SUPERIOR TANK COMPANY INC.®

Structural Calculations

for

State of Alaska-Village Safe Water
Nominal 165,000 Gallon Bolted Steel Water Tank
Anchorage, Alaska

January 18, 2011

Superior Tank
9500 Lucas Ranch Road
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Job No.10963-R

Client:	State of Alaska-Village Safe Water	Location	Anchorage, Alaska
Project:	10963-R	Date:	January 18, 2011
Description	Design Criteria	Designer:	RHA

Nominal 165,000 Gallon Bolted Steel Water Tank

Specifications: **ASCE 7-05; AWWA D103-09**

Tank Properties

Tank Diameter, D:	=	34.167	ft
Tank Height:	=	24.125	ft
Overflow Height, H:	=	19.000	ft

Design Criteria

Wind Loads

Wind Velocity:	=	150	mph [AWWA D103 Figure 20]
Exposure Category	=	D	[AWWA D103 Section 15.1.3]
Gust Factor, G	=	1.0	[AWWA D103 Section 15.1.1]

Seismic Design Parameters

Seismic Use Group	=	I	[AWWA D103 Section 14.2]
I_E:	=	1.00	[AWWA D103 Table 2]
Site Class (Soil):	=	D	[AWWA D103 Table 3]
S_s:	=	1.750	[ASCE 7-05 Figure 22-3]
S_1:	=	0.750	[ASCE 7-05 Figure 22-4]
F_a:	=	1.00	[AWWA D103 Table 5]
F_v:	=	1.50	[AWWA D103 Table 4]
A_i = Impulsive Design Acceleration:	=	0.2779	g
A_c = Convective Design Acceleration:	=	0.1561	g
A_v = Vertical Design Acceleration:	=	0.1630	g

Roof Loading

Roof Live Load:	=	15	psf
Roof Snow Load:	=	30	psf
Roof Plate Dead Load:	=	4.375	psf
Secondary Roof Plate Dead Load	=	4.375	psf
Insulation Dead Load	=	1.458	psf

Liquid Properties

Liquid Wt:	=	62.4	lb/ft^3
Spec. Gravity:	=	1.00	

Material Properties

Plate Material			
ASTM A36	F_y =	36,000	psi
ASTM A572-50	F_y =	50,000	psi
Sheet Material			
ASTM A1011-40	F_y =	40,000	psi
ASTM A1011-30 (Roof Deck)	F_y =	30,000	psi
Structural Material			
ASTM A36	F_y =	36,000	psi
Bolts			
1/2"Dia.SAE Grade 5	F_y =	92,000	psi
Pipe			
ASTM A53-B	F_y =	35,000	psi
Anchor Bolts			
ASTM A36	F_y =	36,000	psi
	F_u =	58,000	psi

Shell Heights

Height of Third Ring :	=	8.04	ft
Height of Second Ring :	=	8.04	ft
Height of Bottom Ring :	=	8.04	ft

Shell Thickness[AWWA D103 Section 5.4]

Shell Thickness Required for Net Tensile Stress, t_{req} = 2.6HDsG/(F_t(s-d_{bh})) [AWWA D103 5-1]

Allowable Tensile Stress, F_t = 0.6F_y(1.0-0.9r+3rd_{bolt}/s) <= 0.6F_y [AWWA D103 5-4]

Height of Liquid, H	=	19.000	ft
Tank Diameter, D	=	34.167	ft
Density of liquid, W	=	62.4	lb/ft^3
Bolt Spacing, s	=	2	in
Specific Gravity, G	=	1.00	
Bolt Hole Diameter, d_{bh}	=	0.531	in
Bolt Diameter, d_{bolt}	=	0.50	in

r = Force Transmitted By Bolt(s) Divided by Tensile Force [AWWA D103 Section 5.5.3]

	Shell Thickness	Bolt Rows	t_{req} (in)	Shell Ht (Ft)	t_{actual} (in)	Material	F_y (ksi)	F_u (ksi)	r	F_t (ksi)
Third Ring :	8 gauge	1	0.017	8.04	0.1644	A-1011-40	40	55	1	20.4
Second Ring:	3/16"	1	0.072	8.04	0.1875	A-36	36	58	1	18.4
Bottom Ring:	1/4"	1	0.125	8.04	0.2500	A-36	36	58	1	18.4

Bolt Design[AWWA D103 Section 5.5]

Vertical Stave Lap Bolt Loading

Hoop Force = HWD/2			
Bolts/Foot = 12/S	=	6	bolts
Bolt Root Diameter, d_k	=	0.4	in
Allowable Bearing, $P_{allow,b}$ = 1.35$F_y d_{bolt} t$			
Allowable Shear, $P_{allow,s}$= 0.25$F_u A_{ts}$			
Ultimate Tensile Stress Bolt, $F_{u,bolt}$	=	120,000	psi SAE Grade 5
n = Number of Bolt Threads Per Inch	=	13	threads/in
A_{ts} =0.78584(d-0.9743/n)^2	=	0.142	in^2
Minimum Edge Distance, s_{min} = P/0.6F_yt [D103 Eq 5-3]			
Edge Distance, s	=	1	in

	Shell Thickness	Bolt Rows	Bolt Grade	H (ft)	Hoop Force (lb/ft)	Bolt Force, P_{bolt} (lb)	Bearing, $P_{allow,b}$ (lb)	Shear P_{allow} (lb)	Min Edge Dist. (in)	
Third Ring :	0.1644	1	5	2.916	3,108	518	4,439	4,259	0.13	**OK**
Second Ring:	0.1875	1	5	10.958	11,681	1,947	4,556	4,259	0.48	**OK**
Bottom Ring:	0.25	1	5	19	20,254	3,376	6,075	4,259	0.63	**OK**

