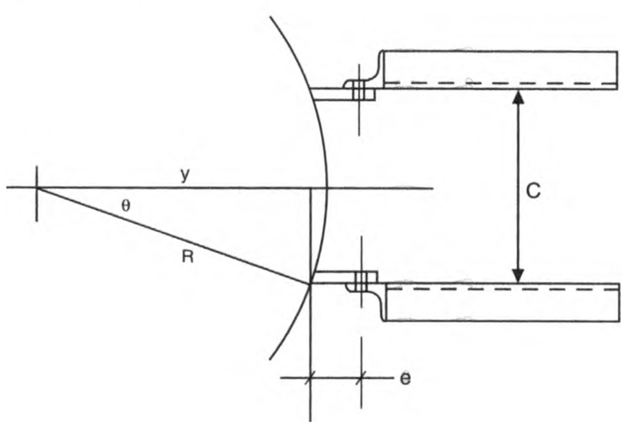
Design of Supports

Notation

A = cross-sectional area of kneebrace, in.^{2}
F = ½ of the total load on the support, lb
R_n = reaction, lb
P = compression load in kneebrace, lb
P_r = radial load in shell, lb
M_1 = moment at shell, in.-lb
M_2 = moment at line of bolts, in.-lb
r = radius of gyration, in.
N = number of bolts in clip
τ = shear load, lbs
E = modulus of elasticity, psi
I = moment of inertia, in.^{4}
Z = section modulus, in.^{3}
K = end connection coefficient
δ = deflection, in.
f_a = axial stress, psi
f_b = bending stress, psi
F_a = allowable axial stress, psi
F_b = allowable bending stress, psi

Cantilever-Type Brackets





- *Dimensions.*

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

$$\therefore\ \theta =$$

$$y = R\cos\theta$$

$$L_1 = (R - y) + L + \frac{1}{2}\ \text{pipe dia}$$

$$E = L_1 - [(R - y) + e]$$

- *Loads.*

$$M_1 = FL_1$$
$$M_2 = FE$$

- *Bracket check.*

$$f_b = \frac{M_1}{Z} < F_b$$

$$\delta = \frac{FL_1^3}{3EI}$$

- *Bolting check.*

Type	f_x
1	0.167 M_2
2	0.1 M_2
3	0.067 M_2

$$f_y = \frac{F}{N}$$

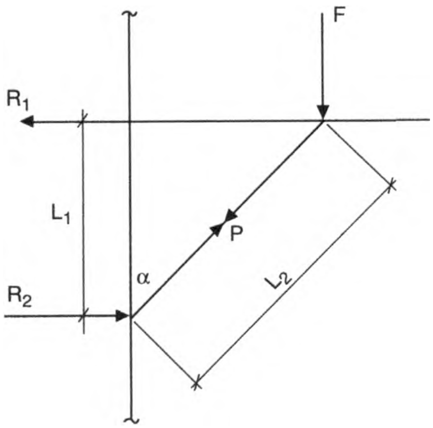
$$f_r = \sqrt{f_x^2 + f_y^2}$$

Compare with allowable shear.

- *Check shell for longitudinal moment, M_2.*

Design of Kneebraced Supports

Case 1



$$R_1 = R_2 = F \tan\alpha$$

$$P = \frac{F}{\cos\alpha} \qquad L_2 = \frac{L_1}{\cos\alpha}$$

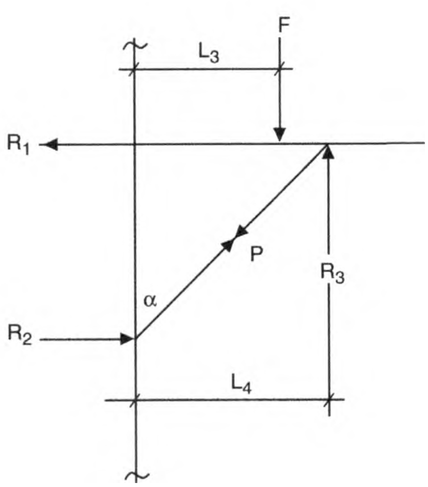
$$f_a = \frac{P}{A} < F_a$$

$$\frac{KL_2}{r} \qquad F_a$$

$$P_r = R_1 \cos\theta$$

$$\tau = \frac{P}{N} \text{ or } \frac{R_1}{N}$$

Case 2



$$R_3 = \frac{L_3 F}{L_4}$$

$$R_1 = R_2 = R_3 \tan\alpha$$

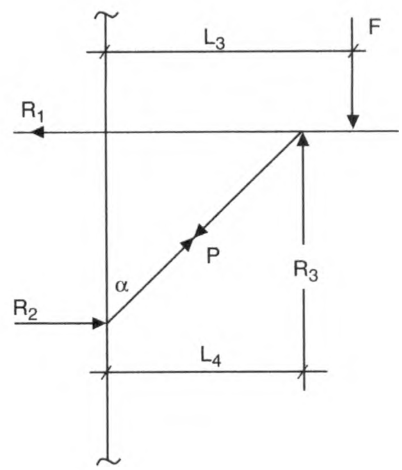
$$P = \frac{R_3}{\cos\alpha}$$

$$f_a = \frac{P}{A} < F_a \qquad f_b = \frac{L_4 - L_3}{Z}$$

$$\frac{KL_2}{r} \qquad F_a$$

$$\tau = \frac{P}{N} \text{ or } \frac{R_1}{N}$$

Case 3



$$R_3 = \frac{L_3 F}{L_4}$$

$$R_1 = R_2 = R_3 \tan\alpha$$

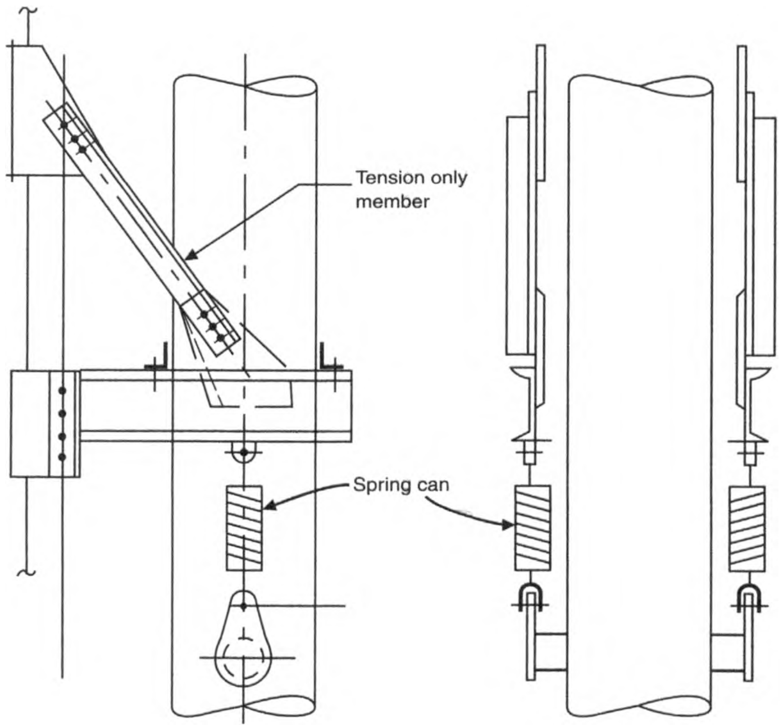
$$P = \frac{R_3}{\cos\alpha}$$

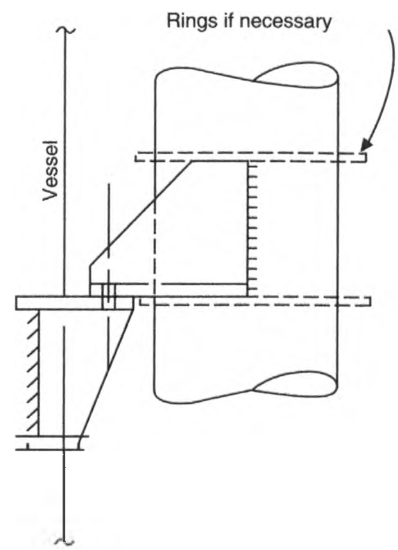
$$f_a = \frac{P}{A} < F_a \qquad f_b = \frac{L_3 - L_4}{Z}$$

$$\frac{KL_2}{r} \quad F_a$$

$$\tau = \frac{P}{N} \text{ or } \frac{R_1}{N}$$

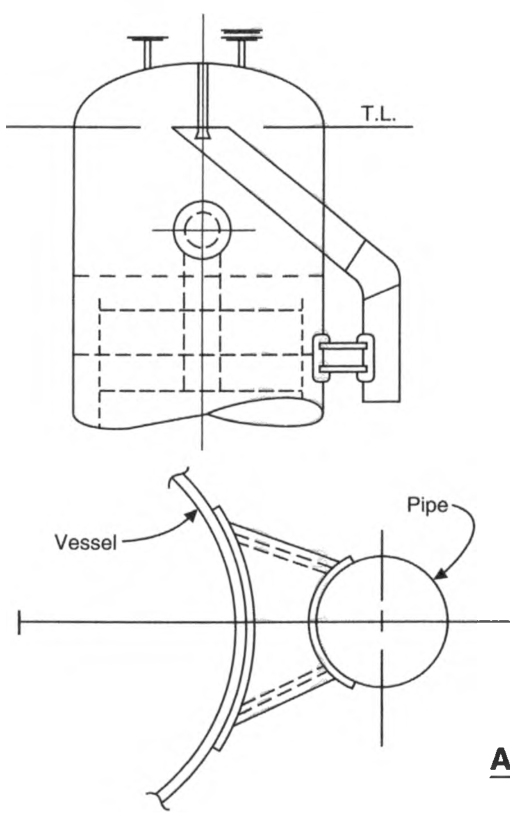
Alternate-Type Supports

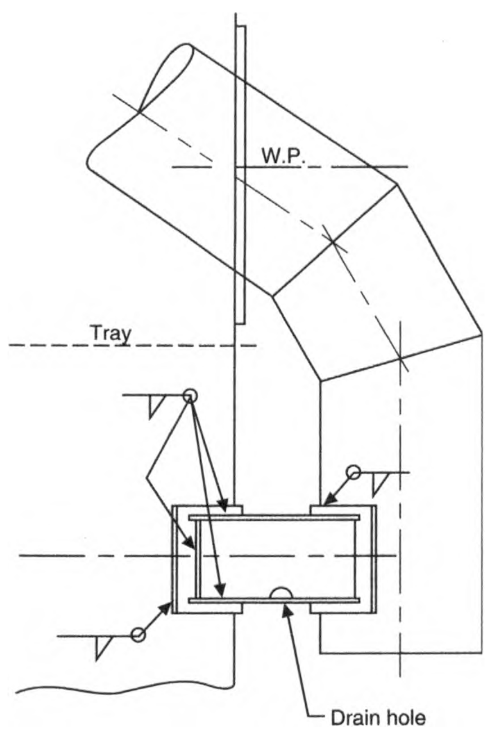




Pipe and Vessel Share Load to Reduce Load on Vessel Shell

Inverted Support, Large Lines with Spring Hangers





All Welded Integral Construction for Overhead Vapor Lines

Notes

1. Allowable deflection brackets should be limited to L/360.
2. Kneebracing should be used only if absolutely necessary.
3. Pipe support should be placed as close as possible to the nozzle to which it attaches. This limits the effect of differential temperature between the pipe and the vessel. If the line is colder than the vessel, the nozzle will tend to pick up the line. For the reverse situation (pipe hotter than vessel), the line tends to go into compression and adds load to the support.
4. The nozzle and the pipe support will share support of the overall line weight. Each will share the load in proportion to its respective stiffness. The procedure is to design the pipe support for the entire load, which is conservative. However, be aware that as the pipe support deflects, more of the load is transferred to the nozzle.
5. The pipe is normally supported by trunnions welded to the pipe. The trunnions can be shimmed to accommodate differences in elevation between the trunnions and the supports.
6. Design/selection of pipe supports:
   - Make preliminary selection of support type based on the sizing in the table.
   - Check allowable bolt loads per chart.
   - Check shell stresses via the applicable local load procedure.
7. The order of preference for overstressed supports, shells, or bolts is as follows:
   - Go to the next largest type of support.
   - If the loads in the bolts exceed that allowable, change the material or size of the bolts.
   - If the brackets are overstressed, increase the bracket size.
8. Use "high-temperature brackets" for kneebraced pipe supports or platform brackets when the design temperature of the vessel exceeds 650°F. This sliding support is utilized for hot, insulated vessels where the support steel is cold. This sliding support prevents the support from dipping as the vessel clips grow apart due to linear thermal expansion of the vessel while the kneebrace remains cold. This condition becomes more pronounced as the vessel becomes hotter and the distance between clips becomes greater.
9. Keep bolts outside of the insulation.
10. Vessel clip thickness should be ⅜ in. for standard clips up to 650°F. Above 650°F, clips should be ½ in. thick.
11. Bolt holes for Type 1, 2, or 3 supports should be $^{13}/_{16}$-in.-diameter holes for ¾-in.-diameter bolts.

Procedure 9-5: Shear Loads in Bolted Connections

Table 9-9
Allowable loads, in kips

Material	Size		⅝ in.	¾ in.	⅞ in.	1 in.	1 ⅛ in.	1 ¼ in.	1 ⅜ in.	1 ½ in.
A-307	Single		3.68	5.30	7.21	9.42	11.9	14.7	17.8	21.2
	Double		7.36	10.6	14.4	18.8	23.8	29.4	35.6	42.4
A-325	Single		7.36	10.6	14.4	18.8	23.8	29.4	35.6	42.4
	Double		14.7	21.2	28.8	37.6	47.7	58.9	71.2	84.8

Values from AISC.

Cases of Bolted Connections

Case 1

n = no. of fasteners in a vertical row

m = no. of fasteners in a horizontal row = 2

I_p = polar moment of inertia about c.g. of fastener group: $I_x + I_y$

$$I_x = 2\left[\frac{nb^2(n^2-1)}{12}\right]$$

$$I_y = n\left[\frac{mD^2(m^2-1)}{12}\right]$$

$$f_x = \frac{(F\ell)(n-1)b}{2I_p}$$

$$f_y = \frac{F}{mn} + \frac{F\ell D}{2I_p}$$

$$f_r = \sqrt{f_x^2 + f_y^2}$$

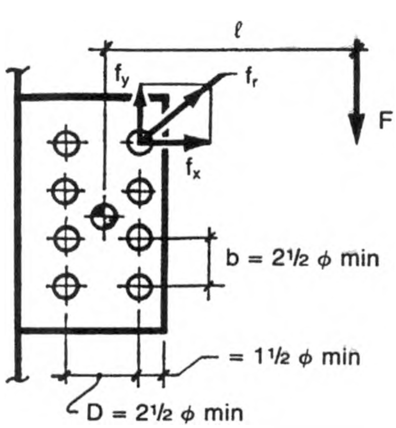


Figure 9-16. Longitudinal clip with double row of n bolts.

Case 2

$$f_x = \frac{F\ell}{e}$$

$$f_y = \frac{F}{2}$$

$$f_r = \sqrt{f_x^2 + f_y^2}$$

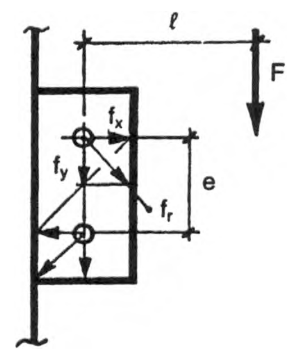


Figure 9-17. Longitudinal clip with two bolts.

Case 3

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

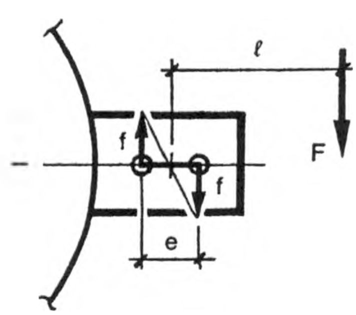


Figure 9-18. Circumferential clip with two bolts.

Case 4

$$f_x = x_n \left[\frac{Fl}{2(x_1^2 + x_2^2 + \ldots + x_n^2)} \right]$$

$$f_y = \frac{F}{n}$$

$$f_r = \sqrt{f_x^2 + f_y^2}$$

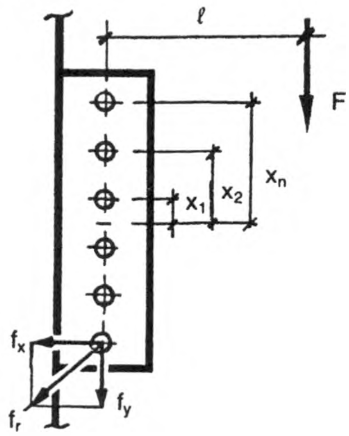


Figure 9-19. Longitudinal clip with single row of n bolts.

Case 5

$$f_x = \frac{Fl}{2b}$$

$$f_y = \frac{Fld}{2(b^2 + d^2)}$$

$$f_r = \sqrt{f_x^2 + f_y^2}$$

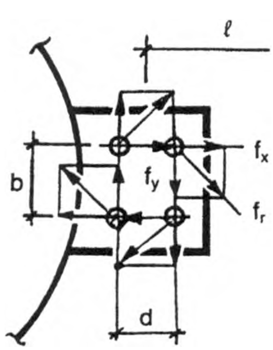


Figure 9-20. Circumferential clip with four bolts.

Shear Loads in Bolted Connections

Shear loads in bolted connections are classified as either "Friction Type" or "Bearing Type". A brief definition is as follows;

Friction Type Connection

A friction type connection is one in which the sole purpose of the bolts is to provide adequate tension such that the two plates being joined will not slip. In this manner the bolts are not technically in shear, but solely in tension. High strength bolts must be used for friction type connections. The following factors are critical to the joint functioning as designed;

a. The condition of the surface finish of the plates being joined on the contacting surfaces.
   1. Uncoated (Class A, B or C)
   2. Hot dipped galvanized and roughened (Class D)
   3. Blast cleaned, zinc rich paint (Class E or F)
   4. Blast cleaned, metalized zinc or alum (Class G or H)
   5. Contact surfaces are coated (Class I)

b. The tightness of the joint based on;
   1. Snug Tightened
   2. Pre-tensioned
   3. Slip Critical

c. Size of hole relative to bolt size. Dimensions given in Table J3.3.
   1. STD: Standard round holes
   2. OVS: Oversize round holes
   3. SSL: Short slotted holes
   4. LSL: Long Slotted holes

Bearing Type Connection

A bearing type connection is one in which the bolts are in shear because there is not significant enough friction in the joint to prevent slip. There are two major classifications of bearing type connections;

a. Connections with threads in shear plane (Type N)
b. Connections without threads in shear plane (Type X)

The following bolt hole/slot types may be used with a bearing type connection;

a. STD: Standard round holes, d + 1/16"
b. NSL: Long Slotted holes

Paint is acceptable for all types of bearing connections.

