Submittal #1.3

Tesoro Savage Petroleum Terminal

PO # 0241-10-16

Port of Vancouver, WA

240'Ø x 50'high Tank 'DRAFT' Design Calculations

- *Unheated – Tanks TK-001 – 4 (rev 1 dated 3/3/15)*
- *Heated – Tanks TK-005 & 6 (rev 1 dated 3/3/15)*
- *Diffuser Calculations (dated 2/19/15)*

Tank Contractor:

T BAILEY, INC.

12441 Bartholomew, Rd.
Anacortes, WA 98221
Voice (360) 293-0682
Fax (360) 293-3893

It is hereby certified that this equipment/material is in compliance with the contract drawings and specifications, coordination with other work on the job, and dimensional suitability.

T BAILEY, INC. By _____ **Date** **03/03/15**

TANK CALCULATIONS

T-Bailey

Tesoro Savage Petroleum Terminal, LLC
Tesoro Savage Vancouver Energy
Port of Vancouver, WA

Crude Oil Un-Insulated Storage Tank

240´ Diameter x 50´ High Cone Roof Tank

Preliminary Design Calculat ons

PROJECT ENGINEER

Hagen Engineering Int rnati nal
202 S. Cedar St. Su te B
Owasso, OK 74055

(651) 3 5-46 6

DRAFT

DESIGN ENGINEER

Doug Bayles, P.E.

HEI Job Number: 15-1006

Standard Compliant Issues

Shell

Shell Course 1 requires impact testing per the standard.
Shell Course 2 requires impact testing per the standard.

Comments

Shell

Joint efficiency for all shell courses = 100%. X-rays are required. Quantity of x-rays shall be per 8.1.2.

Roof

Roof is single lap welded
Internal pressure does not exceed weight of roof. Appendix F calculations not required.
Tank meets the frangibility requirements of 5.10.2.6.
The roof support members shall not be attached to the roof plate.
The roof must be attached to the top angle with a single continuous fillet weld on the top side only that does not exceed 3/16'. No underside welding of roof to top angle (including seal welding) is permitted.

Rafters

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.
For Outer Rafter for Three-Span System rafters, the required section modulus is 13.68. The actual section modulus is 17.1.
For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.
For Intermediate Rafter for Three-Span System rafters, the required section modulus is 15.31. The actual section modulus is 21.3.
For Inner Rafter for Three-Span System rafters, (1) lateral bracing is required equal spaced.
For Inner Rafter for Three-Span System rafters, the required section modulus is 10.37. The actual section modulus is 10.9.
For Outer Intermediate Girder for Three-Span System girders, the required section modulus is 72.75. The actual section modulus is 78.8.
For Inner Intermediate Girder for Three-Span System girders, the required section modulus is 72.52. The actual section modulus is 78.8.

Columns, Base Plates, & Cap Plates

For center column, KL/r = 137.2
Load on center column (71740.73) is less than the allowable load (111759.42).
The required bottom base plate thickness of .2911 is less than or equal to the plate thickness.
The required upper base plate thickness of 1.1547 is less than or equal to the plate thickness.
For Inner Intermediate Column, KL/r = 166.8
For Inner Intermediate Column, load (49410.96) is less than the allowable load (56119.29).
The required bottom base plate thickness of .1837 is less than or equal to the plate thickness.
The required upper base plate thickness of .9623 is less than or equal to the plate thickness.
For Outer Intermediate Column, KL/r = 158.7
For Outer Intermediate Column, load (52270.93) is less than the allowable load (62016.76).
The required bottom base plate thickness of .1129 is less than or equal to the plate thickness.
The required upper base plate thickness of .9697 is less than or equal to the plate thickness.

Wind

Per 5.2.1 (k)(1)(b), wind loads are based on ASCE 7-10.

Shell External Pressure

Per 5.2.1(b), vacuum calculations are not required.

Seismic

Tank is self-anchored
Per E.6.2.1.1.3, butt welded annular plates are required.
Tank is stable against sliding.

Anchor Bolt and Anchor Chair

Tank is not anchored

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Design Data

Tank Diameter	240'- 0.0000" (Outside)
Tank Height	50'- 0.0000"
High Liquid Level	47'- 0.0000"
Normal Liquid Level	46'- 6.0000"
Low Liquid Level	4'- 0.0000"
Total Capacity	15,765,268 gallons
	375,364 barrels
Working Capacity	14,357,570 gallons
	341,847 barrels
Design Standard	API-650 12th Edition, Add. 1, Annex E
Design Metal Temperature	25 °F
Operating Temperature	150 °F
Design Temperature	150 °F
Roof Live Load	25.00 psf
Ground Snow Load	30.00 psf
Roof Design Snow Load (0.84 x Ground Snow Load)	25.20 psf
Additional Roof Loads	0.00 psf
Wind:	
Velocity per ASCE 7-10 Figure 26.5-1A	135 mph
Risk Category	II
Exposure	C

Seismic:

Site Class	E
Importance Factor	1.00
Use Group	I
Ss Value	94.00 %
S_1 Value	41.20 %
Vertical Acceleration	0.00 g
Transitional Period (T_L)	16
Specific Gravity	1.000
Internal Pressure (Design)	Atmospheric
(Operating)	Atmospheric
External Pressure (Design)	1.00 inches of water
(Operating)	Atmospheric

Corrosion Allowance:

Bottom	0.1250 in
Annular Ring	0.1250 in
Shell Courses 1 - 3	0.1875 in
Courses 4 - 4	0.1875 in
Courses 5 - 5	0.1875 in
Roof	0.0000 in
Rafters	0.0000 in
Columns	0.0000 in
Cap Plate	0.0000 in
Base Plate	0.0000 in
Compression Ring	0.0000 in

Summary

Total Weight of Tank: 2,577,353 lbs
Total Operating Weight: 134,095,100 lbs
Total Testing Weight: 134,095,100 lbs

Primary Bottom – See Bottom of Page for Secondary Bottom and Dead Shell

Bottom with Annular Ring Sloped Up 1.0000 in 120:
0.3750 in (A36) PL (weight: 656,750 lbs)

Annular Ring:
0.5000 in (A36 MOD) PL (weight: 50,680 lbs)

Outside projection = 2.0000 in

Total width: 38.0000 in

Shell

Shell Course 01: 1.1964 in (A573-70) PL x 119.5000 in wide (weight: 366,868 lbs)

Shell Course 1 requires impact testing per the standard.

Shell Course 02: 0.9386 in (A573-70) PL x 119.5000 in wide (weight: 287,815 lbs)

Shell Course 2 requires impact testing per the standard.

Shell Course 03: 0.7400 in (A573-70) PL x 119.5000 in wide (weight: 226,916 lbs)

Shell Course 04: 0.6240 in (A36 MOD) PL x 119.5000 in wide (weight: 191,345 lbs)

Shell Course 05: 0.4000 in (A36 MOD) PL x 118.0000 in wide (weight: 121,118 lbs)

Supported Cone Roof

0.1875 in (A36) PL (weight: 347,093 lbs)

Roof Slope: 0.7500 in 12 ø: 3.5763° Roof Rise: 7.5000 ft

Roof is single lap welded

Secondary Bottom

0.2360 in (A36 MOD) PL

Outside projection = 2.0000 in

Dead Shell or Vertical Spacer Plate

Dead Shell: 1.1964 inch (A573-70) PL x 1 ¼″ in

Inner Intermediate Girder for Three-Span System Girder: (8) W18x 46 (A992-GR50) x 30.6147 ft
(weight: 11,266 lbs)

Outer Intermediate Girder for Three-Span System Girder: (16) W18x 46 (A992-GR50) x 31.2145 ft
(weight: 22,974 lbs)

Inner Rafter for Three-Span System Rafter: (48) W10x 12 (A992-GR50) x 37.0956 ft
(weight: 21,367 lbs)

For Inner Rafter for Three-Span System rafters, (1) lateral bracing is required equal spaced.

Intermediate Rafter for Three-Span System Rafter: (80) W12x 19 (A992-GR50) x 44.5686 ft
(weight: 67,744 lbs)

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

Outer Rafter for Three-Span System Rafter: (160) W12x 16 (A992-GR50) x 42.6031 ft
(weight: 109,064 lbs)

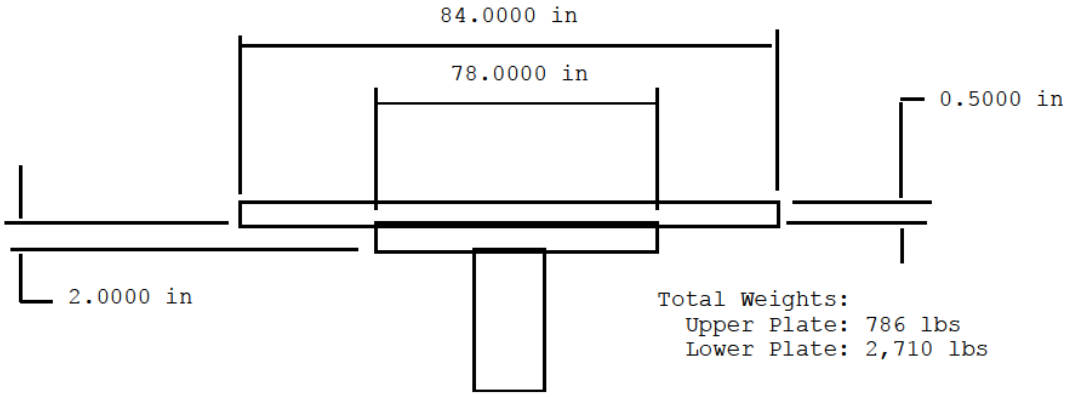
For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

Intermediate Column: (16) 10" Std Wt Pipe (A53-B) x 48.7979 ft
(Location Radius - 80.0000 ft)
(weight each: 1975 lbs, total weight: 31,605 lbs)

Intermediate Column: (8) 10" Std Wt Pipe (A53-B) x 51.2979 ft
(Location Radius - 40.0000 ft)
(weight each: 2077 lbs, total weight: 16,612 lbs)

Center Column: 14" Std Wt Pipe (A53-B) x 55.2817 ft (weight: 3,017 lbs)

Center Column Cap Plate: Material - A36



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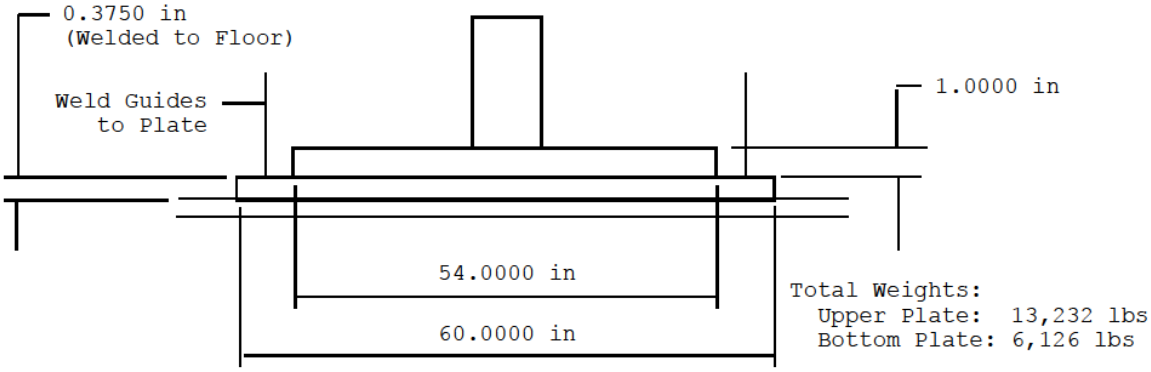
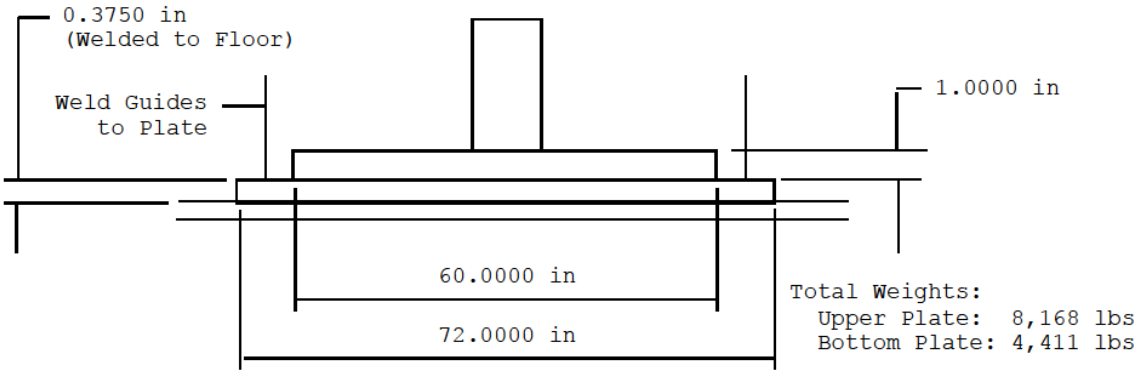
Outer Intermediate Column Base Plate description: Square Base Plate (A36)



Inner Intermediate Column Base Plate description: Square Base Plate (A36)



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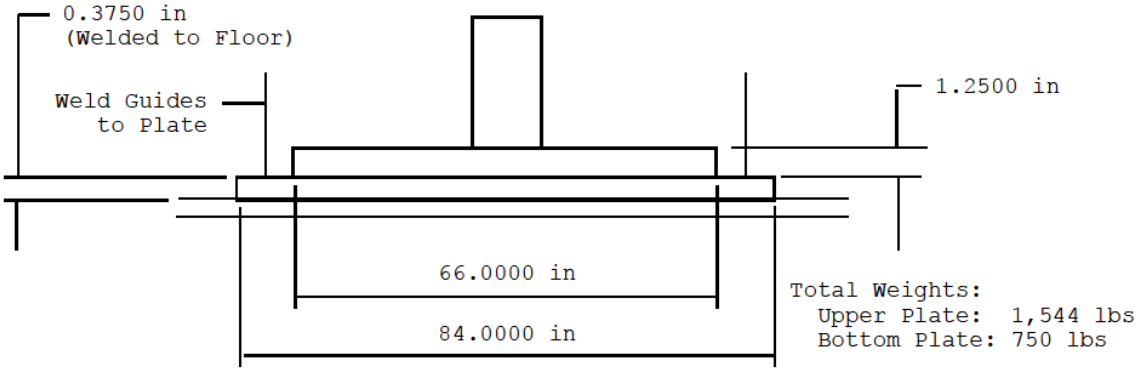
Center Column Base Plate description: Square Base Plate (A36)



Compression Ring

L4 x4 x3/ 8 per Detail d (A36)
Weight: 7,389 lbs

4 inches Vertical leg of rim angle is butt welded to the top of the shell with the horizontal leg turned out

Bottom Design

Bottom with Annular Ring Sloped Up 1.0000 in 120

Material Grade: A36
Thickness (t_b): 0.3750 in
Corrosion Allowance (CA): 0.1250 in
Minimum thickness per 5.4.1 = 0.2360 in

Minimum thickness + corrosion allowance = 0.2360 + 0.1250 = 0.3610 in

Annular Ring

Material Grade: A36 MOD
Thickness (t_b): 0.5000 in
Corrosion Allowance (CA): 0.1250 in
Minimum required outside projection per 5.4.2 = 2.0000 in
Actual outside projection (p) = 2.0000 in
Minimum thickness per 5.4.1 = 0.2360 in

t_d = 1.1963 in t_t = 0.9448 in t = 1.1964 in
CA = 0.1875 in S_d = 28000.0000 psi S_t = 30000.0000 psi

Annular ring width: 38.0000 in

$S_t = (t_t/t)S_t$ = (0.9448 /1.1964) * 30000.0000 = 23690 psi

$S_d = ((t_d - CA)/(t - CA))S_d$ =
((1.1963 - 0.1875)/(1.1964 - 0.1875)) * 28000.0000 = 27998 psi

From Table 5-1, minimum thickness = 0.3438 in

Minimum thickness + corrosion allowance = 0.3438 + 0.1250 = 0.4688 in

Minimum annular ring width inside shell per 5.5.2 = 24.0000 in
F_y = 36000.00 psi
Υ = 0.4333 psi/ft

Per 5.5.2, width = $2t_b \sqrt{(F_y/(2\Upsilon GH))}$ =
(2 * 0.5000)√(36000.00/(2 * 0.4333 * 1.0000 * 47.0000)) = 29.7287 in

Shell Design

Diameter (D) = 240.0000 ft
Specific Gravity (G) = 1.000

Shell Course 01

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 47.0000 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.3 and 5.6.3.2:

$$t_{pd} = \frac{(2.6)(47.0000 - 1.0)(240.0000)(1.0000)}{28000.0000} + 0.1875 = 1.2126 \text{ in}$$

For Design per 5.6.4.4:

$$t_{1d} = \left\{ 1.06 - \frac{(0.4630)(240.0000)}{47.0000} \sqrt{\left(\frac{(47.0000)(1.0000)}{28000.0000} \right)} \right\} \left\{ \frac{(2.6000)(47.0000)(240.0000)(1.0000)}{28000.0000} \right\} + 0.1875 = 1.1963 \text{ in}$$

t_{1d} (need not be greater than t_{pd}) = 1.1963 in

For Hydrotest per 5.6.4.3 and 5.6.3.2:

$$t_{pt} = \frac{(2.6)(47.0000 - 1.0)(240.0000)}{30000.0000} = 0.9568 \text{ in}$$

For Hydrotest per 5.6.4.4:

$$t_{1t} = \left\{ 1.06 - \frac{(0.4630)(240.0000)}{47.0000} \sqrt{\left(\frac{47.0000}{30000.0000} \right)} \right\} \left\{ \frac{(2.6000)(47.0000)(240.0000)}{30000.0000} \right\} = 0.9448 \text{ in}$$

t_{1t} (need not be greater than t_{pt}) = 0.9448 in

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 1.1964 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 1.1964 inches.

Shell Course 1 requires impact testing per the standard.

Shell Course 02

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 37.0417 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.5:

$$h_d = \frac{h_1}{\sqrt{(r\ t_1)}} = \frac{119.5000}{\sqrt{(1440.0000*1.0089)}} = 3.1352$$

For Design per 5.6.4.6:

t_u = 0.7510 t_L = 1.0089
K = t_L/t_u = 1.0089/0.7510 = 1.3434

$$C = \frac{(\sqrt{1.3434})(1.3434-1)}{1 + (1.3434)^{1.5}} = 0.1557$$

$x_1 = (0.61)\sqrt{(1440.0000*0.7510)} + (3.8400)(0.1557)(37.0417) = 42.2015$
$x_2 = (12)(0.1557)(37.0417) = 69.1924$
$x_3 = (1.22)\sqrt{(1440.0000*0.7510)} = 40.1198$
x = smaller of x_1, x_2, and x_3 = 40.1198

$$t_{dx} = \frac{(2.6)(240.0000)(37.0417 - 40.1198/12)(1.0000)}{28000.0000} = 0.7510 \text{ in}$$

$t_{2d} = t_{2a} = t_{dx} + CA = 0.7510 + 0.1875 = 0.9385$ in

For Hydrotest per 5.6.4.5:

$$h_t = \frac{h_1}{\sqrt{(r\ t_1)}} = \frac{119.5000}{\sqrt{(1440.0000*1.1964)}} = 2.8790$$

For Hydrotest per 5.6.4.6:

t_u = 0.7032 t_L = 1.1964
K = t_L/t_u = 1.1964/0.7032 = 1.7014

$$C = \frac{(\sqrt{1.7014})(1.7014-1)}{1 + (1.7014)^{1.5}} = 0.2842$$

$x_1 = (0.61)\sqrt{(1440.0000*0.7032)} + (3.8400)(0.2842)(37.0417) = 59.8353$
$x_2 = (12)(0.2842)(37.0417) = 126.3266$
$x_3 = (1.22)\sqrt{(1440.0000*0.7032)} = 38.8216$
x = smaller of x_1, x_2, and x_3 = 38.8216

$$t_{tx} = \frac{(2.6)(240.0000)(37.0417 - 38.8216/12)}{30000.0000} = 0.7032 \text{ in}$$

$t_{2t} = t_{2a} = t_{dt} = 0.7032$ in

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.9386 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.9386 inches.

Shell Course 2 requires impact testing per the standard.

Shell Course 03

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 27.0833 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.6:

t_u = 0.5404 t_L = 0.7511
K = t_L/t_u = 0.7511/0.5404 = 1.3900

$$C = \frac{(\sqrt{1.3900})(1.3900-1)}{1 + (1.3900)^{1.5}} = 0.1742$$

x_1 = (0.61)√(1440.0000*0.5404) + (3.8400)(0.1742)(27.0833) = 35.1367
x_2 = (12)(0.1742)(27.0833) = 56.6275
x_3 = (1.22)√(1440.0000*0.5404) = 34.0319
x = smaller of x_1, x_2, and x_3 = 34.0319

$$t_{dx} = \frac{(2.6)(240.0000)(27.0833 - 34.0319/12)(1.0000)}{28000.0000} + 0.1875 = 0.7279 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.5062 t_L = 0.9386
K = t_L/t_u = 0.9386/0.5062 = 1.8541

$$C = \frac{(\sqrt{1.8541})(1.8541-1)}{1 + (1.8541)^{1.5}} = 0.3300$$

x_1 = (0.61)√(1440.0000*0.5062) + (3.8400)(0.3300)(27.0833) = 50.7846
x_2 = (12)(0.3300)(27.0833) = 107.2339
x_3 = (1.22)√(1440.0000*0.5062) = 32.9396
x = smaller of x_1, x_2, and x_3 = 32.9396

$$t_{tx} = \frac{(2.6)(240.0000)(27.0833 - 32.9396/12)}{30000.0000} = 0.5062 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.7400 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.7400 inches.

Shell Course 04

Material Grade: A36 MOD

Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 23200.0000 psi

H = 17.1250 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 24900.0000 psi

For Design per 5.6.4.6:

t_u = 0.4031 t_L = 0.5525
K = t_L/t_u = 0.5525/0.4031 = 1.3706

$$C = \frac{(\sqrt{1.3706})(1.3706-1)}{1 + (1.3706)^{1.5}} = 0.1666$$

x_1 = (0.61)√(1440.0000*0.4031) + (3.8400)(0.1666)(17.1250) = 25.6509
x_2 = (12)(0.1666)(17.1250) = 34.2314
x_3 = (1.22)√(1440.0000*0.4031) = 29.3936
x = smaller of x_1, x_2, and x_3 = 25.6509

$$t_{dx} = \frac{(2.6)(240.0000)(17.1250 - 25.6509/12)(1.0000)}{23200.0000} + 0.1875 = 0.5906 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.3703 t_L = 0.7400
K = t_L/t_u = 0.7400/0.3703 = 1.9983

$$C = \frac{(\sqrt{1.9983})(1.9983-1)}{1 + (1.9983)^{1.5}} = 0.3690$$

x_1 = (0.61)√(1440.0000*0.3703) + (3.8400)(0.3690)(17.1250) = 38.3486
x_2 = (12)(0.3690)(17.1250) = 75.8194
x_3 = (1.22)√(1440.0000*0.3703) = 28.1728
x = smaller of x_1, x_2, and x_3 = 28.1728

$$t_{tx} = \frac{(2.6)(240.0000)(17.1250 - 28.1728/12)}{24900.0000} = 0.3703 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.6240 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.6240 inches.