Tank Sheeting Summary

	Thickness (in)	No. of Bolt Rows	Stave Width (in)
Primary Deck Layer	0.1046	1	N/A
Third Ring	0.1644	1	58
Second Ring	0.1875	1	58
Bottom Ring	0.2500	1	58
Primary Bottom Layer	0.1046	1	N/A

Secondary Tank Sheeting Summary

	Thickness (in)	No. of Bolt Rows	Stave Width (in)
Second Deck Layer	0.1046	1	N/A

Design Wind Pressure [AWWA Section 15.1]

$P_w = q_z G C_f \geq 30 C_f$ [AWWA D103 Eq 15-1]

P_w = Wind pressure applied to horizontal projected area (psf)	=	43.89	psf
G = Gust-effect factor; shall be taken as 1.0	=	1.0	
C_f = Force coefficient for cylindrical surface	=	0.60	
q_z = Velocity pressure at height z of the centroid (psf)	=	73.15	psf
where $q_z = 0.00256 K_z I V^2$ [AWWA D103 Eq 15-2]			
K_z = Velocity pressure exposure coefficient at height z of the centroid	=	1.27	[AWWA D103 Table 10]
Exposure	=	D	
z = Height above finished grade (ft)	=	24.13	ft
I = Wind importance factor	=	1.00	
V = Basic wind speed (mph)	=	150	mph

Intermediate Wind Girder Check

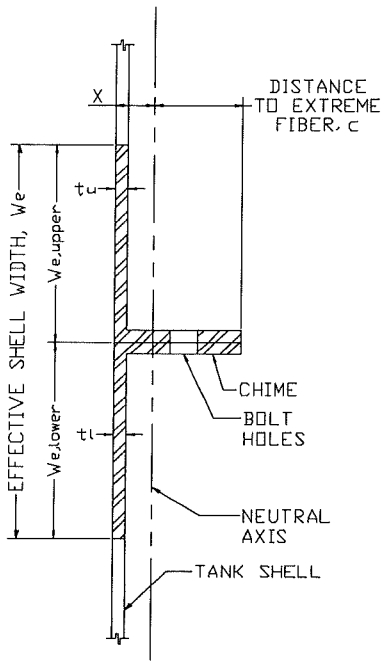
See Intermediate Wind Girder Calcs

Wind Overturning

$M_w = (P_w D H_{shell}^2/2)$	=	436,392	ft-lb
W_s = Weight of Shell	=	22,865	lb
W_{roof} = Weight of Roof	=	9,359	lb
W_{rftr} = Weight of Rafters	=	2,019	lb
W_c = Weight of Contents, Minimum Water Level(6")	=	28,606	lb
$W = W_s + 2/3 W_{roof} + 1/2 W_{rftr} + W_c$	=	58,720	lb
D_{ac} = Diameter of Anchor Circle	=	34.834	ft
N = No. of Anchor Bolts	=	23	
$P_w = 4M_w/ND_{ac} - W'/N$ [D103 Eq. 5-11] = Uplift Per Bolt	=	**-374 lb < 0 lb OK, No Uplift**	

Design Parameters

D = Tank diameter	=	34.167	ft
H = Tank height	=	24.125	ft
V = Wind velocity	=	150	mph
$P_w = q_z G C_f \geq 30 C_f$	=	43.89	psf
H_3 = Third ring height	=	8.04	ft
H_2 = Second ring height	=	8.04	ft
H_1 = Bottom ring height	=	8.04	ft
t_3 = Third ring thickness	=	0.1644	in
t_2 = Second ring thickness	=	0.1875	in
t_1 = Bottom ring thickness	=	0.2500	in



Upper Chime

t_u = Thickness of upper ring	=	0.1644	in
t_L = Thickness of lower ring	=	0.1875	in
l_c = Chime length	=	2.00	in
m = Diameter of hole	=	17/32	in
n = Distance from edge of chime to	=	0.75	in

$W_{e,3} = \text{Min}(16t, 0.78(Rt)^{0.5})$ [D103 Section 15.4.1] = 2.630 in

$W_{e,2} = \text{Min}(16t, 0.78(Rt)^{0.5})$ [D103 Section 15.4.1] = 3.000 in

$W_{e,2\text{-}3} = W_{e,3} + W_{e,2}$ = 5.630 in

Determine Total Moment of Inertia for Chime with effective Tank Shell Width(Subtract Area at Bolt Penetrations)

		A	x	xA	I	d^2	$I + Ad^2$
+	Tank Shell, Lower	0.527	0.09	0.05	0.0015	0.08	0.0452
+	Chime, Lower	0.375	1.00	0.38	0.1250	0.38	0.2685
+	Chime, Upper	0.329	1.00	0.33	0.1096	0.38	0.2354
+	Tank Shell, Upper	0.405	0.08	0.03	0.0009	0.09	0.0372
-	Bolt Hole, Lower	0.100	1.25	0.12	0.0023	0.75	0.0775
-	Bolt Hole, Upper	0.087	1.25	0.11	0.0021	0.75	0.0679
		$\sum A$ = 1.450		$\sum xA$ = 0.55			

X = Centroid of Stiffened Section = $\sum xA / \sum A$ = 0.381 in

$I_{xx} = \sum I + Ad^2$ = 0.441 in^4

c = distance to extreme fiber of compression face = l_c-X = 1.619 in

S = Section Modulus = I/c = 0.272 in^3

Equivalent Plate Thickness to Give Section Modulus of Stiffened Section

$S = bd^2/6$, b = $W_{e,\text{total}}$ (equivalent shell width), d = t_2 (equivalent plate thickness)