The cost to install friction type connections is variable depending on the degree of labor required to produce the joint. In general, the relative costs of each type of joint is as follows (from cheapest to most expensive):

a. Snug tightened (X): 1.0
b. Pre-tensioned (X): 1.2
c. Snug tightened (N): 1.3
d. Pre-tensioned (N): 1.6
e. Slip Critical (N or X): 3.1

Procedure 9-6: Design of Bins and Elevated Tanks [3–9]

The definition of a "bulk storage container" can be quite subjective. The terms "bunkers," "hoppers," and "bins" are commonly used. This procedure is written specifically for cylindrical containers of liquid or bulk material with or without small internal pressures.

There is no set of standards that primarily applies to bins and since they are rarely designed for pressures greater than 15 psi, they do not require code stamps. They can, however, be designed, constructed, and inspected in accordance with certain sections of the ASME Code or combinations of codes.

When determining the structural requirements for bins, the horizontal and vertical force components on the bin walls must be computed. A simple but generally incorrect design method is to assume that the bin is filled with a fluid of the same density as the actual contents and then calculate the "equivalent" hydrostatic pressures. While this is correct for liquids, it is wrong for solid materials. All solid materials tend to bridge or arch, and this arch creates two force components on the bin walls.

The vertical component on the bin wall reduces the weight load on the material below, and pressures do not build up with the depth as much as in the case of liquids. Consequently, the hoop stresses caused by granular or powdered solids are much lower than for liquids of the same density. However, friction between the shell wall and the granular material can cause high longitudinal loads and even longitudinal buckling. These loads must be carefully considered in the case of a "deep bin."

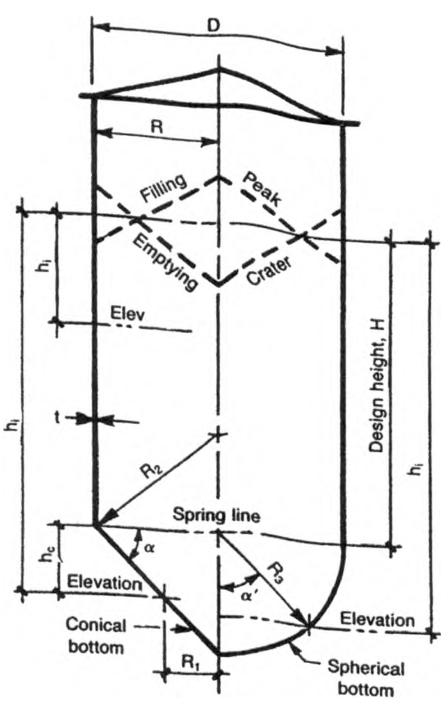
In a "shallow bin," the contents will be entirely supported by the bin bottom. In a "deep bin" or "silo," the support will be shared, partly by the bottom and partly by the bin walls due to friction and arching of material.

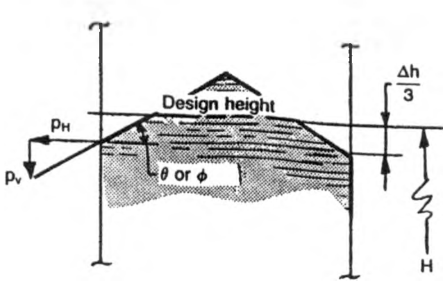
Notation

A = cross-sectional area of bin, ft^2
A_r = area of reinforcement required, $in.^2$
A_a = area of reinforcement available, $in.^2$
A_s = cross-sectional area of strut, $in.^2$
e = common log 2.7183
C.A. = corrosion allowance, in.
E = joint efficiency, 0.35–1.0
F = summation of all vertical downward forces, lb
F_a = allowable compressive stress, psi
f = vertical reactions at support points, lb
h_i = depth of contents to point of evaluation, ft
K_1, K_2 = Rankine factors, ratio of lateral to vertical pressure
M = overturning moment, ft-lb
N = number of supports
P = internal pressure, psi
p_n = pressure normal to surface of cone, psf
p_v = vertical pressure of contents, psf
p_h = horizontal pressure on bin walls, psf
Q = total circumferential force, lb
R_h = hydraulic radius of bin, ft
S = allowable tension stress, psi
T_1, T_{1s} = longitudinal force, lb/ft
T_2, T_{2s} = circumferential force, lb/ft
G = specific gravity of contents
θ = angle of repose of contents, degrees
ϕ = angle of filling, angle of surcharge, friction angle. Equal to θ for free filling or 0 if filled flush, degrees
β = angle of rupture, degrees
μ = friction coefficient, material on material
μ' = friction coefficient, material on bin wall
Δh = height of filling peak, depth of emptying crater, ft
C_s = a function of the area of shell that acts with strut to A_s

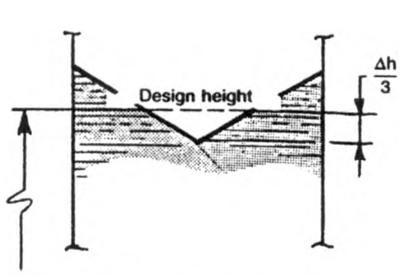
Weights

W = total weight of bin contents, lb
w = density of contents, lb/cu ft
W_T = total weight of bin and contents, lb
W_c = weight of cone and lining below elevation under consideration, lb
W_R = D.L. + L.L. of roof plus applied loads, lb (include weight of any installed plant equipment)
W_s = weight of shell and lining (cylindrical portion only), lb
$W_1 = W + W_c$
W_2 = weight of contents in cylindrical portion of bin, lb, $= \pi R^2 H w$
W_3 = load caused by vertical pressure of contents, lb, $= p_v \pi R^2$





Filling peak



Emptying crater

Figure 9-21. Dimensional data and forces of bin or elevated tank.

W_4 = portion of bin contents carried by bin walls due to friction, lb, $= W_2 - W_3$

$W_5 = W_R + W_4 + W_s$

$W_6 = W_T - W_c - W_{cl}$

W_7 = weight of bin below point of supports plus total weight of contents, lb

W_{cl} = weight of contents in bottom, lb

Bins

1. *Determine if bin is "deep" or "shallow."* The distinction between deep and shallow bins is as follows:
   - In a shallow bin the plane of rupture emerges from the top of the bin.
   - In a deep bin the plane of rupture intersects the opposite bin wall below both the top of the bin and/or the maximum depth of contents.

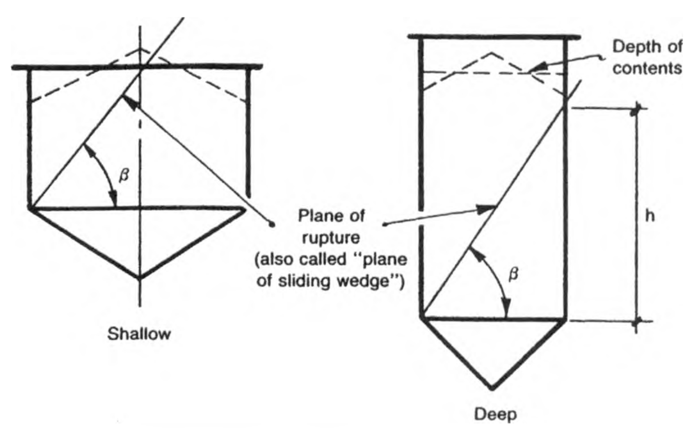


Figure 9-22. Examples illustrating the shallow vs. deep bin.

2. *Determine angle β.*

$$\tan\beta = \mu + \sqrt{\mu + \frac{1+\mu^2}{\mu + \mu'}}$$

If μ and μ' are unknown, compute β as follows:

$$\beta = \frac{90+\theta}{2}$$

and $h = D \tan\beta$.

If h is smaller than the straight side of the bin and below the design depth of the contents, the bin is assumed to be "deep" and the silo theory applies. If h is larger than the straight side of the bin or greater than the design depth of the contents, then the bin should be designed as "shallow." This design procedure is also known as the "sliding wedge" method.

Liquid-Filled Elevated Tanks

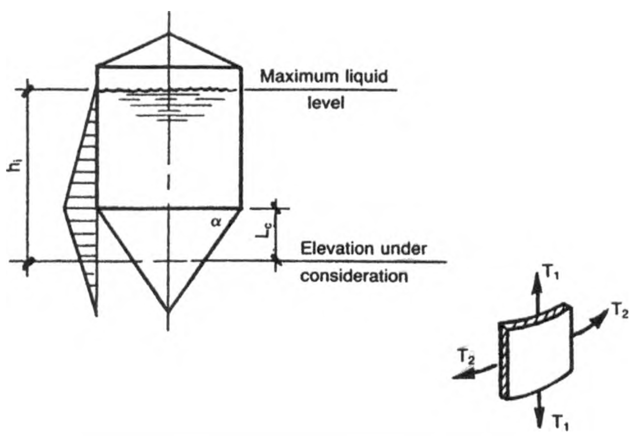


Figure 9-23. Dimensions and loads for a liquid-filled elevated tank.

- *Shell (API 650 & AWWA D100).*

$$t = \frac{2.6DHG}{SE} + C.A.$$

For A-36 material:

API 650: S = 21,000 psi

AWWA D100: S = 15,000 psi

- *Conical bottom (Wozniak).*
  At spring line,

$$T_1 = \frac{wR}{2\sin\alpha}\left(H + \frac{R\tan\alpha}{3}\right)$$

$$T_2 = \frac{wRH}{\sin\alpha}$$

At any elevation below spring line,

$$T_1 = \frac{w}{2\sin\alpha}\left(R - \frac{h_c}{\tan\alpha}\right)\left(H + \frac{2h_c}{3} + \frac{R\tan\alpha}{3}\right)$$

$$T_2 = \frac{wh_i}{\sin\alpha}\left(R - \frac{h_c}{\tan\alpha}\right)$$

$$t_c = \frac{(T_1 \text{ or } T_2)}{12SE\sin\alpha} + C.A.$$

- *Spherical bottom (Wozniak).*
  At spring line,

$$T_1 = wR_3\left[\frac{H}{2} + \frac{R_3}{3}\right]$$

$$T_2 = wR_3\left[\frac{H}{2} - \frac{R_3}{3}\right]$$

At bottom (max. stress),

$$T_1 = T_2 = \frac{wh_iR_3}{2}$$

$$t_s = \frac{(T_1 \text{ or } T_2)}{12SE} + C.A.$$

- *Ring compression at junction (Wozniak).*

$$Q = \frac{R^2w}{2\tan\alpha}\left(H + \frac{R\tan\alpha}{3}\right)$$

Shallow, Granular- or Powder-Filled Bin

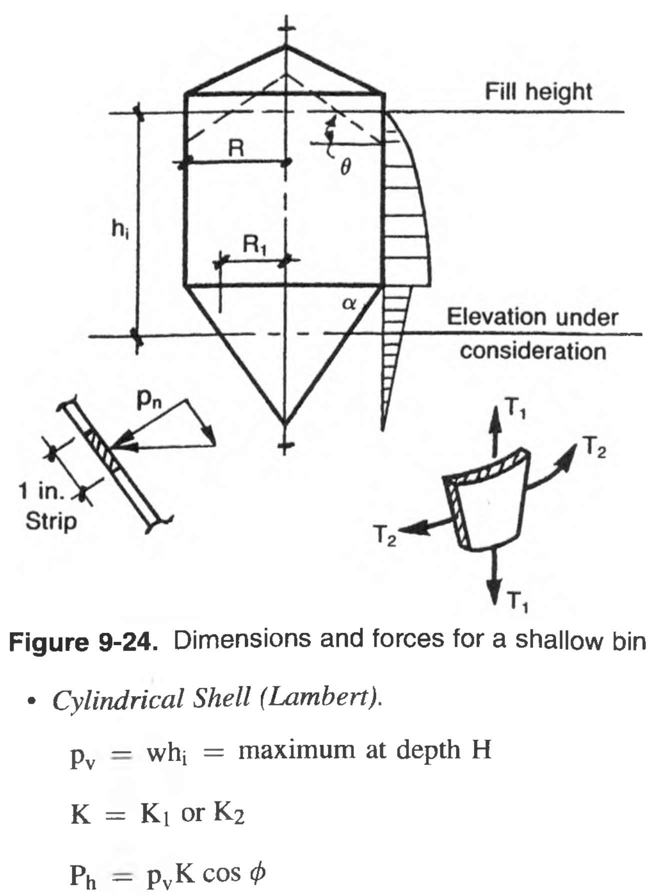


Figure 9-24. Dimensions and forces for a shallow bin.

- *Cylindrical Shell (Lambert).*

$p_v = wh_i =$ maximum at depth H

$K = K_1$ or K_2

$P_h = p_vK\cos\phi$

$T_1 =$ compression only—from weight of shell, roof, and wind loads

Hoop tension, T_2, will govern design of shell for shallow bins

$$T_2 = p_h R$$

$$t = \frac{T_2}{12SE} + C.A.$$

- *Conical bottom (Ketchum).*

$$p_v = wh_i$$

Maximum at depth H =

$$p_n = \frac{p_v \sin^2(\alpha + \theta)}{\sin^3\alpha \left[1 + \frac{\sin\theta}{\sin\alpha}\right]^2}$$

$$W_1 = W + W_c$$

$$T_1 = \frac{W_1}{2\pi R_1 \sin\alpha}$$

$$T_2 = \frac{p_h R_1}{\sin\alpha}$$

$$t_c = \frac{T_1 \text{ or } T_2}{12SE} + C.A.$$

- *Spherical bottom (Ketchum).*

$$T_1 = T_2 = \frac{W_1}{2\pi R_3 \sin^2\alpha'}$$

Note: At $\alpha' = 90°$, $\sin^2\alpha' = 1$

$$t_s = \frac{T_1}{12SE} + C.A.$$

- *Ring compression (Wozniak).*

$$Q = T_1 R \cos\alpha$$

Deep Bins (Silo)—Granular/Powder Filled

- *Shell (Lambert).*

Hydraulic radius

$$R_h = \frac{R}{2}$$

- *Pressures on bin walls, p_v and p_h.*

$$K = K_1 \text{ or } K_2$$

$$e^{\left(\frac{-K\mu' h_i}{R_h}\right)}$$

e = common log 2.7183

$$p_v = \frac{wR_h}{\mu' K}\left[1 - e^{\left(\frac{-K\mu' h_i}{R_h}\right)}\right]$$

$$ph = p_v K$$

- *Weights.*

$$W_2 = \pi R^2 H w$$

$$W_3 = p_v \pi R^2$$

$$W_4 = W_2 - W_3$$

$$W_5 = W_4 + W_R + W_s$$

$$W_R =$$

$$W_s =$$

- *Forces.*

$$T_1 = \frac{-W_5}{\pi D} - \frac{48M}{\pi D}$$

$$T_2 = p_h R$$

Note: For thin, circular steel bins, longitudinal compression will govern. The shell will fail by buckling from vertical drag rather than bursting due to hoop tension.

- *Maximum allowable compressive stress (Boardman formula).*

$$F_a = 2 \times 10^6 \left(\frac{t}{R}\right)\left(1 - \frac{100t}{3R}\right)$$

$$F_a = 10{,}000 \text{ psi maximum}$$

- *Thickness required shell, t.*

$$t = \frac{T}{12F_a}$$

- *Conical bottom (Ketchum).*

Note: Design bottoms to support full load of contents. Vibration will cause lack of side-wall friction.

At spring line,

$$p_v = wH$$

$$p_n = \frac{p_v \sin^2(\alpha + \theta)}{\sin^3\alpha\left[1 + \frac{\sin\theta}{\sin\alpha}\right]^2}$$

$$W_1 = W + W_c$$

$$T_1 = \frac{W_1}{2\pi R \sin\alpha}$$

$$T_2 = \frac{p_n R}{\sin\alpha}$$

$$t = \frac{(T_1 \text{ or } T_2)}{12SE} + C.A.$$

- *Spherical bottom (Ketchum).*

At spring line,

$$T_1 = T_2 = \frac{W_1}{2\pi R_3}$$

$$t = \frac{T_1}{12SE} + C.A.$$

- *Ring compression (Wozniak).*

$$Q = T_1 R \cos\alpha$$

Bins and Tanks with Small Internal Pressures

- *Pressures.*

P_1 = pressure due to gas pressure

P_2 = pressure due to static head of liquid

$$P_2 = \frac{wH}{144}$$

P_3 = pressure due to solid material

$$P_3 = \frac{wHK\cos\phi}{144}$$

P = total pressure

$$P = P_1 + P_2$$

or

$$P_1 + P_3 =$$

- *Shell (API 620).*

$$F = W_T$$

$$W_6 = W_T - W_c - W_{c1}$$

$$A = \pi R^2$$

$$T_{1s} = \frac{R}{2}\left(P + \frac{-W_6 + F}{A}\right)$$

$$T_{2s} = PR$$

$$t = \frac{(T_{1s} \text{ or } T_{2s})}{SE} + C.A.$$

- *Conical bottom (API 620).*

$$T_1 = \frac{R}{2\cos\alpha}\left(P + \frac{-W_6 + F}{A}\right)$$

$$T_2 = \frac{PR}{\sin\alpha}$$

$$t_c = \frac{(T_1 \text{ or } T_2)}{SE} + C.A.$$

- *Ring compression at spring line, Q (API 620).*

$$W_h = 0.6\sqrt{R_2(t_c - C.A.)}$$

$$W_c = 0.6\sqrt{R(t - C.A.)}$$

$$Q = T_2 W_h + T_{2s} W_c - T_1 R_2 \cos\alpha$$

Design of Compression Ring

Per API 620 the horizontal projection of the compression ring juncture shall have a width in a radial direction not less than 0.015 R. The compression ring may be used as a balcony girder (walkway) providing it is at least 3 ft-0 in. wide.

$$R_2 = \frac{R}{\sin\alpha}$$

$$W_h = 0.6\sqrt{R_2(t_c - C.A.)}$$

$$W_c = 0.6\sqrt{R(t - C.A.)}$$

Q = from applicable case =

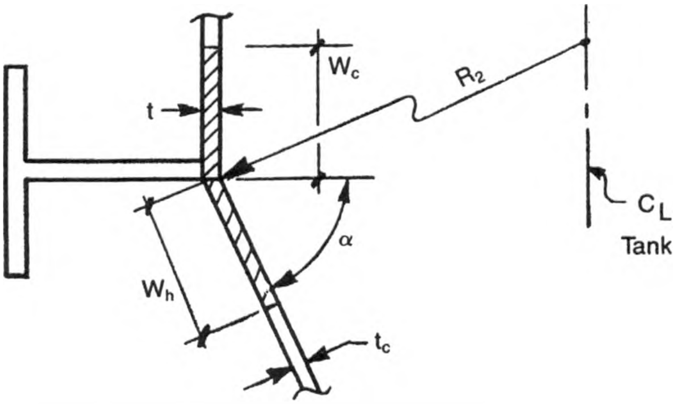


Figure 9-25. Dimensions at junction of cone and cylinder.

$$A_r = \frac{Q}{S}$$

$$A_a = W_c t + W_h t_c$$

- *Additional area required.*

$$A_r - A_a =$$

Struts

Struts are utilized to offset unfavorable high local stresses in the shell immediately above lugs when either lugs or rings are used to support the bin. These high localized stresses may cause local buckling or deformation if struts are not used.