Shell Course 05

Material Grade: A36 MOD

Shell Height: 9.8333 ft

Product Design Stress per Table 5-2 (S_d) = 23200.0000 psi

H = 7.1667 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 24900.0000 psi

For Design per 5.6.4.6:

t_u = 0.1523 t_L = 0.4365
K = t_L/t_u = 0.4365/0.1523 = 2.8667

$$C = \frac{(\sqrt{2.8667})(2.8667-1)}{1 + (2.8667)^{1.5}} = 0.5399$$

x_1 = (0.61)√(1440.0000*0.1523) + (3.8400)(0.5399)(7.1667) = 23.8913
x_2 = (12)(0.5399)(7.1667) = 46.4333
x_3 = (1.22)√(1440.0000*0.1523) = 18.0653
x = smaller of x_1, x_2, and x_3 = 18.0653

$$t_{dx} = \frac{(2.6)(240.0000)(7.1667 - 18.0653/12)(1.0000)}{23200.0000} + 0.1875 = 0.3398 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.1430 t_L = 0.6240
K = t_L/t_u = 0.6240/0.1430 = 4.3626

$$C = \frac{(\sqrt{4.3626})(4.3626-1)}{1 + (4.3626)^{1.5}} = 0.6946$$

x_1 = (0.61)√(1440.0000*0.1430) + (3.8400)(0.6946)(7.1667) = 27.8687
x_2 = (12)(0.6946)(7.1667) = 59.7318
x_3 = (1.22)√(1440.0000*0.1430) = 17.5089
x = smaller of x_1, x_2, and x_3 = 17.5089

$$t_{tx} = \frac{(2.6)(240.0000)(7.1667 - 17.5089/12)}{24900.0000} = 0.1430 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.4000 in

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Supported Cone Roof per API-650

Material Grade: A36
Roof Thickness (t_r): 0.1875 in
Corrosion Allowance (CA): 0.0000 in
Tank Diameter (D): 240.0000 ft

Roof is single lap welded

Roof Slope: 0.7500 in 12
ø: 3.5763°
Roof Rise: 7.5000 ft

Live Load (L_v): 25.0000 lbf/ft²
Snow Load (Sb): 25.2000 lbf/ft²
Dead Load (D_L): 7.6575 lbf/ft²
Ext. Pressure (P_e): 5.1960 lbf/ft²

Per 5.2.2(e)(1), $T = D_L + (L_v \text{ or } Sb) + 0.40P_e =$
$7.6575 + 25.2000 + (0.40)(5.1960) = 34.9359$ lbf/ft²

Per 5.2.2(e)(2), $T = D_L + 0.4(L_v \text{ or } Sb) + P_e =$
$7.6575 + (0.4)(25.2000) + 5.1960 = 22.9335$ lbf/ft²

Use greater of (1) or (2), T = 34.9359 lbf/ft²

Minimum thickness per 5.10.2.2 =0.1875 in

For Unbalanced Snow Load: Su = 1.0 Sb = 1.0 * 25.2000 = 25.2000 lbf/ft²

Per 5.2.2(e)(1), $U = D_L + Su + 0.40P_e =$
$7.6575 + 25.2000 + (0.40)(5.1960) = 34.9359$ lbf/ft²

Per 5.2.2(e)(2), $U = D_L + 0.4(Su) + P_e =$
$7.6575 + (0.4)(25.2000) + 5.1960 = 22.9335$ lbf/ft²

Use greater of (1) or (2), U = 34.9359 lbf/ft²

p = greater of T or U = 34.9359 lbf/ft²

Roof yield strength (Fy) = 36000.0000 psi

Maximum rafter spacing per 5.10.4.4,
$b = (t_r - CA)\sqrt{(1.5Fy * 144/p)}/12 =$
$(0.1875 - 0.0000)\sqrt{(1.5 * 36000.0000 * 144/34.9359)}/12 = 7.3716$ ft
Not to exceed 7.0000 ft
Therefore, b = 7.0000 ft

Quantity of rafters (Q) = 160
Actual rafter spacing (R) = π D/Q = π 240.0000/160 = 4.7124 ft

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Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Reactions for Inner Rafters

Total Rafter Length: 37.0956 ft
Rafter Weight: 12.0000 lbf/ft
Roof DL + LL: 34.9359 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Inner **Spacing (ft)**	Outer **Spacing (ft)**	Constant **Load (lbf/ft)**	Variable **Load (lbf/ft)**	**Inner Reaction (lbf)**	**Outer Reaction (lbf)**
35.5956	0.4581	5.1085	28.0058	162.4653		

(28.0058)(35.5956) + (162.4653)(35.5956)/2 - 2426.1270 = 1462.2849

(28.0058)(35.5956)/2 + (162.4653)(35.5956)/3 = 2426.1270

34.2459	0.4581	4.9322	28.0058	156.3047		

(28.0058)(34.2459) + (156.3047)(34.2459)/2 - 2263.8061 = 1371.6741

(28.0058)(34.2459)/2 + (156.3047)(34.2459)/3 = 2263.8061

33.5999	0.4581	4.8478	28.0058	153.3565		

(28.0058)(33.5999) + (153.3565)(33.5999)/2 - 2188.0846 = 1329.2907

(28.0058)(33.5999)/2 + (153.3565)(33.5999)/3 = 2188.0846

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Reactions for Intermediate Rafters

Total Rafter Length: 44.5686 ft
Rafter Weight: 19.0000 lbf/ft
Roof DL + LL: 34.9359 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Inner **Spacing (ft)**	Outer **Spacing (ft)**	Constant **Load (lbf/ft)**	Variable **Load (lbf/ft)**	**Inner Reaction (lbf)**	**Outer Reaction (lbf)**
42.5686	2.9039	6.2393	120.4520	116.5226		

(120.4520)(42.5686) + (116.5226)(42.5686)/2 - 4217.1360 = 3390.4356

(120.4520)(42.5686)/2 + (116.5226)(42.5686)/3 = 4217.1360

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Reactions for Outer Rafters

Total Rafter Length: 42.6031 ft
Rafter Weight: 16.0000 lbf/ft
Roof DL + LL: 34.9359 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Outer **Spacing** (ft)	Inner **Spacing** (ft)	Constant **Load** (lbf/ft)	Variable **Load** (lbf/ft)	**Inner Reaction** (lbf)	**Outer Reaction** (lbf)
40.3745	4.7124	3.1300	125.3484	55.2828		

(125.3484)(40.3745) + (55.2828)(40.3745)/2 - 3274.4481 = 2902.4450

(125.3484)(40.3745)/2 + (55.2828)(40.3745)/3 = 3274.4481

40.8698	4.7124	3.1106	124.6703	55.9609		

(124.6703)(40.8698) + (55.9609)(40.8698)/2 - 3309.9932 = 2928.8082

(124.6703)(40.8698)/2 + (55.9609)(40.8698)/3 = 3309.9932

41.2378	4.7124	3.0961	124.1663	56.4649		

(124.1663)(41.2378) + (56.4649)(41.2378)/2 - 3336.3389 = 2948.2570

(124.1663)(41.2378)/2 + (56.4649)(41.2378)/3 = 3336.3389

41.4816	4.7124	3.0866	123.8325	56.7987		

(123.8325)(41.4816) + (56.7987)(41.4816)/2 - 3353.7556 = 2961.0715

(123.8325)(41.4816)/2 + (56.7987)(41.4816)/3 = 3353.7556

41.6031	4.7124	3.0818	123.6662	56.9650		

(123.6662)(41.6031) + (56.9650)(41.6031)/2 - 3362.4203 = 2967.4337

(123.6662)(41.6031)/2 + (56.9650)(41.6031)/3 = 3362.4203

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Loadings and Moments for Intermediate Girders

Total Girder Length: 30.6147 ft
Girder Weight: 46.0000 lbf/ft

Description	Load on Girder (lbf)	Distance from Girder Centerline to Load Centerline (ft)	Bending Moment (ft-lb)	Load on Column (lbf)
Girder	1408.28			704.14
		$(46.0000)(30.6147)^2/8 =$	5389.24	
Intermediate Rafter	3409.44	13.6335		3409.44
		$(3409.4356)(30.6147/2 - 13.6335) =$	5706.89	
Intermediate Rafter	3409.44	10.4224		3409.44
		$(3409.4356)(30.6147/2 - 10.4224) =$	16654.75	
Intermediate Rafter	3409.44	7.3508		3409.44
		$(3409.4356)(30.6147/2 - 7.3508) =$	27127.16	
Intermediate Rafter	3409.44	4.3739		3409.44
		$(3409.4356)(30.6147/2 - 4.3739) =$	37276.73	
Intermediate Rafter	3409.44	1.4520		3409.44
		$(3409.4356)(30.6147/2 - 1.4520) =$	47238.97	
Inner Rafter	2438.13	12.5446		2438.13
		$(2438.1270)(30.6147/2 - 12.5446) =$	6735.92	
Inner Rafter	2292.00	7.3508		2292.00
		$(2292.0031)(30.6147/2 - 7.3508) =$	18236.31	
Inner Rafter	2224.03	2.4222		2224.03
		$(2224.0331)(30.6147/2 - 2.4222) =$	28657.04	
		Total:	193023.00	24705.48
			Total load on Column:	49410.96

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Loadings and Moments for Outer Girders

Total Girder Length: 31.2145 ft
Girder Weight: 46.0000 lbf/ft

Description	Load on Girder (lbf)	Distance from Girder Centerline to Load Centerline (ft)	Bending Moment (ft-lb)	Load on Column (lbf)
Girder	1435.86			717.93
		(46.0000)(31.2145)²/8 =	5602.47	
Outer Rafter	2938.10	14.0117		2938.10
		(2938.1015)(31.2145/2 - 14.0117) =	4687.87	
Outer Rafter	2956.54	10.8527		2956.54
		(2956.5410)(31.2145/2 - 10.8527) =	14056.88	
Outer Rafter	2970.10	7.7279		2970.10
		(2970.1007)(31.2145/2 - 7.7279) =	23402.35	
Outer Rafter	2979.01	4.6272		2979.01
		(2979.0144)(31.2145/2 - 4.6272) =	32709.67	
Outer Rafter	2983.43	1.5408		2983.43
		(2983.4337)(31.2145/2 - 1.5408) =	41966.21	
Intermediate Rafter	4236.14	12.4273		4236.14
		(4236.1360)(31.2145/2 - 12.4273) =	13470.64	
Intermediate Rafter	4236.14	6.1752		4236.14
		(4236.1360)(31.2145/2 - 6.1752) =	39955.52	
Intermediate Rafter	4236.14	0.0000		2118.07
		(4236.1360)(31.2145/4) =	33057.17	
		Total:	208908.78	26135.46
		Total load on Column:		52270.93

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Inner Intermediate Girder for Three-Span System

Girder description: W18x 46
Quantity (Q): 8
Material grade: A992-GR50
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 712.0000 in^4
S_x = 78.8000 in³
Area (A) = 13.5000 in²
Radius of Gyration (r) = 7.2500 in

Girder length:
Distance from tank centerline to column centerline (C_o) = 40.0000 ft

Girder length (L) = 2(C_o)*sin(360/(2Q) =
2 * (40.0000) * sin(360/(2 * 8)) = 30.6147 ft

Girder unsupported length L_1 = 6.2393 ft

L/r = 12 * 30.6147/7.2500 = 10.3271

Bending Moment due to rafter loads and girder weight:
M = 193022.9998 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 209739.9815 ft-lb

Per AISC360-10:
Allowable Flexure Strength (M_c) = 209739.9815 ft-lb

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Outer Intermediate Girder for Three-Span System

Girder description: W18x 46
Quantity (Q): 16
Material grade: A992-GR50
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 712.0000 in^4
S_x = 78.8000 in^3
Area (A) = 13.5000 in^2
Radius of Gyration (r) = 7.2500 in

Girder length:
Distance from tank centerline to column centerline (C_o) = 80.0000 ft

Girder length (L) = 2(C_o)*sin(360/(2Q) =
2 * (80.0000) * sin(360/(2 * 16)) = 31.2145 ft

Girder unsupported length L_1 = 3.1300 ft

L/r = 12 * 31.2145/7.2500 = 5.1806

Bending Moment due to rafter loads and girder weight:
M = 208908.7774 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 226297.4052 ft-lb

Per AISC360-10:
Allowable Flexure Strength (M_c) = 226297.4052 ft-lb

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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Inner Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W10x 12
Quantity (Q): 48
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 53.8000 in^4
S_x = 10.9000 in^3
Area (A) = 3.5400 in^2
Radius of Gyration (r) = 3.9000 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 3.5000 ft
Distance from tank centerline to outer edge of rafter (D_o) = 39.0263 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(3.5000/48)$ = 0.4581 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(39.0263/48)$ = 5.1085 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(39.0263 - 3.5000)/cos(3.5763) = 35.5956 ft

For Inner Rafter for Three-Span System rafters, (1) lateral bracing is required equal spaced.

L_1 = L/2 = 35.5956/2 = 17.7978 ft

L/r = 12 * 35.5956/3.9000 = 109.5250

For dead load only:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(7.6575) + 12.0000 = 15.5083 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(7.6575) = 35.6103 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[35.6103/(2*35.5956)x² + (15.5083)x -
(15.5083*35.5956/2 + 35.6103*35.5956/6) = 0

(0.5002)x² + (15.5083)x - (487.2752) = 0
therefore x = 19.3472 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(15.5083)(19.3472)(35.5956 - 19.3472)/2 +
(35.6103)(19.3472)(35.5956² - 19.3472²)/(6*35.5956) = 5317.4402 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 5590.1422 ft-lb

For dead load + live load:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(34.9359) + 12.0000 = 28.0058 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(34.9359) = 162.4653 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[162.4653/(2*35.5956)x² + (28.0058)x -
(28.0058*35.5956/2 + 162.4653*35.5956/6) = 0

(2.2821)x² + (28.0058)x - (1462.2849) = 0
therefore x = 19.9104 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(28.0058)(19.9104)(35.5956 - 19.9104)/2 +
(162.4653)(19.9104)(35.5956² - 19.9104²)/(6*35.5956) = 17559.4155 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 31437.1257 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (28.0058)(35.5956)/2 + (162.4653)(35.5956)/3 = 2426.1270$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(28.0058)(35.5956) + (162.4653)(35.5956)/2 - 2426.1270 = 1462.2849$ lbf

Intermediate Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W12x 19
Quantity (Q): 80
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 130.0000 in^4
S_x = 21.3000 in^3
Area (A) = 5.5700 in^2
Radius of Gyration (r) = 4.8200 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 36.9552 ft
Distance from tank centerline to outer edge of rafter (D_o) = 79.4409 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(36.9552/80)$ = 3.1416 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(79.4409/80)$ = 6.2393 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(79.4409 - 36.9552)/cos(3.5763) = 42.5686 ft

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 42.5686/3 = 14.1895 ft

L/r = 12 * 42.5686/4.8200 = 105.9799

For dead load only:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(7.6575) + 19.0000 = 43.0567 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(7.6575) = 23.7205 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[23.7205/(2*42.5686)x² + (43.0567)x -
(43.0567*42.5686/2 + 23.7205*42.5686/6) = 0

(0.2786)x² + (43.0567)x - (1084.7237) = 0
therefore x = 22.0475 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(43.0567)(22.0475)(42.5686 - 22.0475)/2 +
(23.7205)(22.0475)(42.5686² - 22.0475²)/(6*42.5686) = 12455.3535 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 17330.4213 ft-lb

For dead load + live load:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(34.9359) + 19.0000 = 128.7544 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(34.9359) = 108.2202 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[108.2202/(2*42.5686)x² + (128.7544)x -
(128.7544*42.5686/2 + 108.2202*42.5686/6) = 0

(1.2711)x² + (128.7544)x - (3508.2427) = 0
therefore x = 22.3264 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(128.7544)(22.3264)(42.5686 - 22.3264)/2 +
(108.2202)(22.3264)(42.5686² - 22.3264²)/(6*42.5686) = 41520.9970 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 61626.7465 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (128.7544)(42.5686)/2 + (108.2202)(42.5686)/3 = 4276.0396$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(128.7544)(42.5686) + (108.2202)(42.5686)/2 - 4276.0396 = 3508.2427$ lbf

Outer Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W12x 16
Quantity (Q): 160
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 103.0000 in^4
S_x = 17.1000 in^3
Area (A) = 4.7100 in^2
Radius of Gyration (r) = 4.6700 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 78.4779 ft
Distance from tank centerline to outer edge of rafter (D_o) = 120.0000 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = 2π(78.4779/160) = 3.0818 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = 2π(120.0000/160) = 4.7124 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(120.0000 - 78.4779)/cos(3.5763) = 41.6031 ft

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 41.6031/3 = 13.8677 ft

L/r = 12 * 41.6031/4.6700 = 106.9030

For dead load only:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(7.6575) + 16.0000 = 39.5991 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(7.6575) = 12.4860 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[12.4860/(2*41.6031)x² + (39.5991)x -
(39.5991*41.6031/2 + 12.4860*41.6031/6) = 0

(0.1501)x² + (39.5991)x - (910.2970) = 0
therefore x = 21.2729 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(39.5991)(21.2729)(41.6031 - 21.2729)/2 +
(12.4860)(21.2729)(41.6031² - 21.2729²)/(6*41.6031) = 9923.1198 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 12405.6104 ft-lb

For dead load + live load:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(34.9359) + 16.0000 = 123.6662 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(34.9359) = 56.9650 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[56.9650/(2*41.6031)x² + (123.6662)x -
(123.6662*41.6031/2 + 56.9650*41.6031/6) = 0

(0.6846)x² + (123.6662)x - (2967.4337) = 0
therefore x = 21.4487 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(123.6662)(21.4487)(41.6031 - 21.4487)/2 +
(56.9650)(21.4487)(41.6031² - 21.4487²)/(6*41.6031) = 32949.6511 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 50149.7006 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (123.6662)(41.6031)/2 + (56.9650)(41.6031)/3 = 3362.4203$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(123.6662)(41.6031) + (56.9650)(41.6031)/2 - 3362.4203 = 2967.4337$ lbf

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Inner Rafter for Three-Span System

Rafter description: W10x 12
Quantity (Q): 48
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 53.8000 in^4
S_x = 10.9000 in^3
Area (A) = 3.5400 in^2
Radius of Gyration (r) = 3.9000 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 3.5000 ft
Distance from tank centerline to outer edge of rafter (D_o) = 39.0263 ft

Inner spacing (S_i) = $2\pi D_i/Q = 2\pi(3.5000/48) = 0.4581$ ft
Outer spacing (S_o) = $2\pi D_o/Q = 2\pi(39.0263/48) = 5.1085$ ft

Rafter length (L) = $(D_o - D_i)/\cos(3.5763)$ =
$(39.0263 - 3.5000)/\cos(3.5763) = 35.5956$ ft

For Inner Rafter for Three-Span System rafters, (1) lateral bracing is required equal spaced.

L_1 = L/2 = 35.5956/2 = 17.7978 ft

L/r = 12 * 35.5956/3.9000 = 109.5250

For dead load only:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(7.6575) + 12.0000 = 15.5083 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(7.6575) = 35.6103 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[35.6103/(2*35.5956)x² + (15.5083)x -
(15.5083*35.5956/2 + 35.6103*35.5956/6) = 0

(0.5002)x² + (15.5083)x - (487.2752) = 0
therefore x = 19.3472 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(15.5083)(19.3472)(35.5956 - 19.3472)/2 +
(35.6103)(19.3472)(35.5956² - 19.3472²)/(6*35.5956) = 5317.4402 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 5590.1422 ft-lb

For dead load + live load:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(34.9359) + 12.0000 = 28.0058 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(34.9359) = 162.4653 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[162.4653/(2*35.5956)x² + (28.0058)x -
(28.0058*35.5956/2 + 162.4653*35.5956/6) = 0

(2.2821)x² + (28.0058)x - (1462.2849) = 0
therefore x = 19.9104 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(28.0058)(19.9104)(35.5956 - 19.9104)/2 +
(162.4653)(19.9104)(35.5956² - 19.9104²)/(6*35.5956) = 17559.4155 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 31437.1257 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (28.0058)(35.5956)/2 + (162.4653)(35.5956)/3 = 2426.1270$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(28.0058)(35.5956) + (162.4653)(35.5956)/2 - 2426.1270 = 1462.2849$ lbf

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202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Intermediate Rafter for Three-Span System

Rafter description: W12x 19
Quantity (Q): 80
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 130.0000 in^4
S_x = 21.3000 in^3
Area (A) = 5.5700 in^2
Radius of Gyration (r) = 4.8200 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 36.9552 ft
Distance from tank centerline to outer edge of rafter (D_o) = 79.4409 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(36.9552/80)$ = 3.1416 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(79.4409/80)$ = 6.2393 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(79.4409 - 36.9552)/cos(3.5763) = 42.5686 ft

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 42.5686/3 = 14.1895 ft

L/r = 12 * 42.5686/4.8200 = 105.9799

For dead load only:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(7.6575) + 19.0000 = 43.0567 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(7.6575) = 23.7205 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[23.7205/(2*42.5686)x² + (43.0567)x -
(43.0567*42.5686/2 + 23.7205*42.5686/6) = 0

(0.2786)x² + (43.0567)x - (1084.7237) = 0
therefore x = 22.0475 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(43.0567)(22.0475)(42.5686 - 22.0475)/2 +
(23.7205)(22.0475)(42.5686² - 22.0475²)/(6*42.5686) = 12455.3535 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 17330.4213 ft-lb

For dead load + live load:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(34.9359) + 19.0000 = 128.7544 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(34.9359) = 108.2202 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[108.2202/(2*42.5686)x² + (128.7544)x -
(128.7544*42.5686/2 + 108.2202*42.5686/6) = 0

(1.2711)x² + (128.7544)x - (3508.2427) = 0
therefore x = 22.3264 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(128.7544)(22.3264)(42.5686 - 22.3264)/2 +
(108.2202)(22.3264)(42.5686² - 22.3264²)/(6*42.5686) = 41520.9970 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 61626.7465 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (128.7544)(42.5686)/2 + (108.2202)(42.5686)/3 = 4276.0396$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(128.7544)(42.5686) + (108.2202)(42.5686)/2 - 4276.0396 = 3508.2427$ lbf

Outer Rafter for Three-Span System

Rafter description: W12x 16
Quantity (Q): 160
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 103.0000 in^4
S_x = 17.1000 in^3
Area (A) = 4.7100 in^2
Radius of Gyration (r) = 4.6700 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 78.4779 ft
Distance from tank centerline to outer edge of rafter (D_o) = 120.0000 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(78.4779/160)$ = 3.0818 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(120.0000/160)$ = 4.7124 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(120.0000 - 78.4779)/cos(3.5763) = 41.6031 ft

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 41.6031/3 = 13.8677 ft

L/r = 12 * 41.6031/4.6700 = 106.9030

For dead load only:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load (D_L) = 7.6575 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(7.6575) + 16.0000 = 39.5991 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(7.6575) = 12.4860 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[12.4860/(2*41.6031)x² + (39.5991)x -
(39.5991*41.6031/2 + 12.4860*41.6031/6) = 0

(0.1501)x² + (39.5991)x - (910.2970) = 0
therefore x = 21.2729 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(39.5991)(21.2729)(41.6031 - 21.2729)/2 +
(12.4860)(21.2729)(41.6031² - 21.2729²)/(6*41.6031) = 9923.1198 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 12405.6104 ft-lb

For dead load + live load:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 34.9359 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(34.9359) + 16.0000 = 123.6662 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(34.9359) = 56.9650 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[56.9650/(2*41.6031)x² + (123.6662)x -
(123.6662*41.6031/2 + 56.9650*41.6031/6) = 0

(0.6846)x² + (123.6662)x - (2967.4337) = 0
therefore x = 21.4487 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(123.6662)(21.4487)(41.6031 - 21.4487)/2 +
(56.9650)(21.4487)(41.6031² - 21.4487²)/(6*41.6031) = 32949.6511 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 50149.7006 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (123.6662)(41.6031)/2 + (56.9650)(41.6031)/3 = 3362.4203$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(123.6662)(41.6031) + (56.9650)(41.6031)/2 - 3362.4203 = 2967.4337$ lbf

Inner Intermediate Girder for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Girder description: W18x 46
Quantity (Q): 8
Material grade:
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

I_x = 712.0000 in^4
S_x = 78.8000 in³
Zx = 90.7000 in³
r_x = 7.2500 in
I_y = 22.5000 in^4
S_y = 7.4300 in³
Z_y = 8.0000 in³
r_y = 1.2900 in
J = 1.2200 in^4
C_w = 1720.0000 in^6
A = 13.5000 in²

Unsupported Girder length (L_b) = 74.8713 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 3.0300 in
b/t_f = 5.0083

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 46.9167$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 90.7000 = 4535000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 1.2900 \sqrt{\frac{28800000.0000}{50000.0000}} = 54.4896$ in

$h_o = d - t_f = 18.1000 - 0.6050 = 17.4950$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{22.5000 * 1720.0000}}{78.8000}} = 1.5800$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$1.95 * 1.5800 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{1.2200 * 1.0000}{78.8000 * 17.4950} + \sqrt{\left(\frac{1.2200 * 1.0000}{78.8000 * 17.4950}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 163.6428$ in

$L_p < L_b \leq L_r$

$$(F2\text{-}2)\ M_n = C_b \left[M_p - (M_p - 0.7F_yS_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] =$$

$$1.0000 \left[4535000.0000 - (4535000.0000 - 0.7 * 50000.0000 * 78.8000) \left(\frac{74.8713 - 54.4896}{163.6428 - 54.4896} \right) \right] = 4203189.2296 \text{ in-lb}$$

For dead load + live load:

Minimmum Mn = 4203189.2296 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 4203189.2296/1.67 = 2516879.7782 in-lb
= 2516879.7782/12 = 209739.9815 ft-lb

Outer Intermediate Girder for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Girder description: W18x 46
Quantity (Q): 16
Material grade:
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

I_x = 712.0000 in^4
S_x = 78.8000 in³
Zx = 90.7000 in³
r_x = 7.2500 in
I_y = 22.5000 in^4
S_y = 7.4300 in³
Z_y = 16.0000 in³
r_y = 1.2900 in
J = 1.2200 in^4
C_w = 1720.0000 in^6
A = 13.5000 in²

Unsupported Girder length (L_b) = 37.5597 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 3.0300 in
b/t_f = 5.0083

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 46.9167$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 90.7000 = 4535000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 1.2900 \sqrt{\frac{28800000.0000}{50000.0000}} = 54.4896$ in

The limit state of lateral-torsional buckling does not apply per F2.2(a).