$t_{2\text{-}3} = (S/W_{e,2\text{-}3}*6)^{0.5}$ = = **0.539** in

Lower Chime

t_u = Thickness of upper ring	=	0.1875	in
t_L = Thickness of lower ring	=	0.2500	in
l_c = Chime length	=	2.00	in
m = Diameter of hole	=	17/32	in
n = Distance from edge of chime to CL of hole	=	0.75	in

$W_{e,2} = \text{Min}(16t, 0.78(Rt)^{0.5})$ [D103 Section 15.4.1] = 3.000 in

$W_{e,1} = \text{Min}(16t, 0.78(Rt)^{0.5})$ [D103 Section 15.4.1] = 4.000 in

$W_{e,1\text{-}2} = W_{e,2} + W_{e,1}$ = 7.000 in

Lower Chime(Continued)

Determine Total Moment of Inertia for Chime with effective Tank Shell Width(Subtract Area at Bolt Penetrations)

		A	x	xA	I	d^2	$I + Ad^2$
+	Tank Shell, Lower	0.938	0.13	0.12	0.0049	0.05	0.0551
+	Chime, Lower	0.500	1.00	0.50	0.1667	0.41	0.3738
+	Chime, Upper	0.375	1.00	0.38	0.1250	0.41	0.2803
+	Tank Shell, Upper	0.527	0.09	0.05	0.0015	0.07	0.0379
-	Bolt Hole, Lower	0.133	1.25	0.17	0.0031	0.80	0.1092
-	Bolt Hole, Upper	0.100	1.25	0.12	0.0023	0.80	0.0819
	$\sum A =$	2.107	$\sum xA =$	0.75			

X = Centroid of Stiffened Section = $\sum xA / \sum A$ = 0.356 in

$I_{xx} = \sum I + Ad^2$ = 0.556 in^4

c = distance to extreme fiber of compression face = $I_c - X$ = 1.644 in

S = Section Modulus = I/c = 0.338 in^3

Equivalent Plate Thickness to Give Section Modulus of Stiffened Section

$S = bd^2/6$, $b = W_{e,\,total}$ (equivalent shell width), $d = t_2$ (equivalent plate thickness)

$t_{1-2} = (S/W_{e,1-2}*6)^{0.5}$ = = **0.539** in

Check Requirements for Intermediate Shell Girders(Per D103 Section 15.4.2)

$H_3' = H_3 - W_{e,3}$, Effective height of third ring = 7.8228 ft

t_3 = Thickness of third ring = 0.1644 in

$H_{2\text{-}3}' = W_{e,2\text{-}3}$, Effective height of upper chime = 0.4692 ft

$t_{2\text{-}3}$ = Equivalent thickness of stiffened ring = 0.5388 in

$H_2' = H_2 - 2(W_{e,2})$, Effective height of second ring = 7.5420 ft

t_2 = Thickness of second ring = 0.1875 in

$H_{1\text{-}2}' = W_{e,1\text{-}2}$, Effective height of lower chime = 0.5833 ft

$t_{1\text{-}2}$ = Equivalent thickness of stiffened ring = 0.5385 in

$H_1' = H_1 - W_{e,2}$, Effective height of bottom ring = 7.7087 ft

t_1 = Thickness of bottom ring = 0.2500 in

$t_{avg} = \sum H' * t / \sum H'$

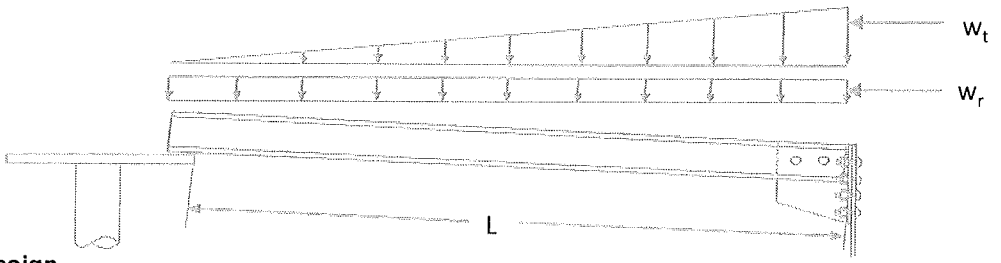
$h_{allow} = 10.625x10^6 t/(P(D/t)^{1.5})$ [D103 EQ 15-5]

AWWA D103 Section 15.4.2 Method Used to Determine If Intermediate Girders Required

	t_{avg}	H	h_{allow}	
3rd Ring	0.1644	7.82	13.28	OK
Top Chime	0.1856	8.29	17.98	OK
2nd Ring	0.1865	15.83	18.21	OK
Lower Chime	0.1990	16.42	21.41	OK
Bottom Ring	0.2153	24.13	26.07	OK

Therefore, No Wind Girder Required

Roof Framing Design



Rafter Design

Try Rafter Member C4x5.4

Section Properties

d = Rafter Depth	4	inch Channel	w_r = Rafter Weight	5.4	lbs/ft
F_y (ASTM A36)	36	ksi	S_x = Section Modulus	1.93	in^3
r_y = Radius of Gyration	0.44	in	I_x = Moment of Inertia	3.85	in^4
Z_x = Plastic Section Modulus	2.29	in^3	I_y = Moment of Inertia	0.31	in^5
h_o = Flange Centroid Dist.	3.70	in	C_w = Warping Constant	0.92	in^6
J = Torsional Constant	0.04	in^4	r_{ts} = Eff. Radius of Gyration	0.528	in
$c = h_o/2\sqrt{(I_y/C_w)}$	1.08		E =	29,000	ksi