- *Height of struts required, q.*

$$q = \frac{\pi R}{N}$$

- *Strut cross-sectional area required, A_s.*

$$A_s = \frac{fC_s}{S}$$

$$\text{where } f = \frac{W_7 R_f + 2M}{NR_f}$$

W_7 = weight of bin below point of supports plus total weight of contents, lb

The total cross-sectional area of single or double struts may be computed by this procedure. To determine C_s assume a value of A_s and a corresponding value of C_s from Figure 9-27. Substitute this value of C_s into the area equation and compute the area required. Repeat this procedure until the proposed A_s and calculated A_s are in agreement.

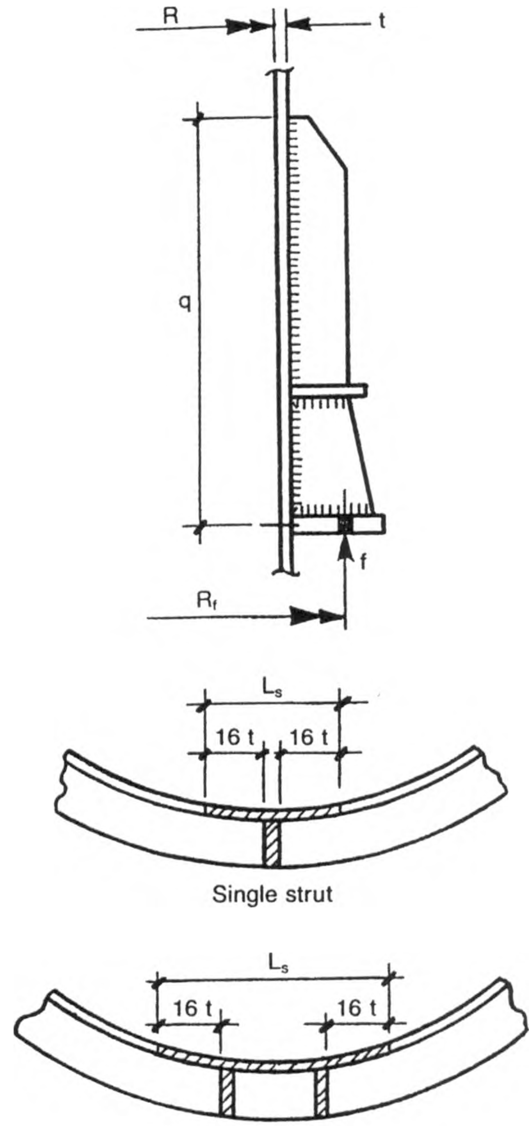


Figure 9-26. Dimensions and arrangement of single and double struts.

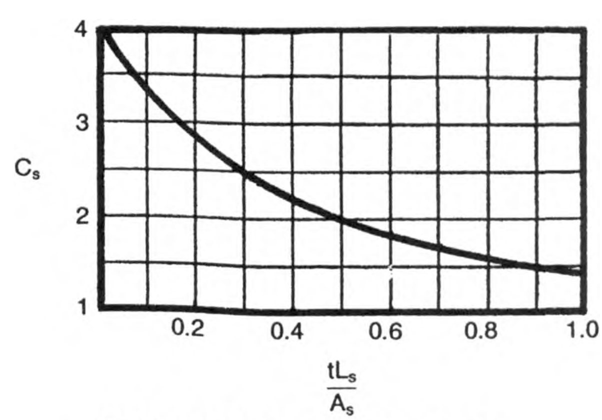


Figure 9-27. Graph of function C_s.

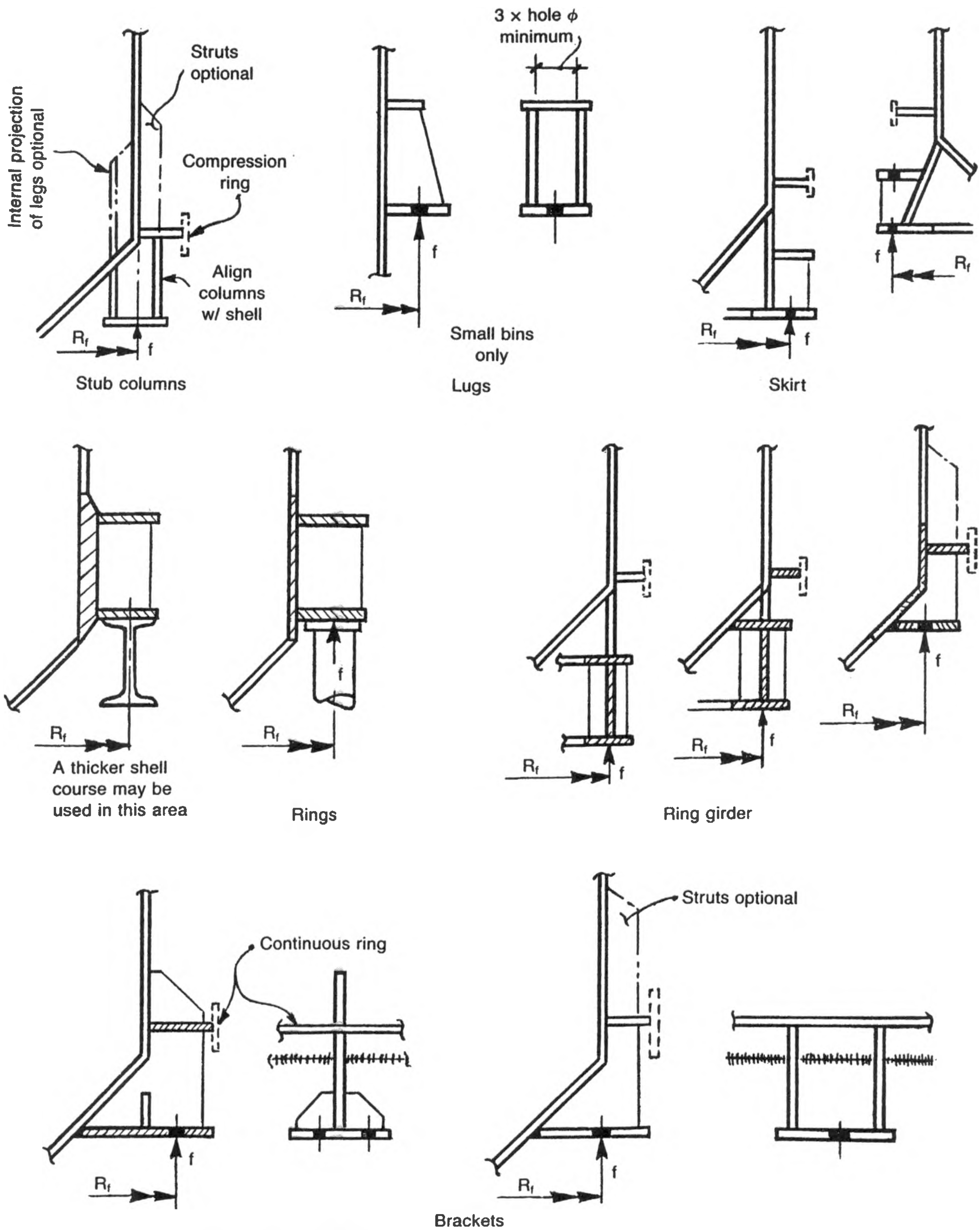


Figure 9-28. Typical support arrangements for bins and elevated tanks.

Supports

Bins may be supported in a variety of ways. Since the bottom cone-cylinder intersection normally requires a compression ring, it is common practice to combine the supports with this ring. This will take advantage of the local stiffness and is convenient for the support design.

Table 9-10
Material properties

			Coefficients of Friction				
				μ' – Contents on Wall			
				On Steel		On Concrete	
Material	Density w	Angle of Repose θ	Contents on Contents μ	μ'	ϕ	μ'	ϕ
Portland cement	90	39°	0.32	0.93		0.54	
Coal (bituminous)	45–55	35°	0.70	0.59	25	0.70	35
Coal (anthracite)	52	27°	0.51	0.45	22	0.51	27
Coke (dry)	28	30°	0.58	0.55	20	0.84	20
Sand	90–110	30°–35°	0.67	0.60	20	0.58	30
Wheat	50–53	25°–28°	0.47	0.41		0.44	
Ash	45	40°	0.84	0.70	25	0.70	35
Clay—dry, fine	100–120	35°	0.70	0.70			
Stone, crushed	100–110	32°–39°	0.70	0.60			
Bauxite ore	85	35°	0.70	0.70			
Corn	44	27.5°	0.52	0.37		0.42	
Peas	50	25°	0.47	0.37		0.44	

If μ' is unknown it may be estimated as follows:
- Mean particle diameter <0.002 in., $\tan^{-1}\mu' = \theta$.
- Mean particle diameter >0.008 in., $\tan^{-1}\mu' = 0.75\,\theta$.

Table 9-11
Rankine factors K_1 and K_2

		Values of K_2 for angles ϕ							
θ	K1	10°	15°	20°	25°	30°	35°	40°	45°
10°	0.7041	1.0000							
12°	0.6558	0.7919							
15°	0.5888	0.6738	1.0000						
17°	0.5475	0.6144	0.7532						
20°	0.4903	0.5394	0.6241	1.0000					
22°	0.4549	0.4958	0.5620	0.7203					
25°	0.4059	0.4376	0.4860	0.5820	1.0000				
27°	0.3755	0.4195	0.4428	0.5178	0.6906				
30°	0.3333	0.3549	0.3743	0.4408	0.5446	1.0000			
35°	0.2709	0.2861	0.3073	0.3423	0.4007	0.5099	1.0000		
40°	0.2174	0.2282	0.2429	0.2665	0.3034	0.3638	0.4549	1.0000	
45°	0.1716	0.1792	0.1896	0.2058	0.2304	0.2679	0.3291	0.4444	1.0000

K_1, no surcharge *K_2, with surcharge*

$$K_1 = \frac{p_h}{p_v} = \frac{1 - \sin\theta}{1 + \sin\theta} = \qquad K_2 = \frac{\cos\phi - \sqrt{\cos^2\phi - \cos^2\theta}}{\cos\phi + \sqrt{\cos^2\phi - \cos^2\theta}} =$$

Notes

1. Rankine factors K_1 and K_2 are ratios of horizontal to vertical pressures. These factors take into account the distribution of forces based on the filling and emptying properties of the material. If the filling angle is different from the angle of repose, then K_2 is used. Remember, even if the material is not heaped to begin with, a crater will develop when emptying. The heaping, filling peak, and emptying crater all affect the distribution of forces.
2. Supports for bins should be designed by an appropriate design procedure. See Chapter 4.
3. In order to assist in the flow of material, the cone angle should be as steep as possible. An angle of 45° can be considered as minimum, 50°–60° preferred.
4. While roofs are not addressed in this procedure, their design loads must be considered since they are translated to the shell and supports. As a minimum, allow 25 psf dead load and 50–75 psf live load plus the weight of any installed plant equipment (mixers, conveyors, etc.).
5. Purging, fluidizing techniques, and general vibration can cause loss of friction between the bin wall and the contents. Therefore its effect must be considered or ignored in accordance with the worst situation: in general, added to longitudinal loads and ignored for circumferential loads.
6. Surcharge: Most bunkers will be surcharged as a result of the normal filling process. If the surcharge is taken into account, the horizontal pressures will be overestimated for average bins. It is therefore more economical to assume the material to be flat and level at the mean height of the surcharge and to design accordingly. Where the bin is very wide in relation to the depth of contents the effects of surcharging need to be considered.

Procedure 9-7: Field-Fabricated Spheres

A sphere is the most efficient pressure vessel because it offers the maximum volume for the least surface area and the required thickness of a sphere is one-half the thickness of a cylinder of the same diameter. The stresses in a sphere are equal in each of the major axes, ignoring the effects of supports. In terms of weight, the proportions are similar. When compared with a cylindrical vessel, for a given volume, a sphere would weigh approximately only half as much. However, spheres are more expensive to fabricate, so they aren't used extensively until larger sizes. In the larger sizes, the higher costs of fabrication are balanced out by larger volumes.

Spheres are typically utilized as "storage" vessels rather than "process" vessels. Spheres are economical for the storage of volatile liquids and gases under pressure, the design pressure being based on some marginal allowance above the vapor pressure of the contents. Spheres are also used for cryogenic applications for the storage of liquified gases.

Products Stored

- Volatile liquids and gases: propane, butane, and natural gas.
- Cryogenic: oxygen, nitrogen, hydrogen, ethylene, helium, and argon.

Codes of Construction

Spheres are built according to ASME, Section VIII, Division 1 or 2, API 620 or BS 5500. In the United States, ASME, Section VIII, Division 1 is the most commonly used code of construction. Internationally spheres are often designed to a higher stress basis upon agreement between the user and the jurisdictional authorities. Spheres below 15 psig design pressure are designed and built to API 620.

The allowable stresses for the design of the supports is based on either AWWA D100 or AISC.

Materials of Construction (MOC)

Typical materials are carbon steel, usually SA-516-70. High-strength steels are commonly used as well (SA-537, Class 1 and 2, and SA-738, Grade B). SA-516-60 may be used to eliminate the need for PWHT in wet H_2S service. For cryogenic applications, the full range of materials has been utilized, from the low-nickel steels, stainless steels, and higher alloys. Spheres of aluminum have also been fabricated.

Liquified gases such as ethylene, oxygen, nitrogen, and hydrogen are typically stored in double-wall spheres, where the inner tank is suspended from the outer tank by straps or cables and the annular space between the tanks is filled with insulation. The outer tank is not subjected to the freezing temperatures and is thus designed as a standard carbon steel sphere.

Size, Thickness, and Capacity Range

Standard sizes range from 1000 barrels to 50,000 barrels in capacity. This relates in size from about 20 feet to 82 feet in diameter. Larger spheres have been built but are considered special designs. In general, thicknesses are limited to 1.5 in. to preclude the requirement for PWHT, however PWHT can be accomplished, even on very large spheres.

Supports

Above approximately 20 feet in diameter, spheres are generally supported on legs or columns evenly spaced around the circumference. The legs are attached at or near the equator. The plates in this zone of leg attachment may be required to be thicker, to compensate for the additional loads imposed on the shell by the supports. An internal stiffening ring or ring girder is often used at the junction of the centerline of columns and the shell to take up the loads imposed by the legs.

The quantity of legs will vary. For gas-filled spheres, assume one leg every third plate, assuming 10-feet-wide plates. For liquid-filled spheres, assume one leg every other plate.

Legs can be either cross-braced or sway-braced. Of the two bracing methods, sway-bracing is the more common. Sway-bracing is for tension-only members. Cross-bracing is used for tension and compression members. When used, cross-bracing is usually pinned at the center to reduce the sizes of the members in compression.

Smaller spheres, less than 20 feet in diameter, can be supported on a skirt. The diameter of the supporting skirt should be 0.7 × the sphere diameter.

Heat Treatment

Carbon steel spheres above 1.5-in. thickness must be PWHT per ASME Code. Other alloys should be checked for thickness requirements. Spheres are often stress relieved for process reasons. Spheres made of high-strength carbon steel in wet H_2S service should be stress relieved regardless of thickness. When PWHT is required, the following precautions should be taken:

a. Loosen cross-bracing to allow for expansion.
b. Jack out columns to keep them level during heating and cooling.
c. Scaffold the entire vessel.
d. Weld thermocouple wires to shell external surface to monitor and record temperature.
e. Typically, internally fire it.
f. Monitor heat/cooling rate and differential temperature.

Accessories

Accessories should include a spiral stairway and a top platform to access instruments, relief valves, and vents. Manways should be used on both the top and bottom of the sphere. Nozzles should be kept as close as practical to the center of the sphere to minimize platforming requirements.

Methods of Fabrication

Field-fabricated spheres are made in one of two methods. Smaller spheres can be made by the expanded cube, soccer ball method, while larger ones are made by the orange peel method. The orange peel method consists of petals and cap plates top and bottom.

Typically all shell pieces are pressed and trimmed in the shop and assembled to the maximum shipping sizes allowable. Often, the top portion of the posts are fit up and welded in the shop to their respective petals.

Field Hydrotests

Typically the bracing on the support columns is not tightened fully until the hydrotest. While the sphere is full of water and the legs are at their maximum compression, the bracing is tightened so that once the sphere is emptied, all of the bracing goes into tension and there is the assurance that they remain in tension during service.

Settlement between the legs must be monitored during hydrotest to detect any uneven settlement between the posts. Any uneven settlement of over ½ in. between any pair of adjacent legs can cause distortion and damage to the sphere. Foundation requirements should take this requirement into consideration.

Notes

1. Spheres that operate either hot or cold will expand or contract differentially with respect to the support

columns or posts. The moment and shear forces resulting from this differential expansion must be accounted for in the design of the legs.

2. The minimum clearance between the bottom of the vessel and grade is 2ft 6in.
3. The weights shown in the tables include the weight of the sphere with an allowance for thinning (1/16 in.) and corrosion (1/8 in.) plus plate overtolerance. A clearance of 3 ft was assumed between the bottom of the sphere and the bottom of the base plate. The weights include columns, base plates, and bracing, plus a spiral stairway and top platform. Column weights were estimated from the quantities and sizes listed in the table.
4. For estimating purposes, the following percentages of the sphere shell weight should be added for the various categories:
   - Columns and base plates: 6–14%. For thicker, heavier spheres, the lower percentage should be used. For larger, thinner spheres, the higher percentage should be used.
   - Sway rods/bracing: 1–9%. Use the lower value for wind only and higher values where seismic governs. The highest value should be used for the highest seismic area.
   - Stairway, platform, and nozzles: 2–5%. Apply the lower value for minimal requirements and the higher where the requirements are more stringent.