For dead load + live load:

Minimmum $M_n = 4535000.0000$ in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega b = 4535000.0000/1.67 = 2715568.8623$ in-lb
$= 2715568.8623/12 = 226297.4052$ ft-lb

Hagen Engineering International
202 S. Cedar St. Suite B
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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Inner Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W10x 12
Quantity (Q): 48
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

I_x = 53.8000 in^4
S_x = 10.9000 in³
Zx = 12.6000 in³
r_x = 3.9000 in
I_y = 2.1800 in^4
S_y = 1.1000 in³
Z_y = 1.7400 in³
r_y = 0.7850 in
J = 0.0547 in^4
C_w = 50.9000 in^6
A = 3.5400 in²

Unsupported Rafter length (L_b) = 213.5737 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 1.9800 in
b/t_f = 9.4286

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Non-Compact

Web classification (AISC 16.1 B4)

$h/t_w = 49.7368$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 12.6000 = 630000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.7850 \sqrt{\frac{28800000.0000}{50000.0000}} = 33.1584$ in

$h_o = d - t_f = 9.8700 - 0.2100 = 9.6600$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.1800 * 50.9000}}{10.9000}} = 0.9831$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$1.95 * 0.9831 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.0547 * 1.0000}{10.9000 * 9.6600} + \sqrt{\left(\frac{0.0547 * 1.0000}{10.9000 * 9.6600}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 96.2291$ in

$$(F2\text{-}4)\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\,\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} =$$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\frac{213.5737}{0.9831}\right)^2}\sqrt{1 + 0.078\,\frac{0.0547 * 1.0000}{10.9000 * 9.6600}\left(\frac{213.5737}{0.9831}\right)^2}$$

$$= 10277.6560 \text{ psi}$$

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 10277.6560 * 10.9000 = 112026.4500$ in-lb $\leq M_p$

Compression Flange Local Buckling:

Per F3.2:

$\lambda = b_f/(2t_f) = 3.9600/(2 * 0.2100) = 9.4286$

$k_c = 4/\sqrt{(h/t_w)} = 4/\sqrt{(9.8700/0.1900)} = 377245.5090$
but not less than 0.35 nor greater than 0.76
therefore, $k_c = 0.5550$

$\lambda_{pf} = \lambda_p = 9.1200$

$\lambda_{rf} = \lambda_r = 24.0000$

$$(F3\text{-}1)\ M_n = M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) =$$

$$630000.0000 - (630000.0000 - 0.7 * 50000.0000 * 10.9000)$$

$$\left(\frac{9.4286 - 9.1200}{9.1200 - 24.0000}\right) = 629346.7742 \text{ in-lb}$$

For dead load only:

Minimmum Mn = 112026.4500 in-lb

Allowable Flexure Strength = M_c = Mn/Ωb = 112026.4500/1.67 = 67081.7066 in-lb
= 67081.7066/12 = 5590.1422 ft-lb

For dead load + live load:

Minimmum Mn = 630000.0000 in-lb

Allowable Flexure Strength = M_c = Mn/Ωb = 630000.0000/1.67 = 377245.5090 in-lb
= 377245.5090/12 = 31437.1257 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Intermediate Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W12x 19
Quantity (Q): 80
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

I_x = 130.0000 in^4
S_x = 21.3000 in³
Zx = 24.7000 in³
r_x = 4.8200 in
I_y = 3.7600 in^4
S_y = 1.8800 in³
Z_y = 2.9800 in³
r_y = 0.8220 in
J = 0.1800 in^4
C_w = 131.0000 in^6
A = 5.5700 in²

Unsupported Rafter length (L_b) = 170.2744 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 2.0050 in
b/t_f = 5.7286

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 48.9362$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 24.7000 = 1235000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.8220 \sqrt{\frac{28800000.0000}{50000.0000}} = 34.7213$ in

$h_o = d - t_f = 12.2000 - 0.3500 = 11.8500$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{3.7600 * 131.0000}}{21.3000}} = 1.0208$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$$1.95 * 1.0208 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.1800 * 1.0000}{21.3000 * 11.8500} + \sqrt{\left(\frac{0.1800 * 1.0000}{21.3000 * 11.8500}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 102.9701 \text{ in}$$

$$(F2\text{-}4)\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\,\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} =$$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\frac{170.2744}{1.0208}\right)^2}\sqrt{1 + 0.078\,\frac{0.1800 * 1.0000}{21.3000 * 11.8500}\left(\frac{170.2744}{1.0208}\right)^2}$$

$$= 16305.2415 \text{ psi}$$

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 16305.2415 * 21.3000 = 347301.6433$ in-lb $\leq M_p$

For dead load only:

Minimmum M_n = 347301.6433 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 347301.6433/1.67 = 207965.0559 in-lb
= 207965.0559/12 = 17330.4213 ft-lb

For dead load + live load:

Minimmum M_n = 1235000.0000 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 1235000.0000/1.67 = 739520.9581 in-lb
= 739520.9581/12 = 61626.7465 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Hagen Engineering International
202 S. Cedar St. Suite B
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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Outer Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W12x 16
Quantity (Q): 160
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

I_x = 103.0000 in^4
S_x = 17.1000 in³
Zx = 20.1000 in³
r_x = 4.6700 in
I_y = 2.8200 in^4
S_y = 1.4100 in³
Z_y = 2.2600 in³
r_y = 0.7730 in
J = 0.1030 in^4
C_w = 96.9000 in^6
A = 4.7100 in²

Unsupported Rafter length (L_b) = 166.4123 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 1.9950 in
b/t_f = 7.5283

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 52.1364$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 20.1000 = 1005000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.7730 \sqrt{\frac{28800000.0000}{50000.0000}} = 32.6515$ in

$h_o = d - t_f = 12.0000 - 0.2650 = 11.7350$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.8200 * 96.9000}}{17.1000}} = 0.9832$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$$1.95 * 0.9832 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.1030 * 1.0000}{17.1000 * 11.7350} + \sqrt{\left(\frac{0.1030 * 1.0000}{17.1000 * 11.7350}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 96.1502 \text{ in}$$

(F2-4) $F_{cr} = \dfrac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \dfrac{J_c}{S_x h_o} \left(\dfrac{L_b}{r_{ts}}\right)^2} =$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\dfrac{166.4123}{0.9832}\right)^2} \sqrt{1 + 0.078 \frac{0.1030 * 1.0000}{17.1000 * 11.7350} \left(\frac{166.4123}{0.9832}\right)^2}$$

= 14538.5049 psi

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 14538.5049 * 17.1000 = 248608.4333$ in-lb $\leq M_p$

For dead load only:

Minimmum M_n = 248608.4333 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 248608.4333/1.67 = 148867.3253 in-lb
= 148867.3253/12 = 12405.6104 ft-lb

For dead load + live load:

Minimmum M_n = 1005000.0000 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 1005000.0000/1.67 = 601796.4072 in-lb
= 601796.4072/12 = 50149.7006 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Intermediate Column for Supported Cone Roof

(16) Columns located on 80.0000 ft Radius

Column description: 10" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 10.7500 in
Wall thickness (t_w) = 0.3650 in
Mill tolerance (m) = 12.50%

therefore $tw = t_w(1 - m) = 0.3650(1 - 0.1250) = 0.3194$ in

$t = t_w = 0.3194$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 10.7500 - 2*0.3194 = 10.1113$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(10.7500^2 - 10.1113^2)/4 = 10.4655$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(10.7500^2 + 10.1113^2)/4} = 3.6895$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((10.7500)^4 - (10.1113)^4)/10.7500 = 26.5046$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Column Location Radius (R_2) = 80.0000 ft

Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 80.0000)\tan(0.0624) = 2.5000$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(Rise)/Run = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 12.2000 in
Girder depth (L_6) = 18.1000 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 - L_6 =$
$(2.5000 + -1.0000 + 50.0000)12 - 12.2000 - 18.1000 = 585.5750$ in

KL/r = (1.00)(585.5750)/3.6895 = 158.7134

For Outer Intermediate Column, KL/r = 158.7

Calculate column load:

P (from girder loads) = 52270.9273 lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:

$\Omega_c = 1.67$

$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(158.7134)^2} = 11284.0454 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 11284.0454 = 9896.1078$ psi

(E3.1) $P_n = F_{cr}A = 9896.1078 * 10.4655 = 103567.9884$ lbf

$P_c = P_n/\Omega_c = 103567.9884/1.67 = 62016.7595$ lbf

For Outer Intermediate Column, load (52270.93) is less than the allowable load (62016.76).

Intermediate Column for Supported Cone Roof

(8) Columns located on 40.0000 ft Radius

Column description: 10" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 10.7500 in
Wall thickness (t_w) = 0.3650 in
Mill tolerance (m) = 12.50%

therefore $t_w = t_w(1 - m) = 0.3650(1 - 0.1250) = 0.3194$ in

$t = t_w = 0.3194$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 10.7500 - 2*0.3194 = 10.1113$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(10.7500^2 - 10.1113^2)/4 = 10.4655$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(10.7500^2 + 10.1113^2)/4} = 3.6895$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((10.7500)^4 - (10.1113)^4)/10.7500 = 26.5046$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Column Location Radius (R_2) = 40.0000 ft

Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 40.0000)\tan(0.0624) = 5.0000$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(\text{Rise})/\text{Run} = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 12.2000 in
Girder depth (L_6) = 18.1000 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 - L_6 =$
$(5.0000 + -1.0000 + 50.0000)12 - 12.2000 - 18.1000 = 615.5750$ in

KL/r = (1.00)(615.5750)/3.6895 = 166.8446

For Inner Intermediate Column, KL/r = 166.8

Calculate column load:

P (from girder loads) = 49410.9574 lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:

$\Omega_c = 1.67$

$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(166.8446)^2} = 10210.9919 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 10210.9919 = 8955.0399$ psi

(E3.1) $P_n = F_{cr}A = 8955.0399 * 10.4655 = 93719.2162$ lbf

$P_c = P_n/\Omega_c = 93719.2162/1.67 = 56119.2911$ lbf

For Inner Intermediate Column, load (49410.96) is less than the allowable load (56119.29).

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Center Column for Supported Cone Roof

Column description: 14" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 14.0000 in
Wall thickness (t_w) = 0.3750 in
Mill tolerance (m) = 12.50%

therefore $t_w = t_w(1 - m) = 0.3750(1 - 0.1250) = 0.3281$ in

$t = t_w = 0.3281$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 14.0000 - 2*0.3281 = 13.3438$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(14.0000^2 - 13.3438^2)/4 = 14.0934$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(14.0000^2 + 13.3438^2)/4} = 4.8351$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((14.0000)^4 - (13.3438)^4)/14.0000 = 47.0692$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Cap Radius (R_2) = 3.5000 ft
Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 3.5000)\tan(0.0624) = 7.2813$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(\text{Rise})/\text{Run} = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 9.8700 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 =$
$(7.2813 + -1.0000 + 50.0000)12 - 9.8700 = 663.3800$ in

KL/r = (1.00)(663.3800)/4.8351 = 137.2000

For center column, KL/r = 137.2

Calculate column load:

Quantity of rafters (Q) = 48
Total load from Reactions for Inner Rafters (R_i) = 66611.9960 lbf
Rafter extension on cap (L_6) = 6.0000 in
Rafter weight (R_w) = 12.0000 lbf
Roof dead load + live load (D_L) = 34.9359 lbf/ft²
Weight of cap plate (C_w) = 3496.2470 lbf

$P = R_i + QR_w(L_6/12) + \pi R_2^2 D_L + C_w =$
$66611.9960 + (48)(12.0000)(6.0000/12) +$
$\pi(3.5000)^2(34.9359) + 3496.2470 = 71740.7340$ lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:
$\Omega_c = 1.67$
$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(137.2000)^2} = 15100.2361 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 15100.2361 = 13242.9071$ psi

(E3.1) $P_n = F_{cr}A = 13242.9071 * 14.0934 = 186638.2285$ lbf

$P_c = P_n/\Omega_c = 186638.2285/1.67 = 111759.4183$ lbf

Load on center column (71740.73) is less than the allowable load (111759.42).

Center Column for Supported Cone Roof (Unbalanced Snow Load)

Column description: 14" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Cap Radius (R_2) = 3.5000 ft

KL/r = (1.00)(663.3800)/4.8351 = 137.2000

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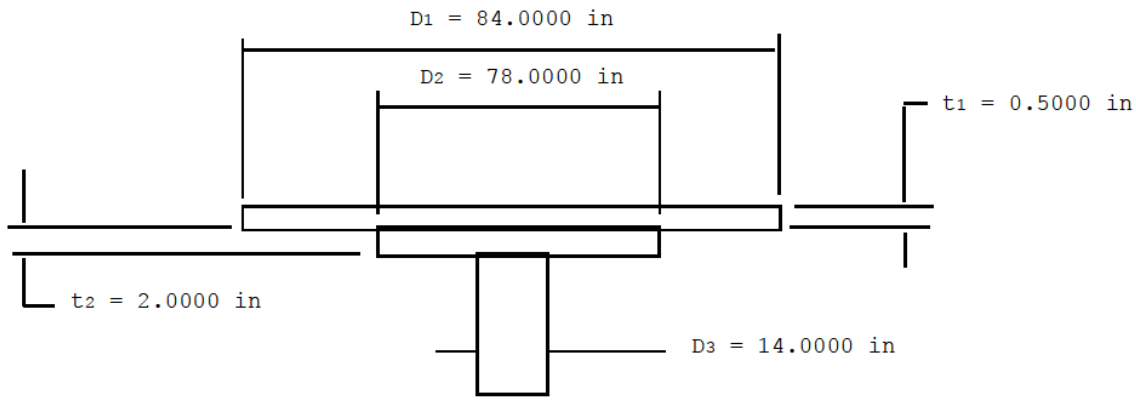
Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Center Column Cap Plate

Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 68244.4870 lbf



Top Plate:
$E = D_1/2 = 84.0000/2 = 42.0000$ in
$G = D_2/2 = 78.0000/2 = 39.0000$ in

$F_b = P_a[1.17 + 10*\log(E/G) - 1.17G^2/E^2]/(7 * (t_1 - CA)^2) =$
$68244.4870 * [1.17 + 10*\log(42.0000/39.0000) -$
$1.17*(39.0000)^2/(42.0000)^2]/(7 * (0.5000 - 0.0000)^2) = 18836.2702$ psi

$F_a = 0.75F_y = 0.75 * 36000.0000 = 27000.0000$ psi

Secondary Plate:
$E = D_2/2 = 84.0000/2 = 39.0000$ in
$G = D_3/2 = 14.0000/2 = 7.0000$ in

$F_b = P_a[1.17 + 10*\log(E/G) - 1.17G^2/E^2]/(7 * (t_2 - CA)^2) =$
$68244.4870 * [1.17 + 10*\log(39.0000/7.0000) -$
$1.17*(7.0000)^2/(39.0000)^2]/(7 * (2.0000 - 0.0000)^2) = 20941.2434$ psi

$F_a = 0.75F_y = 0.75 * 36000.0000 = 27000.0000$ psi

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Customer:
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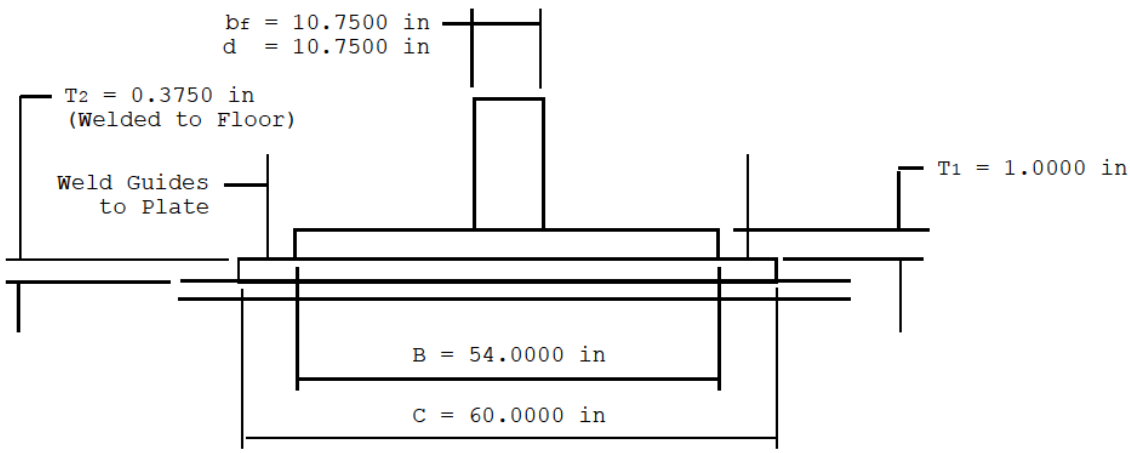
Outer Intermediate Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 55073.2770 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (54.0000 - 0.7071 * 10.7500)/2 = 23.1993$ in

$m = (B - 0.7071 * d)/2 = (54.0000 - 0.7071 * 10.7500)/2 = 23.1993$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(10.7500 * 10.7500)/4} = 2.6875$ in

ℓ = greater of n, m, and n' = 23.1993 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$23.1993\sqrt{[3.33 * 55073.2770/(36000.0000 * (54.0000)^2]} + 0.0000$
$= 0.9697$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (60.0000 - 54.0000)/2 = 3.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$3.0000\sqrt{[3.33 * 55073.2770/(36000.0000 * (60.0000)^2]} + 0.0000$
$= 0.1129$ in

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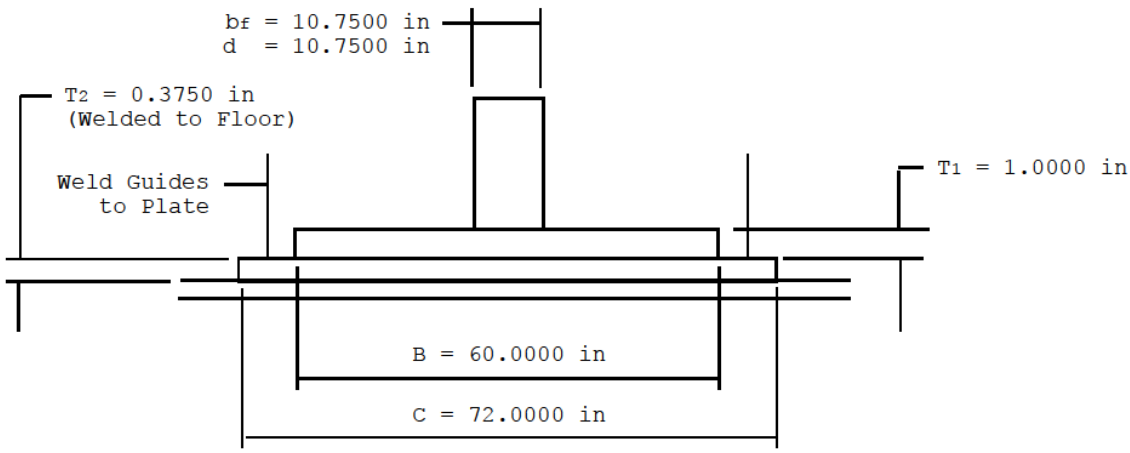
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Inner Intermediate Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 52508.4971 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (60.0000 - 0.7071 * 10.7500)/2 = 26.1993$ in

$m = (B - 0.7071 * d)/2 = (60.0000 - 0.7071 * 10.7500)/2 = 26.1993$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(10.7500 * 10.7500)/4} = 2.6875$ in

ℓ = greater of n, m, and n' = 26.1993 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$26.1993\sqrt{[3.33 * 52508.4971/(36000.0000 * (60.0000)^2]} + 0.0000$
$= 0.9623$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (72.0000 - 60.0000)/2 = 6.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$6.0000\sqrt{[3.33 * 52508.4971/(36000.0000 * (72.0000)^2]} + 0.0000$
$= 0.1837$ in

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
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240 x 50 CRT Un-Insulated
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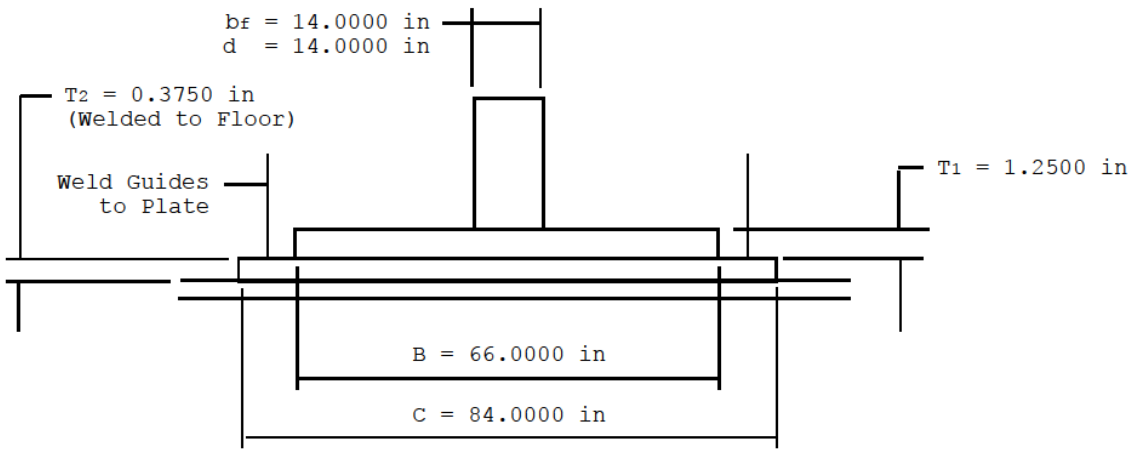
Center Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 79798.5170 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (66.0000 - 0.7071 * 14.0000)/2 = 28.0503$ in

$m = (B - 0.7071 * d)/2 = (66.0000 - 0.7071 * 14.0000)/2 = 28.0503$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(14.0000 * 14.0000)/4} = 3.5000$ in

ℓ = greater of n, m, and n' = 28.0503 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$28.0503\sqrt{[3.33 * 79798.5170/(36000.0000 * (66.0000)^2]} + 0.0000$

$= 1.1547$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (84.0000 - 66.0000)/2 = 9.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$9.0000\sqrt{[3.33 * 79798.5170/(36000.0000 * (84.0000)^2]} + 0.0000$
$= 0.2911$ in

Compression Ring

L4 x4 x3/ 8 per Detail d

4 inches Vertical leg of rim angle is butt welded to the top of the shell with the horizontal leg turned out

Material: A36

API-650 Figure F-2

R_c = 1440.0000 in

Roof Angle (ø) = 3.5763°

$R_2 = R_c/(\sin ø)$ = 1440.0000/(sin 3.5763) = 23084.9561 in

t_h = 0.1875 in

$w_h = 0.3 \sqrt{(R_2 t_h)} = 0.3 \sqrt{(23084.9561 * 0.1875)} = 19.7372$ in
not to exceed 12.0000 in
therefore w_h = 12.0000 in

Available area from roof (A_r) = $t_h w_h$ = 0.1875 * 12.0000 = 2.2500 in²

t_c = 0.4000 in

$w_c = 0.6 \sqrt{(R_c t_c)} = 0.6 \sqrt{(1440.0000 * 0.4000)} = 14.4000$ in

Distance from top of angle to top of shell (b) = 4.0000 in
Available area from shell (A_s) = (w_c - b)t_c = (14.4000 - 4.0000) * 0.4000 = 4.1600 in²

Yield strength (F_y) = 36000.0000 psi

t_a = 0.3750 in
Horizontal projection of angle (a_1) = 3.6250 in
Maximum horizontal projection = $t_a * 3000/\sqrt{(F_y)}$= 0.3750 * 3000/$\sqrt{(36000.0000)}$
= 5.9293 in
therefore a_1 = 3.6250 in
Vertical projection of angle (a_2) = 4.0000 in
Maximum vertical projection = w_c = 14.4000 in
therefore a_2 = 4.0000 in
Available area from compression ring (A_c) = (a_1 + a_2)t_a =
(3.6250 + 4.0000)0.3750 = 2.8594 in²

Total available area (A_t) = A_r + A_s + A_c = 2.2500 + 4.1600 + 2.8594 = 9.2694 in²

Check in Corroded Condition

Roof corrosion allowance = 0.0000 in
t_h = Roof thickness - CA = 0.1875 - 0.0000 = 0.1875 in

$w_h = 0.3 \sqrt{(R_2 t_h)} = 0.3 \sqrt{(23084.9561 * 0.1875)} = 19.7372$ in
not to exceed 12.0000 in
therefore w_h = 12.0000 in

Available area from roof (A_r) = $t_h w_h$ = 0.1875 * 12.0000 = 2.2500 in²

Shell corrosiona allowance = 0.1875 in
t_c = shell thickness - CA = 0.4000 - 0.1875 = 0.2125 in

$w_c = 0.6 \sqrt{(R_c t_c)} = 0.6 \sqrt{(1440.0000 * 0.2125)} = 10.4957$ in

Distance from top of angle to top of shell (b) = 4.0000 in
Available area from shell (A_s) = (w_c - b)t_c = (10.4957 - 4.0000) * 0.2125 = 1.3803 in²

Angle corrosion allowance = 0.0000 in
t_a = Angle thickness - CA = 0.3750 - 0.0000 = 0.3750 in
Horizontal projection of angle (a_1) = 3.6250 in
Maximum horizontal projection = $t_a * 3000/\sqrt{(F_y)}$= 0.3750 * 3000/$\sqrt{(36000.0000)}$
= 5.9293 in
therefore a_1 = 3.6250 in
Vertical projection of angle (a_2) = 4.0000 in
Maximum vertical projection = w_c = 10.4957 in
therefore a_2 = 4.0000 in
Available area from compression ring (A_c) = (a_1 + a_2)t_a =
(3.6250 + 4.0000)0.3750 = 2.8594 in²

Total available area (A_t) = A_r + A_s + A_c = 2.2500 + 1.3803 + 2.8594 = 6.4897 in²

Tank meets the frangibility requirements of 5.10.2.6.

DLs = 1254076.8087 lbs
DLs/(2π * F_y * tan ø) = 1254076.8087/(2π * 36000.0000 * tan 3.5763) = 88.7078 in²

The roof support members shall not be attached to the roof plate.

The roof must be attached to the top angle with a single continuous fillet weld on the top side only that does not exceed 3/16'. No underside welding of roof to top angle (including seal welding) is permitted.

Wind Calculations

Wind Velocity per ASCE 7-10 Figure 26.5-1A = 135 mph
Tank diameter (D) = 240.0000 ft
Thinnest shell course thickness (t) = 0.4000 in

Exposure C

Risk Category = II

Per 5.2.1 (k)(1)(b), wind loads are based on ASCE 7-10.