Roof Configuration & Loading

No. Rafters	=	23	
Roof Slope	=	1	:12
θ = arctan(rise/run)	=	4.764	°
L = Design Rafter Length	=	16.254	ft
L_a = Actual Rafter Length	=	16.310	ft
l_t = Max Tributary Width to Rafter	=	4.67	ft
$w_{DL} = (DL)*l_t$ =Max Tributary Dead Load to Rafter	=	52.31	lb/ft
$w_{LL} = (LL)*l_t$ =Max Tributary Live Load to Rafter	=	70.00	lb/ft
$w_S = (S)*l_t$ =Max Tributary Snow Load to Rafter	=	140.01	lb/ft

Design Moment

LC 1 (DL+LL)

x = Distance Along Beam to Point of Maximum Moment	=	9.30	ft
$M_u = (2W/(3L^2)(L2-x2)+w_r*(L-x)$	=	2,247	ft-lb
	=	27.0	k-in

LC 2 (DL+S)

x = Distance Along Beam to Point of Maximum Moment	=	9.33	ft
$M_u = (2W/(3L^2)(L2-x2)+w_r*(L-x)$	=	3,434	ft-lb
	=	41.2	k-in

Yielding Limit State [AISC 13th Edition Section F2.1]

$M_n = M_p = F_yZ_x$	=	**82.44**	**k-in**

Lateral-Torsional Buckling Limit State [AISC 13th Edition Section F2.1]

L_b =Unbraced Length	=	0.000	in
$L_p = 1.76r_y\sqrt{(E/F_y)}$ [AISC 13th ED. Eq. F2-5]	=	22.179	in > 0.00 in

$L_b < L_p$, thus limit state of Lateral Torsional Buckling Does not Apply

Design Flexural Strength

M_n =	=	82.44	k-in
Ω_b (ASD) =	=	1.67	
M_n/Ω_b =	=	**49.37**	**lb-in > 41.2 lb-in OK**
$M_u/(M_n/\Omega_b)$ =	=	0.835	

Use 23-C4x5.4 Rafters

Max. Rafter Spacing [AWWA D103 Eq. 13]

t_{rp} = Roof Plate Thickness	=	0.1046	in
F_y = Yield Strength of Roof Material	=	40,000	ksi
W_{DL+LL} = Dead Load and Live Load on Roof Surface	=	25.208	psf
W_{DL+S} = Dead Load and Snow Load on Roof Surface	=	40.208	psf
$L_{max} = ((300F_y t_{rp}^2)/W_{DL+S})^{0.5}$ <= 60 inches	=	57.1	in
Rafter Spacing	=	**56.0**	**in < 57.1 OK**

Column Design

d = Column Dia.	=	5	in (Sch 40 Pipe)
L = Column Length	=	25.04	ft
r = radius of gyration	=	1.88	in
K,(Pinned-Pinned, See AISC 9th Ed. Table C-C2.1)	=	1	
Kl/r ≤ 175 [AWWA D100 Section 5.7.3]	=	**160**	**< 175 OK**
Roof Load Tributary to Column = 1/3(DL+LL)	=	8,010	lb
R_{DL} = DL Reaction of Rafters at Column	=	1,009	lb
W_c = Column wt.	=	366	lb
Rafter Plate	=	75	lb
P = Total Load (DL+LL) to Column	=	9,460	lb
P_{allow} (Per AISC 9th Ed. Table on Sheet 3-36)	=	**25,257**	**lb >9,460 lbs OK**

Use 5" dia. Sch. 40 Column

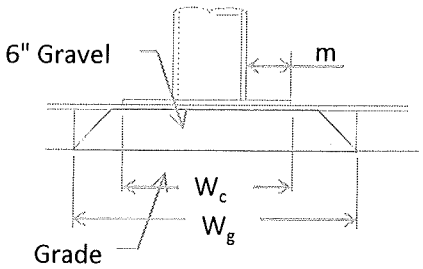
Rafter Plate Design

$t^2 = 3W/2PiS_r[2a^2(m+1)Lna/b+a^2(m-1)-b^2(m-1)]/[a^2(m+1)+b^2(m-1)]$ (Roark - 4th Ed. 10-22)

Rafter Plate Dia., d_{rp}	=	21.25	in
Load to Rafter Plate, W = P-W_c	=	9,094	lbs
S_r	=	27,000	psi
m	=	3.33	
b	=	3.157	
a	=	10.625	in
t_{req} =	=	**0.669**	**in < 0.750in OK**

Use 0.750 x 21.25 dia. Rafter Plate

Base Plate Design



Column width (d) =	=	5.563	in
Allowable Bearing on Insulation =	=	5,760	psf
Act. Col. Load =	=	9,460	lb
Area Req'd at Floor =	=	1.64	ft^2
Width (W_c) at Base of Column =	=	18	in
Area at Floor =	=	**2.25**	**sq ft > 1.64 sq ft O**
$f_p = P/(W_c^2)$	=	29.20	psi
m = Cantilever distance of plate = (W_c - (0.8 * d)) / 2	=	6.77	in
t_p = Base plate thickness = $2m(f_p/F_y)^{1/2}$	=	**0.386**	**in < 0.500 in OK**