Notation

A = surface area, sq ft
d = OD of column legs, in.
D = diameter, ft
D_m = mean vessel diameter, ft
E = joint efficiency
E_m = modulus of elasticity, psi
N = number of support columns
n = number of equal volumes
P = internal pressure, psig
P_a = maximum allowable external pressure, psi
P_m = MAWP, psig
R = radius, ft
R_c = radius, corroded, in
S = allowable stress, psi
t = thickness, new, in.
t_c = thickness, corroded, in.
t_p = thickness of pipe leg, in.
t_{rv} = thickness required for full vacuum, in.
V = volume, cu ft
W = weight, lb
w = unit weight of plate, psf

Conversion Factors

7.481 gallons/cu ft
0.1781 barrels/cu ft
5.614 cu ft/barrel
35.31 cu ft/cu meter
6.29 barrels/cu meter
42 gallons/barrel

Formulas

$$V = \frac{\pi D^3}{6} \quad \text{or} \quad V = \frac{4\pi R^3}{3}$$

$$V_n = \frac{\pi D^3}{6n} \quad \text{or} \quad V_n = \frac{4\pi R^3}{3n}$$

$$V_1 = \frac{\pi h_1^2}{3}(3R - h_1)$$

$$V_2 = \frac{\pi h_1}{6}\left(3r_1^2 + 3r_2^2 + h_2^2\right)$$

$$D = \sqrt[3]{\frac{6V}{\pi}}$$

$$A = \pi D^2 \quad \text{or} \quad A = 4\pi R^2$$

$$A_n = \pi D h_n \quad \text{or} \quad A_n = 2\pi R h_n$$

$$r_1 = \sqrt{2Rh_1 - h_1^2}$$

$$r_2 = \sqrt{R^2 - h_3^2}$$

$$\sin\alpha = \frac{r_1}{R} \quad \alpha$$

$$W = \pi D_m^2 w$$

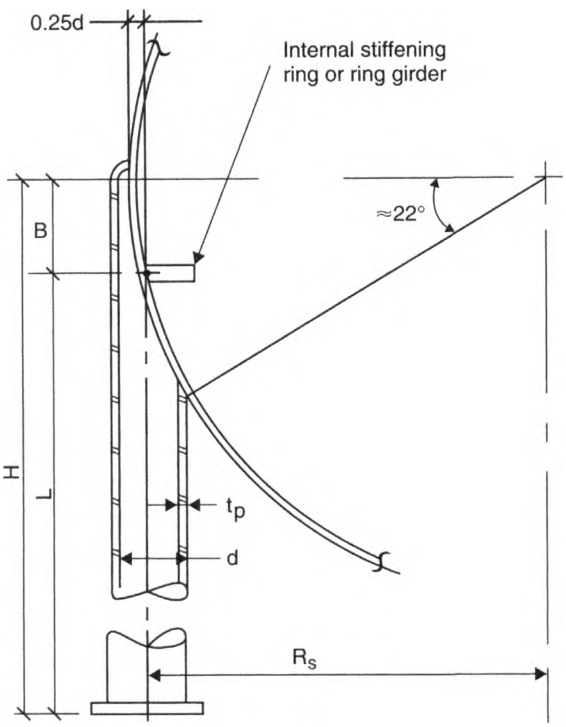
$$P_m = \frac{2SEt_c}{R_i + 0.2t_c}$$

$$P_a = \frac{0.0625E_m}{\left(\frac{R_o}{t_c}\right)^2}$$

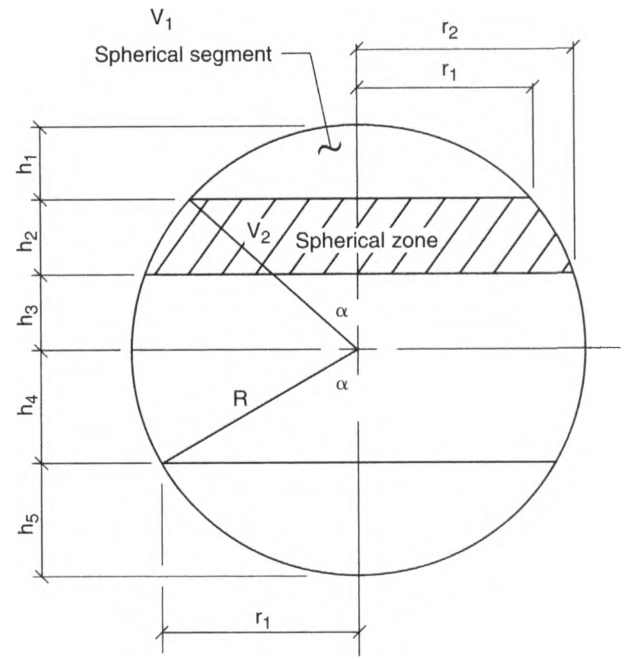
$$t_r = \frac{PR_c}{2SE - 0.2P} \quad \text{(Division 1)}$$

$$t_r = R_c\left(e^{0.5P/SE} - 1\right) \quad \text{(Division 2)}$$

Typical Leg Attachment



Dimensional Data



Liquid Level in a Sphere

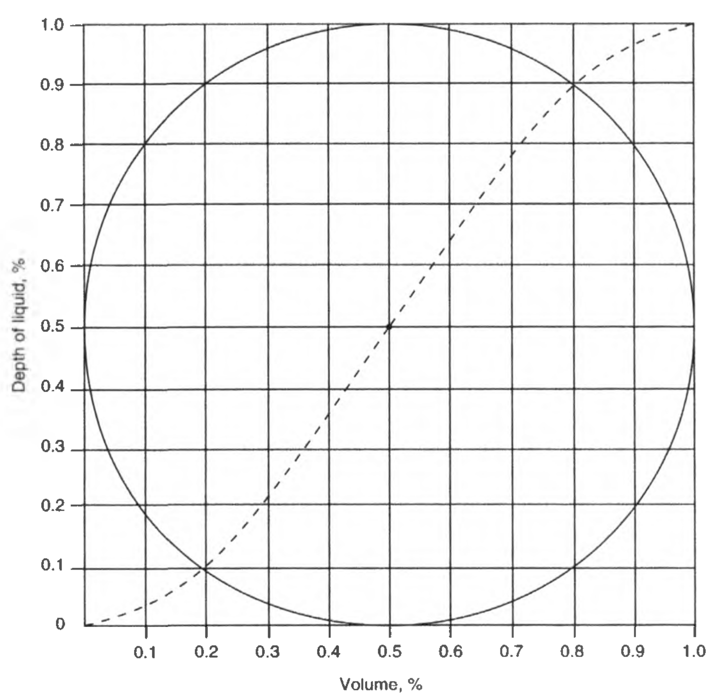


Table 9-12
Dimensions for "n" quantity of equal volumes

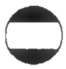
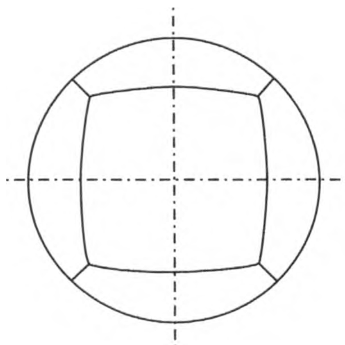
Figure	n	V_n	r_1	r_2	h_1	h_2	h_3
	3	$\frac{\pi D^3}{18}$	0.487D	—	0.387D	0.226D	—
	4	$\frac{\pi D^3}{24}$	0.469D	—	0.326D	0.174D	—
	5	$\frac{\pi D^3}{30}$	0.453D	0.496D	0.287D	0.146D	0.067D
	6	$\frac{\pi D^3}{36}$	0.436D	0.487D	0.254D	0.133D	0.113D

Table 9-13
Volumes and surface areas for various depths of liquid

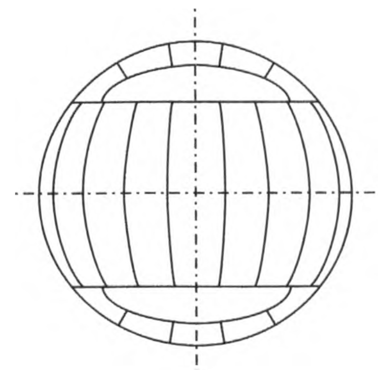
h_4	h_5	α	r_1	V_5	V_4	A_5	A_4
0.05D	0.45D	25.84	0.218D	$0.0038D^3$	$0.2580D^3$	$0.1571D^2$	$1.4137D^2$
0.10D	0.40D	36.87	0.300D	$0.0147D^3$	$0.2471D^3$	$0.3142D^2$	$1.2567D^2$
0.15D	0.35D	45.57	0.357D	$0.0318D^3$	$0.2300D^3$	$0.4712D^2$	$1.1000D^2$
0.20D	0.30D	53.13	0.400D	$0.0545D^3$	$0.2073D^3$	$0.6283D^2$	$0.9425D^2$
0.25D	0.25D	60.0	0.433D	$0.0818D^3$	$0.1800D^3$	$0.7854D^2$	$0.7854D^2$
0.30D	0.20D	66.42	0.458D	$0.1131D^3$	$0.1487D^3$	$0.9425D^2$	$0.6283D^2$
0.35D	0.15D	72.54	0.477D	$0.1475D^3$	$0.1143D^3$	$1.1000D^2$	$0.4712D^2$
0.40D	0.10D	78.46	0.490D	$0.1843D^3$	$0.0775D^3$	$1.2567D^2$	$0.3141D^2$
0.45D	0.05D	84.26	0.498D	$0.2227D^3$	$0.0391D^3$	$1.4137D^2$	$0.1571D^2$
0.50D	0D	90.0	0.500D	$0.2618D^3$	$0D^3$	$1.5708D^2$	$0D^2$

Types of Spheres



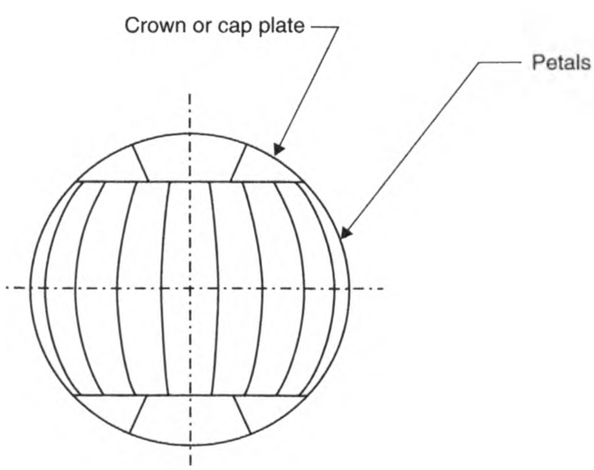
Expanded Cube, Square Segment, or Soccer Ball Type

- Small spheres only
- Sizes less than about 20 feet in diameter
- Volumes less than 750 bbls



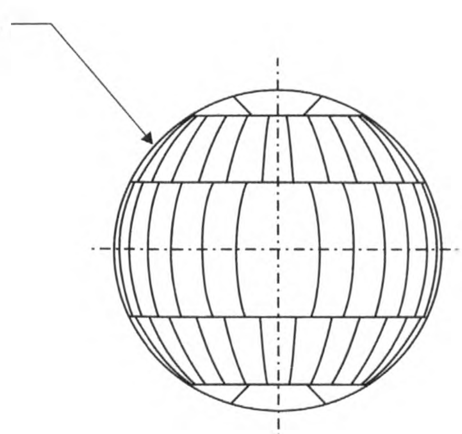
Partial Soccer Ball Type

- Combines orange peel and soccer ball types
- Sizes 30 to 62 feet in diameter
- Volumes 2200 to 22,000 bbls



Meridian, Orange Peel, or Watermelon Type (3-Course Version)

- Consists of crown plates and petal plates
- Sizes 20 to 32 feet in diameter
- Volumes 750 to 3000 bbls



Meridian, Orange Peel, or Watermelon Type (5-Course Version)

- Consists of crown plates and petal plates
- Sizes up to 62 feet in diameter
- Volumes to 22,000 bbls

Table 9-14
Data for 50-psig sphere

D	t	Volume bbl—nom	bbls	ft^3	A	W	N	d	t_p	P_q	t_{rv}
20 ft-0 in.	0.3125	750	746	4188	1256	23.5	4	16	0.25	4.4	0.5
22 ft-3 in.	0.375	1000	1027	5767	1555	32.8	4	16	0.25	6.32	0.5625
25 ft-0 in.	0.375	1500	1457	8181	1963	41	4	16	0.25	5.01	0.5625
25 ft-6 in.	0.375	1500	1546	8682	2043	42.7	4	16	0.25	4.82	0.5625
28 ft-0 in.	0.375	2000	2047	11,494	2463	52.2	5	16	0.25	4	0.625
30 ft-3 in.	0.4375	2500	2581	14,494	2875	68.8	5	16	0.25	5.35	0.6875
32 ft-0 in.	0.4375	3000	3055	17,157	3217	78	6	18	0.25	4.78	0.6875
35 ft-0 in.	0.4375	3000	3998	22,449	3848	93.4	6	18	0.25	4	0.75
35 ft-3 in.	0.4375	4000	4084	22,934	3904	94.7	6	20	0.25	2.52	0.75
38 ft-0 in.	0.5	5000	5116	28,731	4536	123	6	22	0.25	4.88	0.8125
40 ft-0 in.	0.5	6000	5968	33,510	5027	138	6	22	0.25	4.41	0.8125
40 ft-6 in.	0.5	6000	6195	34,783	5153	142.3	7	24	0.25	4.3	0.875
43 ft-6 in.	0.5625	7500	7676	43,099	5945	181	7	24	0.29	5.07	0.875
45 ft-0 in.	0.5625	8500	8497	47,713	6362	193.6	7	24	0.29	4.74	0.9375
48 ft-0 in.	0.5625	10,000	10,313	57,906	7238	222.2	8	28	0.3	4.17	1
50 ft-0 in.	0.625	11,500	11,656	65,450	7854	269.4	8	28	0.3	5.01	1
51 ft-0 in.	0.625	12,500	12,370	69,456	8171	280.2	9	30	0.29	4.82	1
54 ft-9 in.	0.625	15,000	15,304	85,931	9417	326.8	9	32	0.344	4.18	1.0625
55 ft-0 in.	0.625	15,000	15,515	87,114	9503	330.6	9	32	0.344	4.15	1.125
60 ft-0 in.	0.6875	20,000	20,142	113,097	11,310	430.5	9	32	0.344	4.41	1.1875
60 ft-6 in.	0.6875	20,000	20,650	115,948	11,500	438.2	10	34	0.38	4.34	1.1875
62 ft-0 in.	0.6875	22,000	22,225	124,788	12,076	458.8	10	34	0.38	4.13	1.25
65 ft-0 in.	0.75	25,000	25,610	143,793	13,273	551.5	11	36	0.406	4.64	1.25
69 ft-0 in.	0.75	30,000	30,634	172,007	14,957	629.2	11	40	0.438	4.12	1.375
76 ft-0 in.	0.8125	40,000	40,936	229,847	18,146	874.1	12	42	0.503	4.11	1.5
81 ft-10 in.	0.875	50,000	51,104	286,939	21,038	1105	13	42	0.594	3.54	1.625
87 ft-0 in.	0.9375	60,000	61,407	344,791	23,779	1460	14	48	0.75	4.38	1.75

Note: Values are based on the following:

1. Material SA-516-70, S = 20,000 psi.
2. Joint efficiency, E = 0.85.
3. Corrosion allowance, c.a. = 0.125.

Table 9-15
Weights of spheres, kips

Dia. (ft)	Thickness (in.)											
	0.375	0.4375	0.5	0.5625	0.625	0.6875	0.75	0.8125	0.875	0.9375	1	1.125
20 ft-0 in.	26.8	30	[33.3]	36.5	39.8	43	46.3	49.5	52.7	55.9		
22 ft-6 in.	32.8	36.8	40.9	[45]	49	53.1	57.2	61.2	65.3	69.3		
25 ft-0 in.	41	46	51	[56]	61	66.1	71.1	76.1	81.1	86		
27 ft-6 in.	***48***	54.1	60.1	66.2	[72.3]	78.3	84.4	90.4	96.5	103		
30 ft-0 in.	60	66	73.2	80.4	87.6	[94.8]	102	109	117	124	131	
32 ft-6 in.	71.5	80	88.5	97	105	[114]	122	131	139	148	156	
35 ft-0 in.	81.1	***93.4***	103	113	123	133	[143]	152	162	172	182	202
37 ft-6 in.	98.3	110	121	132	143	155	166	[177]	189	200	211	234
40 ft-0 in.	105	122	***138***	151	164	177	189	[202]	215	228	241	266
42 ft-6 in.	129	143	158	172	187	201	216	230	[245]	259	274	303
45 ft-0 in.	145	161	177	194	210	226	242	259	275	[291]	307	340
47 ft-6 in.	161	179	197	***215***	233	251	269	287	305	324	[342]	378
50 ft-0 in.		209	229	249	269	289	309	330	350	370	[390]	430
52 ft-6 in.		234	256	278	300	322	344	366	388	411	433	[477]
55 ft-0 in.			282	306	***331***	355	379	403	428	452	476	[525]
57 ft-6 in.			313	340	366	393	419	446	472	499	525	578
60 ft-0 in.				373	402	431	459	488	517	546	575	633
62 ft-6 in.				399	431	***462***	493	525	556	587	619	650
65 ft-0 in.					484	518	552	585	619	653	687	755
69 ft-0 in.					553	591	***629***	667	706	744	782	858
76 ft-0 in.						782	828	***874***	920	967	1013	1106
81 ft-10 in.						944	998	1051	***1105***	1159	1212	1320
87 ft-0 in.							1278	1339	1400	***1460***	1521	1642

Notes:

1. Values that are underlined indicate 50-psig internal pressure design.
2. Values in brackets [ ] indicate full vacuum design.