For Shell:
Centroid of shell C_s = 25.0000 ft
z = C_s but not less than 15.00 ft
z = 25.0000 ft from bottom of tank
Per ASCE7, zg = 900.00 ft
α = 9.50
K_z = 2.01(25.00/900.00)^(2/9.50) = 0.945

K_d = 0.95
K_{zt} = 1.00

h/D = 50.00/240.00 = 0.21

Per 5.2.1(k)(1)(b), V = (0.78)(135) = 105.30 mph

$q_z = 0.00256 K_z K_{zt} K_d V^2 =$
$(0.00256)(0.945)(1.00)(0.95)(105.30)^2 = 25.49$ lbf/ft²

C_f = 0.50

G = 0.85
$P_w = q_z G C_f = (25.49)(0.85)(0.5000) = 10.8333$ lbf/ft²
but not less than 10.0000 lbf/ft²
= 10.8333 lbf/ft²

For Roof:
Top of roof C_s = 57.50 ft
z = C_s but not less than 15.00 ft
z = 57.50 ft from bottom of tank
Per ASCE7, zg = 900.00 ft
α = 9.50
K_z = 2.01(57.50/900.00)^(2/9.50) = 1.126

$K_d = 0.85$
$K_{zt} = 1.00$
$V = 105.30$ mph

$q_z = 0.00256K_zK_{zt}K_dV^2 =$
$(0.00256)(1.126)(1.00)(0.85)(105.30)^2 = 27.1783$ lbf/ft²

$GC_p = 0.90$
$U_w = q_zGC_p = (27.1783)(0.90) = 24.4605$ lbf/ft²

$P_{wr} = q_zGC_f = (27.1783)(0.85)(0.5000) = 11.5508$ lbf/ft²
but not less than 10.0000 lbf/ft²
= 11.55 lbf/ft²

Per 5.9.7.1:

Per ASCE7, $K_z = 0.95$
$K_d = 0.95$
$K_{zt} = 1.00$
$V = 120.00$ mph
$G = 0.85$
$P = 5.00$ lbf/ft²
$P_w = 0.00256K_zK_{zt}K_dV^2IG + P =$
$(0.00256)(0.95)(1.00)(0.95)(120.00)^2(1.00)(0.85) + 5.00 = 33.1383$ lbf/ft²

Multiplier (m) = $P_w/36.00 = 33.1383/36.00 = 0.9205$
not less than 1.0, therefore m = 1.0000

Per 5.9.7.1: $H_1 = 600000.00t[\sqrt{(t/D)^3}](120/V)^2 =$
$(600000.00)(0.4000)[\sqrt{(0.4000 / 240.0000)^3}](120/105.30)^2 = 21.2075$ ft

Per 5.9.7.2: $t_{uniform} = t = 0.4000$ in

For each shell course, $W_{tr} = W \sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5}$

(Shell courses are numbered from bottom to top)

Shell Course	W (ft)	T_{actual} (in)	W_{tr} (ft)
Angle	0.3333	0.3750	0.3917
5	9.8333	0.4000	9.8333
4	9.9583	0.6240	3.2762
3	9.9583	0.7400	2.1392
2	9.9583	0.9386	1.1807
1	9.9583	1.1964	0.6436
		Total Transposed Width:	17.4648

Overturning moment:

Total shell vertical height (H) = 50.0000 ft
Vertical roof area (A_r) = 900.0000 ft²
Distance to roof (H_r) = 57.5000 ft

Overturning Moment (M_{ws}) = ($P_wDH^2/2 + P_{wr}A_rH_r$) =
= [(10.8333)(240.0000)(50.0000)²/2 + (11.5508)(900.0000)(57.5000)]
= 3847756.9440 ft-lbs

Tank does not contain internal pressure. Therefore, MPi = 0

Check for tank stability without anchors per 5.11.2

Weight of shell (D_{LS}) = 1254076.8087 lbs
$M_{DL} = D_{LS} * D/2$ = 1254076.8087 * 240.0000 / 2 = 150489217.0455 ft-lbs
Weight of roof (D_{LR}) = 347093.0752 lbs
$M_{DLR} = D_{LR} * D/2$ = 347093.0752 * 240.0000 / 2 = 41651169.0299 ft-lbs

Bottom thickness used (t_b) = 0.3750 in
Bottom material yield strength (F_{by}) = 36000.0000 psi
$w_L = 4.6700t_b \sqrt{(F_{by}H)}$ = (4.6700)(0.3750)√(36000.0000 * 47.0000) = 2277.9717 lbf/ft

not to exceed 0.450HD = 0.450 * 47.0000 * 240.0000 = 5076.0000 lbf/ft
therefore, w_L = 2277.9717 lbf/ft

For tank to be stable: $M_{ws} + 0.40M_{pi} < M_{DL}/1.5 + M_{DLR}$

$M_{ws} + 0.40M_{pi}$ = (3847756.9440) + (0.40)(0.0000) = 3847756.9440 ft-lbs
$M_{DL}/1.5 + M_{DLR}$ = 150489217.0455/1.5 + 41651169.0299 = 141977313.7269 ft-lbs

Check for Sliding (tank empty):

Sliding Force (V_w) = [($P_wDH + P_{wr}A_r$)] =
[((10.8333)(240.0000)(50.0000) +
(11.55)(900.00))] = 140,395.87 lbs

Resistance to Sliding (V_s) = 0.4(Ws + Wr + Wf) =
0.4(1201451.6953 + 545268.2652 + 707430.0397) = 981,660.00 lbs

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Weights

	New (lbf)	Corroded (lbf)
Product:	131517746.9188	131517746.9188
Total Shell Plates:	1194062.6694	907306.2601
Total Bottom Plates:	707430.0397	475843.3218
Total Roof Plates:	347093.0752	347093.0752
Total Live Load:	1130973.3553	1130973.3553
Total Snow Load:	1140021.1421	1140021.1421
Total Additional Dead Load:	0.0000	0.0000
Shell Insulation:	0.0000	0.0000
Roof Insulation:	0.0000	0.0000
Total Rafters:	198175.1899	198175.1899
Total Roof and Framing:	545268.2652	545268.2652
Total Shell Appurtenances:	0.0000	0.0000
Total Roof Appurtenances:	0.0000	0.0000
Total Roof Appurtenances acting on Shell	0.0000	0.0000
Live Load acting on Shell:	343312.6708	343312.6708
Snow Load acting on Shell:	346059.1721	346059.1721
Total of Roof acting on Shell:	105156.6711	105156.6711
Total of Roof Structural acting on Shell:	52625.1134	52625.1134
Total Compression Ring and Shell Stiffener	7389.0259	7389.0259

Seismic Calculations

Tank diameter (D): 240.0000 ft
Product Height (H): 47.0000 ft
Specific Gravity (G): 1.0000

Tank is self-anchored

Mapped spectral response acceleration (S_S): 0.9400 g
Mapped spectral response acceleration (S_1): 0.4120 g
Mapped spectral response acceleration (S_0) = $0.4S_S$ = (0.4)(0.9400) = 0.3760 g
Mapped spectral response acceleration (S_P) = $0.8S_1$ = (0.8)(0.4120) = 0.3296 g

Site Class per E.4.4 : E
Seismic Use Group per E.3.1 : I

From Table E-1, F_a = 0.9720
From Table E-2, F_v = 2.4000
From Table E-5, I = 1.0000
From Table E-4, R_{wi} = 3.5000
From Table E-4, R_{wc} = 2.0000

Scaling factor per E.2, Q = 2/3

$S_{DS} = QF_aS_S$ = (2/3)(0.9720)(0.9400) = 0.6091
$S_{D1} = QF_vS_1$ = (2/3)(2.4000)(0.4120) = 0.6592

Per E.6.1.3, Av = 0.7 x Purchaser specified vertical acceleration
Vertical earthquake acceleration coefficient (A_v): 0.0000 g

E.4.5.2 Convective (Sloshing) Period

E.4.5.2-c:

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}} = \frac{0.578}{\sqrt{\tanh\left(\frac{(3.68)(47.0000)}{240.0000}\right)}} = 0.7357$$

Per E4.5.2-b: $T_c = 1.0K_s\sqrt{D}$ = (1.0)(0.7357)√240.0000 = 11.3967

E.4.6.1 Spectral Acceleration Coefficients

Impulsive spectral acceleration parameter:

Per E.4.6.1-1:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 0.6091\left(\frac{1.00}{3.5000}\right) = 0.1740$$

per E.4.6.1-3:

$$A_i \geq 0.5S_1\left(\frac{I}{R_{wi}}\right) = (0.5)(0.4120)\left(\frac{1.00}{3.5000}\right) = 0.0589$$

However, per E.4.6.1-2, $A_i \geq 0.0070$
Therefore, $A_i = 0.1740$

Convective spectral acceleration parameter:

$T_L = 16.0000$
$K = 1.5$

$T_c \leq T_L$, Per E.4.6.1-4

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = (1.5)(0.6592)\left(\frac{1}{11.3967}\right)\left(\frac{1.00}{2.0000}\right) = 0.0434$$

However, $A_c \leq A_i$
Therefore, $A_c = 0.0434$

Per E.6.1.1

D/H = 240.0000/47.0000 = 5.1064

Weights:
W_s = 1201452 lbs
W_f = 707430 lbs
W_r = Roof weight + 10% of snow = 659270 lbs
W_P = 131517747 lbs

$D/H \geq 1.333$

Per E.6.1.1-1,

$$W_i = \frac{\tanh(0.866D/H)}{0.866D/H} W_P = \frac{\tanh(0.866 * 5.1064)}{0.866 * 5.1064}(131517747) = 29732254 \text{ lbs}$$

Per E.6.1.1-3, $W_c = 0.230D/H \tanh(3.67H/D)W_P =$
$(0.230)(5.1064)[\tanh(3.67/5.1064)](131517747) = 95166223$ lbs

Per E.6.1-2, $V_i = A_i(W_s + W_r + W_f + W_i) =$
$(0.1740)(1201452 + 659270 + 707430 + 29732254) = 5621378$ lbs
Per E.6.1-3, $V_c = A_cW_c = (0.0434)(95166223) = 4128419$ lbs
Per E.6.1-1, $V = \sqrt{(V_i^2 + V_c^2)} = \sqrt{[(5621378)^2 + (4128419)^2]} = 6974506$ lbs

Per E.6.1.2.1

$D/H \geq 1.333$

Per E.6.1.2.1-1, $X_i = 0.375H = (0.375)(47.0000) = 17.6250$ ft

Per E.6.1.2.1-3,

$$X_c = \left[1.0 - \frac{\cosh(3.67H/D) - 1}{3.67H/D \sinh(3.67H/D)} \right] H =$$

$$\left[1.0 - \frac{\cosh(3.67/5.1064) - 1}{(3.67/5.1064)\sinh(3.67/5.1064)} \right] (47.0000) = 24.4619 \text{ ft}$$

Per E.6.1.2.2

$D/H \geq 1.333$

Per E.6.1.2.2-1,

$$X_{is} = 0.375 \left[1.0 + 1.333 \left(\frac{(0.866D/H)}{\tanh(0.866D/H)} - 1.0 \right) \right] H =$$

$$0.375 \left[1.0 + 1.333 \left(\frac{(0.866 * 5.1064)}{\tanh(0.866 * 5.1064)} - 1.0 \right) \right] (47.0000) =$$

98.0549 ft

Per E.6.1.2.2-3,

$$X_{cs} = \left[1.0 - \frac{\cosh(3.67H/D) - 1.937}{3.67H/D \sinh(3.67H/D)} \right] H =$$

$$\left[1.0 - \frac{\cosh(3.67/5.1064) - 1.937}{(3.67/5.1064)\sinh(3.67/5.1064)} \right] (47.0000) = 102.7989 \text{ ft}$$

Per E.6.1.5

Per E.6.1.5-1, $M_{rw} = \sqrt{\{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2\}} =$
$\sqrt{\{[0.1740(29732254*17.6250 + 1201452*19.9796 + 659270*52.5000)]^2 + [0.0434(95166223*24.4619)]^2\}} = 143111329$ ft-lbs

Per E.6.1.5-2, $M_s = \sqrt{\{[A_i(W_iX_{is} + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_{cs})]^2\}} =$
$\sqrt{\{[0.1740(29732254*98.0549 + 1201452*19.9796 + 659270*52.5000)]^2 + [0.0434(95166223*102.7989)]^2\}} = 669328931$ ft-lbs

Self-anchored tank (new condition)

Per E.6.2.1.1.1

$W_s = 1201452$ lbs
$w_{rs} = 255.1621$ lbf/ft
$t_a = 0.5000$ in
$t_s = 1.0088$ in
$G_e = G(1.0 - 0.4A_v) = 1.0000(1.0 - (0.4)(0.0000)) = 1.0000$
Per E.6.2.1.1.1-2, $w_t = W_s/(\pi D) + w_{rs} = 1201452/(\pi * 240.0000) + 255.1621$
$= 1848.6369$ lbf/ft

Check annular ring:

Per E.6.2.1.1.2-1b, $L = 0.21600 t_a \sqrt{(F_y/HG_e)}$ =

$(0.21600)(0.5000)\sqrt{[36000.0000/(47.0000 * 1.0000)]} = 2.9890$ ft

$\leq 0.035D = (0.035)(240.0000) = 8.4000$ ft

therefore, L = 2.9890 ft

$L_s = 2.9003$ ft

Per E.6.2.1.1-2b,

$w_a = 36.5HG_eL_s = (36.5)(47.0000)(1.0000)(2.9003) = 4975.4647$ lbf/ft

$W_{int} = 0.0000$ lbf/ft

Per E.6.2.1.1.1-1, $J = \dfrac{M_{rw}}{D^2[w_t(1.0 - 0.4A_v) + w_a - 0.40w_{int}]} =$

$$\frac{143111329}{(240.0000)^2[1848.6369(1.0 - (0.4)(0.0000)) + 4975.4647 - 0.40 * 0.0000]} = 0.364$$

Shell compression per E.6.2.2.1-1b where J < 0.785

$\sigma_c = [w_t(1 + 0.4A_v) + 1.273M_{rw}/D^2]/(12t_s) =$

$[1848.6369(1 + (0.4)(0.0000)) + (1.273)(143111329)/(240.0000)^2]/(12 * 1.0088) =$

413.9752 psi

Self-anchored tank (corroded condition)

Per E.6.2.1.1.1

$W_s = 914695$ lbs

$w_{rs} = 255.1621$ lbf/ft

$t_a = 0.3750$ in

$t_s = 1.0088$ in

$G_e = G(1.0 - 0.4A_v) = 1.0000(1.0 - (0.4)(0.0000)) = 1.0000$

Per E.6.2.1.1.1-2, $w_t = W_s/(\pi D) + w_{rs} = 914695/(\pi * 240.0000) + 255.1621$

$= 1468.3144$ lbf/ft

Check annular ring:

Per E.6.2.1.1.2-1b, $L = 0.21600 t_a \sqrt{(F_y/HG_e)}$ =

$(0.21600)(0.3750)\sqrt{[36000.0000/(47.0000 * 1.0000)]} = 2.2418$ ft

$\leq 0.035D = (0.035)(240.0000) = 8.4000$ ft

therefore, L = 2.2418 ft

Per E.6.2.1.1-1b,

$w_a = 7.9t_a\sqrt{(F_yHG_e)} =$

$(7.9)(0.3750)\sqrt{[(36000.0000)(47.0000)(1.0000)]} = 3853.5282$ lbf/ft

$\leq 1.28HDG_e = (1.28)(47.0000)(240.0000)(1.0000) = 14438.4000$ lbf/ft

therefore, $w_a = 3853.5282$ lbf/ft

Per E.6.2.1.1.1-1, $J = \dfrac{M_{rw}}{D^2[w_t(1.0 - 0.4A_v) + w_a - 0.40w_{int}]} =$

$$\frac{143111329}{(240.0000)^2[1468.3144(1.0 - (0.4)(0.0000)) + 3853.5282 - 0.40 * 0.0000]} = 0.467$$

Shell compression per E.6.2.2.1-1b where J < 0.785

$\sigma_c = [w_t(1 + 0.4A_v) + 1.273M_{rw}/D^2]/(12t_s) =$
$[1468.3144(1 + (0.4)(0.0000)) + (1.273)(143111329)/(240.0000)^2]/(12 * 1.0088) =$
383 psi

Allowable Compression Stresses in Shell per E.6.2.2.3

For new condition:
$GHD^2/t^2 = (1.0000)(47.0000)(240.0000)^2/(1.0088)^2 = 2660092$

For $GHD^2/t^2 \geq 1000000$
Per E.6.2.2.3-1b, $F_c = 1000000t_s/D = (1000000)(1.0088)/240.0000 = 4203$ psi

For corroded condition:
$GHD^2/t^2 = (1.0000)(47.0000)(240.0000)^2/(1.0088)^2 = 2660092$

For $GHD^2/t^2 \geq 1000000$
Per E.6.2.2.3-1b, $F_c = 1000000t_s/D = (1000000)(1.0088)/240.0000 = 4203$ psi

Freeboard requirements per E.7.2

$T_c > 4$
Per E.7.2-3, $A_f = KS_{D1}I(4/T_c^2) = (1.5)(0.6592)(1.0000)(4/11.3967^2) = 0.0305$

Wave Height Per E.7.2-1: $\delta_s = 0.42DA_f = (0.42)(240.0000)(0.0305) = 3.0695$ ft
Per Table E-7, minimum freeboard $= 0.7\delta_s = (0.7)(3.0695) = 2.1487$ ft

Freeboard equal to 0.7 times the calculated wave height is recommended but not required. The required minimum freeboard = 2.149. The available freeboard = 2.000.

Per E.6.2.1.1.3, butt welded annular plates are required.

Estimated Tank Uplift per E.7.3.1-1b
$t_b = 0.3750$ in
$y_u = F_yL^2/(83300)(t_b) = (36000)(2.2418)^2/(83300)(0.3750) = 5.7916$ in

Tank resistance to sliding per E.7.6
Per E.7.6-1, $V_s = 0.4000(W_s + W_r + W_f + W_p)(1.0 - 0.4A_v) =$
$0.4000(914695 + 659270 + 475843 + 131517747)(1.0 - 0.4 * 0.0000) = 53427022$ lbs
$V = 6974506$ lbs

Tank is stable against sliding.

Hoop Stress per E.6.1.4

G = 1.00
D = 240.0000 ft
H = 47.0000 ft
A_i = 0.1740
A_c = 0.0434
A_v = 0.0000
D/H = 5.1064

Shell Course 01

Y = 47.0000 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{47.0000}{47.0000} - 0.5 \left(\frac{47.0000}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 4415.7164 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 47.0000)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 1926.4797 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((47.0000)-1)(1.00) = 28704.0000$ lbf/in

Shell thickness (t) = 1.0089 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[28704.0000 + \sqrt{\{(4415.7164)^2 + (1926.4797)^2\}}]/1.0089 = 33225.9519$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = (σ_T * t)/Sa =
(33225.9519 * 1.0089)/37240.0000 = 0.9002 in

Shell Course 02

Y = 37.0417 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{37.0417}{47.0000} - 0.5 \left(\frac{37.0417}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 4217.4821 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 37.0417)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 1948.9818 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((37.0417)-1)(1.00) = 22490.0000$ lbf/in

Shell thickness (t) = 0.7511 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[22490.0000 + \sqrt{\{(4217.4821)^2 + (1948.9818)^2\}}]/0.7511 = 36128.3981$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(36128.3981 * 0.7511)/37240.0000 = 0.7287 in

Shell Course 03

Y = 27.0833 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{27.0833}{47.0000} - 0.5 \left(\frac{27.0833}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 3622.7793 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 27.0833)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 2017.0140 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((27.0833)-1)(1.00) = 16276.0000$ lbf/in

Shell thickness (t) = 0.5525 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{N_i^2 + N_c^2}]/t =$
$[16276.0000 + \sqrt{(3622.7793)^2 + (2017.0140)^2}]/0.5525 = 36963.6728$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(36963.6728 * 0.5525)/37240.0000 = 0.5484 in

Shell Course 04

Y = 17.1250 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{17.1250}{47.0000} - 0.5 \left(\frac{17.1250}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 2631.6078 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 17.1250)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 2132.1656 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((17.1250)-1)(1.00) = 10062.0000$ lbf/in

Shell thickness (t) = 0.4365 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[10062.0000 + \sqrt{\{(2631.6078)^2 + (2132.1656)^2\}}]/0.4365 = 30810.9021$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(23200.0000)(1.0000) = 30856.0000 psi
or
0.90FyE = (0.90)(36000.0000)(1.0000) = 32400.0000 psi
Therefore, Allowable stress = 30856.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(30810.9021 * 0.4365)/30856.0000 = 0.4359 in

Shell Course 05

Y = 7.1667 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{7.1667}{47.0000} - 0.5 \left(\frac{7.1667}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 1243.9678 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 7.1667)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 2297.1265 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((7.1667)-1)(1.00) = 3848.0000$ lbf/in

Shell thickness (t) = 0.2125 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[3848.0000 + \sqrt{\{(1243.9678)^2 + (2297.1265)^2\}}]/0.2125 = 30401.5330$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(23200.0000)(1.0000) = 30856.0000 psi
or
0.90FyE = (0.90)(36000.0000)(1.0000) = 32400.0000 psi
Therefore, Allowable stress = 30856.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(30401.5330 * 0.2125)/30856.0000 = 0.2094 in

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202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Un-Insulated
Date: 03-02-2015 Rev.: 1

Foundation Loads

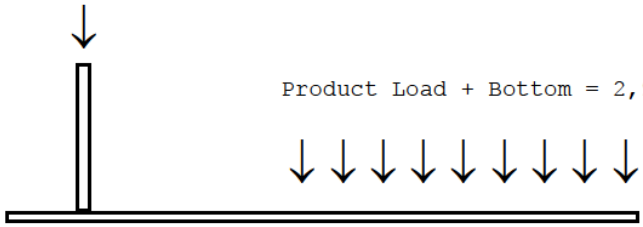
Dead Load = 1,803 lbf/ft
Dead Load + Live Load = 2,258 lbf/ft
Dead Load + Snow Load = 2,262 lbf/ft
Wind Load + Dead Load = 1,888 lbf/ft

(Wind Foundation Moment = 3,847,757 ft-lbs)
(Wind Horizontal Force = 130,000 lbs)

Seismic Load + Dead Load (ringwall foundation) = 5,011 lbf/ft
Seismic Load + Dead Load (slab foundation) = 16,641 lbf/ft

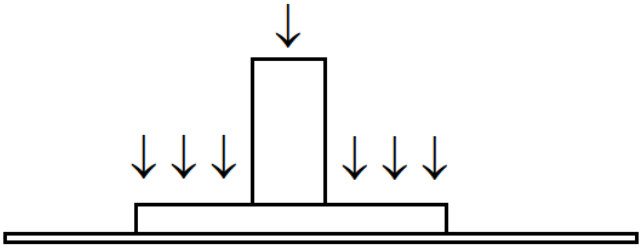
(Note: For Seismic the dead load has a portion of the vertical acceleration added to it and includes a portion of the snow load)
(Seismic Overturning Moment (ringwall) (M_{rw}) = 143,111,329 ft-lbs)
(Seismic Overturning Moment (slab) (M_s) = 669,328,931 ft-lbs)

(Seismic Horizontal Force = 6,974,506 lbs)



Product Load + Bottom = 2,948 lbf/ft²

Center Column Load = 82,093 lbs



Base Plate Area = 30.25 ft²
Base Plate Load = 2,714 lbf/ft²

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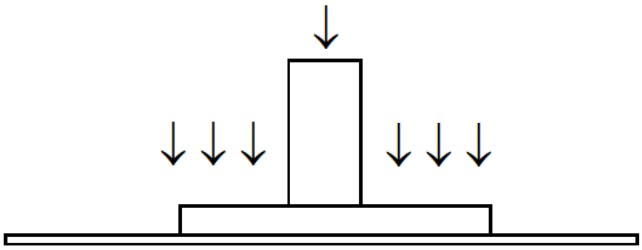
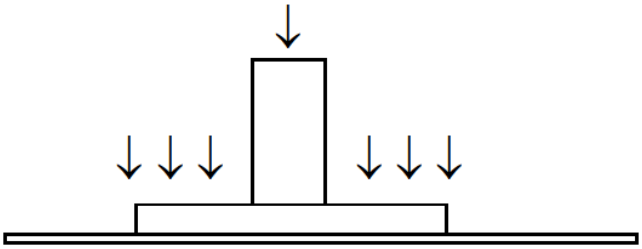
Intermediate Column Load = 54,081 lbs



Base Plate Area = 25.00 ft²
Base Plate Load = 2,163 lbf/ft²

Outer Column Load = 56,283 lbs



Base Plate Area = 20.25 ft²
Base Plate Load = 2,779 lbf/ft²

Total Weight of Tank: 2,577,353 lbs
Total Operating Weight: 134,095,100 lbs
Total Testing Weight: 134,095,100 lbs

TANK CALCULATIONS

T-Bailey

Tesoro Savage Petroleum Terminal, LLC
Tesoro Savage Vancouver Energy
Port of Vancouver, WA

Crude Oil Insulated Storage Tank

240´ Diameter x 50´ High Cone Roof Tank

Preliminary Design Calculat ons

PROJECT ENGINEER

Hagen Engineering Int rnati nal
202 S. Cedar St. Su te B
Owasso, OK 74055

(651) 3 5-46 6

DRAFT

DESIGN ENGINEER

Doug Bayles, P.E.

HEI Job Number: 15-1006

Standard Compliant Issues

Shell

Shell Course 1 requires impact testing per the standard.
Shell Course 2 requires impact testing per the standard.

Comments

Shell

Joint efficiency for all shell courses = 100%. X-rays are required. Quantity of x-rays shall be per 8.1.2.

Roof

Roof is single lap welded
Internal pressure does not exceed weight of roof. Appendix F calculations not required.
Tank meets the frangibility requirements of 5.10.2.6.
The roof support members shall not be attached to the roof plate.
The roof must be attached to the top angle with a single continuous fillet weld on the top side only that does not exceed 3/16'. No underside welding of roof to top angle (including seal welding) is permitted.

Rafters

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.
For Outer Rafter for Three-Span System rafters, the required section modulus is 14.59. The actual section modulus is 17.1.
For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.
For Intermediate Rafter for Three-Span System rafters, the required section modulus is 16.33. The actual section modulus is 21.3.
For Inner Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.
For Inner Rafter for Three-Span System rafters, the required section modulus is 6.21. The actual section modulus is 10.9.
For Outer Intermediate Girder for Three-Span System girders, the required section modulus is 74.15. The actual section modulus is 78.8.
For Inner Intermediate Girder for Three-Span System girders, the required section modulus is 73.91. The actual section modulus is 78.8.

Columns, Base Plates, & Cap Plates

For center column, KL/r = 137.2
Load on center column (73036.26) is less than the allowable load (111759.42).
The required bottom base plate thickness of .2934 is less than or equal to the plate thickness.
The required upper base plate thickness of 1.164 is less than or equal to the plate thickness.
For Inner Intermediate Column, KL/r = 166.8
For Inner Intermediate Column, load (50357.05) is less than the allowable load (56119.29).
The required bottom base plate thickness of .1853 is less than or equal to the plate thickness.
The required upper base plate thickness of .971 is less than or equal to the plate thickness.
For Outer Intermediate Column, KL/r = 158.7
For Outer Intermediate Column, load (53279.88) is less than the allowable load (62016.76).
The required bottom base plate thickness of .1139 is less than or equal to the plate thickness.
The required upper base plate thickness of .9785 is less than or equal to the plate thickness.

Wind

Per 5.2.1 (k)(1)(b), wind loads are based on ASCE 7-10.

Shell External Pressure

Per 5.2.1(b), vacuum calculations are not required.

Seismic

Tank is self-anchored
Per E.6.2.1.1.3, butt welded annular plates are required.
Tank is stable against sliding.

Anchor Bolt and Anchor Chair

Tank is not anchored

Hagen Engineering International
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Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Design Data

Tank Diameter	240'- 0.0000" (Outside)
Tank Height	50'- 0.0000"
High Liquid Level	47'- 0.0000"
Normal Liquid Level	46'- 6.0000"
Low Liquid Level	4'- 0.0000"
Total Capacity	15,765,268 gallons
	375,364 barrels
Working Capacity	14,357,570 gallons
	341,847 barrels
Design Standard	API-650 12th Edition, Add. 1, Annex E
Design Metal Temperature	25 °F
Operating Temperature	150 °F
Design Temperature	150 °F
Roof Live Load	25.00 psf
Ground Snow Load	30.00 psf
Roof Design Snow Load (0.84 x Ground Snow Load)	25.20 psf
Additional Roof Loads	0.00 psf
Wind:	
Velocity per ASCE 7-10 Figure 26.5-1A	135 mph
Risk Category	II
Exposure	C

Seismic:

Site Class	E
Importance Factor	1.00
Use Group	I
Ss Value	94.00 %
S_1 Value	41.20 %
Vertical Acceleration	0.00 g
Transitional Period (T_L)	16
Specific Gravity	1.000
Internal Pressure (Design)	Atmospheric
(Operating)	Atmospheric
External Pressure (Design)	1.00 inches of water
(Operating)	Atmospheric

Corrosion Allowance:

Bottom	0.1250 in
Annular Ring	0.1250 in
Shell Courses 1 - 3	0.1875 in
Courses 4 - 4	0.1875 in
Courses 5 - 5	0.1875 in
Roof	0.0000 in
Rafters	0.0000 in
Columns	0.0000 in
Cap Plate	0.0000 in
Base Plate	0.0000 in
Compression Ring	0.0000 in

Shell is insulated with 3.0000 in thick insulation (Weight - 0.7800 psf)

Roof is insulated with 3.0000 in thick insulation (Weight - 0.7800 psf)

Summary

Total Weight of Insulation: 64,761 lbs

Total Weight of Tank: 2,577,353 lbs
Total Operating Weight: 134,159,860 lbs
Total Testing Weight: 134,159,860 lbs

Primary Bottom – See Bottom of Page for Secondary Bottom and Dead Shell

Bottom with Annular Ring Sloped Up 1.0000 in 120:
0.3750 in (A36) PL (weight: 656,750 lbs)

Annular Ring:
0.5000 in (A36 MOD) PL (weight: 50,680 lbs)

Outside projection = 2.0000 in

Total width: 38.0000 in

Shell

Shell Course 01: 1.1964 in (A573-70) PL x 119.5000 in wide (weight: 366,868 lbs)

Shell Course 1 requires impact testing per the standard.