*** Use * 18 in square x 0.500 in thick baseplate**

Design Response Spectra - General Procedure

Site Location: Anchorage, Alaska

Site Class:	= D	
Seismic Importance Factor, I_E:	= 1.00	
S_s=	= 1.750 g	(Mapped MCE spectral response acceleration, 5% damped, at 0.2 sec period)
S_1 =	= 0.750 g	(Mapped MCE spectral response acceleration, 5% damped, at 1.0 sec period)
F_a = Short Period Site Coefficient	= 1.00	
F_v = Long Period Site Coefficient	= 1.50	
$S_{MS} = F_a S_s$ (Eq 14-5)	= 1.750 g	(Site Specific MCE spectral response acceleration, 5% damped, at 0.2 sec period)
$S_{M1} = F_v S_1$ (Eq 14-6)	= 1.125 g	(Site Specific MCE spectral response acceleration, 5% damped, at 1.0 sec period)
$S_{DS} = US_{MS}$ (Eq 14-7)	= 1.167 g	(Design Acceleration, 5% damped, at 0.2 sec period)
$S_{D1} = US_{M1}$ (Eq 14-8)	= 0.750 g	(Design Acceleration, 5% damped, at 1.0 sec period)
U = (AWWA D103 Section 14.2.7.3)	= 2/3	

Impulsive Acceleration

S_{ai} = Design spectral response acceleration for impulsive components, 5 percent damped, at the natural period of the structure T_i

$T_S = S_{D1}/S_{DS}$	= 0.643	
T_L = Region-dependent transition period for longer-period ground motion (Fig. 17)	= 8	s
T_i = Natural period of the structure	= 0.000	s [Assume 0 per D100 13.5]
R_i = Response Mod. Factor (Table 6)	= 3.0	
R_c = Response Mod. Factor (Table 6)	= 1.5	
For $0 <= T_i <= T_S$ (Eq. 14-9)		
$S_{ai} = S_{DS}$	= 1.167	g
For $T_S < T_i < T_L$ (Eq. 14-10)		
$S_{ai} = S_{D1}/T_i <= S_{DS}$	= N/A	
For $Ti > T_L$ (Eq. 14-11)		
$S_{ai} = T_L S_{D1}/T_i^2$	= N/A	
$A_i = S_{ai} I_E/1.4R_i >= 0.36 S_1 I_E/R_i$ [Eq 14-16]	= 0.278	g

Convective Acceleration

S_{ac} = Design spectral response acceleration for the convective component, 0.5 percent damped, at the first mode sloshing wave period T_c

T_c = First mode sloshing wave period

$T_c = 2Pi \times (D / 3.68g \times \tanh(3.68H/D))^{1/2}$	= 3.431	Seconds
K = Damping scaling factor to convert spectrum from 5 percent to 0.5 percent damping	= 1.5	
For $T_c <= T_L$ [Eq. 14-12]		
$S_{ac} = KS_{D1}/T_c <= S_{DS}$	= 0.328	g
For $T_c >= T_L$ [Eq. 14-13]		
$S_{ac} = KT_L S_{D1}/T_c^2$	= N/A	
$A_c = S_{ac} I_E/1.4R_c$ [Eq. 14-17]	= 0.156	g

Shell Weight

Shell - Third Ring Wt.	=	6,245	lb
Shell - Second Ring Wt.	=	7,123	lb
Shell - Bottom Ring Wt.	=	9,497	lb
W_s = Total Shell Wt.	=	22,865	lb

Roof Structure Weight

Roof Wt.	=	9,359	lb
Total Rafter Wt.	=	2,019	lb
W_r =Total Roof & Rafter Wt.	=	11,378	lb

Summary of Design Seismic Loads

D/H	=	1.8	
A_i = Impulsive design acceleration	=	0.2779	g
A_c = Convective Design Acceleration	=	0.1561	g
W_s = Total Weight of Tank Shell	=	22,865	lb
W_r = Weight of Roof and Rafters	=	11,378	lb

Summary of Design Seismic Loads(Continued)

W_i/W_t = tanh(00866D/H)/(0.866D/H) (Eq 14-20)	=	0.587	
W_i = Effective Impulsive Weight	=	638,196	lb
W_c /W_T = 0.230D/H*tanh(3.67H/D) (Eq 14-22)	=	0.400	
W_c = Effective Compulsive Weight	=	435,139	lb
W_T = Weight of Tank Contents	=	1,087,031	lb
X_s = Height from Bottom of Shell to Center of Gravity of Shell	=	10.92	ft.
H_t = Total Height of the Shell	=	24.125	ft
X_i/H = 0.375 [Eq. 14-24]	=	0.3750	
X_i = Height from Bottom of Shell to Centroid of Lateral Seismic Force Applied to Impulsive Weight W_i	=	7.13	ft
X_c/H = (1.0-(cosh(3.67H/D)-1)/(3.67H/Dsinh(3.67H/D))[Eq. 14-26]	=	0.6227	
X_c = Height from Bottom of Shell to Centroid of Lateral Seismic Force Applied to Convective Weight W_c	=	11.83	ft
W_f = Weight of Tank Bottom	=	3,916	lb

Design Overturning Moment and Base Shear at Bottom of the Shell

$M_s = \sqrt{[(A_i)(W_sX_s+W_rH_t+W_iX_i)^2+(A_cW_cX_c)^2]}$ [D103 Eq. 14-18]	=	1,622,986	ft-lb
$V_f = \sqrt{[(A_i)(W_s+W_r+W_i+W_f)^2+(A_cW_c)^2]}$ [D103 Eq. 14-27]	=	199,835	lb
t_b init	=	0.1046	in
$w_{L\ init} = 7.9t_b\sqrt{FyHG} \le 1.28HDG$ [D103 Eq. 14-33]	=	720	plf
$w_{L\ max} = 1.28HDG$	=	831	plf
$w_{rs} = 2/3(W_r) + 1/2(W_{rftr})$	=	7,252	lb
$w_t = (W_s + w_{rs})/(\pi D)$ [Eq. 14-37]	=	280.58	plf

Overturning Ratio

$J \le 0.785$	No calculated uplift under design seismic overturning moment. Tank is self-anchored.
$0.785<J\le1.54$	Uplift occurs but tank is stable for the design load providing shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular plate if L < 0.035D is not controlling or add mechanical anchorage.