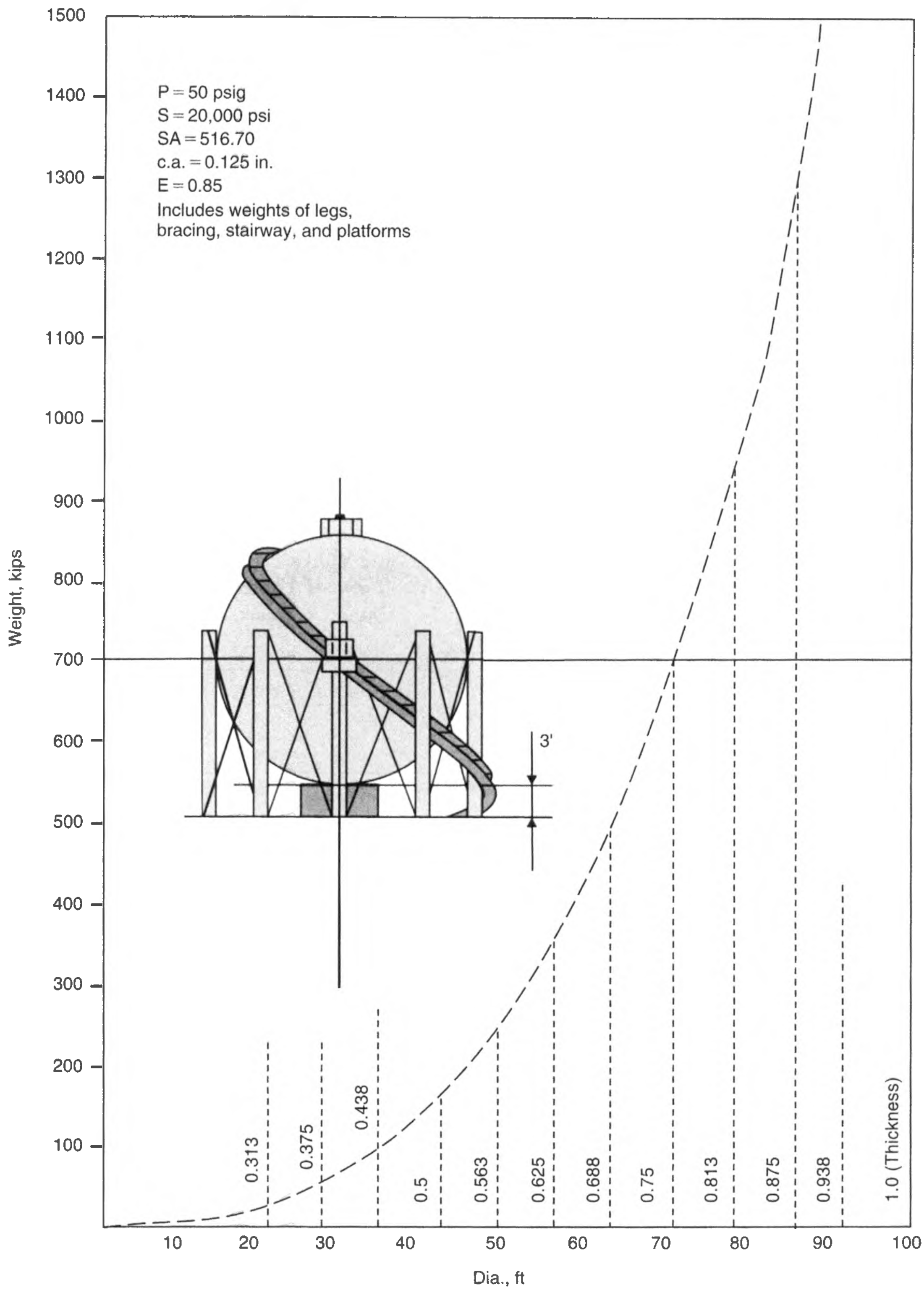
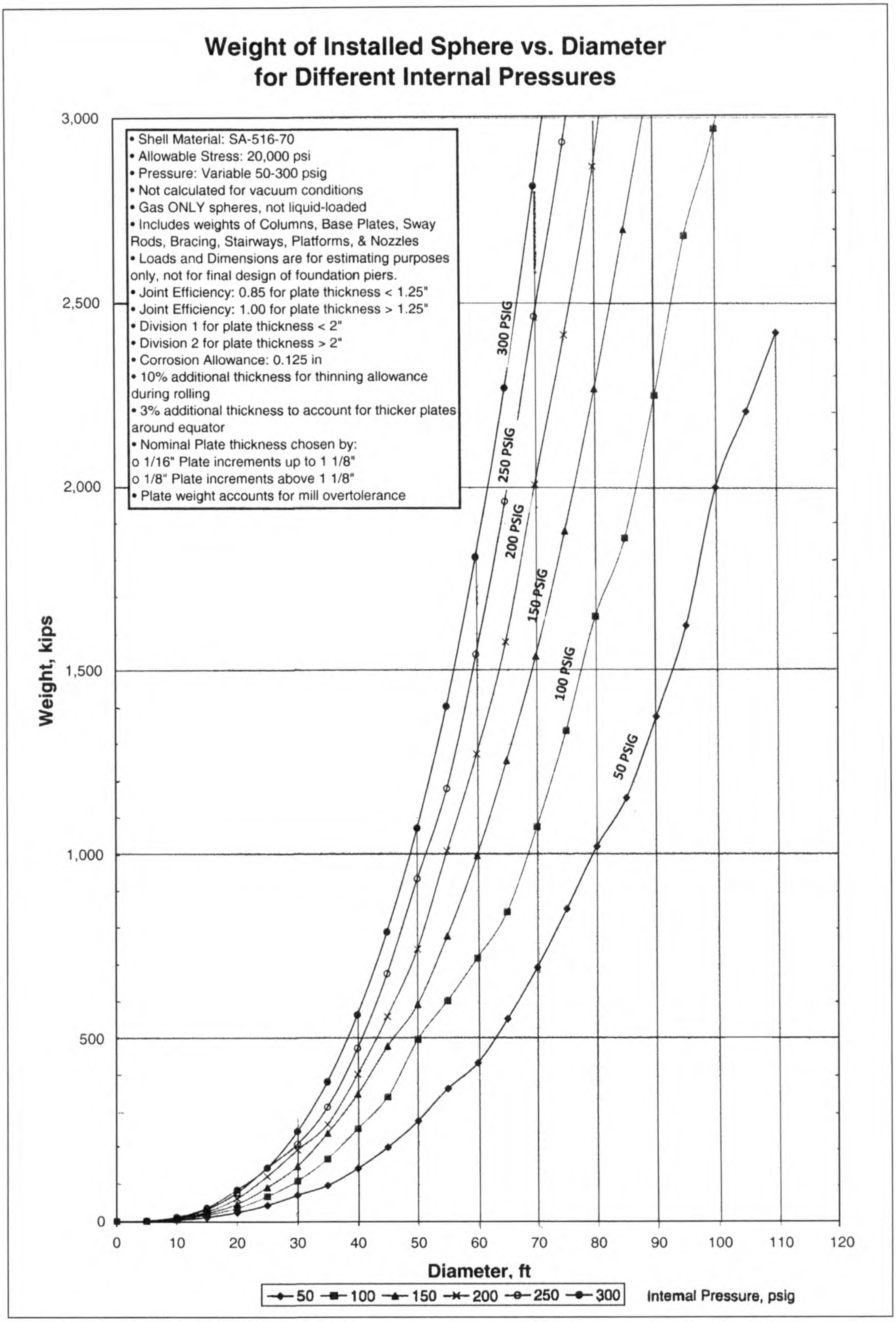


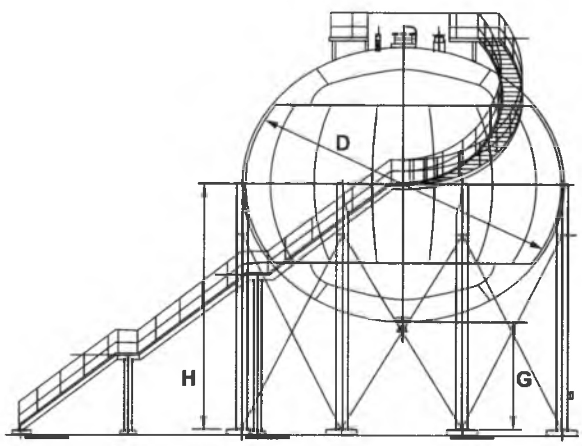
Figure 9-29. Weight of sphere.


Weight of Installed Sphere vs. Diameter
for Different Internal Pressures
• Shell Material: SA-516-70
• Allowable Stress: 20,000 psi
• Pressure: Variable 50-300 psig
• Not calculated for vacuum conditions
• Gas ONLY spheres, not liquid-loaded
• Includes weights of Columns, Base Plates, Sway Rods, Bracing, Stairways, Platforms, & Nozzles
• Loads and Dimensions are for estimating purposes only, not for final design of foundation piers.
• Joint Efficiency: 0.85 for plate thickness < 1.25"
• Joint Efficiency: 1.00 for plate thickness > 1.25"
• Division 1 for plate thickness < 2"
• Division 2 for plate thickness > 2"
• Corrosion Allowance: 0.125 in
• 10% additional thickness for thinning allowance during rolling
• 3% additional thickness to account for thicker plates around equator
• Nominal Plate thickness chosen by:
o 1/16" Plate increments up to 1 1/8"
o 1/8" Plate increments above 1 1/8"
• Plate weight accounts for mill overtolerance
300 PSIG
250 PSIG
200 PSIG
150 PSIG
100 PSIG
50 PSIG
Weight, kips
0
500
1,000
1,500
2,000
2,500
3,000
0 10 20 30 40 50 60 70 80 90 100 110 120
Diameter, ft
50 100 150 200 250 300
Internal Pressure, psig

SPHERE DATA SHEET

EQUIPMENT ITEM NO. :

EQUIPMENT NAME :



DIMENSIONS	
D	
H	
G	

WEIGHTS	
FABRICATED	
EMPTY	
OPERATING	
TEST	
SHELL	
COLUMNS	
BRACING	
CONTENTS	
WATER	
FIREPROOFING	

DESIGN DATA		MATERIALS	
Design Pressure - Internal		Shell	
Design Pressure - External		Columns	
Design Temperature - Internal		Column Bracing	
Design Temperature - External		Flanges	
MDMT		Nozzle Necks- Pipe	
Specific Gravity		Bolting	
Capacity		Gaskets	
Corrosion Allowance			
Joint Efficiency		Base Plates	
PWHT		**CODES**	
Contents		Design	
Insulation/Thk		Seismic	
Fireproofing - Legs		Wind	
Service (Sour, Lethal, Cyclic, etc.)		Structural	

Design of Spheres

Nomenclature

A_b = Area, brace, in^2
A_c = Area, column, in^2
A_{cr} = Area required, column
A_s = Area, shell, in^2
A_g = Area, girder, in^2
A_T = Area, total, in^2
A_{br} = Area, brace, required, in^2
A_{sn} = Surface area of shell section, Ft^2
A_{cn} = Cross sectional area, Ft^2
C_a = Corrosion allowance, in
D_c = Centerline diameter of columns, Ft
d_c = Inside diameter of column, in
E = Joint efficiency
E_m = 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_L = Axial load on column due to live load, ie. wind or seismic, lbs
F_T = Alowable stress, tension, psi
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_g = Moment of inertia, girder, in^4
I_s = Moment of inertia, shell, in^4
I_T = Moment of inertia, combined shell and girder total, in^4
I_c = Moment of inertia, column, in^4
k = End connection coefficient, columns
L' = Theoretical length of shell resisting loads, in
M_o = Overturning moment, Ft-Lbs
M_B = Internal bending moment in girder section between columns due to horizontal force, in-Lbs
M_C = Internal bending moment in girder section between columns due to vertical force, in-Lbs
M_P = Internal bending moment in post plate at column due to vertical force, in-Lbs
M_S = Internal bending moment in post plate between columns due to vertical force, 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, 2 for pinned at center
P = Internal pressure, PSIG
P_x = External pressure, PSIG
P_h = Pressure due to static head of liquid, PSI
P_n = Total pressure at a given elevation, n, PSI
Q = Maximum axial force in column, Lbs
R = Inside radius, corroded, Ft
r = Inside radius, corroded, in
r_c = Radius of gyration, column, in
r_b = Radius of gyration, brace, in
r_o = Outside radius of sphere, in
r_g = Radius of gyration, girder, in
S = Allowable stress, shell, PSI
S_C = Combined stress, compression, PSI
S_g = Specific gravity, contents
S_r = Slenderness ratio
S_T = Combined stress, tension, PSI
T = Period of vibration, Sec's
T' = Greater of T_1 or T_2, Lbs / in
T_1 = Meridional load, Lb/in
T_2 = Latitudinal load, Lb/in
t = Thickness, sphere, in
t_c = Thickness, column, in
t_r = Thickness required, shell, in
V = Base shear, Lbs
V_S = Volume, Ft^3
V_{sn} = Partial volumes of section, Ft^3
V_n = Horizontal force per column, Lbs
W_o = Weight, operating, Lbs
W_w = Weight, water, Lbs
W_p = Weight, product, Lbs
W_s = Weight, steel, Lbs
w = Unit weight of liquid, PCF
Z_g = Section modulus, girder, in^3
Z_s = Section modulus, shell, in^3
Z_T = Section modulus, combined shell and girder, in^3
ΔL = Change in length of brace, in
δ = Lateral deflection of sphere, in

Table 9-16
Summary of loads at support locations

QTY of Columns	LEG No.	Case 1: At Columns		Case 2: Between Columns	
		Horiz (V_n)	Vertical (Q)	Horiz (V_n)	Vertical (Q)
6	1	.0833 V	$F_D + F_L$	.125 V	$F_D + .866\ F_L$
	2	.2083 V	$F_D + .5\ F_L$	.25 V	F_D
	3	.2083 V	$F_D - .5\ F_L$	.125 V	$F_D - .866\ F_L$
	4	.0833 V	$F_D - F_L$	.125 V	$F_D - .866\ F_L$
	5	.2083 V	$F_D - .5F_L$	.25 V	F_D
	6	.2083 V	$F_D + .5\ F_L$	.125 V	$F_D + .866\ F_L$
8	1	.0366 V	$F_D + F_L$	.0625 V	$F_D + .9239\ F_L$
	2	.125 V	$F_D + .707\ F_L$	.1875 V	$F_D + .3827\ F_L$
	3	.2134 V	F_D	.1875 V	$F_D - .3827\ F_L$
	4	.125 V	$F_D - .707\ F_L$	.0625 V	$F_D - .9239\ F_L$
	5	.0366 V	$F_D - F_L$	.0625 V	$F_D - .9239\ F_L$
	6	.125 V	$F_D - .707\ F_L$	.1875 V	$F_D - .3827\ F_L$
	7	.2134 V	F_D	.1875 V	$F_D + .3827\ F_L$
	8	.125 V	$F_D + .707\ F_L$	.0625 V	$F_D + .9239\ F_L$
10	1	.0191 V	$F_D + F_L$	.0346 V	$F_D + .9511\ F_L$
	2	.0750 V	$F_D + .809\ F_L$	.125 V	$F_D + .5878\ F_L$
	3	.1655 V	$F_D + .309\ F_L$	.1809 V	F_D
	4	.1655 V	$F_D - .309\ F_L$	.125 V	$F_D - .5878\ F_L$
	5	.0750 V	$F_D - .809\ F_L$	.0346 V	$F_D - .9511\ F_L$
	6	.0191 V	$F_D - F_L$	.0346 V	$F_D - .9511\ F_L$
	7	.0750 V	$F_D - .809\ F_L$	.125 V	$F_D - .5878\ F_L$
	8	.1655 V	$F_D - .309\ F_L$	.1809 V	F_D
	9	.1655 V	$F_D + .309\ F_L$	.125 V	$F_D + .5878\ F_L$
	10	.0750V	$F_D + .809\ F_L$	.0346 V	$F_D + .9511\ F_L$
12	1	.0112 V	$F_D + F_L$	.0209 V	$F_D + .9659\ F_L$
	2	.0472 V	$F_D + .866\ F_L$	.0834 V	$F_D + .7071\ F_L$
	3	.1194 V	$F_D + .5\ F_L$	.1458 V	$F_D + .2588\ F_L$
	4	.1555 V	F_D	.1458 V	$F_D - .2588\ F_L$
	5	.1194 V	$F_D - .5\ F_L$	.0834 V	$F_D - .7071\ F_L$
	6	.0472 V	$F_D - .866\ F_L$	.0209 V	$F_D - .9659\ F_L$
	7	.0112 V	$F_D - F_L$	.0209 V	$F_D - .9659\ F_L$
	8	.0472 V	$F_D - .866\ F_L$	.0834 V	$F_D - .7071\ F_L$
	9	.1194 V	$F_D - .5\ F_L$	.1458 V	$F_D - .2588\ F_L$
	10	.1555 V	F_D	.1458 V	$F_D - .2588\ F_L$
	11	.1194 V	$F_D + .5\ F_L$	.0834 V	$F_D - .7071\ F_L$
	12	.0472 V	$F_D + .866\ F_L$	.0209 V	$F_D + .9659\ F_L$
16	1	.0048 V	$F_D + F_L$	.0091 V	$F_D + .9808\ F_L$
	2	.0217 V	$F_D + .9239\ F_L$	.0404 V	$F_D + .8315\ F_L$
	3	.0625 V	$F_D + .7071\ F_L$	.0846 V	$F_D + .5556\ F_L$
	4	.1034 V	$F_D + .3827\ F_L$	.1158 V	$F_D + .1951\ F_L$
	5	.1202 V	F_D	.1158 V	$F_D - .1951\ F_L$
	6	.1034 V	$F_D - .3827\ F_L$	.0846 V	$F_D - .5556\ F_L$
	7	.0625 V	$F_D - .7071\ F_L$	.0404 V	$F_D - .8315\ F_L$
	8	.0217 V	$F_D - .9239\ F_L$	.0091 V	$F_D - .9808\ F_L$
	9	.0048 V	$F_D - F_L$	.0091 V	$F_D - .9808\ F_L$
	10	.0217 V	$F_D - .9239\ F_L$	.0404 V	$F_D - .8315\ F_L$
	11	.0625 V	$F_D - .7071\ F_L$	.0846 V	$F_D - .5556\ F_L$
	12	.1034 V	$F_D - .3827\ F_L$	.1158 V	$F_D - .1951\ F_L$
	13	.1202 V	F_D	.1158 V	$F_D + .1951\ F_L$
	14	.1034 V	$F_D + .3827\ F_L$	.0846 V	$F_D + .5556\ F_L$
	15	.0625 V	$F_D + .7071\ F_L$	.0404 V	$F_D + .8315\ F_L$
	16	.0217 V	$F_D + .9239\ F_L$	.0091 V	$F_D + .9808\ F_L$

Table 9-17
Internal bending moments in girder section

No. of Columns	Due to Vertical Force, Q (Note 1)		Due to Horizontal Force, V (Note 2)	
	M_s	M_c	M_P	M_B
4	(-) .1366 QR_C	+ .0705 QR_C	+ .0683 VR_n	(-) .049 VR_n
6	(-) .0889 QR_C	+ .0451 QR_C	+ .0164 VR_n	(-) .013 VR_n
8	(-) .0662 QR_C	+ .0333 QR_C	+ .0061 VR_n	(-) .0058 VR_n
10	(-) .0527 QR_C	+ .0265 QR_C	+ .0030 VR_n	(-) .0029 VR_n
12	(-) .0438 QR_C	+ .0228 QR_C	+ .0016 VR_n	(-) .0016 VR_n
16	(-) .0328 QR_C	+ .0165 QR_C	+ .0007 VR_n	(-) .0007 VR_n

Notes:

1. R_C is in inches in order to get moment in in-Lbs
2. $R_n = R_C$ if no interal girder is used. $R_n = R_1$ if a girder is used.