Shell Course 02: 0.9386 in (A573-70) PL x 119.5000 in wide (weight: 287,815 lbs)

Shell Course 2 requires impact testing per the standard.

Shell Course 03: 0.7400 in (A573-70) PL x 119.5000 in wide (weight: 226,916 lbs)

Shell Course 04: 0.6240 in (A36 MOD) PL x 119.5000 in wide (weight: 191,345 lbs)

Shell Course 05: 0.4000 in (A36 MOD) PL x 118.0000 in wide (weight: 121,118 lbs)

Supported Cone Roof

0.1875 in (A36) PL (weight: 347,093 lbs)

Roof Slope: 0.7500 in 12 ø: 3.5763° Roof Rise: 7.5000 ft

Roof is single lap welded

Secondary Bottom

0.2360 in (A36 MOD) PL

Outside projection = 2.0000 in

Dead Shell or Vertical Spacer Plate

Dead Shell: 1.1964 inch (A573-70) PL x 1 ¼″ in

Inner Intermediate Girder for Three-Span System Girder: (8) W18x 46 (A992-GR50) x 30.6147 ft
(weight: 11,266 lbs)

Outer Intermediate Girder for Three-Span System Girder: (16) W18x 46 (A992-GR50) x 31.2145 ft
(weight: 22,974 lbs)

Inner Rafter for Three-Span System Rafter: (48) W10x 12 (A992-GR50) x 37.0956 ft
(weight: 21,367 lbs)

For Inner Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

Intermediate Rafter for Three-Span System Rafter: (80) W12x 19 (A992-GR50) x 44.5686 ft
(weight: 67,744 lbs)

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

Outer Rafter for Three-Span System Rafter: (160) W12x 16 (A992-GR50) x 42.6031 ft
(weight: 109,064 lbs)

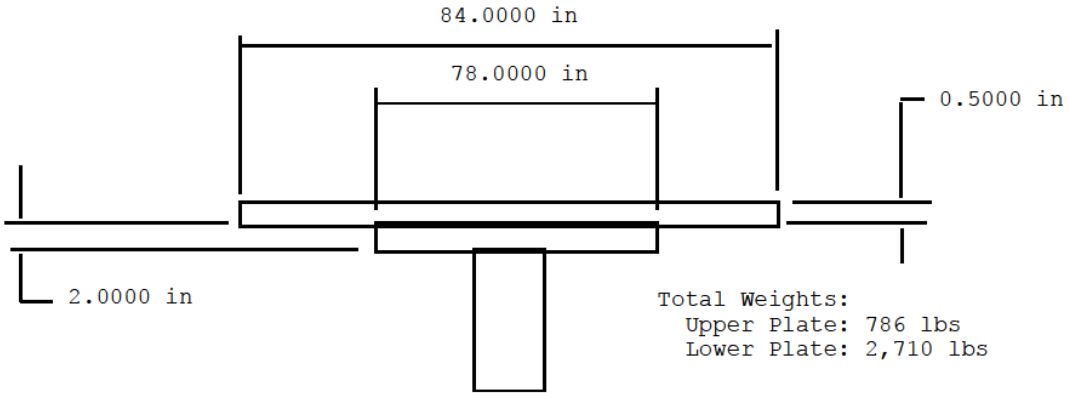
For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

Intermediate Column: (16) 10" Std Wt Pipe (A53-B) x 48.7979 ft
(Location Radius - 80.0000 ft)
(weight each: 1975 lbs, total weight: 31,605 lbs)

Intermediate Column: (8) 10" Std Wt Pipe (A53-B) x 51.2979 ft
(Location Radius - 40.0000 ft)
(weight each: 2077 lbs, total weight: 16,612 lbs)

Center Column: 14" Std Wt Pipe (A53-B) x 55.2817 ft (weight: 3,017 lbs)

Center Column Cap Plate: Material - A36



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240 x 50 CRT Insulated
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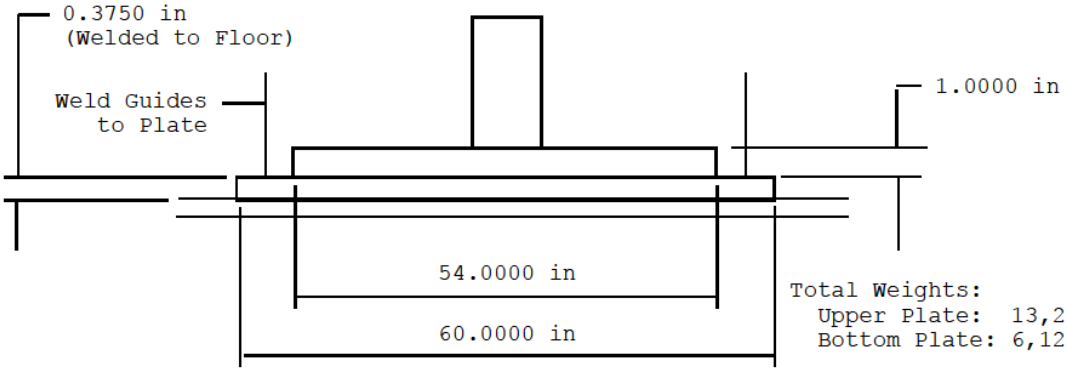
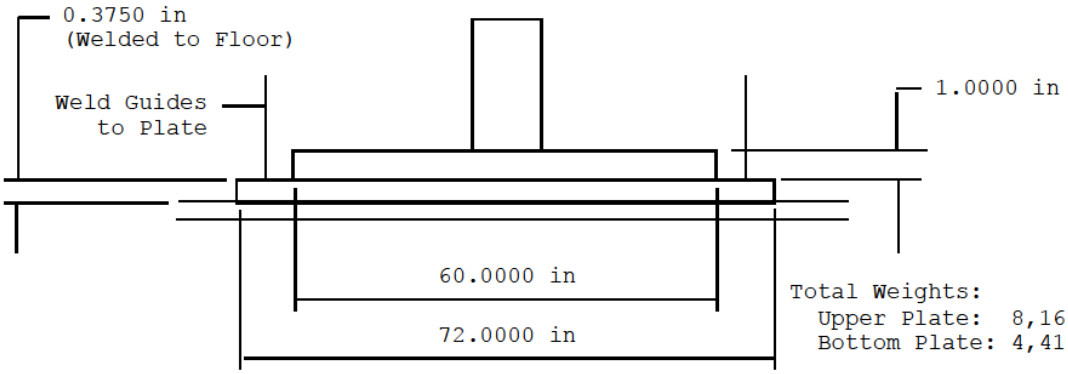
Outer Intermediate Column Base Plate description: Square Base Plate (A36)



Total Weights:
Upper Plate: 13,232 lbs
Bottom Plate: 6,126 lbs

Inner Intermediate Column Base Plate description: Square Base Plate (A36)



Total Weights:
Upper Plate: 8,168 lbs
Bottom Plate: 4,411 lbs

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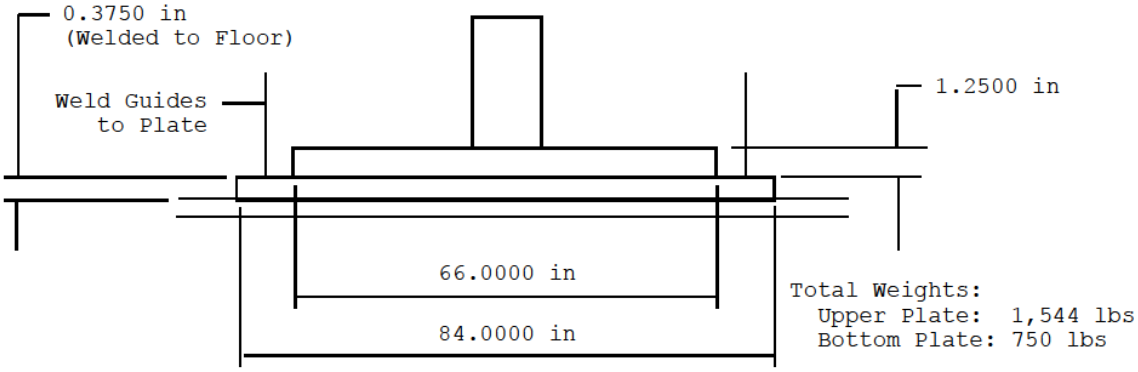
Date: 03-03-2015 Rev.: 1

Center Column Base Plate description: Square Base Plate (A36)



Total Weights:
Upper Plate: 1,544 lbs
Bottom Plate: 750 lbs

Compression Ring

L4 x4 x3/ 8 per Detail d (A36)
Weight: 7,389 lbs

4 inches Vertical leg of rim angle is butt welded to the top of the shell with the horizontal leg turned out

Bottom Design

Bottom with Annular Ring Sloped Up 1.0000 in 120

Material Grade: A36
Thickness (t_b): 0.3750 in
Corrosion Allowance (CA): 0.1250 in
Minimum thickness per 5.4.1 = 0.2360 in

Minimum thickness + corrosion allowance = 0.2360 + 0.1250 = 0.3610 in

Annular Ring

Material Grade: A36 MOD
Thickness (t_b): 0.5000 in
Corrosion Allowance (CA): 0.1250 in
Minimum required outside projection per 5.4.2 = 2.0000 in
Actual outside projection (p) = 2.0000 in
Minimum thickness per 5.4.1 = 0.2360 in

t_d = 1.1963 in t_t = 0.9448 in t = 1.1964 in
CA = 0.1875 in S_d = 28000.0000 psi S_t = 30000.0000 psi

Annular ring width: 38.0000 in

$S_t = (t_t/t)S_t$ = (0.9448 /1.1964) * 30000.0000 = 23690 psi

$S_d = ((t_d - CA)/(t - CA))S_d$ =
((1.1963 - 0.1875)/(1.1964 - 0.1875)) * 28000.0000 = 27998 psi

From Table 5-1, minimum thickness = 0.3438 in

Minimum thickness + corrosion allowance = 0.3438 + 0.1250 = 0.4688 in

Minimum annular ring width inside shell per 5.5.2 = 24.0000 in
F_y = 36000.00 psi
Υ = 0.4333 psi/ft

Per 5.5.2, width = $2t_b \sqrt{(F_y/(2\Upsilon GH))}$ =
(2 * 0.5000)√(36000.00/(2 * 0.4333 * 1.0000 * 47.0000)) = 29.7287 in

Shell Design

Diameter (D) = 240.0000 ft
Specific Gravity (G) = 1.000

Shell Course 01

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 47.0000 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.3 and 5.6.3.2:

$$t_{pd} = \frac{(2.6)(47.0000 - 1.0)(240.0000)(1.0000)}{28000.0000} + 0.1875 = 1.2126 \text{ in}$$

For Design per 5.6.4.4:

$$t_{1d} = \left\{ 1.06 - \frac{(0.4630)(240.0000)}{47.0000} \sqrt{\left(\frac{(47.0000)(1.0000)}{28000.0000} \right)} \right\} \left\{ \frac{(2.6000)(47.0000)(240.0000)(1.0000)}{28000.0000} \right\} + 0.1875 = 1.1963 \text{ in}$$

t_{1d} (need not be greater than t_{pd}) = 1.1963 in

For Hydrotest per 5.6.4.3 and 5.6.3.2:

$$t_{pt} = \frac{(2.6)(47.0000 - 1.0)(240.0000)}{30000.0000} = 0.9568 \text{ in}$$

For Hydrotest per 5.6.4.4:

$$t_{1t} = \left\{ 1.06 - \frac{(0.4630)(240.0000)}{47.0000} \sqrt{\left(\frac{47.0000}{30000.0000} \right)} \right\} \left\{ \frac{(2.6000)(47.0000)(240.0000)}{30000.0000} \right\} = 0.9448 \text{ in}$$

t_{1t} (need not be greater than t_{pt}) = 0.9448 in

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 1.1964 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 1.1964 inches.

Shell Course 1 requires impact testing per the standard.

Shell Course 02

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 37.0417 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.5:

$$h_d = \frac{h_1}{\sqrt{(r\ t_1)}} = \frac{119.5000}{\sqrt{(1440.0000*1.0089)}} = 3.1352$$

For Design per 5.6.4.6:

t_u = 0.7510 t_L = 1.0089
K = t_L/t_u = 1.0089/0.7510 = 1.3434

$$C = \frac{(\sqrt{1.3434})(1.3434-1)}{1 + (1.3434)^{1.5}} = 0.1557$$

$x_1 = (0.61)\sqrt{(1440.0000*0.7510)} + (3.8400)(0.1557)(37.0417) = 42.2015$
$x_2 = (12)(0.1557)(37.0417) = 69.1924$
$x_3 = (1.22)\sqrt{(1440.0000*0.7510)} = 40.1198$
x = smaller of x_1, x_2, and x_3 = 40.1198

$$t_{dx} = \frac{(2.6)(240.0000)(37.0417 - 40.1198/12)(1.0000)}{28000.0000} = 0.7510 \text{ in}$$

$t_{2d} = t_{2a} = t_{dx} + CA = 0.7510 + 0.1875 = 0.9385$ in

For Hydrotest per 5.6.4.5:

$$h_t = \frac{h_1}{\sqrt{(r\ t_1)}} = \frac{119.5000}{\sqrt{(1440.0000*1.1964)}} = 2.8790$$

For Hydrotest per 5.6.4.6:

t_u = 0.7032 t_L = 1.1964
K = t_L/t_u = 1.1964/0.7032 = 1.7014

$$C = \frac{(\sqrt{1.7014})(1.7014-1)}{1 + (1.7014)^{1.5}} = 0.2842$$

$x_1 = (0.61)\sqrt{(1440.0000*0.7032)} + (3.8400)(0.2842)(37.0417) = 59.8353$
$x_2 = (12)(0.2842)(37.0417) = 126.3266$
$x_3 = (1.22)\sqrt{(1440.0000*0.7032)} = 38.8216$
x = smaller of x_1, x_2, and x_3 = 38.8216

$$t_{tx} = \frac{(2.6)(240.0000)(37.0417 - 38.8216/12)}{30000.0000} = 0.7032 \text{ in}$$

$t_{2t} = t_{2a} = t_{dt} = 0.7032$ in

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.9386 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.9386 inches.

Shell Course 2 requires impact testing per the standard.

Shell Course 03

Material Grade: A573-70 Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 28000.0000 psi

H = 27.0833 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 30000.0000 psi

For Design per 5.6.4.6:

t_u = 0.5404 t_L = 0.7511
K = t_L/t_u = 0.7511/0.5404 = 1.3900

$$C = \frac{(\sqrt{1.3900})(1.3900-1)}{1 + (1.3900)^{1.5}} = 0.1742$$

x_1 = (0.61)√(1440.0000*0.5404) + (3.8400)(0.1742)(27.0833) = 35.1367
x_2 = (12)(0.1742)(27.0833) = 56.6275
x_3 = (1.22)√(1440.0000*0.5404) = 34.0319
x = smaller of x_1, x_2, and x_3 = 34.0319

$$t_{dx} = \frac{(2.6)(240.0000)(27.0833 - 34.0319/12)(1.0000)}{28000.0000} + 0.1875 = 0.7279 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.5062 t_L = 0.9386
K = t_L/t_u = 0.9386/0.5062 = 1.8541

$$C = \frac{(\sqrt{1.8541})(1.8541-1)}{1 + (1.8541)^{1.5}} = 0.3300$$

x_1 = (0.61)√(1440.0000*0.5062) + (3.8400)(0.3300)(27.0833) = 50.7846
x_2 = (12)(0.3300)(27.0833) = 107.2339
x_3 = (1.22)√(1440.0000*0.5062) = 32.9396
x = smaller of x_1, x_2, and x_3 = 32.9396

$$t_{tx} = \frac{(2.6)(240.0000)(27.0833 - 32.9396/12)}{30000.0000} = 0.5062 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.7400 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.7400 inches.

Shell Course 04

Material Grade: A36 MOD

Shell Height: 9.9583 ft

Product Design Stress per Table 5-2 (S_d) = 23200.0000 psi

H = 17.1250 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 24900.0000 psi

For Design per 5.6.4.6:

t_u = 0.4031 t_L = 0.5525

K = t_L/t_u = 0.5525/0.4031 = 1.3706

$$C = \frac{(\sqrt{1.3706})(1.3706-1)}{1 + (1.3706)^{1.5}} = 0.1666$$

x_1 = (0.61)√(1440.0000*0.4031) + (3.8400)(0.1666)(17.1250) = 25.6509

x_2 = (12)(0.1666)(17.1250) = 34.2314

x_3 = (1.22)√(1440.0000*0.4031) = 29.3936

x = smaller of x_1, x_2, and x_3 = 25.6509

$$t_{dx} = \frac{(2.6)(240.0000)(17.1250 - 25.6509/12)(1.0000)}{23200.0000} + 0.1875 = 0.5906 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.3703 t_L = 0.7400

K = t_L/t_u = 0.7400/0.3703 = 1.9983

$$C = \frac{(\sqrt{1.9983})(1.9983-1)}{1 + (1.9983)^{1.5}} = 0.3690$$

x_1 = (0.61)√(1440.0000*0.3703) + (3.8400)(0.3690)(17.1250) = 38.3486

x_2 = (12)(0.3690)(17.1250) = 75.8194

x_3 = (1.22)√(1440.0000*0.3703) = 28.1728

x = smaller of x_1, x_2, and x_3 = 28.1728

$$t_{tx} = \frac{(2.6)(240.0000)(17.1250 - 28.1728/12)}{24900.0000} = 0.3703 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.6240 in

Use actual thickness for subsequent calculations.
Per 5.6.4.2 the required design thickness = 0.6240 inches.

Shell Course 05

Material Grade: A36 MOD

Shell Height: 9.8333 ft

Product Design Stress per Table 5-2 (S_d) = 23200.0000 psi

H = 7.1667 ft

Corrosion Allowance (CA): 0.1875 in

Hydrostatic Test Stress per Table 5-2 (S_t) = 24900.0000 psi

For Design per 5.6.4.6:

t_u = 0.1523 t_L = 0.4365
K = t_L/t_u = 0.4365/0.1523 = 2.8667

$$C = \frac{(\sqrt{2.8667})(2.8667-1)}{1 + (2.8667)^{1.5}} = 0.5399$$

x_1 = (0.61)√(1440.0000*0.1523) + (3.8400)(0.5399)(7.1667) = 23.8913
x_2 = (12)(0.5399)(7.1667) = 46.4333
x_3 = (1.22)√(1440.0000*0.1523) = 18.0653
x = smaller of x_1, x_2, and x_3 = 18.0653

$$t_{dx} = \frac{(2.6)(240.0000)(7.1667 - 18.0653/12)(1.0000)}{23200.0000} + 0.1875 = 0.3398 \text{ in}$$

For Hydrotest per 5.6.4.6:

t_u = 0.1430 t_L = 0.6240
K = t_L/t_u = 0.6240/0.1430 = 4.3626

$$C = \frac{(\sqrt{4.3626})(4.3626-1)}{1 + (4.3626)^{1.5}} = 0.6946$$

x_1 = (0.61)√(1440.0000*0.1430) + (3.8400)(0.6946)(7.1667) = 27.8687
x_2 = (12)(0.6946)(7.1667) = 59.7318
x_3 = (1.22)√(1440.0000*0.1430) = 17.5089
x = smaller of x_1, x_2, and x_3 = 17.5089

$$t_{tx} = \frac{(2.6)(240.0000)(7.1667 - 17.5089/12)}{24900.0000} = 0.1430 \text{ in}$$

(Per 5.6.1.1 - min. t = 0.3750 in)
Actual t = 0.4000 in

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Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Supported Cone Roof per API-650

Material Grade: A36
Roof Thickness (t_r): 0.1875 in
Corrosion Allowance (CA): 0.0000 in
Tank Diameter (D): 240.0000 ft

Roof is single lap welded

Roof Slope: 0.7500 in 12
ø: 3.5763°
Roof Rise: 7.5000 ft

Live Load (L_v): 25.0000 lbf/ft²
Snow Load (Sb): 25.2000 lbf/ft²
Dead Load (D_L): 8.4375 lbf/ft²
Ext. Pressure (P_e): 5.1960 lbf/ft²

Per 5.2.2(e)(1), $T = D_L + (L_v \text{ or } Sb) + 0.40P_e =$
$8.4375 + 25.2000 + (0.40)(5.1960) = 35.7159$ lbf/ft²

Per 5.2.2(e)(2), $T = D_L + 0.4(L_v \text{ or } Sb) + P_e =$
$8.4375 + (0.4)(25.2000) + 5.1960 = 23.7135$ lbf/ft²

Use greater of (1) or (2), T = 35.7159 lbf/ft²

Minimum thickness per 5.10.2.2 =0.1875 in

For Unbalanced Snow Load: Su = 1.0 Sb = 1.0 * 25.2000 = 25.2000 lbf/ft²

Per 5.2.2(e)(1), $U = D_L + Su + 0.40P_e =$
$8.4375 + 25.2000 + (0.40)(5.1960) = 35.7159$ lbf/ft²

Per 5.2.2(e)(2), $U = D_L + 0.4(Su) + P_e =$
$8.4375 + (0.4)(25.2000) + 5.1960 = 23.7135$ lbf/ft²

Use greater of (1) or (2), U = 35.7159 lbf/ft²

p = greater of T or U = 35.7159 lbf/ft²

Roof yield strength (Fy) = 36000.0000 psi

Maximum rafter spacing per 5.10.4.4,
$b = (t_r - CA)\sqrt{(1.5Fy * 144/p)}/12 =$
$(0.1875 - 0.0000)\sqrt{(1.5 * 36000.0000 * 144/35.7159)}/12 = 7.2907$ ft
Not to exceed 7.0000 ft
Therefore, b = 7.0000 ft

Quantity of rafters (Q) = 160
Actual rafter spacing (R) = π D/Q = π 240.0000/160 = 4.7124 ft

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Reactions for Inner Rafters

Total Rafter Length: 37.0956 ft
Rafter Weight: 12.0000 lbf/ft
Roof DL + LL: 35.7159 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Inner **Spacing (ft)**	Outer **Spacing (ft)**	Constant **Load (lbf/ft)**	Variable **Load (lbf/ft)**	**Inner Reaction (lbf)**	**Outer Reaction (lbf)**
35.5956	0.4581	5.1085	28.3632	166.0926		

(28.3632)(35.5956) + (166.0926)(35.5956)/2 - 2475.5258 = 1490.1643

(28.3632)(35.5956)/2 + (166.0926)(35.5956)/3 = 2475.5258

34.2459	0.4581	4.9322	28.3632	159.7945		

(28.3632)(34.2459) + (159.7945)(34.2459)/2 - 2309.7616 = 1397.7114

(28.3632)(34.2459)/2 + (159.7945)(34.2459)/3 = 2309.7616

33.5999	0.4581	4.8478	28.3632	156.7804		

(28.3632)(33.5999) + (156.7804)(33.5999)/2 - 2232.4360 = 1354.4683

(28.3632)(33.5999)/2 + (156.7804)(33.5999)/3 = 2232.4360

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Reactions for Intermediate Rafters

Total Rafter Length: 44.5686 ft
Rafter Weight: 19.0000 lbf/ft
Roof DL + LL: 35.7159 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Inner **Spacing (ft)**	Outer **Spacing (ft)**	Constant **Load (lbf/ft)**	Variable **Load (lbf/ft)**	**Inner Reaction (lbf)**	**Outer Reaction (lbf)**
42.5686	2.9039	6.2393	122.7170	119.1241		

(122.7170)(42.5686) + (119.1241)(42.5686)/2 - 4302.2614 = 3457.1036

(122.7170)(42.5686)/2 + (119.1241)(42.5686)/3 = 4302.2614

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Reactions for Outer Rafters

Total Rafter Length: 42.6031 ft
Rafter Weight: 16.0000 lbf/ft
Roof DL + LL: 35.7159 lbf/ft²

Outer Reaction = (CL)(RL)/2 +(VL)(RL)/3

Inner Reaction = (CL)(RL) + (VL)(RL)/2 - OR

Rafter Unsupported Length (ft)	Outer **Spacing** (ft)	Inner **Spacing** (ft)	Constant **Load** (lbf/ft)	Variable **Load** (lbf/ft)	**Inner Reaction** (lbf)	**Outer Reaction** (lbf)
40.3745	4.7124	3.1300	127.7898	56.5171		

(127.7898)(40.3745) + (56.5171)(40.3745)/2 - 3340.3440 = 2960.0353

(127.7898)(40.3745)/2 + (56.5171)(40.3745)/3 = 3340.3440

40.8698	4.7124	3.1106	127.0965	57.2104		

(127.0965)(40.8698) + (57.2104)(40.8698)/2 - 3376.5943 = 2986.8986

(127.0965)(40.8698)/2 + (57.2104)(40.8698)/3 = 3376.5943

41.2378	4.7124	3.0961	126.5813	57.7256		

(126.5813)(41.2378) + (57.7256)(41.2378)/2 - 3403.4624 = 3006.7160

(126.5813)(41.2378)/2 + (57.7256)(41.2378)/3 = 3403.4624

41.4816	4.7124	3.0866	126.2400	58.0669		

(126.2400)(41.4816) + (58.0669)(41.4816)/2 - 3421.2245 = 3019.7730

(126.2400)(41.4816)/2 + (58.0669)(41.4816)/3 = 3421.2245

41.6031	4.7124	3.0818	126.0700	58.2369		

(126.0700)(41.6031) + (58.2369)(41.6031)/2 - 3430.0609 = 3026.2556

(126.0700)(41.6031)/2 + (58.2369)(41.6031)/3 = 3430.0609

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Job Number:
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Loadings and Moments for Intermediate Girders