$A_v = 0.14S_{DS}$	=	0.163	g
$J = M_s/(D^2(w_t(1-0.4A_v)+w_L))$ [D103 Eq. 14-32]	=	1.415	Uplift Occurs, No Anchorage Required!

Shell Compression-No Uplift or Tank is Anchored

t/R =	=	0.00122	
$\sigma_c = (w_t(1+0.4A_v)+1.273M_s/(D^2))$ x 1/(12t) [D100 Eq. 14-35]	=	690	psi
$\sigma_e = 1.333\sigma_a = 1.333f_s$ [D100 Eq 14-44]			
$f_s = 1.3333(15{,}000)(2/3)(100t/R)*[2-(2/3)(100t/R)] \leq 15{,}000$ [D103 Eq. 5-2]			
f_s =	=	**3,121**	**psi > 690 psi OK**

Hydrodynamic Seismic Hoop Tensile Stress

Definitions

N_i = Impulsive hoop membrane force in tank shell

N_c = Convective hoop membrane force in tank shell

σ_T = Total combined hoop stress in the shell

σ_s = Hoop stress in theshell due to impulsive and convective forces of stored liquid

σ_h = Product hydrostatic hoop stress in the shell

Bottom Ring:

t = 0.25 in

Y = 19 ft <0.75D

D/H >= 1.333

$N_i = (4.5A_i(GDH)(Y/H - .5((Y/H)^2))(TANH(.866(D/H)))$[D103 Eq. 14-3	=	371.3	lb/in ←Governs

D/H < 1.333 and Y < 0.75D:

$N_i = (2.77A_iGD^2)(Y/0.75D - .5((Y/0.75D)^2)))$[D103 Eq. 14-40]	=	419.2	lb/in

D/H < 1.333 and Y >= 0.75D:

$N_i = 1.39A_iGD^2$ [D103 Eq. 14-41]	=	450.9	lb/in
$N_c = 0.98A_cGD^2(\cosh(3.68(H-Y)/D)/COSH(3.68(H/D)))$[Eq. 14-42]	=	45.4	lb/in
$N_h = 2.6YDG$	=	1,688	lb/in
a_v = Vertical Acceleration	=	0.16	g
$\sigma_s = \sqrt{(N_i^2 + N_c^2 + (N_h \times A_v)^2)}/t_s$ [D103 Eq. 14-38]]	=	1,857	psi
$\sigma_h = N_h/t_s$	=	6,751	psi
$\sigma_t = \sigma_s + \sigma_h$	=	8,609	psi
$F_{allow} = 1.333F_t$	=	**24,474**	**psi > 8,609 psi OK**

Second Ring:

t = 0.1875 in

Y = 10.958 ft <0.75D

D/H >= 1.333

$N_i = (4.5A_i(GDH)(Y/H - .5((Y/H)^2))(TANH(.866(D/H)))$[D103 Eq. 14-3	=	304.8	lb/in ←Governs

D/H < 1.333 and Y < 0.75D:

$N_i = (2.77A_iGD^2)(Y/0.75D - .5((Y/0.75D)^2)))$[D103 Eq. 14-40]	=	302.1	lb/in

D/H < 1.333 and Y >= 0.75D:

$N_i = 1.39A_iGD^2$ [D103 Eq. 14-41]	=	450.9	lb/in
$N_c = 0.98A_cGD^2(\cosh(3.68(H-Y)/D)/COSH(3.68(H/D)))$[Eq. 14-42]	=	63.52	lb/in
$N_h = 2.6YDG$	=	973	lb/in
a_v = Vertical Acceleration	=	0.16	g
$\sigma_s = \sqrt{(N_i^2 + N_c^2 + (N_h \times A_v)^2)}/t_s$ [D103 Eq. 14-38]]	=	1,864	psi
$\sigma_h = N_h/t_s$	=	5,191	psi
$\sigma_t = \sigma_s + \sigma_h$	=	7,055	psi
$F_{allow} = 1.333F_t$	=	**24,474**	**psi > 7,055psi OK**

Third Ring

t = 0.1644

Y = 2.916 ft <0.75D

D/H >= 1.333

$N_i = (4.5A_i(GDH)(Y/H - .5((Y/H)^2))(TANH(.866(D/H)))$[D103 Eq. 14-3	=	105.2	lb/in ←Governs
D/H < 1.333 and Y < 0.75D:			
$N_i = (2.77A_iGD^2)(Y/0.75D - .5((Y/0.75D)^2)))$[D103 Eq. 14-40]	=	96.4	lb/in
D/H < 1.333 and Y >= 0.75D:			
$N_i = 1.39A_iGD^2$ [D103 Eq. 14-41]	=	450.9	lb/in
$N_c = 0.98A_cGD^2(\cosh(3.68(H-Y)/D)/COSH(3.68(H/D)))$[Eq. 14-42]	=	132.36	lb/in
$N_h = 2.6YDG$	=	259.0	lb/in
a_v = Vertical Acceleration	=	0.16	g
$\sigma_s = \sqrt{(N_i^2 + N_c^2 + (N_h \times A_v)^2)}/t_s$ [D103 Eq. 14-38]]	=	1,060	psi
$\sigma_h = N_h/t_s$	=	1,575	psi
$\sigma_t = \sigma_s + \sigma_h$	=	2,636	psi
$F_{allow} = 1.333F_t$	=	**27,193**	**psi > 2,636psi OK**