Table 9-18
Angles

QTY OF COLUMNS	α	ϕ
4	45	90
6	60	60
8	67.5	45
10	72	36
12	75	30
16	77.5	25

Sphere-Dimensions & Data

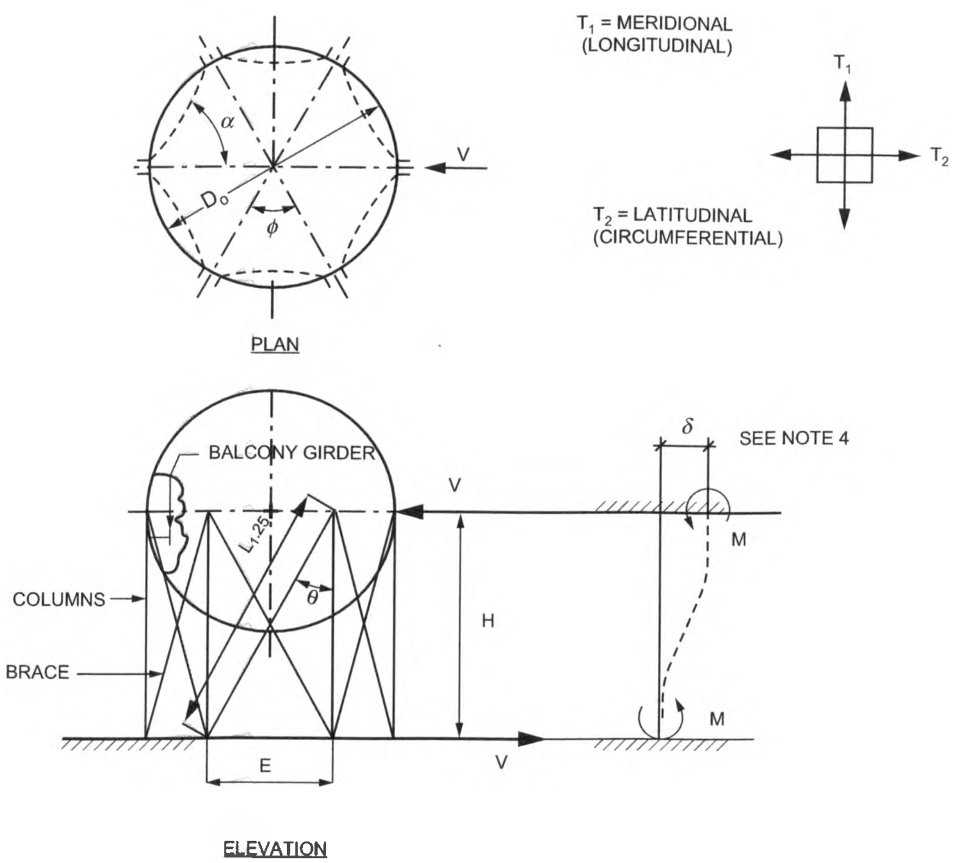


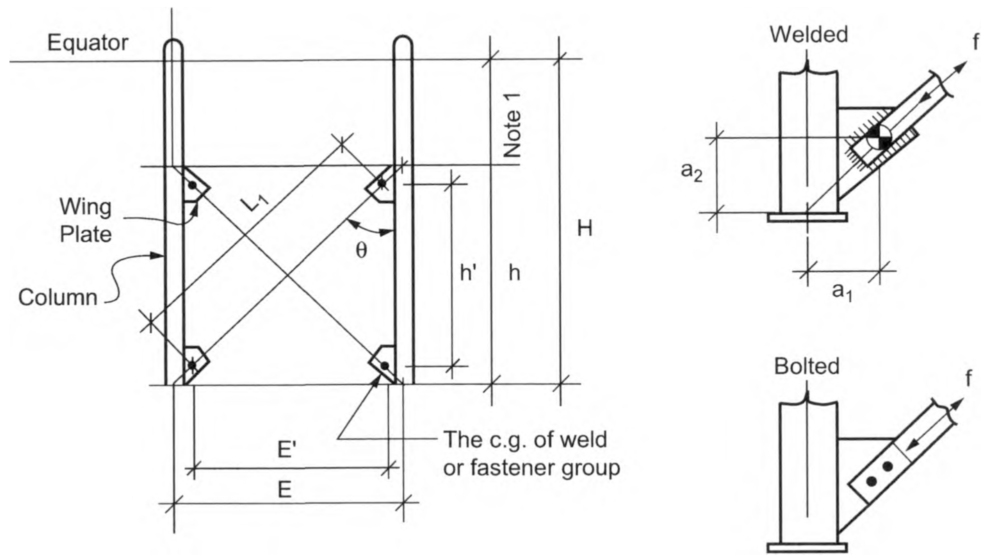
Table 9.19. Quantity and size of Support Columns

Capacity (BBLS)	Inside Dia (Ft-in)	Qty Columns	Column Size (OD X Thk (in)	Capacity (BBLS)	Inside Dia (Ft-in)	Qty Columns	Column Size (OD X Thk (in)
1,000	22-3	4	16 X .25	10,000	48-0	8	28 X .30
1,500	25-6	4	16 X .25	12,500	51-0	9	30 X .29
2,000	28-0	5	16 X .25	15,000	54-9	9	32 X .344
2,500	30-3	5	16 X .25	20,000	60-6	10	34 X .38
3,000	32-0	6	18 X .25	25,000	65-0	11	36 X .406
4,000	35-3	6	20 X .25	30,000	69-0	11	40 X .438
5,000	38-0	6	22 X .25	40,000	76-0	12	42 X .503
6,000	40-6	7	24 X .25	50,000	81-10	13	42 X .594
7,500	43-6	7	24 X .29				

Notes:

1. The column sizes shown are based on a sphere filled with water and normal wind/seismic loading. Higher pressure or significantly higher loadings will result in larger diameter columns or thicker columns.
2. The quantity of columns shown are based on 10 feet wide plates and a liquid sphere where columns are assumed for every other plate. For gas filled spheres, assume one column for every third plate.

Establish Leg & Brace Dimensions



Note 1: This dimesion shall be set by the designer/manufacturer. The clips are typically installed below dimension "B" as a minimum.

DIMENSIONS:	
ϕ =	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 =	θ =
a_2 =	Find Length of Brace, L_1
$E' = E - 2\,a_1$ =	$L_1 = E' / \text{Sin}\ \theta$ =
H =	L_1 =

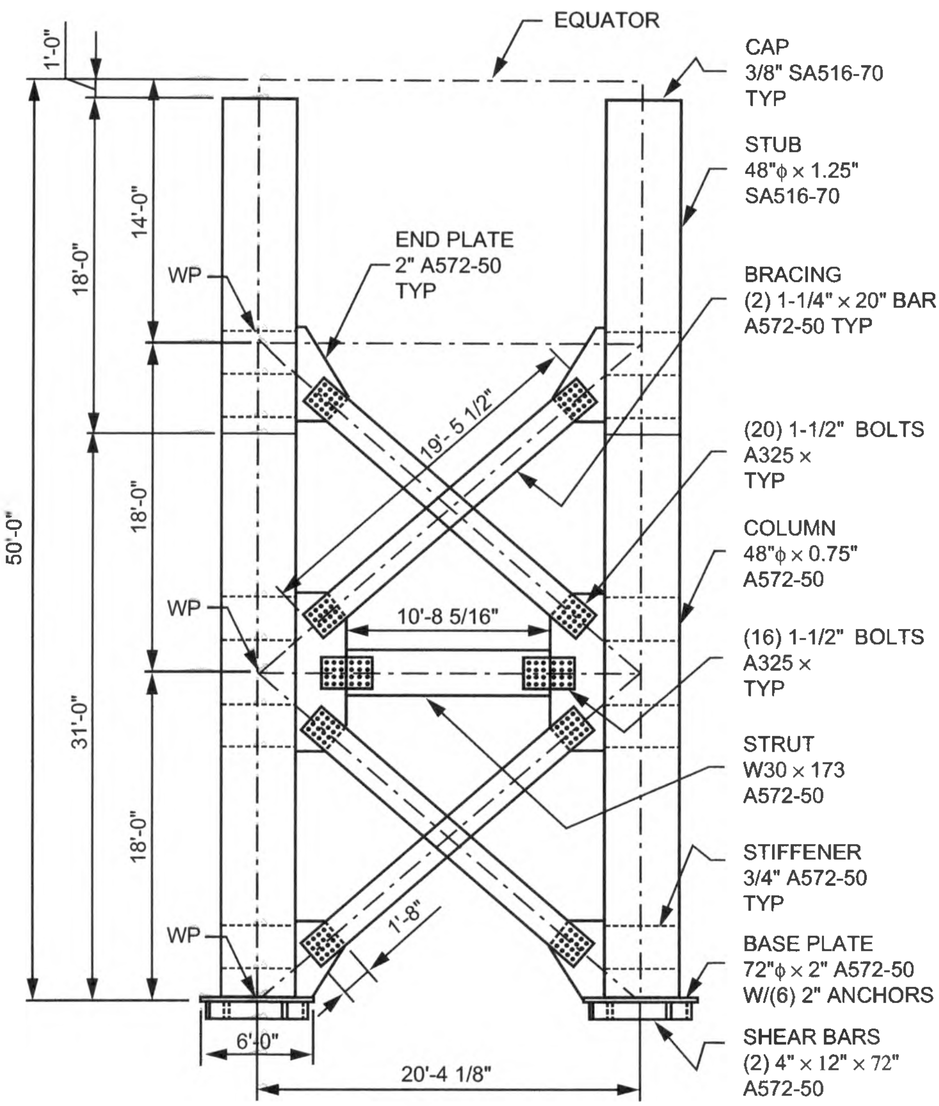
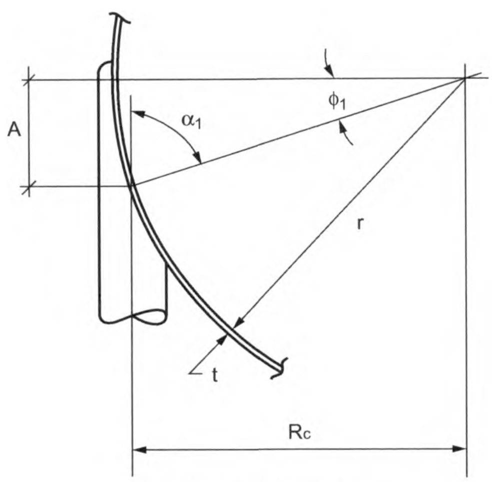


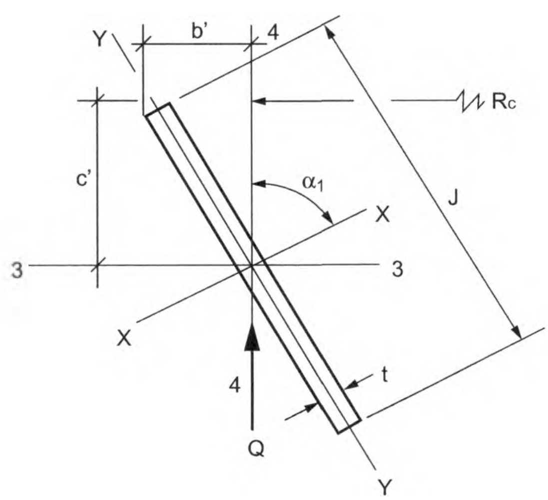
Figure 9-30. Two sets of cross bracing with horizontal struts for an 87' diameter sphere.

Properties of Shell Section (Without Girder) - Resisting Horizontal Loads

FIGURES



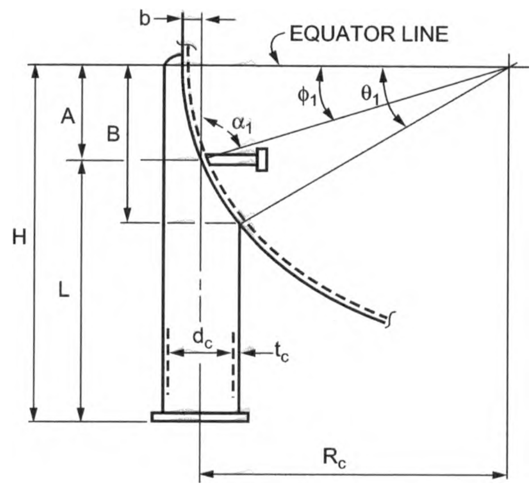
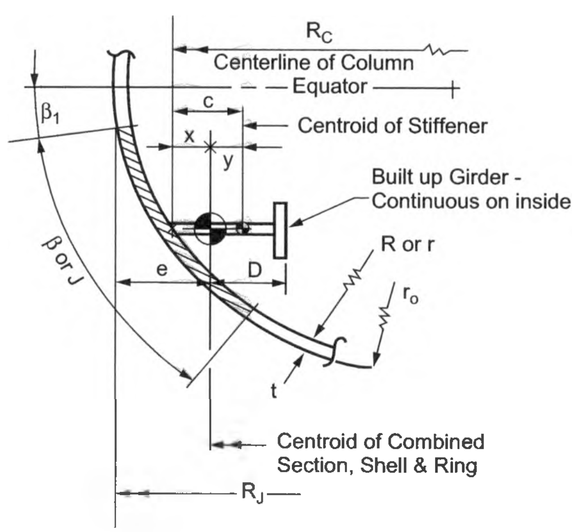
GENERAL ELEVATION



IDEALIZED SHELL SECTION
ACTING AS A GIRDER

EQUATIONS/DIMENSIONS
$r =$
$t =$
$R_c =$
$\text{Cos } \varphi_1 = (R_c / r) =$
$\varphi_1 =$
$\alpha_1 = 90 - \varphi_1 =$
$A = r \text{ Sin } \varphi_1 =$
J = Greater of following....
1. $2\,(r\,t)^{1/2} =$
2. $32\,t =$
USE, $J =$
$b' = .5\,L' \text{ Cos } \alpha_1 + .5\,t \text{ Sin } \alpha_1 =$
$c' = .5\,J \text{ Sin } \alpha_1 + .5\,t \text{ Cos } \alpha_1 =$
$A_s = J\,t =$
$I_X = t\,J^3 / 12 =$
$I_Y = t^3\,J / 12 =$
$I_3 = I_X \text{ Sin}^2 \alpha_1 + I_Y \text{ Cos}^2 \alpha_1 =$
$I_4 = I_X \text{ Cos}^2 \alpha_1 + I_Y \text{ Sin}^2 \alpha_1 =$
$Z_3 = I_3 / c' =$
$Z_4 = I_4 / b' =$

Dimensional Data for Combined Girder-Shell Section

FIGURES	EQUATIONS/DIMENSIONS
	$R =$
	$r =$
	$t =$
	$r_o = r + t =$
	$R_c =$
	$d_c =$
	$t_c =$
	$H =$
	$\cos\theta_1 = (R_c - .5d_c - t_c) / r =$
	$\theta_1 =$
	$B = r \sin\theta_1 =$
	$\cos\phi_1 = (R_c / r) =$
	$\phi_1 =$
	$\alpha_1 = 90 - \phi_1 =$
	$A = r \sin\phi_1 =$
	J = Greater of following....
	1. $2 (r t)^{1/2} =$
	2. $32 t =$
	USE, $J =$
	$x = A_g\, C / A_t$
	$y = C - x$
	$e = R_J - R$
	$R_1 = R_c - x =$
	$\beta = (180 J) / (\pi r_o)$
	$\beta_1 = \varphi_1 - \beta$
	$R_J = r_o \cos\beta_1$

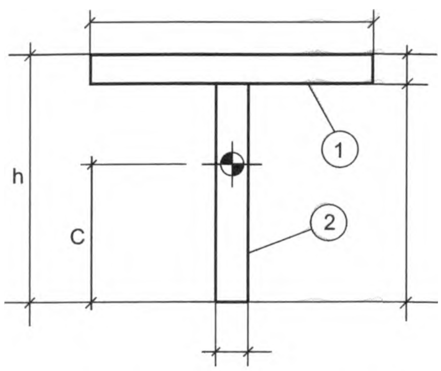
Sphere - Seismic Design			
	Method 1	**Method 2**	**Method 3**
V_n = Horizontal shear per leg	Worst case from table dependent on number of legs and direction of seisnic force (between legs or through legs).	NA	$V_n = V/N$
f = Max force in brace	$f = V_n / n \sin\theta$	$f = 2 W_o / 2 N \sin\theta$	$f = V_n / \sin\theta$
ΔL = Change in length of brace	$\Delta L = (f L_1) / (E_m A_b)$	$\Delta L = (f L_1)/(E_m A_b)$	$\Delta L = (2 W_o L_1)/(2 N E_m A_b \sin\theta)$
δ = Lateral deflection of sphere	$\delta = \Delta L / \sin\theta$	$\delta = \Delta L / \sin\theta$	$\delta = \Delta L / \sin\theta$
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; $T_a = C_t h_n^x$

Design of Girder

Properties of Stiffener Alone



DIMENSIONS OF INTERNAL GIRDER

Part	A	y	A y	A y^2	I
1					
2					
Σ					

$$C = \Sigma A\, y/\Sigma A$$

$$I_g = \Sigma A\, y^2 + \Sigma I - C\Sigma A\, y$$

Properties of Combined Section

- Area of Shell, A_s

$$A_s = J\, t$$

- Area of combined section, A_t

$$A_t = A_s + A_g$$

- Misc dimensions...

$$X = A_g\, C/A_t$$

$$y = C - X$$

$$D = h - X$$

- Moment of inertia of combined section, I_T

$$I_T = I_g + I_g + A_S\, X^2 + A_g\, y^2$$

- Section modulus, Z_T

$$Z_T = I_T/e$$

- Radius of gyration of girder, r_g

$$r_g = (I_T/A_T)^{1/2}$$

- Length of girder section exposed to load, L_g

$$L_g = (2\,\pi\, R_1)/N\, r_g$$

- Slenderness ratio, S_r

$$S_r = L_g/r_g$$

- Allowable stress;

Tension = lesser of...
1.2 (.6) F_y or 1.2 S

Compression = Lesser of ...
1. Factor B X 1.2
2. 1.2 (1.8) (10^6) (t / r)
3. AISC allowable buckling stress based on slenderness ratio

- Factor A

$$A = (.125\ t)\, r_o$$

Design of Columns

- Base Shear, V
Use worst case of wind or seismic
V = ___________

- Overturning Moment, M_o

$$M_o = H\, V$$

- Maximum Dead load, F_D

$$F_D = (-)\, W_o/N$$

- Maximum Live Load, F_L

$$F_L = +/(-)\, 48\, M_o/N\, D_c$$

- Maximum Column Load, Q
  Select worst case from Table or use;

  $Q = F_D +/- F_L$

  Q max compression $= Q_C =$

  Q max tension $= Q_T =$

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

- Leg selection;

A preliminary selection can be made and then checked.