Total Girder Length: 30.6147 ft
Girder Weight: 46.0000 lbf/ft

Description	Load on Girder (lbf)	Distance from Girder Centerline to Load Centerline (ft)	Bending Moment (ft-lb)	Load on Column (lbf)
Girder	1408.28			704.14
		$(46.0000)(30.6147)^2/8 =$	5389.24	
Intermediate Rafter	3476.10	13.6335		3476.10
		$(3476.1036)(30.6147/2 - 13.6335) =$	5818.48	
Intermediate Rafter	3476.10	10.4224		3476.10
		$(3476.1036)(30.6147/2 - 10.4224) =$	16980.41	
Intermediate Rafter	3476.10	7.3508		3476.10
		$(3476.1036)(30.6147/2 - 7.3508) =$	27657.60	
Intermediate Rafter	3476.10	4.3739		3476.10
		$(3476.1036)(30.6147/2 - 4.3739) =$	38005.64	
Intermediate Rafter	3476.10	1.4520		3476.10
		$(3476.1036)(30.6147/2 - 1.4520) =$	48162.68	
Inner Rafter	2487.53	12.5446		2487.53
		$(2487.5258)(30.6147/2 - 12.5446) =$	6872.40	
Inner Rafter	2337.96	7.3508		2337.96
		$(2337.9586)(30.6147/2 - 7.3508) =$	18601.96	
Inner Rafter	2268.38	2.4222		2268.38
		$(2268.3846)(30.6147/2 - 2.4222) =$	29228.51	
		Total:	196716.92	25178.52
			Total load on Column:	50357.05

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Loadings and Moments for Outer Girders

Total Girder Length: 31.2145 ft
Girder Weight: 46.0000 lbf/ft

Description	Load on Girder (lbf)	Distance from Girder Centerline to Load Centerline (ft)	Bending Moment (ft-lb)	Load on Column (lbf)
Girder	1435.86			717.93
		(46.0000)(31.2145)²/8 =	5602.47	
Outer Rafter	2995.69	14.0117		2995.69
		(2995.6918)(31.2145/2 - 14.0117) =	4779.76	
Outer Rafter	3014.63	10.8527		3014.63
		(3014.6315)(31.2145/2 - 10.8527) =	14333.07	
Outer Rafter	3028.56	7.7279		3028.56
		(3028.5597)(31.2145/2 - 7.7279) =	23862.97	
Outer Rafter	3037.72	4.6272		3037.72
		(3037.7159)(31.2145/2 - 4.6272) =	33354.21	
Outer Rafter	3042.26	1.5408		3042.26
		(3042.2556)(31.2145/2 - 1.5408) =	42793.63	
Intermediate Rafter	4321.26	12.4273		4321.26
		(4321.2614)(31.2145/2 - 12.4273) =	13741.33	
Intermediate Rafter	4321.26	6.1752		4321.26
		(4321.2614)(31.2145/2 - 6.1752) =	40758.43	
Intermediate Rafter	4321.26	0.0000		2160.63
		(4321.2614)(31.2145/4) =	33721.45	
		Total:	212947.32	26639.94
		Total load on Column:		53279.88

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Inner Intermediate Girder for Three-Span System

Girder description: W18x 46
Quantity (Q): 8
Material grade: A992-GR50
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 712.0000 in^4
S_x = 78.8000 in³
Area (A) = 13.5000 in²
Radius of Gyration (r) = 7.2500 in

Girder length:
Distance from tank centerline to column centerline (C_o) = 40.0000 ft

Girder length (L) = 2(C_o)*sin(360/(2Q) =
2 * (40.0000) * sin(360/(2 * 8)) = 30.6147 ft

Girder unsupported length L_1 = 6.2393 ft

L/r = 12 * 30.6147/7.2500 = 10.3271

Bending Moment due to rafter loads and girder weight:
M = 196716.9169 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 209739.9815 ft-lb

Per AISC360-10:
Allowable Flexure Strength (M_c) = 209739.9815 ft-lb

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Outer Intermediate Girder for Three-Span System

Girder description: W18x 46
Quantity (Q): 16
Material grade: A992-GR50
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 712.0000 in^4
S_x = 78.8000 in^3
Area (A) = 13.5000 in^2
Radius of Gyration (r) = 7.2500 in

Girder length:
Distance from tank centerline to column centerline (C_o) = 80.0000 ft

Girder length (L) = 2(C_o)*sin(360/(2Q) =
2 * (80.0000) * sin(360/(2 * 16)) = 31.2145 ft

Girder unsupported length L_1 = 3.1300 ft

L/r = 12 * 31.2145/7.2500 = 5.1806

Bending Moment due to rafter loads and girder weight:
M = 212947.3180 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 226297.4052 ft-lb

Per AISC360-10:
Allowable Flexure Strength (M_c) = 226297.4052 ft-lb

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Inner Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W10x 12
Quantity (Q): 48
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 53.8000 in^4
S_x = 10.9000 in^3
Area (A) = 3.5400 in^2
Radius of Gyration (r) = 3.9000 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 3.5000 ft
Distance from tank centerline to outer edge of rafter (D_o) = 39.0263 ft

Inner spacing (S_i) = $2\pi D_i/Q = 2\pi(3.5000/48) = 0.4581$ ft
Outer spacing (S_o) = $2\pi D_o/Q = 2\pi(39.0263/48) = 5.1085$ ft

Rafter length (L) = $(D_o - D_i)/\cos(3.5763)$ =
$(39.0263 - 3.5000)/\cos(3.5763) = 35.5956$ ft

For Inner Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

$L_1 = L/3 = 35.5956/3 = 11.8652$ ft

L/r = 12 * 35.5956/3.9000 = 109.5250

For dead load only:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(8.4375) + 12.0000 = 15.8656 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(8.4375) = 39.2376 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[39.2376/(2*35.5956)x² + (15.8656)x -
(15.8656*35.5956/2 + 39.2376*35.5956/6) = 0

(0.5512)x² + (15.8656)x - (515.1546) = 0
therefore x = 19.3981 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(15.8656)(19.3981)(35.5956 - 19.3981)/2 +
(39.2376)(19.3981)(35.5956² - 19.3981²)/(6*35.5956) = 5667.0008 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 10024.3703 ft-lb

For dead load + live load:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(35.7159) + 12.0000 = 28.3632 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(35.7159) = 166.0926 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[166.0926/(2*35.5956)x² + (28.3632)x -
(28.3632*35.5956/2 + 166.0926*35.5956/6) = 0

(2.3330)x² + (28.3632)x - (1490.1643) = 0
therefore x = 19.9151 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(28.3632)(19.9151)(35.5956 - 19.9151)/2 +
(166.0926)(19.9151)(35.5956² - 19.9151²)/(6*35.5956) = 17909.6235 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 31437.1257 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (28.3632)(35.5956)/2 + (166.0926)(35.5956)/3 = 2475.5258$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(28.3632)(35.5956) + (166.0926)(35.5956)/2 - 2475.5258 = 1490.1643$ lbf

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Intermediate Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W12x 19
Quantity (Q): 80
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 130.0000 in^4
S_x = 21.3000 in^3
Area (A) = 5.5700 in^2
Radius of Gyration (r) = 4.8200 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 36.9552 ft
Distance from tank centerline to outer edge of rafter (D_o) = 79.4409 ft

Inner spacing (S_i) = $2\pi D_i/Q = 2\pi(36.9552/80) = 3.1416$ ft
Outer spacing (S_o) = $2\pi D_o/Q = 2\pi(79.4409/80) = 6.2393$ ft

Rafter length (L) = $(D_o - D_i)/\cos(3.5763)$ =
$(79.4409 - 36.9552)/\cos(3.5763) = 42.5686$ ft

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

$L_1 = L/3 = 42.5686/3 = 14.1895$ ft

$L/r = 12 * 42.5686/4.8200 = 105.9799$

For dead load only:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(8.4375) + 19.0000 = 45.5072 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(8.4375) = 26.1367 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[26.1367/(2*42.5686)x² + (45.5072)x -
(45.5072*42.5686/2 + 26.1367*42.5686/6) = 0

(0.3070)x² + (45.5072)x - (1154.0219) = 0
therefore x = 22.0725 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(45.5072)(22.0725)(42.5686 - 22.0725)/2 +
(26.1367)(22.0725)(42.5686² - 22.0725²)/(6*42.5686) = 13286.2704 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 17330.4213 ft-lb

For dead load + live load:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(35.7159) + 19.0000 = 131.2048 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(35.7159) = 110.6364 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[110.6364/(2*42.5686)x² + (131.2048)x -
(131.2048*42.5686/2 + 110.6364*42.5686/6) = 0

(1.2995)x² + (131.2048)x - (3577.5409) = 0
therefore x = 22.3288 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(131.2048)(22.3288)(42.5686 - 22.3288)/2 +
(110.6364)(22.3288)(42.5686² - 22.3288²)/(6*42.5686) = 42352.1637 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 61626.7465 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (131.2048)(42.5686)/2 + (110.6364)(42.5686)/3 = 4362.4801$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(131.2048)(42.5686) + (110.6364)(42.5686)/2 - 4362.4801 = 3577.5409$ lbf

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Outer Rafter for Three-Span System for Unbalanced Snow Load

Rafter description: W12x 16
Quantity (Q): 160
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 103.0000 in^4
S_x = 17.1000 in^3
Area (A) = 4.7100 in^2
Radius of Gyration (r) = 4.6700 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 78.4779 ft
Distance from tank centerline to outer edge of rafter (D_o) = 120.0000 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(78.4779/160)$ = 3.0818 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(120.0000/160)$ = 4.7124 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(120.0000 - 78.4779)/cos(3.5763) = 41.6031 ft

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 41.6031/3 = 13.8677 ft

L/r = 12 * 41.6031/4.6700 = 106.9030

For dead load only:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(8.4375) + 16.0000 = 42.0029 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(8.4375) = 13.7578 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[13.7578/(2*41.6031)x² + (42.0029)x -
(42.0029*41.6031/2 + 13.7578*41.6031/6) = 0

(0.1653)x² + (42.0029)x - (969.1189) = 0
therefore x = 21.2886 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(42.0029)(21.2886)(41.6031 - 21.2886)/2 +
(13.7578)(21.2886)(41.6031² - 21.2886²)/(6*41.6031) = 10581.4802 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 12405.6104 ft-lb

For dead load + live load:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(35.7159) + 16.0000 = 126.0700 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(35.7159) = 58.2369 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[58.2369/(2*41.6031)x² + (126.0700)x -
(126.0700*41.6031/2 + 58.2369*41.6031/6) = 0

(0.6999)x² + (126.0700)x - (3026.2556) = 0
therefore x = 21.4501 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(126.0700)(21.4501)(41.6031 - 21.4501)/2 +
(58.2369)(21.4501)(41.6031² - 21.4501²)/(6*41.6031) = 33608.0934 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 50149.7006 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (126.0700)(41.6031)/2 + (58.2369)(41.6031)/3 = 3430.0609$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(126.0700)(41.6031) + (58.2369)(41.6031)/2 - 3430.0609 = 3026.2556$ lbf

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Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Inner Rafter for Three-Span System

Rafter description: W10x 12
Quantity (Q): 48
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 53.8000 in^4
S_x = 10.9000 in^3
Area (A) = 3.5400 in^2
Radius of Gyration (r) = 3.9000 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 3.5000 ft
Distance from tank centerline to outer edge of rafter (D_o) = 39.0263 ft

Inner spacing (S_i) = $2\pi D_i/Q = 2\pi(3.5000/48) = 0.4581$ ft
Outer spacing (S_o) = $2\pi D_o/Q = 2\pi(39.0263/48) = 5.1085$ ft

Rafter length (L) = $(D_o - D_i)/\cos(3.5763)$ =
$(39.0263 - 3.5000)/\cos(3.5763) = 35.5956$ ft

For Inner Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

$L_1 = L/3 = 35.5956/3 = 11.8652$ ft

L/r = 12 * 35.5956/3.9000 = 109.5250

For dead load only:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(8.4375) + 12.0000 = 15.8656 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(8.4375) = 39.2376 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[39.2376/(2*35.5956)x² + (15.8656)x -
(15.8656*35.5956/2 + 39.2376*35.5956/6) = 0

(0.5512)x² + (15.8656)x - (515.1546) = 0
therefore x = 19.3981 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(15.8656)(19.3981)(35.5956 - 19.3981)/2 +
(39.2376)(19.3981)(35.5956² - 19.3981²)/(6*35.5956) = 5667.0008 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 10024.3703 ft-lb

For dead load + live load:

Rafter load (R_w) = 12.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (0.4581)(35.7159) + 12.0000 = 28.3632 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (5.1085 - 0.4581)(35.7159) = 166.0926 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[166.0926/(2*35.5956)x² + (28.3632)x -
(28.3632*35.5956/2 + 166.0926*35.5956/6) = 0

(2.3330)x² + (28.3632)x - (1490.1643) = 0
therefore x = 19.9151 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(28.3632)(19.9151)(35.5956 - 19.9151)/2 +
(166.0926)(19.9151)(35.5956² - 19.9151²)/(6*35.5956) = 17909.6235 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 31437.1257 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (28.3632)(35.5956)/2 + (166.0926)(35.5956)/3 = 2475.5258$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(28.3632)(35.5956) + (166.0926)(35.5956)/2 - 2475.5258 = 1490.1643$ lbf

Intermediate Rafter for Three-Span System

Rafter description: W12x 19
Quantity (Q): 80
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 130.0000 in^4
S_x = 21.3000 in^3
Area (A) = 5.5700 in^2
Radius of Gyration (r) = 4.8200 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 36.9552 ft
Distance from tank centerline to outer edge of rafter (D_o) = 79.4409 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = $2\pi(36.9552/80)$ = 3.1416 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = $2\pi(79.4409/80)$ = 6.2393 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(79.4409 - 36.9552)/cos(3.5763) = 42.5686 ft

For Intermediate Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 42.5686/3 = 14.1895 ft

L/r = 12 * 42.5686/4.8200 = 105.9799

For dead load only:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(8.4375) + 19.0000 = 45.5072 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(8.4375) = 26.1367 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[26.1367/(2*42.5686)x² + (45.5072)x -
(45.5072*42.5686/2 + 26.1367*42.5686/6) = 0

(0.3070)x² + (45.5072)x - (1154.0219) = 0
therefore x = 22.0725 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(45.5072)(22.0725)(42.5686 - 22.0725)/2 +
(26.1367)(22.0725)(42.5686² - 22.0725²)/(6*42.5686) = 13286.2704 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 17330.4213 ft-lb

For dead load + live load:

Rafter load (R_w) = 19.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.1416)(35.7159) + 19.0000 = 131.2048 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (6.2393 - 3.1416)(35.7159) = 110.6364 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[110.6364/(2*42.5686)x² + (131.2048)x -
(131.2048*42.5686/2 + 110.6364*42.5686/6) = 0

(1.2995)x² + (131.2048)x - (3577.5409) = 0
therefore x = 22.3288 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(131.2048)(22.3288)(42.5686 - 22.3288)/2 +
(110.6364)(22.3288)(42.5686² - 22.3288²)/(6*42.5686) = 42352.1637 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 61626.7465 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (131.2048)(42.5686)/2 + (110.6364)(42.5686)/3 = 4362.4801$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(131.2048)(42.5686) + (110.6364)(42.5686)/2 - 4362.4801 = 3577.5409$ lbf

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Outer Rafter for Three-Span System

Rafter description: W12x 16
Quantity (Q): 160
Material grade: A992-GR50
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

Minimum thickness per 5.10.2.4 = 0.1700 in

I = 103.0000 in^4
S_x = 17.1000 in^3
Area (A) = 4.7100 in^2
Radius of Gyration (r) = 4.6700 in

Spacing and lengths:
Distance from tank centerline to inner edge of rafter (D_i) = 78.4779 ft
Distance from tank centerline to outer edge of rafter (D_o) = 120.0000 ft

Inner spacing (S_i) = $2\pi D_i/Q$ = 2π(78.4779/160) = 3.0818 ft
Outer spacing (S_o) = $2\pi D_o/Q$ = 2π(120.0000/160) = 4.7124 ft

Rafter length (L) = (D_o - D_i)/cos(3.5763) =
(120.0000 - 78.4779)/cos(3.5763) = 41.6031 ft

For Outer Rafter for Three-Span System rafters, (2) lateral bracing is required equal spaced.

L_1 = L/3 = 41.6031/3 = 13.8677 ft

L/r = 12 * 41.6031/4.6700 = 106.9030

For dead load only:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load (D_L) = 8.4375 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(8.4375) + 16.0000 = 42.0029 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(8.4375) = 13.7578 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[13.7578/(2*41.6031)x² + (42.0029)x -
(42.0029*41.6031/2 + 13.7578*41.6031/6) = 0

(0.1653)x² + (42.0029)x - (969.1189) = 0
therefore x = 21.2886 ft

$M = (C_L)(x)(L - x)/2 + (V_L)(x)(L^2 - x^2)/(6L) =$
(42.0029)(21.2886)(41.6031 - 21.2886)/2 +
(13.7578)(21.2886)(41.6031² - 21.2886²)/(6*41.6031) = 10581.4802 ft-lb

Allowable bending moment:

Per AISC360-10:
Allowable Flexure Strength (M_c) = 12405.6104 ft-lb

For dead load + live load:

Rafter load (R_w) = 16.0000 lbf/ft
Dead load + Live load per App. R (D_L) = 35.7159 lbf/ft²

Constant load (C_L) = (S_i)(D_L) + R_w = (3.0818)(35.7159) + 16.0000 = 126.0700 lbf/ft
Variable load (V_L) = (S_o - S_i)(D_L) = (4.7124 - 3.0818)(35.7159) = 58.2369 lbf/ft

Determine location (x) where shear = 0 and calculate moment and stress

$(V_L/2L)x^2 + (C_L)x - (C_LL/2 + V_LL/6) = 0$
[58.2369/(2*41.6031)x² + (126.0700)x -
(126.0700*41.6031/2 + 58.2369*41.6031/6) = 0

(0.6999)x² + (126.0700)x - (3026.2556) = 0
therefore x = 21.4501 ft

$M = (CL)(x)(L - x)/2 + (VL)(x)(L^2 - x^2)/(6L) =$
(126.0700)(21.4501)(41.6031 - 21.4501)/2 +
(58.2369)(21.4501)(41.6031² - 21.4501²)/(6*41.6031) = 33608.0934 ft-lb

Determine allowable stress:

Per 5.10.4.3 the roof provides lateral support to the rafters.

Per AISC360-10:
Allowable Flexure Strength (M_C) = 50149.7006 ft-lb

Determine reaction at end of rafters:
At outer end:
$R_O = C_L L/2 + V_L L/3 = (126.0700)(41.6031)/2 + (58.2369)(41.6031)/3 = 3430.0609$ lbf
$R_i = C_L L + V_L L/2 - R_O =$
$(126.0700)(41.6031) + (58.2369)(41.6031)/2 - 3430.0609 = 3026.2556$ lbf

Hagen Engineering International
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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Inner Intermediate Girder for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Girder description: W18x 46
Quantity (Q): 8
Material grade:
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

I_x = 712.0000 in^4
S_x = 78.8000 in³
Zx = 90.7000 in³
r_x = 7.2500 in
I_y = 22.5000 in^4
S_y = 7.4300 in³
Z_y = 8.0000 in³
r_y = 1.2900 in
J = 1.2200 in^4
C_w = 1720.0000 in^6
A = 13.5000 in²

Unsupported Girder length (L_b) = 74.8713 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 3.0300 in
b/t_f = 5.0083

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 46.9167$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 90.7000 = 4535000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 1.2900 \sqrt{\frac{28800000.0000}{50000.0000}} = 54.4896$ in

$h_o = d - t_f = 18.1000 - 0.6050 = 17.4950$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{22.5000 * 1720.0000}}{78.8000}} = 1.5800$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$1.95 * 1.5800 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{1.2200 * 1.0000}{78.8000 * 17.4950} + \sqrt{\left(\frac{1.2200 * 1.0000}{78.8000 * 17.4950}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 163.6428$ in

$L_p < L_b \leq L_r$

$$(F2\text{-}2)\ M_n = C_b \left[M_p - (M_p - 0.7F_yS_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] =$$

$$1.0000 \left[4535000.0000 - (4535000.0000 - 0.7 * 50000.0000 * 78.8000) \left(\frac{74.8713 - 54.4896}{163.6428 - 54.4896} \right) \right] = 4203189.2296 \text{ in-lb}$$

For dead load + live load:

Minimmum Mn = 4203189.2296 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 4203189.2296/1.67 = 2516879.7782 in-lb
= 2516879.7782/12 = 209739.9815 ft-lb

Outer Intermediate Girder for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Girder description: W18x 46
Quantity (Q): 16
Material grade:
Yield strength (Fy) = 50000 psi
Corrosion allowance (CA) = 0.0000 in

Girder dimensions:
Flange width (b_f) = 6.0600 in
Flange thickness (t_f) = 0.6050 in
Beam Depth (d) = 18.1000 in
Web thickness (t_w) = 0.3600 in

I_x = 712.0000 in^4
S_x = 78.8000 in³
Zx = 90.7000 in³
r_x = 7.2500 in
I_y = 22.5000 in^4
S_y = 7.4300 in³
Z_y = 16.0000 in³
r_y = 1.2900 in
J = 1.2200 in^4
C_w = 1720.0000 in^6
A = 13.5000 in²

Unsupported Girder length (L_b) = 37.5597 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 3.0300 in
b/t_f = 5.0083

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 46.9167$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 90.7000 = 4535000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 1.2900 \sqrt{\frac{28800000.0000}{50000.0000}} = 54.4896$ in

The limit state of lateral-torsional buckling does not apply per F2.2(a).

For dead load + live load:

Minimmum $M_n = 4535000.0000$ in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega b = 4535000.0000/1.67 = 2715568.8623$ in-lb
$= 2715568.8623/12 = 226297.4052$ ft-lb

Hagen Engineering International
202 S. Cedar St. Suite B
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Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Inner Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W10x 12
Quantity (Q): 48
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9600 in
Flange thickness (t_f) = 0.2100 in
Beam Depth (d) = 9.8700 in
Web thickness (t_w) = 0.1900 in

I_x = 53.8000 in^4
S_x = 10.9000 in³
Zx = 12.6000 in³
r_x = 3.9000 in
I_y = 2.1800 in^4
S_y = 1.1000 in³
Z_y = 1.7400 in³
r_y = 0.7850 in
J = 0.0547 in^4
C_w = 50.9000 in^6
A = 3.5400 in²

Unsupported Rafter length (L_b) = 142.3825 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 1.9800 in
b/t_f = 9.4286

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Non-Compact

Web classification (AISC 16.1 B4)

$h/t_w = 49.7368$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 12.6000 = 630000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.7850 \sqrt{\frac{28800000.0000}{50000.0000}} = 33.1584$ in

$h_o = d - t_f = 9.8700 - 0.2100 = 9.6600$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.1800 * 50.9000}}{10.9000}} = 0.9831$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$1.95 * 0.9831 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.0547 * 1.0000}{10.9000 * 9.6600} + \sqrt{\left(\frac{0.0547 * 1.0000}{10.9000 * 9.6600}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 96.2291$ in

$$(F2\text{-}4)\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\,\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} =$$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\frac{142.3825}{0.9831}\right)^2}\sqrt{1 + 0.078\,\frac{0.0547 * 1.0000}{10.9000 * 9.6600}\left(\frac{142.3825}{0.9831}\right)^2}$$

$$= 18430.1266 \text{ psi}$$

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 18430.1266 * 10.9000 = 200888.3801$ in-lb $\leq M_p$

Compression Flange Local Buckling:

Per F3.2:

$\lambda = b_f/(2t_f) = 3.9600/(2 * 0.2100) = 9.4286$

$k_c = 4/\sqrt{(h/t_w)} = 4/\sqrt{(9.8700/0.1900)} = 377245.5090$
but not less than 0.35 nor greater than 0.76
therefore, $k_c = 0.5550$

$\lambda_{pf} = \lambda_p = 9.1200$

$\lambda_{rf} = \lambda_r = 24.0000$

$$(F3\text{-}1)\ M_n = M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) =$$

$$630000.0000 - (630000.0000 - 0.7 * 50000.0000 * 10.9000)$$

$$\left(\frac{9.4286 - 9.1200}{9.1200 - 24.0000}\right) = 629346.7742 \text{ in-lb}$$

For dead load only:

Minimmum Mn = 200888.3801 in-lb

Allowable Flexure Strength = M_c = Mn/Ωb = 200888.3801/1.67 = 120292.4432 in-lb
= 120292.4432/12 = 10024.3703 ft-lb

For dead load + live load:

Minimmum Mn = 630000.0000 in-lb

Allowable Flexure Strength = M_c = Mn/Ωb = 630000.0000/1.67 = 377245.5090 in-lb
= 377245.5090/12 = 31437.1257 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Intermediate Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W12x 19
Quantity (Q): 80
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 4.0100 in
Flange thickness (t_f) = 0.3500 in
Beam Depth (d) = 12.2000 in
Web thickness (t_w) = 0.2350 in

I_x = 130.0000 in^4
S_x = 21.3000 in³
Zx = 24.7000 in³
r_x = 4.8200 in
I_y = 3.7600 in^4
S_y = 1.8800 in³
Z_y = 2.9800 in³
r_y = 0.8220 in
J = 0.1800 in^4
C_w = 131.0000 in^6
A = 5.5700 in²

Unsupported Rafter length (L_b) = 170.2744 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 2.0050 in
b/t_f = 5.7286

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 48.9362$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 24.7000 = 1235000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.8220 \sqrt{\frac{28800000.0000}{50000.0000}} = 34.7213$ in

$h_o = d - t_f = 12.2000 - 0.3500 = 11.8500$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{3.7600 * 131.0000}}{21.3000}} = 1.0208$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$$1.95 * 1.0208 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.1800 * 1.0000}{21.3000 * 11.8500} + \sqrt{\left(\frac{0.1800 * 1.0000}{21.3000 * 11.8500}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 102.9701 \text{ in}$$

$$(F2\text{-}4)\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\,\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} =$$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\frac{170.2744}{1.0208}\right)^2}\sqrt{1 + 0.078\,\frac{0.1800 * 1.0000}{21.3000 * 11.8500}\left(\frac{170.2744}{1.0208}\right)^2}$$

$$= 16305.2415 \text{ psi}$$

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 16305.2415 * 21.3000 = 347301.6433$ in-lb $\leq M_p$

For dead load only:

Minimmum M_n = 347301.6433 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 347301.6433/1.67 = 207965.0559 in-lb
= 207965.0559/12 = 17330.4213 ft-lb

For dead load + live load:

Minimmum M_n = 1235000.0000 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 1235000.0000/1.67 = 739520.9581 in-lb
= 739520.9581/12 = 61626.7465 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Outer Rafter for Three-Span System

ALLOWABLE FLEXURE STRENGTH PER AISC360-10

Rafter description: W12x 16
Quantity (Q): 160
Material grade: The limit state of lateral-torsional buckling does not apply per F2.2(a).
Yield strength (Fy) = 50000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Rafter dimensions:
Flange width (b_f) = 3.9900 in
Flange thickness (t_f) = 0.2650 in
Beam Depth (d) = 12.0000 in
Web thickness (t_w) = 0.2200 in

I_x = 103.0000 in^4
S_x = 17.1000 in³
Zx = 20.1000 in³
r_x = 4.6700 in
I_y = 2.8200 in^4
S_y = 1.4100 in³
Z_y = 2.2600 in³
r_y = 0.7730 in
J = 0.1030 in^4
C_w = 96.9000 in^6
A = 4.7100 in²

Unsupported Rafter length (L_b) = 166.4123 in

Determine allowable stress:

E = 28800000.0000 psi

Flange classificati0n (AISC 16.1 B4)

b = 1.9950 in
b/t_f = 7.5283

$\lambda_p = 0.38\sqrt{(E/F_y)} = 0.38\sqrt{(28800000.0000/50000.0000)} = 9.1200$
$\lambda_r = 1.0\sqrt{(E/F_y)} = 1.0\sqrt{(28800000.0000/50000.0000)} = 24.0000$

Flanges are Compact

Web classification (AISC 16.1 B4)

$h/t_w = 52.1364$

$\lambda_p = 3.76\sqrt{(E/F_y)} = 3.76\sqrt{(28800000.0000/50000.0000)} = 90.2400$
$\lambda_r = 5.70\sqrt{(E/F_y)} = 5.70\sqrt{(28800000.0000/50000.0000)} = 136.8000$

Webs are Compact

(Per AISC 16.1 F1 for ASD) $\Omega b = 1.67$

Yielding:
(F2-1) $M_n = M_p = F_y * Z_x = 50000.0000 * 20.1000 = 1005000.0000$ in-lb

Lateral Torsional Buckling:

(F2-5) $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 * 0.7730 \sqrt{\frac{28800000.0000}{50000.0000}} = 32.6515$ in

$h_o = d - t_f = 12.0000 - 0.2650 = 11.7350$ in

(F2-8a) $c = 1.0$

(F2-7) $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.8200 * 96.9000}}{17.1000}} = 0.9832$ in

(F2-6) $L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{Sxho} + \sqrt{\left(\frac{Jc}{Sxho}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} =$

$$1.95 * 0.9832 \frac{28800000.0000}{0.7*50000.0000} \sqrt{\frac{0.1030 * 1.0000}{17.1000 * 11.7350} + \sqrt{\left(\frac{0.1030 * 1.0000}{17.1000 * 11.7350}\right)^2 + 6.76\left(\frac{0.7 * 50000.0000}{28800000.0000}\right)^2}} = 96.1502 \text{ in}$$

$$(F2\text{-}4)\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\,\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} =$$

$$\frac{1.0000 * \pi^2 * 28800000.0000}{\left(\frac{166.4123}{0.9832}\right)^2}\sqrt{1 + 0.078\,\frac{0.1030 * 1.0000}{17.1000 * 11.7350}\left(\frac{166.4123}{0.9832}\right)^2}$$

$$= 14538.5049 \text{ psi}$$

$L_b > L_r$

(F2-3) $M_n = F_{cr} * S_x = 14538.5049 * 17.1000 = 248608.4333$ in-lb $\leq M_p$

For dead load only:

Minimmum M_n = 248608.4333 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 248608.4333/1.67 = 148867.3253 in-lb
= 148867.3253/12 = 12405.6104 ft-lb

For dead load + live load:

Minimmum M_n = 1005000.0000 in-lb

Allowable Flexure Strength = $M_c = M_n/\Omega_b$ = 1005000.0000/1.67 = 601796.4072 in-lb
= 601796.4072/12 = 50149.7006 ft-lb

The limit state of lateral-torsional buckling does not apply per F2.2(a).