Anchorage Requirements

Liquid Wt:	=	62.4	lb/ft^3
Min. Concrete, f'_c:	=	2,500	psi
Rebar:	A.S.T.M. A-615, Grade 60		
F_t :	=	24,000	psi

Seismic Uplift

N = No. of Anchor Bolts	=	23	ea
M_s =	=	1,623	ft-k
$W' = W_s + (1/2W_{rftr} + 2/3W_{roof})$	=	30,117	lb
D_{ac} = Diameter of Anchor Circle	=	34.83	ft
$P_s = 4M_s/Nd_{ac} - W'/N$ [D103 Eq. 5-9]	**=**	**6,794**	**lb > 869 lb, Seismic Uplift**

Wind Uplift

M_w = (See Page 3)	=	436,392	ft-lb
$P_w = 4M_w/ND_{ac}-W'/N$ [D103 Eq. 5-10] = Uplift Per Bolt	**=**	**869**	**lb**

Anchor Design [ACI 318 Appendix D]

Anchor Properties

d_o = Anchor Bolt Diameter	=	0.875	in
$h_{ef} = l_e$ = Embedment Depth	=	8.125	in
h = Member Thickness	=	24	in
$c_{a1} = c_{a,min}$ = Distance from center of Anchor to Edge of Concrete	=	8	in < 1.5 hef
c_{a2} =	=	12.1875	in

Check minimum edge distance and concrete thickness

c_{min} = [ACI 316 D8.2]	=	3.00	in < 8 in OK
$h_{min} = h_{ef} + 2d_0$	=	9.875	in < 24 in OK

Calculate Steel Strength

n_t = thread per inch	=	7	threads/in
$A_{se} = \pi/4(d_o-0.9743/n_t)^2$	=	0.4252	in^2
f_{uta} =	=	60,000	psi
$N_{sa} = A_{se}f_{uta}$	=	25,514	lb

Determine Concrete Breakout Strength

$A_{nc} = (3* h_{ef})(c_{a2}+c_{a1})$	=	492	in^2
$A_{nc0} = (9* h_{ef}^2)$ [ACI 318 Eq. D-6]	=	594	in^2
$\Psi_{ec,N}$ = [ACI 318 Sec. D.5.2.4, No eccentricity = 1.0]	=	1.00	
$\Psi_{ed,N} = 1$ if $c_{a,min} > 1.5h_{ef}$ [ACI 318 D-10]			
$= 0.7+0.3\ c_{a,min}/(1.5_{hef})$ if $c_{a,min} < 1.5h_{ef}$[ACI 318 D-11]	=	0.897	
$\Psi_{c,N}$ = (uncracked concrete assumed)[ACI 318 D.5.]	=	1.25	
$\Psi_{cp,N}$ = 1 for cast-in anchors [ACI 318 D.5.2.7]	=	1	
k_c = [ACI 318 D.5.2.2]	=	24	
$N_b = k_c\sqrt{(f'_c)}*h_{ef}^{1.5}$ [ACI 318 Eq. D-7]	=	27,792	lb
$N_{cb} = A_{nc}/A_{nc0}*\Psi_{ec,N}\Psi_{c,N}\Psi_{cp,N}N_b$ [ACI 318 Equation D-4]	=	25,806	lb

Pullout Strength of Anchor in Tension

$N_{pn} = \psi_{c,P} N_p$ [ACI 318 Eq. D-14]

$\psi_{cp,N}$ = 1.4 [ACI 318 D.5.3.6]	=	1.4
D_{ap} = Anchor plate diameter	=	4.0 in
t_{ap} = Anchor plate thickness	=	0.375 in
A_{brg} = Bearing area of head	=	0.89 in^2
A_{brg-2} = Bearing area of anchor plate	=	2.56 in^2
A_{brg} = Governing Bearing Area	=	2.56 in^2
$N_p = 8A_{brg}f'_c$ [ACI 318 Eq. D-15]	=	51,112 lb

Side Face Blowout Strength(when $c_{min} < 0.4h_{ef}$)

$0.4h_{ef}$ =	=	3.3 in < 8 in OK

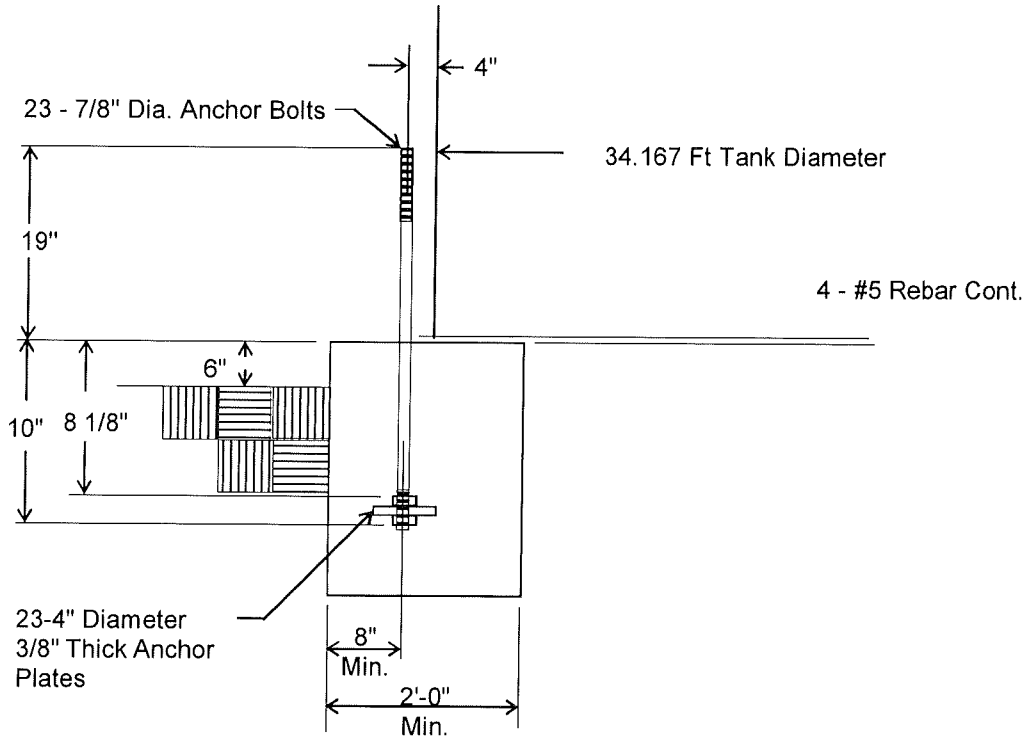
$c_{min} < 0.4h_{ef}$, Side-Face Blowout Check Not Required