Selection: __________
$A_c =$ ______________
$r_c =$ ______________

Compression Case

- Axial Stress, f_a

  $f_a = Q_C/A_C < F_a$

- Slenderness ratio, S_r

  $S_r = K\,h/r_c =$

- Allowable axial stress, F_a

  $F_a =$

Tension Case

- Tension stress, f_T

  $f_T = Q_T/A_C < F_T$

- Allowable tension stress, F_T

  $F_T = 1.2\,(.6)\,F_y$

Cross Bracing

Note: Loads in cross bracing are tension and compression.

Compression Case

- Required moment of inertia, i_r

  Case 1; Pinned at center ...

  $i_r = (f\,L_1^2)/(4\,\pi^2\,E_m)$

  Case 2: Not pinned at center ...

  $i_r = (f\,L_1^2)/(\pi^2\,E_m)$

- Use: _____________

  $I_b =$ ___________
  $r_b =$ ___________
  $A_b =$ ___________

- Slenderness ratio, S_r

  $S_r = K\,L_1/n'\,r_b$

  $n' = 1$ for not pinned; 2 for pinned

- Allowable axial stress, F_a

  Based on applicable slenderness ratio;

  $F_a =$ ____________

- Axial stress, f_a

  $f_a = f/A_b < F_a$

Tension Case

- Tension stress, f_T

  $f_T = f/A_b < F_T$

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

  $F_T = 1.2\,(.6)\,F_y =$

- Use: ___________

Post Connection Plate

There are two major steps required in the design of the post connection plates. The steps are as follows;

Step 1: Determine if an internal girder is required.

Step 2: If a ring girder is required, design the girder to resist all load conditions.

Note: A girder is only required to resist horizontal loads. If stresses from vertical loads are excessive, than the shell thickness must be increased for the post connection plates.

Step 1:

a. Determine the properties of the shell in the girder section (Use worksheet)
b. Determine all the moments in the shell due to external loads, M_S, M_C, M_P and M_B, (Use worksheet)
c. Determine the stresses in the shell due to external loads (Use worksheet)
d. If the stresses in the shell due to horizontal loading are exceeded, either add a ring girder or increase the shell thickness. If a ring girder is added, proceed to Step 2.
e. If the stresses in the shell due to vertical loading are acceptable, than the design is acceptable as is. If the shell stresses are exceeded, the shell thickness must be increased until the stresses are below allowable.

Step 2: The shell stresses due to horizontal loading are exceeded and a ring girder must be added.

a. Approximate the size of the ring girder required from the following equation:

   $$Z_r = M_P/F_T$$

   Where M_P and F_T are the same as those calculated in Step 1.
b. Based on the section modulus required, select a member size that will approximate the value required.
c. Complete the worksheet with girder to determine the exact properties.
d. Check the stresses in the combined shell-girder section as follows:

   $$f_T = M_P/Z_T$$

Combined Stresses

The stress at any point in the sphere depends on whether the plane under consideration is above or below the LOS (Line of Support).

In the following equations, T' is the greater of T_1 or T_2

Tension Other than support region:

$$S_T = T'/t\,E$$

At support location:

$$S_T = (T'/t\,E) + f_T$$

Compression Other than support region:

$$S_C = T'/t$$

At support location:

$$S_C = (T'/t) - f_C$$

Spheres; Shell Thickness Procedure

The thickness required for a sphere varies at every elevation. The required thickness is a function of the pressure, liquid level, loadings, cross sectional area and weights of shell and liquid at any particular elevation. Since the weights and pressures are varying at every elevation, the required thickness is also changing.

However, the thickness will only be changed in step increments to accommodate fabrication limitations and actual sizes of plates selected. Adding extra seams to accommodate potential step breaks in plate thickness may not yield the most economical design since the welding costs related to these extra seams may outweigh the savings in plate thickness. These are shop decisions and the elevations where thickness breaks occur should not be arbitrarily established by the designer.

Typically, the designer will check the required shell thickness at elevations where a seam is required. Actual plate thicknesses used will include a thinning allowance to accommodate thinning which occurs in the pressing of the plates. The plates do not get lighter. The actual plate thickness utilized is dependent upon the sizes of the plate available from the shop, mill or warehouse and shop limitation regarding the pressing of plates.

The weight estimates should include a thinning allowance to account for the thicker plates utilized. Thinning allowance can be estimated as follows:

Thickness	Thinning Allowance
Up to 1"	.188"
1" to 2"	.375"
2" to 3"	.625"
3" to 4"	.75"
4" to 5"	1"
Over 5"	1.5"

In addition, the "post plates" may be made thicker to accommodate local loads and horizontal loads due to wind or seismic. Since the columns are not attached to every plate in the support band, the plates between the post plates may also be increased in thickness, but not as much as the post plates themselves. Once again, this is a shop decision based on the overall capabilities of the shop, as well as economic decisions.

So, a sphere can be made of a variety of plate thicknesses, both from top to bottom, but also around the equator where the support columns are attached. Occasionally, the post plates are the only plates in the sphere thick enough to require PWHT. In such a case, the PWHT will be carried out in the shop. Field PWHT will not be required. If the spheres are large enough, or multiple spheres are required, then mill runs for the exact thickness of material required are selected.

Stress in Post Connection Plates Due to External Loads				
	At Support Locations		Between Support Locations	
	Moment	Stress	Moment	Stress
Due to Vertical Force, Q	M_S = (Compression)	f_C =	M_C = (Tension)	f_T =
Due to Horizontal Force, V	M_P = (Tension)	f_T =	M_B = (Compression)	f_C =

Notes:

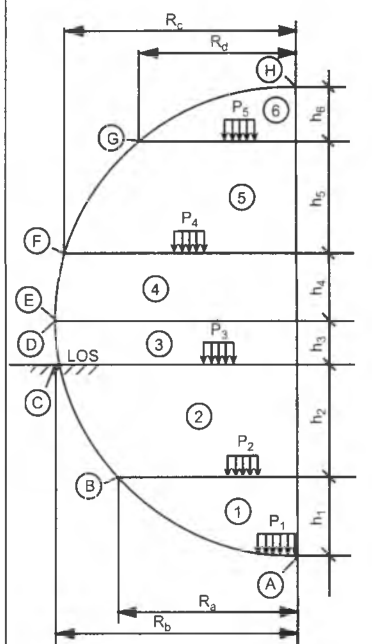
1. The post connection plates are the shell sections to which the "posts" or columns are attached. The post connection plates form a circumferential band all around the circumference at the location where the posts are attached. These plates take the horizontal and vertical loads imposed by dead and live loads. The post plates are typically thicker than all the other plates of sphere, whether a girder is provided or not. However a girder is usually the most economical way of distributing the loads encountered at this location.
2. The moments above are from the Table and are dependent on the number of legs.
3. Allowable stresses:

   Tension; $F_T = 1.2\ (.6)\ Fy =$
   Compression, $F_C = 1.2\ (1.8)\ (10^6)\ (t/r) =$

4. DATA REQUIRED:

N =	R_C =	Z_3 =	V =	F_y =
Q =	t =	Z_4 =	r =	

SPHERE - WEIGHTS, VOLUMES, AREAS, PRESSURES & LOADS



FIVE COURSE SPHERE SHOWN FOR EXAMPLE

h_n	R_n	t_n'	V_n	A_{sn}	W_L	W_S	W_T	Σh_n	P_n	A_{cn}
Σ										

EQUATIONS

$A_{sn} = 2\pi R_n h_n$	$R_a = [2 R h_1 - h_1^2]^{1/2}$	$V_1 = 1.047 h_1^2 (3R - h_1)$	GIVEN:
$A_{cn} = \pi R_n^2$	$R_b = [R^2 - h_3^2]^{1/2}$	$V_2 = .523 h_2 (3R_a^2 + 3R_b^2 + h_2^2)$	R =
$A_{S1} = 2\pi R_a h_1$	$R_c = [R^2 - h_4^2]^{1/2}$	$V_3 = .523 h_3 (3R_b^2 + 3R^2 + h_3^2)$	r =
$A_{S2} = 2\pi R_b h_2$	$R_d = [2 R h_6 - h_6^2]^{1/2}$	$V_4 = .523 h_4 (3R^2 + 3R_c^2 + h_4^2)$	S_g =
$A_{S3} = 2\pi R h_3$	$P_n = P + P_h$ or $P_h - P_X$	$V_5 = .523 h_5 (3R_c^2 + 3R_d^2 + h_5^2)$	$w = 62.4 S_g$ =
$A_{S4} = 2\pi R h_4$	$P_h = .433 \Sigma h_n S_g$	$V_6 = 1.047 h_6^2 (3R - h_6)$	P =
$A_{S5} = 2\pi R_c h_5$	$W_L = V_n w$		P_x =
$A_{S6} = 2\pi R_d h_6$	$W_S = 144 (.2833) t_n A_{sn}$	$W_T = W_L + W_S$	

CALCULATION OF THICKNESS

DESIGN POINT OR LEVEL	P_n	W_{Tn}	A_{Cn}	W_{Tn} / A_{cn}	T_1	T_2	t_n	EQUATIONS
H								$T_1 = T_2 = P_1 r / 2$ = (Note 1)
G-G								Any elevation above the LOS $T_1 = .5 r [P_n - W_{Tn} / A_{cn}]$
F-F								$T_2 = .5 r [P_n + W_{Tn} / A_{cn}]$ t_r = Greater of ....
E-E								(Greater of T_1 or T_2) / SE (Greater of T_1 or T_2) / F_c
D-D								
C-C (LOS)								Any elevation at or below the LOS $T_1 = .5 r [P_n + W_{Tn} / A_{cn}]$ $T_2 = .5 r [P_n - W_{Tn} / A_{cn}]$
B-B								t_r = Greater of .... (Greater of T_1 or T_2) / SE (Greater of T_1 or T_2) / F_c
A								$T_1 = T_2 = P_1 r / 2$ = (Note 1)

NOTES:

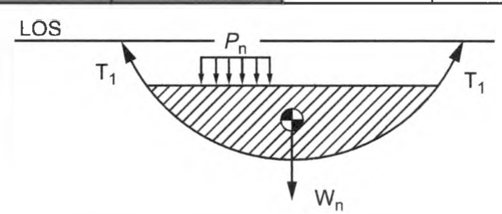
1) Formulas shown are for API 620 Sphere;

For ASME VIII-1, Use...

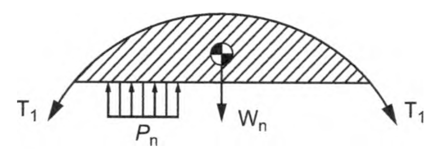
$t_r = (P_n r) / (2 S E - .2 P_n)$ =

For ASME VIII - 2, Use...

$t_r = r (e^{.5P/S} - 1)$ =



CASE 1: PLANE BELOW LINE OF SUPPORT (LOS)

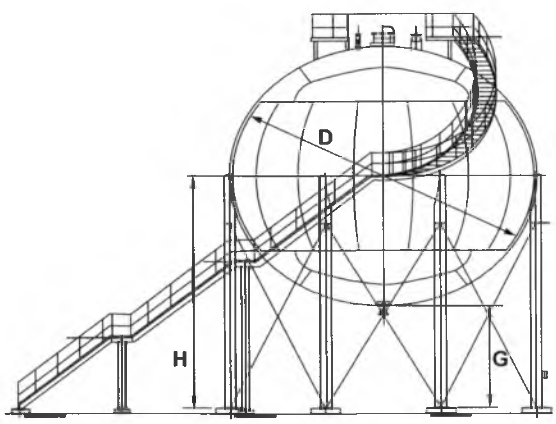


CASE 2: PLANE ABOVE LINE OF SUPPORT (LOS)

SPHERE DATA SHEET (EXAMPLE)

EQUIPMENT ITEM NO. : 745-V-101 A/B

EQUIPMENT NAME : NGL SPHERE



DIMENSIONS	
D	39.37 FT ID
H	35.1 FT
G	15.1 FT

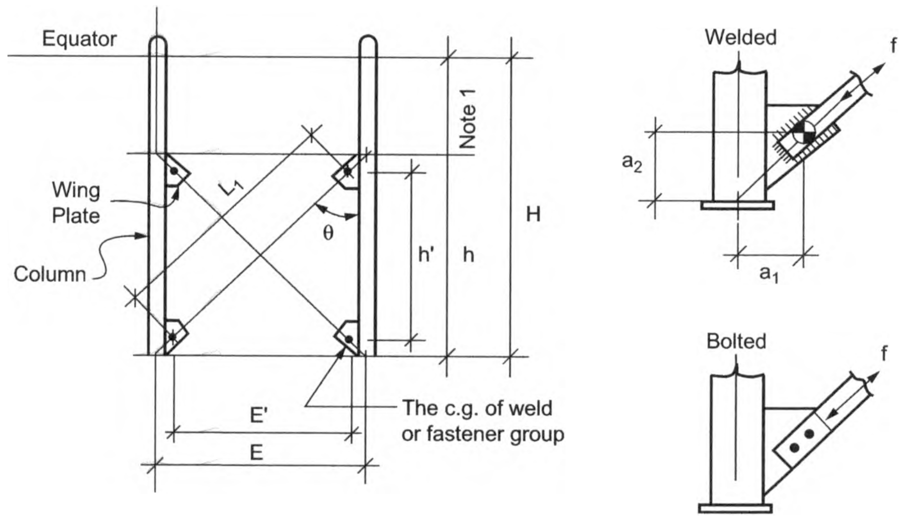
WEIGHTS	
FABRICATED	N.A.
EMPTY	818 KIPS
OPERATING	1,817 KIPS
TEST	2,807 KIPS
SHELL	704 KIPS
COLUMNS	23 KIPS
BRACING	59 KIPS
CONTENTS	1,018 KIPS
WATER	1,994 KIPS
FIREPROOFING	32 KIPS

DESIGN DATA	
Design Pressure - Internal	580 PSIG
Design Pressure - External	7.5 PSIG
Design Temperature - Internal	212 ^{O}F
Design Temperature - External	150 ^{O}F
MDMT	(-) 29 ^{O}F
Specific Gravity	0.517
Capacity	5690 BBLS
Corrosion Allowance	0.125 IN
Joint Efficiency	1
PWHT	YES
Contents	NGL
Insulation/Thk	NO
Fireproofing - Legs	YES, 2 IN
Service (Sour, Lethal, Cyclic, etc.)	N.A.

MATERIALS	
Shell	SA-516-65
Columns	SA-333-6
Column Bracing	SA-53-B
Flanges	SA-350-LF2
Nozzle Necks- Pipe	SA-333-6
Bolting	SA-193-B7
Gaskets	SPIRAL WOUND- STYLE CG
1/8" THK - 316 SST W/ GRAPHITE	
Base Plates	SA-36

CODES	
Design	ASME VIII-2
Seismic	ASCE 7 -05
Wind	ASCE 7 -05
Structural	AISC

Establish Leg & Brace Dimensions

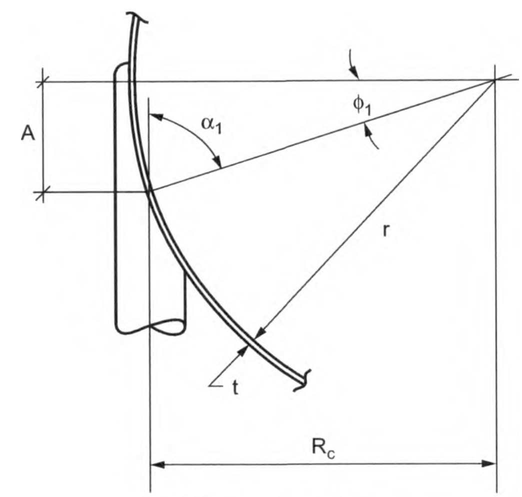


Note 1: This dimension shall be by the designer/manufacturer. The clips are typically installed below dimension "B" as a minimum.