Per 5.10.4.3 the roof provides lateral support to the rafters.

Intermediate Column for Supported Cone Roof

(16) Columns located on 80.0000 ft Radius

Column description: 10" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 10.7500 in
Wall thickness (t_w) = 0.3650 in
Mill tolerance (m) = 12.50%

therefore $tw = t_w(1 - m) = 0.3650(1 - 0.1250) = 0.3194$ in

$t = t_w = 0.3194$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 10.7500 - 2*0.3194 = 10.1113$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(10.7500^2 - 10.1113^2)/4 = 10.4655$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(10.7500^2 + 10.1113^2)/4} = 3.6895$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((10.7500)^4 - (10.1113)^4)/10.7500 = 26.5046$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Column Location Radius (R_2) = 80.0000 ft

Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 80.0000)\tan(0.0624) = 2.5000$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(Rise)/Run = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 12.2000 in
Girder depth (L_6) = 18.1000 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 - L_6 =$
$(2.5000 + -1.0000 + 50.0000)12 - 12.2000 - 18.1000 = 585.5750$ in

KL/r = (1.00)(585.5750)/3.6895 = 158.7134

For Outer Intermediate Column, KL/r = 158.7

Calculate column load:

P (from girder loads) = 53279.8807 lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:

$\Omega_c = 1.67$

$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(158.7134)^2} = 11284.0454 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 11284.0454 = 9896.1078$ psi

(E3.1) $P_n = F_{cr}A = 9896.1078 * 10.4655 = 103567.9884$ lbf

$P_c = P_n/\Omega_c = 103567.9884/1.67 = 62016.7595$ lbf

For Outer Intermediate Column, load (53279.88) is less than the allowable load (62016.76).

Intermediate Column for Supported Cone Roof

(8) Columns located on 40.0000 ft Radius

Column description: 10" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 10.7500 in
Wall thickness (t_w) = 0.3650 in
Mill tolerance (m) = 12.50%

therefore $t_w = t_w(1 - m) = 0.3650(1 - 0.1250) = 0.3194$ in

$t = t_w = 0.3194$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 10.7500 - 2*0.3194 = 10.1113$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(10.7500^2 - 10.1113^2)/4 = 10.4655$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(10.7500^2 + 10.1113^2)/4} = 3.6895$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((10.7500)^4 - (10.1113)^4)/10.7500 = 26.5046$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Column Location Radius (R_2) = 40.0000 ft

Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 40.0000)\tan(0.0624) = 5.0000$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(\text{Rise})/\text{Run} = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 12.2000 in
Girder depth (L_6) = 18.1000 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 - L_6 =$
$(5.0000 + -1.0000 + 50.0000)12 - 12.2000 - 18.1000 = 615.5750$ in

KL/r = (1.00)(615.5750)/3.6895 = 166.8446

For Inner Intermediate Column, KL/r = 166.8

Calculate column load:

P (from girder loads) = 50357.0490 lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:

$\Omega_c = 1.67$

$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(166.8446)^2} = 10210.9919 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 10210.9919 = 8955.0399$ psi

(E3.1) $P_n = F_{cr}A = 8955.0399 * 10.4655 = 93719.2162$ lbf

$P_c = P_n/\Omega_c = 93719.2162/1.67 = 56119.2911$ lbf

For Inner Intermediate Column, load (50357.05) is less than the allowable load (56119.29).

Center Column for Supported Cone Roof

Column description: 14" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Outside Diameter (D_o) = 14.0000 in
Wall thickness (t_w) = 0.3750 in
Mill tolerance (m) = 12.50%

therefore $t_w = t_w(1 - m) = 0.3750(1 - 0.1250) = 0.3281$ in

$t = t_w = 0.3281$ in

Pipe Inside Diameter (D_i) = $D_o - 2t = 14.0000 - 2*0.3281 = 13.3438$ in

Area (A) = $\pi(Do^2 - Di^2)/4 = \pi(14.0000^2 - 13.3438^2)/4 = 14.0934$ in^2
Radius of Gyration (r) = $\sqrt{(Do^2 + Di^2)/4} =$
$\sqrt{(14.0000^2 + 13.3438^2)/4} = 4.8351$ in

Section Modulus (q_s) = $0.098175(D_o/^4 - d_1^4)/D_o =$
$0.098175((14.0000)^4 - (13.3438)^4)/14.0000 = 47.0692$ in^3

K = 1.00

Calculate column length:
Tank Radius (R_1) = 120.0000 ft

Cap Radius (R_2) = 3.5000 ft
Roof Rise (L_1) = $(R_1 - R_2)\tan(0.0624) = (120.0000 - 3.5000)\tan(0.0624) = 7.2813$ ft

Bottom with Annular Ring Sloped Up 1.0000 in 120

Bottom Slope (L_2) = $-(R_1)(\text{Rise})/\text{Run} = -(120.0000)(1.0000)/120.0000 = -1.0000$ ft

Tank vertical height (L3) = 50.0000 ft

Rafter depth (L_5) = 9.8700 in

Therefore, $L = (L_1 + L_2 + L_3)12 - L_5 =$
$(7.2813 + -1.0000 + 50.0000)12 - 9.8700 = 663.3800$ in

KL/r = (1.00)(663.3800)/4.8351 = 137.2000

For center column, KL/r = 137.2

Calculate column load:

Quantity of rafters (Q) = 48
Total load from Reactions for Inner Rafters (R_i) = 67877.5041 lbf
Rafter extension on cap (L_6) = 6.0000 in
Rafter weight (R_w) = 12.0000 lbf
Roof dead load + live load (D_L) = 35.7159 lbf/ft²
Weight of cap plate (C_w) = 3496.2470 lbf

$P = R_i + QR_w(L_6/12) + \pi R_2^2 D_L + C_w =$
$67877.5041 + (48)(12.0000)(6.0000/12) +$
$\pi(3.5000)^2(35.7159) + 3496.2470 = 73036.2601$ lbf

Modulus of Elasticity at 150.0000° (E): 28800000.0000 psi

DETERMINE ALLOWABLE LOAD PER AISC 360-10 E3:
$\Omega_c = 1.67$
$V = 4.71\sqrt{(E/F_y)} = 4.71\sqrt{[(28800000.0000)/35000.0000]} = 135.1086$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(28800000.0000)}{(137.2000)^2} = 15100.2361 \text{ psi}$$

(E3-3) $F_{cr} = 0.877F_e = 0.877 * 15100.2361 = 13242.9071$ psi

(E3.1) $P_n = F_{cr}A = 13242.9071 * 14.0934 = 186638.2285$ lbf

$P_c = P_n/\Omega_c = 186638.2285/1.67 = 111759.4183$ lbf

Load on center column (73036.26) is less than the allowable load (111759.42).

Center Column for Supported Cone Roof (Unbalanced Snow Load)

Column description: 14" Std Wt Pipe
Material grade: A53-B
Yield strength (Fy) = 35000.0000 psi
Corrosion allowance (CA) = 0.0000 in

Cap Radius (R_2) = 3.5000 ft

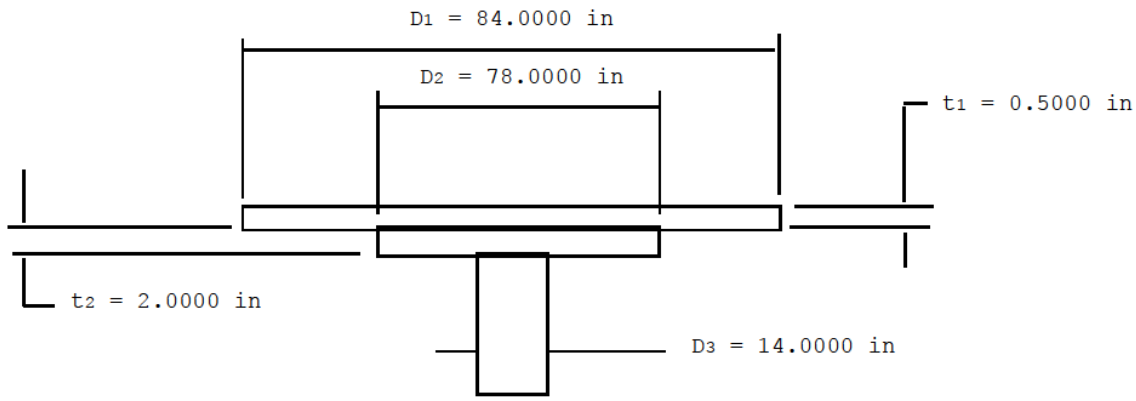
KL/r = (1.00)(663.3800)/4.8351 = 137.2000

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Center Column Cap Plate

Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 69540.0130 lbf



Top Plate:
$E = D_1/2 = 84.0000/2 = 42.0000$ in
$G = D_2/2 = 78.0000/2 = 39.0000$ in

$F_b = P_a[1.17 + 10*\log(E/G) - 1.17G^2/E^2]/(7 * (t_1 - CA)^2) =$
$69540.0130 * [1.17 + 10*\log(42.0000/39.0000) -$
$1.17*(39.0000)^2/(42.0000)^2]/(7 * (0.5000 - 0.0000)^2) = 19193.8504$ psi

$F_a = 0.75F_y = 0.75 * 36000.0000 = 27000.0000$ psi

Secondary Plate:
$E = D_2/2 = 84.0000/2 = 39.0000$ in
$G = D_3/2 = 14.0000/2 = 7.0000$ in

$F_b = P_a[1.17 + 10*\log(E/G) - 1.17G^2/E^2]/(7 * (t_2 - CA)^2) =$
$69540.0130 * [1.17 + 10*\log(39.0000/7.0000) -$
$1.17*(7.0000)^2/(39.0000)^2]/(7 * (2.0000 - 0.0000)^2) = 21338.7836$ psi

$F_a = 0.75F_y = 0.75 * 36000.0000 = 27000.0000$ psi

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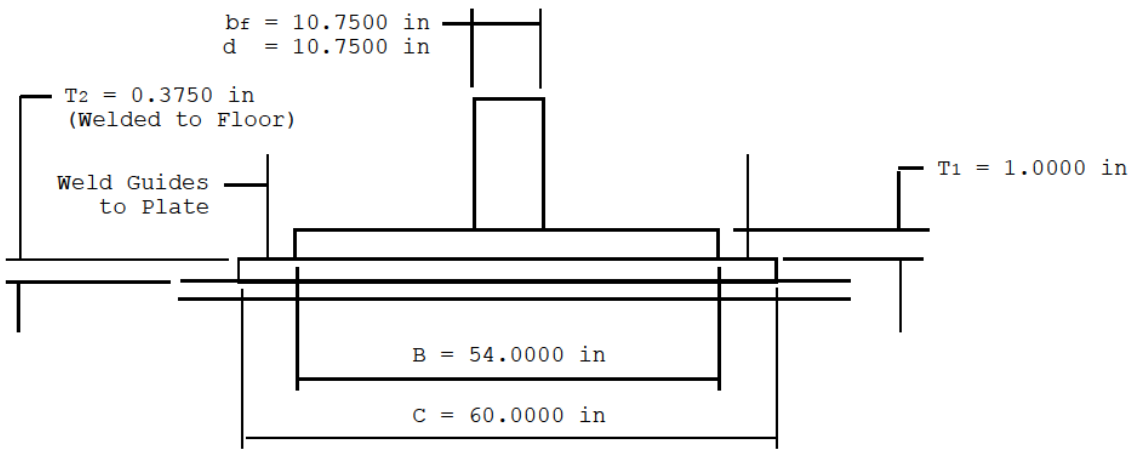
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Outer Intermediate Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 56082.2304 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (54.0000 - 0.7071 * 10.7500)/2 = 23.1993$ in

$m = (B - 0.7071 * d)/2 = (54.0000 - 0.7071 * 10.7500)/2 = 23.1993$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(10.7500 * 10.7500)/4} = 2.6875$ in

ℓ = greater of n, m, and n' = 23.1993 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$23.1993\sqrt{[3.33 * 56082.2304/(36000.0000 * (54.0000)^2]} + 0.0000$
$= 0.9785$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (60.0000 - 54.0000)/2 = 3.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$3.0000\sqrt{[3.33 * 56082.2304/(36000.0000 * (60.0000)^2]} + 0.0000$
$= 0.1139$ in

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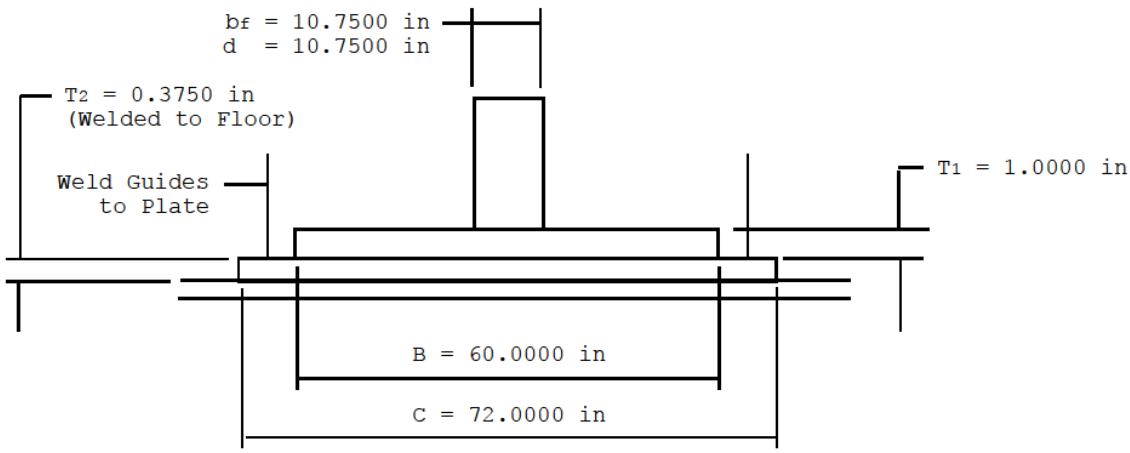
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Inner Intermediate Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 53454.5887 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (60.0000 - 0.7071 * 10.7500)/2 = 26.1993$ in

$m = (B - 0.7071 * d)/2 = (60.0000 - 0.7071 * 10.7500)/2 = 26.1993$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(10.7500 * 10.7500)/4} = 2.6875$ in

ℓ = greater of n, m, and n' = 26.1993 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$26.1993\sqrt{[3.33 * 53454.5887/(36000.0000 * (60.0000)^2]} + 0.0000$

$= 0.9710$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (72.0000 - 60.0000)/2 = 6.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$6.0000\sqrt{[3.33 * 53454.5887/(36000.0000 * (72.0000)^2]} + 0.0000$
$= 0.1853$ in

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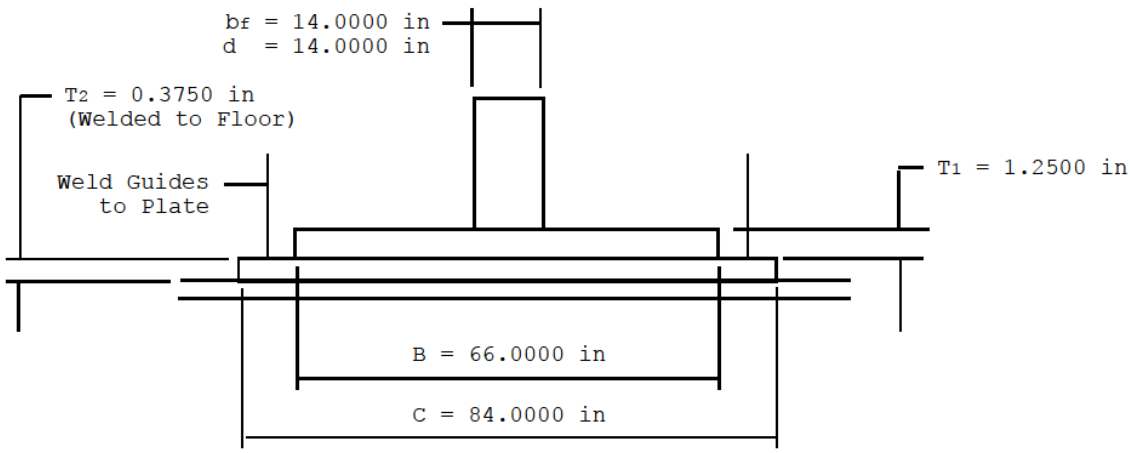
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Center Column Base Plate

Base plate description: Square Base Plate
Material grade: A36
Yield strength (F_y) = 36000 psi
Corrosion allowance (CA) = 0.0000 in
Total Column Load (P_a) = 81094.0430 lbf
Soil Bearing Capacity = 3500.0000 lbf/ft²



Per AISC 13th Edition, Part 14

For Base Plate:

$n = (B - 0.7071 * b_f)/2 = (66.0000 - 0.7071 * 14.0000)/2 = 28.0503$ in

$m = (B - 0.7071 * d)/2 = (66.0000 - 0.7071 * 14.0000)/2 = 28.0503$ in

$n' = \sqrt{(d * b_f)/4} = \sqrt{(14.0000 * 14.0000)/4} = 3.5000$ in

ℓ = greater of n, m, and n' = 28.0503 in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (B)^2]} + CA =$

$28.0503\sqrt{[3.33 * 81094.0430/(36000.0000 * (66.0000)^2]} + 0.0000$
$= 1.1640$ in

Per AISC 13th Edition, Part 14

For Sealed Wear Plate:

$\ell = n = m = (C - B)/2 = (84.0000 - 66.0000)/2 = 9.0000$ in

$t_{min} = \ell\sqrt{[3.33P_a/(F_y * (C)^2]} + CA =$

$9.0000\sqrt{[3.33 * 81094.0430/(36000.0000 * (84.0000)^2]} + 0.0000$
$= 0.2934$ in

Compression Ring

L4 x4 x3/ 8 per Detail d

4 inches Vertical leg of rim angle is butt welded to the top of the shell with the horizontal leg turned out

Material: A36

API-650 Figure F-2

R_c = 1440.0000 in

Roof Angle (ø) = 3.5763°

$R_2 = R_c/(\sin ø)$ = 1440.0000/(sin 3.5763) = 23084.9561 in

t_h = 0.1875 in

$w_h = 0.3 \sqrt{(R_2 t_h)} = 0.3 \sqrt{(23084.9561 * 0.1875)} = 19.7372$ in
not to exceed 12.0000 in
therefore w_h = 12.0000 in

Available area from roof (A_r) = $t_h w_h$ = 0.1875 * 12.0000 = 2.2500 in²

t_c = 0.4000 in

$w_c = 0.6 \sqrt{(R_c t_c)} = 0.6 \sqrt{(1440.0000 * 0.4000)} = 14.4000$ in

Distance from top of angle to top of shell (b) = 4.0000 in
Available area from shell (A_s) = $(w_c - b)t_c$ = (14.4000 - 4.0000) * 0.4000 = 4.1600 in²

Yield strength (F_y) = 36000.0000 psi

t_a = 0.3750 in
Horizontal projection of angle (a_1) = 3.6250 in
Maximum horizontal projection = $t_a * 3000/\sqrt{(F_y)}$ = 0.3750 * 3000/√(36000.0000) = 5.9293 in
therefore a_1 = 3.6250 in
Vertical projection of angle (a_2) = 4.0000 in
Maximum vertical projection = w_c = 14.4000 in
therefore a_2 = 4.0000 in
Available area from compression ring (A_c) = $(a_1 + a_2)t_a$ = (3.6250 + 4.0000)0.3750 = 2.8594 in²

Total available area (A_t) = $A_r + A_s + A_c$ = 2.2500 + 4.1600 + 2.8594 = 9.2694 in²

Check in Corroded Condition

Roof corrosion allowance = 0.0000 in
t_h = Roof thickness - CA = 0.1875 - 0.0000 = 0.1875 in

$w_h = 0.3 \sqrt{(R_2 t_h)} = 0.3 \sqrt{(23084.9561 * 0.1875)} = 19.7372$ in
not to exceed 12.0000 in
therefore w_h = 12.0000 in

Available area from roof (A_r) = $t_h w_h$ = 0.1875 * 12.0000 = 2.2500 in²

Shell corrosiona allowance = 0.1875 in
t_c = shell thickness - CA = 0.4000 - 0.1875 = 0.2125 in

$w_c = 0.6 \sqrt{(R_c t_c)} = 0.6 \sqrt{(1440.0000 * 0.2125)} = 10.4957$ in

Distance from top of angle to top of shell (b) = 4.0000 in
Available area from shell (A_s) = (w_c - b)t_c = (10.4957 - 4.0000) * 0.2125 = 1.3803 in²

Angle corrosion allowance = 0.0000 in
t_a = Angle thickness - CA = 0.3750 - 0.0000 = 0.3750 in
Horizontal projection of angle (a_1) = 3.6250 in
Maximum horizontal projection = $t_a * 3000/\sqrt{(F_y)}$= 0.3750 * 3000/$\sqrt{(36000.0000)}$
= 5.9293 in
therefore a_1 = 3.6250 in
Vertical projection of angle (a_2) = 4.0000 in
Maximum vertical projection = w_c = 10.4957 in
therefore a_2 = 4.0000 in
Available area from compression ring (A_c) = (a_1 + a_2)t_a =
(3.6250 + 4.0000)0.3750 = 2.8594 in²

Total available area (A_t) = A_r + A_s + A_c = 2.2500 + 1.3803 + 2.8594 = 6.4897 in²

Tank meets the frangibility requirements of 5.10.2.6.

DLs = 1254076.8087 lbs
DLs/(2π * F_y * tan ø) = 1254076.8087/(2π * 36000.0000 * tan 3.5763) = 88.7078 in²

The roof support members shall not be attached to the roof plate.

The roof must be attached to the top angle with a single continuous fillet weld on the top side only that does not exceed 3/16'. No underside welding of roof to top angle (including seal welding) is permitted.

Wind Calculations

Wind Velocity per ASCE 7-10 Figure 26.5-1A = 135 mph
Tank diameter (D) = 240.0000 ft
Thinnest shell course thickness (t) = 0.4000 in

Exposure C

Risk Category = II

Tank shell has 3.0000 in of insulation. Therefore, Di = 240.5000 ft

Tank roof has 3.0000 in of insulation.

Per 5.2.1 (k)(1)(b), wind loads are based on ASCE 7-10.