Determine Controlling Strength

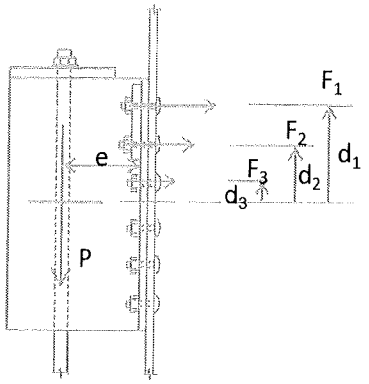
$N_u = 1.4P$	=	9,511 lb
N_n = Min $\|N_{sa}; N_{cb}; N_p\|$	=	25,514 lb
Governing Anchor Strength	=	Steel Failure
Φ = [ACI 318 Appendix D Section D.4.4]	=	0.75
$0.75\Phi N_n$ = [ACI 318 Section D.3.3.3 For Seismic Design Category D, E, and F]	=	**14,352 lb > 9,511 lb OK**

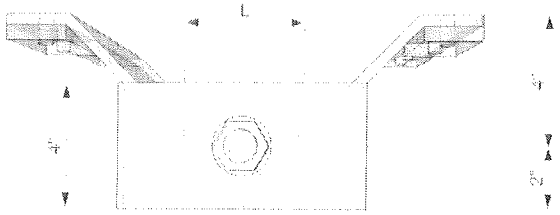
Thus, Use: **0.875 in Diameter Anchor Bolts, Embed 8 1/8 in, Project 19 in above foundation with 4.0 in. Diameter x 3/8 in thick Anchor Plate**

Foundation Ringwall



Anchor Chair Design





Tension (P) = Min($A_{se}F_y$,16M_s/Nd-W/N) [D103 Section 5.9.8.2]	=	6,794	lb
(e) =	=	4	in
No. of Bolts =	=	12	ea
d_1 =	=	5	in
d_2 =	=	3	in
d_3 =	=	1	in
2Sd = ($d_1^2 + d_2^2 + d_3^2$) x 2 =	=	70	in^2
F_1 = Pe*d_1/(Sd*2)	=	971	lb
F_2 = Pe*d_2/(Sd*2)	=	582	lb
F_3 = Pe*d_3/Sd*2)	=	194	lb
Bolt Shear (V) = P/No. of Bolts	=	566	lb/bolt

1/2" Dia. , Grade 5 Bolt (ASTM A-325)

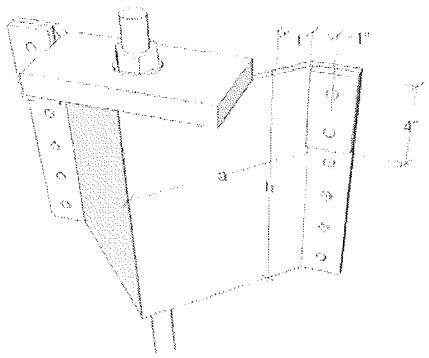
T_{allow} = 44 ksi x 0.20 in^2	=	8.8	k
V_{allow} = 17.5 ksi x 0.20 in^2	=	3.5	k
Unity Check			
$P/T_{allow} + V/V_{allow} + P_{bolt}/P_{allow} \leq 1.33$			
$\frac{971}{8,800} + \frac{566}{3,500} + \frac{3,376}{4,259}$	=	**1.065**	**< 1.33 OK**

Hold Down Plate

M = P* L/4	=	6,794	in-lb
t = 6M/(4F_b*1.33)	=	0.533	in
Therefore, Use: **0.625 Inch Plate**			

Reinforcing Plate

T = (F_1 + F_2)/4"	=	388	lb/in
M = T x L	=	388	in-lb
t = 6M/(1.33F_b)	=	0.255	in
Therefore, Use: **0.3125 in plate**			

Bent Plate Chair

TRY:	Plate Thickness (t) =	0.1046	in		
	Chair Height (h) =	12	in		
$V = P/2$				=	3,397 lb
$f_v = V/(th)$				=	2,706 lb
$a/h = 6"*(2)^{1/2}/12$				=	0.707
h/t =				=	115
F_v =				=	**14,500 lb > 2,706 lb OK**
(All. Shear Stress in Plate Girders, Table 1-36, AISC Manual, 9th ed.)					
$L = 4" *2^{1/2}$				=	5.66 in
$M = P/2*L$				=	19,216 in-lb
$f_b = M/(th^2/6)$				=	**7,655 psi < 20,000 psi OK**