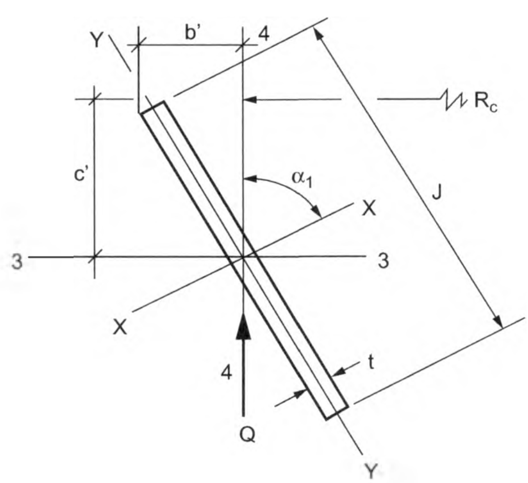
DIMENSIONS:	
ϕ = 45°	h = 27.915 ft
R_c = 19.65 ft	$h' = h - 2a_2$ = 23.315 ft
$E = 2[\sin .5\phi(R_c)]$ =	***Find Angle*** ϑ;
E = 15.04 ft	$\tan\theta = E'/h'$ = 12.38/ 23.315 = .531
a_1 = 1.33 ft	θ = 27.97°
a_2 = 2.3 ft	***Find Length of Brace,*** L_1
$E' = E - 2a_1$ = 12.38 ft	$L_1 = E'/\sin\theta$ = 12.38/ Sin 27.97 =
H = 35.1 ft	L_1 = 26.4 ft

Properties of Shell (Without Girder)- Resisting Horizontal Loads

FIGURES



GENERAL ELEVATION



IDEALIZED SHELL SECTION
ACTING AS A GIRDER

EQUATIONS/DIMENSIONS
r = 236.33 in(Corr)
t = 3.375 in(Corr)
R_c = 235.83 in
Cos ϕ_1 = (R_c / r) = .9979
ϕ_1 = 3.72°
α_1 = 90 - ϕ_1 = 86.27°
A = r Sin ϕ_1 = 15.33 in
J = Greater of following....
1. 2 (r t) $^{1/2}$ = 56.48 in
2. 32 t = 108 in
USE, J = 108 in
b' = .5 L' Cos α_1 + .5 t Sin α_1 = 5.19 in
c' = .5 J Sin α_1 + .5 t Cos α_1 = 54 in
A_s = J t = 364.5 in^2
I_X = t J^3 / 12 = 354, 294 in^4
I_Y = t^3 J / 12 = 346 in^4
I_3 = I_X Sin^2 α_1 + I_Y Cos^2 α_1 = 352,797 in^4
I_4 = I_X Cos^2 α_1 + I_Y Sin^2 α_1 = 1844 in^4
Z_3 = I_3 / c' = 6533 in^3
Z_4 = I_4 / b' = 355 in^3

Data

$P = 580$ PSIG
$P_X = 7.5$ PSIG
D.L.L. = Full
$S_g = .517$
$w = 32.26$ PCF
$D = 39.37$ ft
$R = 19.685$ ft
$r = 236.22$ in
$D_c = 39.3$ ft
$R_c = 19.65$ ft
$H = 35.1$ ft
$g = 386$ in/sec^2
$E = 28.8 \times 10^6$ PSI
S @ 212 °F = 21,300 PSI
$F_y = 31.9$ KSI
$W_o = 1817$ kips

Properties of Columns

Size: 16 in Sch 40
$N = 8$
$d_c = 15$ in ID
$t_c = 0.5$ in
$A_c = 24.35$ in^2
$I_c = 732$ in^4
$r_c = 5.48$ in
$\varphi = 45°$
$\alpha = 67.5°$

Properties of Bracing

Size: 4 in Sch 40
$A_b = 3.17$ in^2
$I_b = 7.23$ in
$r_b = 1.51$ in
$n = 2$ (Cross Bracing)
$L_1 = 26.4$ ft

Wind/Seismic

Code: ASCE 7-05

- Base Shear, Wind; V

$$V = q_z\, G\, C_f\, A_f$$
$$= 36.24\,(.85)\,.7\,(1630) = 35,151 \text{ Lbs}$$

- Base Shear, Seismic: V

$$V = C_h\, W_o\, I$$
$$= .0694\,(1817 \text{ kips})\, 1.5 = 126.1 \text{ kips}$$
$$= 126.1 \text{ kips}$$

Therefore Seismic Governs;

- Overturning Moment, M_o

$$M_o = H\,V = 35.1\,(126.1 \text{ kips})$$
$$= 4426 \text{ Ft} - \text{kips}$$

Design of Columns

- Maximum Dead load, F_D

$$F_D = (-)\,W/N = 1817/8 = (-)\,227.13 \text{ kips}$$

- Maximum Live Load, F_L

$$F_L = +/(-)\,48\,M_o/N\,D_c =$$
$$= 48\,(4226)/8\,(39.3 \times 12) = +/(-)\,6.71 \text{ kips}$$

- Maximum Column Load, Q
  (Equation from Table)

$$Q = F_D - F_L = (-)\,227.13 - 6.71 = 233.84 \text{ kips}$$

Slenderness ratio, S_r

$$S_r = K\,h/r_c = 1.0\,(12 \times 27.915)/5.48$$
$$= 61.12$$

- Allowable Compressive Stress, F_c

$$F_c = 17.33 \text{ KSI}$$

- Axial Stress, f_a

$$f_a = Q/A_c = 233.84^K/24.35 = 9.6 \text{ KSI}$$

- There is no tension in columns to consider!

Design of Bracing

- Max Horizontal Force per Column, V_n
  (Equation from Table)

$$V_n = .2134\,V = .2134\,(126.1 \text{ kips}) = 26.9 \text{ kips}$$

- Maximum Force in Brace, f

$$f = V_n/n \,\text{Sin}\,\theta = 26.9/2\,(\text{Sin}\,27.96)$$
$$= 28.68\,\text{kips}$$

- Required moment of Inertia, I_r

$$I_r = \left(f\,L_1^2\right)/\left(4\,\pi^2\,E_m\right)$$
$$= 28,680\,(26.4 \times 12)^2 / 4\,\pi^2\,(28.8\,(10^6)$$
$$= 2.53\,\text{in}^4 < 7.27\,\text{in}^4$$

- Slenderness Ratio, S_r

$$S_r = K\,L_1/2\,r_b = 1\,(26.4 \times 12)/2 \times 1.51$$
$$= 105$$

- Allowable Axial Stress, F_a

$$F_a = 12.33\,\text{KSI}$$

- Axial Stress, f_a

$$f_a = f/A_b = 28.68/3.17 = 8.42\,\text{KSI}$$

Allowable Stresses

Girder / Post Connection Plates

- Tension: F_T, Lesser of following;

$$F_T = 1.2\,(.6)\,F_y = 1.2\,(.6)\,35\,\text{KSI} = 25.2\,\text{KSI}$$
$$\text{Or } 1.2\,S = 1.2\,(21.3\,\text{KSI}) = 25.56\,\text{KSI}$$

- Girder properties:

$$I_T = 1844\,\text{in}^4$$
$$A_T = 364.5\,\text{in}^2$$

- Radius of Gyration, r_g

$$r_g = (I_T/A_T)^{1/2} = (1844/364.5)^{1/2} = 2.245\,\text{in}$$

- Length of Girder, L_g

$$L_g = 2\,\pi\,(R_1) = 2\,\pi\,(12 \times 19.65\,\text{ft}) = 1481\,\text{in}$$

- Slenderness Ratio, S_r

$$S_r = K\,L_g/(N\,r_g) = 1\,(1481)/(8 \times 2.245\,\text{in})$$
$$= 82.46$$

- Allowable Compressive Stress, F_c

$$F_c = 15.13\,\text{KSI}$$

- $F_T = \text{Tension} = 1.2\,S = 1.2\,(21.3) =$
$= 25.56\,\text{KSI}$

- Allowable Compressive Stress, F_c

F_C = Lesser of following....

1. Factor 'B' from ASME Code;

$$A = .125\,t/r_o = .125\,(3.375)/239.595$$
$$= .00176$$
$$B = 14,000\,\text{PSI}$$

2. $F_C = 1.8\,(10^6)\,(t/r_o)$
$= 1.8\,(10^6)\,.01409$
$= 25,355\,\text{PSI}$

Therefore $F_C = 14{,}000$ PSI

Combined Stress

Tension;
Worst case Plane B-B;

$$S_T = T'/t = 69572/3.375 = 20,613\,\text{PSI}$$

At support location (Plane C-C);

$$S_T = T'/t + f_T = 69495/3.375 + 281$$
$$= 20,832\,\text{PSI}$$
$$S = 21,300\,\text{PSI OK}$$

Sphere - Seismic Design (Sample Problem)			
DATA: $V = 126.1$ $N = 8$ $n = 2$ (Cross braced legs) $D = 39.37$ ft $\theta = 27.97°$ $E_m = 28.8 \times 10^6$ PSI $g = 386$ in/sec^2 $W_o = 1817$ kips $A_b = 3.17$ in^2			
	Method 1	Method 2	Method 3
V_n = Horizontal shear per leg	$V_n = .2134\ V = .2134$ (126.1 kips) = 26.9 kips	NA	$V_n = V/N = 126.1 / 8 = 15.76$ kips
f = Max force in brace	$f = V_n / n \sin\theta = 26.9/2 (\sin 27.97°) = 28.67$ kips	$f = 2\ W_o / 2\ N \sin\theta = 2$ (1817 kips) / 2 (8) Sin 27.97° = 484.26 kips	$f = V_n / \sin\theta = 15.76$ kips/ Sin 27.97° = 33.6 kips
ΔL = Change in length of brace	$\Delta L = (f\ L_1) / (E_m\ A_b) = 28670\ (12)\ 26.4 / 28.8\ (10^6)\ 3.17 = 0.1$ in	$\Delta L = (f\ L_1) / (E_m\ A_b) = 484{,}260\ (12)\ 26.4 / 28.8\ (10^6)\ 3.17 = 1.68$ in	$\Delta L = (2\ W_o\ L_1) / (2\ N\ E_m\ A_b \sin\theta) = 2\ (1{,}817{,}000)\ 12\ (26.4) / 2\ (8)\ 28.8\ (10^6) \sin 27.97 = 1.68$ in
δ = Lateral deflection of sphere	$\delta = \Delta L / \sin\theta = .1 / \sin 27.97 = 0.213$ in	$\delta = \Delta L / \sin\theta = 1.68 / \sin 27.97 = 3.58$ in	$\delta = \Delta L / \sin\theta = 1.68 / \sin 27.97 = 3.58$ in
T = Period of vibration	$T = 2\pi (\delta / g)^{1/2} = 2 (\pi) [.213 / 386]^{1/2} = .147$ Sec's	$T = 2\pi (\delta / g)^{1/2} = 2 (\pi) [3.58 / 386]^{1/2} = .605$ Sec's	$T = 2\pi (\delta / g)^{1/2} = 2 (\pi) [3.58 / 386]^{1/2} = .605$ Sec's

Notes:

1. Approx POV per ASCE7-05; $T_a = C_t h_n^x = .02 (55)^{.75} = .404$ Sec's

Stress in Post Connection Plates Due to External Loads (Example)				
	At Support Locations		Between Support Locations	
	Moment	Stress	Moment	Stress
Due to Vertical Force, Q	M_S = (Compression) = (-) .0662 Q R_C = (-) 3,650,697 in-Lbs	$f_C = M_S / Z_3$ = 558 PSI	M_C = (Tension) = + .0333 Q R_C = + 1,836,378 in-Lbs	$f_T = M_C / Z_3$ = 281 PSI
Due to Horizontal Force, V	M_P = (Tension) = + .0061 V R_C = + 181,403 in-Lbs	$f_T = M_P / Z_4$ = 510 PSI	M_B = (Compression) = (-) .0058 V R_C = (-) 172,481 in-Lbs	$f_C = M_B / Z_4$ = 485 PSI

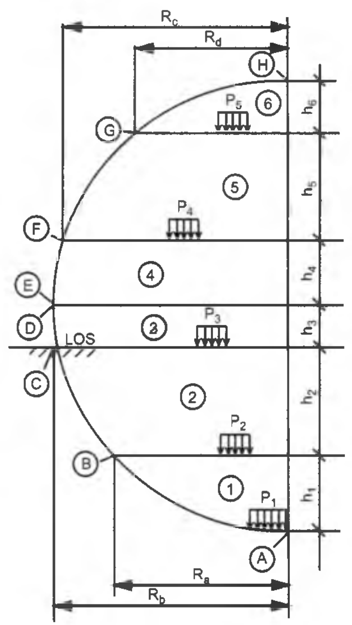
Notes:

1. The post connection plates are the shell sections to which the "posts" or columns are attached. The post connection plates form a circumferential band all around the circumference at the location where the posts are attached. These plates take the horizontal and vertical loads imposed by dead and live loads. The post plates are typically thicker than all the other plates of sphere, whether a girder is provided or not. However a girder is usually the most economical way of distributing the loads encountered at this location.
2. The moments above are from the Table and are dependent on the number of legs.
3. Allowable stresses;

   Tension; F_T = 1.2 (.6) F_y = 1.2 (.6) 31.9 KSI = 23 KSI
   Compression, F_C = 1.2 (1.8) (10^6) (t / r) = 1.2 (1.8) (10^6) (3.375 / 236.33) = 30,846 PSI
4. DATA REQUIRED:

N = 8	R_C = 235.83 in	Z_3 = 6533 in^3	V = 126,100 Lbs	F_y = 31.9 KSI
Q = 233,840 Lbs	t = 3.375 in	Z_4 = 355 in^3	r = 236.33 in	

Sphere - Weights, Volumes, Areas, Pressures & Loads



FIVE COURSE SPHERE SHOWN FOR EXAMPLE

	h_n	R_n	t_n'	V_n	A_{sn}	W_L	W_S	W_T	Σh_n	P_n	A_{cn}
										580	
	5	157.2	3.5	1415	412	45,650	58,834	105,000	5	581.1	77,635
	9	226.1	3.5	7816	1065	252,144	152,082	404,300	14	583.1	160,600
	5.69	236.1	3.5	6718	703	216,722	100,388	317,100	19.69	584.4	175,270
	1.28	236.2	3.5	1551	158	50,035	22,562	72,600	21	584.7	175,270
	13.41	235.7	3.5	12984	1587	418,863	226,624	645,500	34.37	587.7	174,530
	5	157.2	3.5	1415	412	45,650	58,834	105,000	39.37	588.8	77,635
Σ				31,900	4337	1030 K	620 K	1650 K			

EQUATIONS

$A_{sn} = 2\pi R_n h_n$	$R_a = [2 R h_1 - h_1^2]^{1/2}$	$V_1 = 1.047 h_1^2 (3R - h_1)$	GIVEN:
$A_{cn} = \pi R_n^2$	$R_b = [R^2 - h_3^2]^{1/2}$	$V_2 = .523 h_2 (3 R_a^2 + 3 R_b^2 + h_2^2)$	R = 19.685 ft
$A_{s1} = 2\pi R_a h_1$	$R_c = [R^2 - h_4^2]^{1/2}$	$V_3 = .523 h_3 (3 R_b^2 + 3 R^2 + h_3^2)$	r = 236.22 in
$A_{s2} = 2\pi R_b h_2$	$R_d = [2 R h_6 - h_6^2]^{1/2}$	$V_4 = .523 h_4 (3 R^2 + 3 R_c^2 + h_4^2)$	S_g = .517
$A_{s3} = 2\pi R h_3$	$P_n = P + P_h$ or $P_h - P_X$	$V_5 = .523 h_5 (3 R_c^2 + 3 R_d^2 + h_5^2)$	$w = 62.4 S_g$ = 32.26 PCF
$A_{s4} = 2\pi R h_4$	$P_h = .433 \Sigma h_n S_g$	$V_6 = 1.047 h_6^2 (3R - h_6)$	P = 580 PSIG
$A_{s5} = 2\pi R_c h_5$	$W_L = V_n w$		P_x = 7.5 PSIG
$A_{s6} = 2\pi R_d h_6$	$W_S = 144 (.2833) t_n A_{sn}$	$W_T = W_L + W_S$	

Calculation of Thickness

DESIGN POINT OR LEVEL	P_n	W_{Tn}	A_{Cn}	W_{Tn} / A_{cn}	T_1	T_2	t_r	EQUATIONS
H	580				68503.8	68503.8	3.22	$T_1 = T_2 = P_1 r / 2$ = (Note 1)
G-G	581.1	105,000	77,635	1.35	68474.3	68793.2	3.23	Any elevation above the LOS $T_1 = .5\ r\ [\ P_n - W_{Tn} / A_{cn}\ ]$
F-F	583.1	404,300	160,600	2.52	68572.3	69167.6	3.25	$T_2 = .5\ r\ [\ P_n + W_{Tn} / A_{cn}\ ]$ t_r = Greater of ....
E-E	584.4	317,000	175,270	1.81	68809.7	69237.3	3.25	(Greater of T_1 or T_2) / SE (Greater of T_1 or T_2) / F_c
D-D	584.4	72,600	175,270	0.41	68975.1	69071.9	3.24	
C-C (LOS)	584.7	645,500	174,530	3.7	69495.9	68621.9	3.26	Any elevation at or below the LOS $T_1 = .5\ r\ [\ P_n + W_{Tn} / A_{cn}\ ]$ $T_2 = .5\ r\ [\ P_n - W_{Tn} / A_{cn}\ ]$
B-B	587.7	105,000	77,635	1.35	69572.7	69253.8	3.27	t_r = Greater of .... (Greater of T_1 or T_2) / SE (Greater of T_1 or T_2) / F_c
A	588.1				69460.5	69460.5	3.26	$T_1 = T_2 = P_1 r / 2$ = (Note 1)

NOTES:

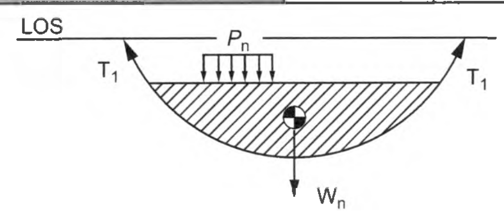
1) Formulas shown are for API 620 Sphere;

For ASME VIII-1, Use...

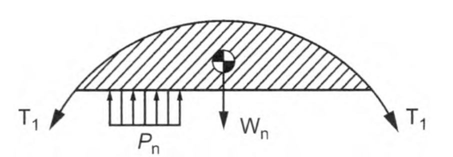
$t_r = (P_n r) / (2 S E - .2 P_n) =$

For ASME VIII - 2, Use...

$t_r = r (e^{.5P/S} - 1) =$



CASE 1: PLANE BELOW LINE OF SUPPORT (LOS)



CASE 2: PLANE ABOVE LINE OF SUPPORT (LOS)

Notes

1. A sphere supported at the equator will have maximum ring compression when the liquid level is at the equator and no gas pressure.
2. Horizontal components of thrust will cancel out horizontal components of shear when column acts through the centroid of the circular girder, and the columns are vertical.
3. Worst case for external pressure is when the sphere is half full (maximum ring compression).
4. Cold or hot insulated spheres will expand or contract differentially with respect to the base plate elevation. If the column base is bolted and grouted to its foundation pier, the following moment and shear force will occur;

$$V = 2\,M/L \text{ and } M = \left(6\,E_m\,I\,\delta\right)/L^2$$

   This moment and shear will add compressive stresses to those already produced by the weight of steel, product and lateral loads.
5. Spheres with long columns may require two or more sets of tie rods or cross bracing. When more than one level of bracing is required, a compression strut at the common wing gusset is required.
6. Columns for spheres that are high pressure, liquid loaded, heavy wall or in high seismic areas are best with cross bracing. Conversely, columns for spheres in low seismic areas, gas filled or light wall are best with sway bracing.

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