For Shell:
Centroid of shell C_s = 25.1250 ft
z = C_s but not less than 15.00 ft
z = 25.1250 ft from bottom of tank
Per ASCE7, zg = 900.00 ft
α = 9.50
K_z = 2.01(25.13/900.00)^(2/9.50) = 0.946

K_d = 0.95
K_{zt} = 1.00

h/D = 50.00/240.00 = 0.21

Per 5.2.1(k)(1)(b), V = (0.78)(135) = 105.30 mph

$q_z = 0.00256 K_z K_{zt} K_d V^2$ =
$(0.00256)(0.946)(1.00)(0.95)(105.30)^2 = 25.52$ lbf/ft²

C_f = 0.50

G = 0.85
$P_w = q_z G C_f$ = (25.52)(0.85)(0.5000) = 10.8447 lbf/ft²
but not less than 10.0000 lbf/ft²
= 10.8447 lbf/ft²

For Roof:
Top of roof C_s = 57.75 ft
z = C_s but not less than 15.00 ft
z = 57.75 ft from bottom of tank
Per ASCE7, zg = 900.00 ft
α = 9.50
K_z = 2.01(57.75/900.00)^(2/9.50) = 1.127

$K_d = 0.85$
$K_{zt} = 1.00$
$V = 105.30$ mph

$q_z = 0.00256K_zK_{zt}K_dV^2 =$
$(0.00256)(1.127)(1.00)(0.85)(105.30)^2 = 27.2031$ lbf/ft²

$GC_p = 0.90$
$U_w = q_zGC_p = (27.2031)(0.90) = 24.4828$ lbf/ft²

$P_{wr} = q_zGC_f = (27.2031)(0.85)(0.5000) = 11.5613$ lbf/ft²
but not less than 10.0000 lbf/ft²
= 11.56 lbf/ft²

Per 5.9.7.1:

Per ASCE7, $K_z = 0.95$
$K_d = 0.95$
$K_{zt} = 1.00$
$V = 120.00$ mph
$G = 0.85$
$P = 5.00$ lbf/ft²
$P_w = 0.00256K_zK_{zt}K_dV^2IG + P =$
$(0.00256)(0.95)(1.00)(0.95)(120.00)^2(1.00)(0.85) + 5.00 = 33.1679$ lbf/ft²

Multiplier (m) = $P_w/36.00 = 33.1679/36.00 = 0.9213$
not less than 1.0, therefore m = 1.0000

Per 5.9.7.1: $H_1 = 600000.00t[\sqrt{(t/D)^3}](120/V)^2 =$
$(600000.00)(0.4000)[\sqrt{(0.4000 / 240.0000)^3}](120/105.30)^2 = 21.2075$ ft

Per 5.9.7.2: $t_{uniform} = t = 0.4000$ in

For each shell course, $W_{tr} = W \sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5}$

(Shell courses are numbered from bottom to top)

Shell Course	W (ft)	T_{actual} (in)	W_{tr} (ft)
Angle	0.3333	0.3750	0.3917
5	9.8333	0.4000	9.8333
4	9.9583	0.6240	3.2762
3	9.9583	0.7400	2.1392
2	9.9583	0.9386	1.1807
1	9.9583	1.1964	0.6436
		Total Transposed Width:	17.4648

Overturning moment:

Total shell vertical height (Hi) = 50.2500 ft
Vertical roof area (A_r) = 900.0000 ft²
Distance to roof (H_r) = 57.7500 ft

Overturning Moment (M_{ws}) = ($P_w D_i H_i^2/2 + P_{wr} A_r H_r$) =
= [(10.8447)(240.5000)(50.2500)²/2 + (11.5613)(900.0000)(57.7500)]
= 3893780.1993 ft-lbs

Tank does not contain internal pressure. Therefore, MPi = 0

Check for tank stability without anchors per 5.11.2

Weight of shell (D_{LS}) = 1254076.8087 lbs
$M_{DL} = D_{LS} * D/2$ = 1254076.8087 * 240.0000 / 2 = 150489217.0455 ft-lbs
Weight of roof (D_{LR}) = 347093.0752 lbs
$M_{DLR} = D_{LR} * D/2$ = 347093.0752 * 240.0000 / 2 = 41651169.0299 ft-lbs

Bottom thickness used (t_b) = 0.3750 in
Bottom material yield strength (F_{by}) = 36000.0000 psi
$w_L = 4.6700 t_b \sqrt{(F_{by} H)}$ = (4.6700)(0.3750)√(36000.0000 * 47.0000) = 2277.9717 lbf/ft

not to exceed 0.450HD = 0.450 * 47.0000 * 240.0000 = 5076.0000 lbf/ft
therefore, w_L = 2277.9717 lbf/ft

For tank to be stable: $M_{ws} + 0.40 M_{pi} < M_{DL}/1.5 + M_{DLR}$

$M_{ws} + 0.40 M_{pi}$ = (3893780.1993) + (0.40)(0.0000) = 3893780.1993 ft-lbs
$M_{DL}/1.5 + M_{DLR}$ = 150489217.0455/1.5 + 41651169.0299 = 141977313.7269 ft-lbs

Check for Sliding (tank empty):

Sliding Force (V_w) = [($P_w D_i H_i + P_{wr} A_r$)] =
[((10.8447)(240.5000)(50.2500) +
(11.56)(900.00))] = 141,465.10 lbs

Resistance to Sliding (V_s) = 0.4(Ws + Wr + Wf) =
0.4(1201451.6953 + 545268.2652 + 707430.0397) = 981,660.00 lbs

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Weights

	New (lbf)	Corroded (lbf)
Product:	131517746.9188	131517746.9188
Total Shell Plates:	1194062.6694	907306.2601
Total Bottom Plates:	707430.0397	475843.3218
Total Roof Plates:	347093.0752	347093.0752
Total Live Load:	1130973.3553	1130973.3553
Total Snow Load:	1140021.1421	1140021.1421
Total Additional Dead Load:	0.0000	0.0000
Shell Insulation:	29405.3072	29405.3072
Roof Insulation:	35355.2202	35355.2202
Total Rafters:	198175.1899	198175.1899
Total Roof and Framing:	580623.4854	580623.4854
Total Shell Appurtenances:	0.0000	0.0000
Total Roof Appurtenances:	0.0000	0.0000
Total Roof Appurtenances acting on Shell	0.0000	0.0000
Live Load acting on Shell:	343312.6708	343312.6708
Snow Load acting on Shell:	346059.1721	346059.1721
Total of Roof acting on Shell:	105156.6711	105156.6711
Total of Roof Structural acting on Shell:	52625.1134	52625.1134
Total Compression Ring and Shell Stiffener	7389.0259	7389.0259

Seismic Calculations

Tank diameter (D): 240.0000 ft
Product Height (H): 47.0000 ft
Specific Gravity (G): 1.0000

Tank is self-anchored

Mapped spectral response acceleration (S_S): 0.9400 g
Mapped spectral response acceleration (S_1): 0.4120 g
Mapped spectral response acceleration (S_0) = $0.4S_S$ = (0.4)(0.9400) = 0.3760 g
Mapped spectral response acceleration (S_P) = $0.8S_1$ = (0.8)(0.4120) = 0.3296 g

Site Class per E.4.4 : E
Seismic Use Group per E.3.1 : I

From Table E-1, F_a = 0.9720
From Table E-2, F_v = 2.4000
From Table E-5, I = 1.0000
From Table E-4, R_{wi} = 3.5000
From Table E-4, R_{wc} = 2.0000

Scaling factor per E.2, Q = 2/3

$S_{DS} = QF_aS_S$ = (2/3)(0.9720)(0.9400) = 0.6091
$S_{D1} = QF_vS_1$ = (2/3)(2.4000)(0.4120) = 0.6592

Per E.6.1.3, Av = 0.7 x Purchaser specified vertical acceleration
Vertical earthquake acceleration coefficient (A_v): 0.0000 g

E.4.5.2 Convective (Sloshing) Period

E.4.5.2-c:

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}} = \frac{0.578}{\sqrt{\tanh\left(\frac{(3.68)(47.0000)}{240.0000}\right)}} = 0.7357$$

Per E4.5.2-b: $T_c = 1.0K_s\sqrt{D}$ = (1.0)(0.7357)√240.0000 = 11.3967

E.4.6.1 Spectral Acceleration Coefficients

Impulsive spectral acceleration parameter:

Per E.4.6.1-1:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 0.6091\left(\frac{1.00}{3.5000}\right) = 0.1740$$

per E.4.6.1-3:

$$A_i \geq 0.5S_1\left(\frac{I}{R_{wi}}\right) = (0.5)(0.4120)\left(\frac{1.00}{3.5000}\right) = 0.0589$$

However, per E.4.6.1-2, $A_i \geq 0.0070$
Therefore, $A_i = 0.1740$

Convective spectral acceleration parameter:

$T_L = 16.0000$
$K = 1.5$

$T_c \leq T_L$, Per E.4.6.1-4

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = (1.5)(0.6592)\left(\frac{1}{11.3967}\right)\left(\frac{1.00}{2.0000}\right) = 0.0434$$

However, $A_c \leq A_i$
Therefore, $A_c = 0.0434$

Per E.6.1.1

D/H = 240.0000/47.0000 = 5.1064

Weights:
W_s = 1230857 lbs
W_f = 707430 lbs
W_r = Roof weight + 10% of snow = 694626 lbs
W_P = 131517747 lbs

D/H ≥ 1.333

Per E.6.1.1-1,

$$W_i = \frac{\tanh(0.866D/H)}{0.866D/H} W_P = \frac{\tanh(0.866 * 5.1064)}{0.866 * 5.1064}(131517747) = 29732254 \text{ lbs}$$

Per E.6.1.1-3, $W_c = 0.230D/H \tanh(3.67H/D)W_P =$
(0.230)(5.1064)[tanh(3.67/5.1064)](131517747) = 95166223 lbs

Per E.6.1-2, $V_i = A_i(W_s + W_r + W_f + W_i) =$
(0.1740)(1230857 + 694626 + 707430 + 29732254) = 5632649 lbs
Per E.6.1-3, $V_c = A_cW_c$ = (0.0434)(95166223) = 4128419 lbs
Per E.6.1-1, $V = \sqrt{(V_i^2 + V_c^2)} = \sqrt{[(5632649)^2 + (4128419)^2]}$ = 6983593 lbs

Per E.6.1.2.1

D/H ≥ 1.333

Per E.6.1.2.1-1, $X_i = 0.375H$ = (0.375)(47.0000) = 17.6250 ft

Per E.6.1.2.1-3,

$$X_c = \left[1.0 - \frac{\cosh(3.67H/D) - 1}{3.67H/D \sinh(3.67H/D)} \right] H =$$

$$\left[1.0 - \frac{\cosh(3.67/5.1064) - 1}{(3.67/5.1064)\sinh(3.67/5.1064)} \right] (47.0000) = 24.4619 \text{ ft}$$

Per E.6.1.2.2

$D/H \geq 1.333$

Per E.6.1.2.2-1,

$$X_{is} = 0.375 \left[1.0 + 1.333 \left(\frac{(0.866D/H)}{\tanh(0.866D/H)} - 1.0 \right) \right] H =$$

$$0.375 \left[1.0 + 1.333 \left(\frac{(0.866 * 5.1064)}{\tanh(0.866 * 5.1064)} - 1.0 \right) \right] (47.0000) =$$

98.0549 ft

Per E.6.1.2.2-3,

$$X_{cs} = \left[1.0 - \frac{\cosh(3.67H/D) - 1.937}{3.67H/D \sinh(3.67H/D)} \right] H =$$

$$\left[1.0 - \frac{\cosh(3.67/5.1064) - 1.937}{(3.67/5.1064)\sinh(3.67/5.1064)} \right] (47.0000) = 102.7989 \text{ ft}$$

Per E.6.1.5

Per E.6.1.5-1, $M_{rw} = \sqrt{\{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2\}} =$
$\sqrt{\{[0.1740(29732254*17.6250 + 1230857*19.9796 + 694626*52.5000)]^2 + [0.0434(95166223*24.4619)]^2\}} = 143412972$ ft-lbs

Per E.6.1.5-2, $M_s = \sqrt{\{[A_i(W_iX_{is} + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_{cs})]^2\}} =$
$\sqrt{\{[0.1740(29732254*98.0549 + 1230857*19.9796 + 694626*52.5000)]^2 + [0.0434(95166223*102.7989)]^2\}} = 669657846$ ft-lbs

Self-anchored tank (new condition)

Per E.6.2.1.1.1

$W_s = 1201452$ lbs
$w_{rs} = 255.1621$ lbf/ft
$t_a = 0.5000$ in
$t_s = 1.0088$ in
$G_e = G(1.0 - 0.4A_v) = 1.0000(1.0 - (0.4)(0.0000)) = 1.0000$
Per E.6.2.1.1.1-2, $w_t = W_s/(\pi D) + w_{rs} = 1201452/(\pi * 240.0000) + 255.1621$
$= 1848.6369$ lbf/ft

Check annular ring:

Per E.6.2.1.1.2-1b, $L = 0.21600 t_a \sqrt{(F_y/HG_e)}$ =

$(0.21600)(0.5000)\sqrt{[36000.0000/(47.0000 * 1.0000)]} = 2.9890$ ft

$\leq 0.035D = (0.035)(240.0000) = 8.4000$ ft

therefore, L = 2.9890 ft

$L_s = 2.9003$ ft

Per E.6.2.1.1-2b,

$w_a = 36.5HG_eL_s = (36.5)(47.0000)(1.0000)(2.9003) = 4975.4647$ lbf/ft

$W_{int} = 0.0000$ lbf/ft

Per E.6.2.1.1.1-1, $J = \dfrac{M_{rw}}{D^2[w_t(1.0 - 0.4A_v) + w_a - 0.40w_{int}]} =$

$$\frac{143412972}{(240.0000)^2[1848.6369(1.0 - (0.4)(0.0000)) + 4975.4647 - 0.40 * 0.0000]} = 0.365$$

Shell compression per E.6.2.2.1-1b where J < 0.785

$\sigma_c = [w_t(1 + 0.4A_v) + 1.273M_{rw}/D^2]/(12t_s) =$

$[1901.8433(1 + (0.4)(0.0000)) + (1.273)(143412972)/(240.0000)^2]/(12 * 1.0088) =$

418.9210 psi

Self-anchored tank (corroded condition)

Per E.6.2.1.1.1

$W_s = 914695$ lbs

$w_{rs} = 255.1621$ lbf/ft

$t_a = 0.3750$ in

$t_s = 1.0088$ in

$G_e = G(1.0 - 0.4A_v) = 1.0000(1.0 - (0.4)(0.0000)) = 1.0000$

Per E.6.2.1.1.1-2, $w_t = W_s/(\pi D) + w_{rs} = 914695/(\pi * 240.0000) + 255.1621$

$= 1468.3144$ lbf/ft

Check annular ring:

Per E.6.2.1.1.2-1b, $L = 0.21600 t_a \sqrt{(F_y/HG_e)}$ =

$(0.21600)(0.3750)\sqrt{[36000.0000/(47.0000 * 1.0000)]} = 2.2418$ ft

$\leq 0.035D = (0.035)(240.0000) = 8.4000$ ft

therefore, L = 2.2418 ft

Per E.6.2.1.1-1b,

$w_a = 7.9t_a\sqrt{(F_yHG_e)} =$

$(7.9)(0.3750)\sqrt{[(36000.0000)(47.0000)(1.0000)]} = 3853.5282$ lbf/ft

$\leq 1.28HDG_e = (1.28)(47.0000)(240.0000)(1.0000) = 14438.4000$ lbf/ft

therefore, $w_a = 3853.5282$ lbf/ft

Per E.6.2.1.1.1-1, $J = \dfrac{M_{rw}}{D^2[w_t(1.0 - 0.4A_v) + w_a - 0.40w_{int}]} =$

$$\frac{143412972}{(240.0000)^2[1468.3144(1.0 - (0.4)(0.0000)) + 3853.5282 - 0.40 * 0.0000]} = 0.468$$

Shell compression per E.6.2.2.1-1b where J < 0.785

$\sigma_c = [w_t(1 + 0.4A_v) + 1.273M_{rw}/D^2]/(12t_s) =$
$[1521.5208(1 + (0.4)(0.0000)) + (1.273)(143412972)/(240.0000)^2]/(12 * 1.0088) =$
388 psi

Allowable Compression Stresses in Shell per E.6.2.2.3

For new condition:
$GHD^2/t^2 = (1.0000)(47.0000)(240.0000)^2/(1.0088)^2 = 2660092$

For $GHD^2/t^2 \geq 1000000$
Per E.6.2.2.3-1b, $F_c = 1000000t_s/D = (1000000)(1.0088)/240.0000 = 4203$ psi

For corroded condition:
$GHD^2/t^2 = (1.0000)(47.0000)(240.0000)^2/(1.0088)^2 = 2660092$

For $GHD^2/t^2 \geq 1000000$
Per E.6.2.2.3-1b, $F_c = 1000000t_s/D = (1000000)(1.0088)/240.0000 = 4203$ psi

Freeboard requirements per E.7.2

$T_c > 4$
Per E.7.2-3, $A_f = KS_{D1}I(4/T_c^2) = (1.5)(0.6592)(1.0000)(4/11.3967^2) = 0.0305$

Wave Height Per E.7.2-1: $\delta_s = 0.42DA_f = (0.42)(240.0000)(0.0305) = 3.0695$ ft
Per Table E-7, minimum freeboard $= 0.7\delta_s = (0.7)(3.0695) = 2.1487$ ft

Freeboard equal to 0.7 times the calculated wave height is recommended but not required. The required minimum freeboard = 2.149. The available freeboard = 2.000.

Per E.6.2.1.1.3, butt welded annular plates are required.

Estimated Tank Uplift per E.7.3.1-1b
$t_b = 0.3750$ in
$y_u = F_yL^2/(83300)(t_b) = (36000)(2.2418)^2/(83300)(0.3750) = 5.7916$ in

Tank resistance to sliding per E.7.6
Per E.7.6-1, $V_s = 0.4000(W_s + W_r + W_f + W_p)(1.0 - 0.4A_v) =$
$0.4000(914695 + 659270 + 475843 + 131517747)(1.0 - 0.4 * 0.0000) = 53427022$ lbs
$V = 6983593$ lbs

Tank is stable against sliding.

Hoop Stress per E.6.1.4

G = 1.00
D = 240.0000 ft
H = 47.0000 ft
A_i = 0.1740
A_c = 0.0434
A_v = 0.0000
D/H = 5.1064

Shell Course 01

Y = 47.0000 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866 D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{47.0000}{47.0000} - 0.5 \left(\frac{47.0000}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 4415.7164 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68 H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 47.0000)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 1926.4797 \text{ lbf/in}$$

$N_h = 2.60 D(Y-1)G = (2.60)(240.0000)((47.0000)-1)(1.00) = 28704.0000$ lbf/in

Shell thickness (t) = 1.0089 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[28704.0000 + \sqrt{\{(4415.7164)^2 + (1926.4797)^2\}}]/1.0089 = 33225.9519$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = (σ_T * t)/Sa =
(33225.9519 * 1.0089)/37240.0000 = 0.9002 in

Shell Course 02

Y = 37.0417 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{37.0417}{47.0000} - 0.5 \left(\frac{37.0417}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 4217.4821 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 37.0417)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 1948.9818 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((37.0417)-1)(1.00) = 22490.0000$ lbf/in

Shell thickness (t) = 0.7511 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[22490.0000 + \sqrt{\{(4217.4821)^2 + (1948.9818)^2\}}]/0.7511 = 36128.3981$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(36128.3981 * 0.7511)/37240.0000 = 0.7287 in

Shell Course 03

Y = 27.0833 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{27.0833}{47.0000} - 0.5 \left(\frac{27.0833}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 3622.7793 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 27.0833)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 2017.0140 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((27.0833)-1)(1.00) = 16276.0000$ lbf/in

Shell thickness (t) = 0.5525 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{N_i^2 + N_c^2}]/t =$
$[16276.0000 + \sqrt{(3622.7793)^2 + (2017.0140)^2}]/0.5525 = 36963.6728$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(28000.0000)(1.0000) = 37240.0000 psi
or
0.90FyE = (0.90)(42000.0000)(1.0000) = 37800.0000 psi
Therefore, Allowable stress = 37240.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(36963.6728 * 0.5525)/37240.0000 = 0.5484 in

Shell Course 04

Y = 17.1250 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{17.1250}{47.0000} - 0.5 \left(\frac{17.1250}{47.0000} \right)^2 \right]$

tanh(0.866 * 240.0000 / 47.0000) = 2631.6078 lbf/in

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 17.1250)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$

= 2132.1656 lbf/in

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((17.1250)-1)(1.00) = 10062.0000$ lbf/in

Shell thickness (t) = 0.4365 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{N_i^2 + N_c^2}]/t =$
$[10062.0000 + \sqrt{(2631.6078)^2 + (2132.1656)^2}]/0.4365 = 30810.9021$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(23200.0000)(1.0000) = 30856.0000 psi
or
0.90FyE = (0.90)(36000.0000)(1.0000) = 32400.0000 psi
Therefore, Allowable stress = 30856.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(30810.9021 * 0.4365)/30856.0000 = 0.4359 in

Shell Course 05

Y = 7.1667 ft

D/H ≥ 1.333

Per E.6.1.4-1b, $N_i = 4.50 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh(0.866D/H) =$

$$(4.50)(0.1740)(1.00)(240.0000)(47.0000) \left[\frac{7.1667}{47.0000} - 0.5 \left(\frac{7.1667}{47.0000} \right)^2 \right]$$

$$\tanh(0.866 * 240.0000 / 47.0000) = 1243.9678 \text{ lbf/in}$$

Per E.6.1.4-4b, $N_c = \frac{0.98 A_c G D^2 \cosh[3.68(H - Y)/D]}{\cosh[3.68H/D]} =$

$$\frac{(0.98)(0.0434)(1.0000)(240.0000)^2 \cosh[3.68(47.0000 - 7.1667)/240.0000]}{\cosh[(3.68)(47.0000/240.0000)]}$$

$$= 2297.1265 \text{ lbf/in}$$

$N_h = 2.60D(Y-1)G = (2.60)(240.0000)((7.1667)-1)(1.00) = 3848.0000$ lbf/in

Shell thickness (t) = 0.2125 in

Per E.6.1.4-5, $\sigma_T = [N_h + \sqrt{\{N_i^2 + N_c^2\}}]/t =$
$[3848.0000 + \sqrt{\{(1243.9678)^2 + (2297.1265)^2\}}]/0.2125 = 30401.5330$ psi

Allowable stress per E.6.2.4
Lesser of:
1.33SE = (1.33)(23200.0000)(1.0000) = 30856.0000 psi
or
0.90FyE = (0.90)(36000.0000)(1.0000) = 32400.0000 psi
Therefore, Allowable stress = 30856.0000 psi

Minimum required thickness = $(\sigma_T * t)/S_a =$
(30401.5330 * 0.2125)/30856.0000 = 0.2094 in

Hagen Engineering International
202 S. Cedar St. Suite B
Owasso, OK 74055

Customer:
Tesoro Savage Terminal
Job Number:
240 x 50 CRT Insulated
Date: 03-03-2015 Rev.: 1

Foundation Loads

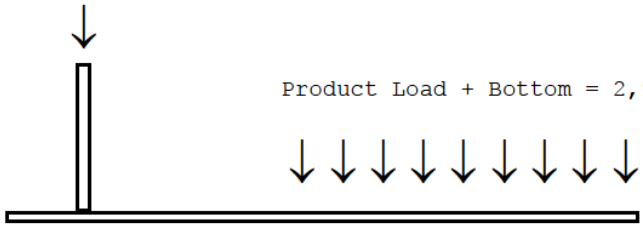
Dead Load = 1,856 lbf/ft
Dead Load + Live Load = 2,311 lbf/ft
Dead Load + Snow Load = 2,315 lbf/ft
Wind Load + Dead Load = 1,942 lbf/ft

(Wind Foundation Moment = 3,893,780 ft-lbs)
(Wind Horizontal Force = 131,060 lbs)

Seismic Load + Dead Load (ringwall foundation) = 5,071 lbf/ft
Seismic Load + Dead Load (slab foundation) = 16,702 lbf/ft

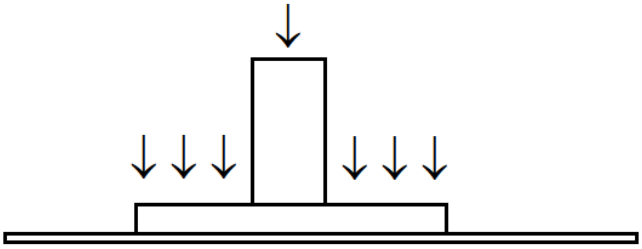
(Note: For Seismic the dead load has a portion of the vertical acceleration added to it and includes a portion of the snow load)
(Seismic Overturning Moment (ringwall) (M_{rw}) = 143,412,972 ft-lbs)
(Seismic Overturning Moment (slab) (M_s) = 669,657,846 ft-lbs)

(Seismic Horizontal Force = 6,983,593 lbs)



Product Load + Bottom = 2,948 lbf/ft²

Center Column Load = 83,389 lbs



Base Plate Area = 30.25 ft²
Base Plate Load = 2,757 lbf/ft²

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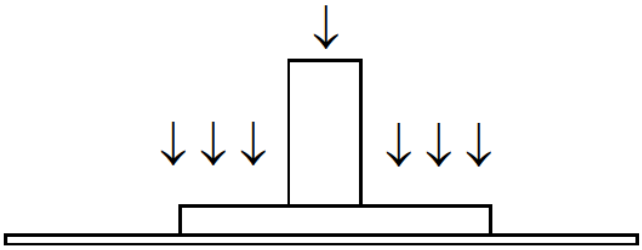
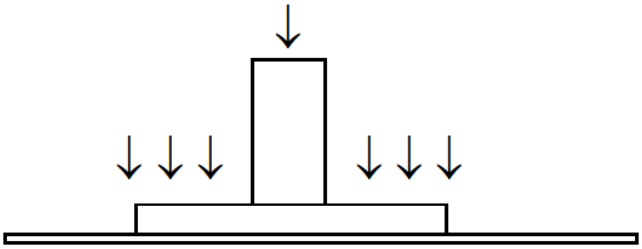
Intermediate Column Load = 55,027 lbs



Base Plate Area = 25.00 ft²
Base Plate Load = 2,201 lbf/ft²

Outer Column Load = 57,292 lbs



Base Plate Area = 20.25 ft²
Base Plate Load = 2,829 lbf/ft²

Total Weight of Tank: 2,577,353 lbs
Total Operating Weight: 134,159,860 lbs
Total Testing Weight: 134,159,860 lbs

Defined Units

$D := 240 \cdot ft + 0 \cdot in$ Diameter of Tank

$Q := 19600 \cdot gpm$ Flow rate

$w := 4 \cdot in$ Width of diffuser slot

$L := 12 \cdot in$ Length of diffuser slot

$A := \frac{\pi}{4} \cdot w^2 + w \cdot (L - w)$ Area of diffuser slot

$N_s := 50$ Number of diffuser slots

$N_r := 2$ Number of diffuser rows

$V := \frac{Q}{N_s \cdot A}$

$V = 2.82 \cdot fps$ Exit velocity at diffuser slot

$d_{min} := \frac{8 \cdot w}{\pi}$

$d_{min} = 10.2 \cdot in$ Minimum diameter of diffuser pipe

$L := 6 \cdot in + \frac{N_s}{N_r} \cdot L + \left(\frac{N_s}{N_r} - 1\right) \cdot 6 \cdot in + 6 \cdot in$

$L = 38.0\, ft$ Length of diffuser pipe

$\frac{L}{\frac{1}{6} \cdot D} = 95.0 \cdot \%$ Length of diffuser pipe check