ATTACHMENT B

Letter from C.R. Steinhardt (WPSC)

To

Document Control Desk (NRC)

Dated

August 15, 1997

Stevenson & Associates Calculation C-018, Rev. 1

Client: Wisconsin Public Service Company Calculation No. C-018

Title: Seismic Analysis and/or Outlier Resolution of Tanks and Heat Exchangers

Project: Kewaunee USI A-46 / IPEEE

Method: Hand Calculation

Acceptance Criteria: GIP, ASCE 4-86, AISC 8th Edition, ACI 318-89

Remarks:

REVISIONS

No.	Description	By	Date	Chk.	Date	App.	Date
0	Original Issue	MSLi	6/9/94	TMT	6/9/94	WD	6/9/94
1	Revised P. 6, 7, 21 and added P. B-1 of App. 6	MSLi	8/6/97	TMT WD	8/6/97 8/7/97	WD	8/7/97

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CALCULATION COVER SHEET

FIGURE 1.3

CONTRACT NO.

91C2683

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CLIENT WPSC JOB No. 91C2683 SHEET 1 OF 28

SUBJECT Kewaunee A46 / IPEEE
Refueling Water Storage Tank
153-021
Calculation C-018, Appendix G

REVISIONS
0 MSL 5/20/94
TMT 5/24/94

This calculation is to assess the integrity of Refueling Water Storage Tank (RWST) during the seismic event of Safety Shutdown Earthquake (SSE) anchored at 0.13 g (A-46 Outlier Resolution) and to compute the median Fragility of the tank.

Two analyses, upper bound and lower bound of the stiffness of the top ring, are performed.

For upper bound, it is assumed that the top ring is rigid because of the tank roof. Most of the seismic inertia loads will be concentrated at the top ring and two standard bumper structures because the relative high rigidity of the top ring compares to the rigidity of the two middle rings.

For lower bound, it is assumed that the top ring has the same rigidity as those of the two middle rings. Higher seismic inertia loads will be transferred to the two middle ring structures.

Therefore, the top ring and the standard bumper structures are checked to keep their integrity based on the upper bound case and the two middle ring structures are checked to keep their integrity based on the lower bound case.

The top roof diaphragm has sine flexibility and the seismic inertia forces acting on the structural components will be between the upper bound and lower bound cases. Following calculation is conservative.

Additional torsional effect of spectral acceleration is not considered in the following calculation. It is because the SSE FRS is an enveloped FRS. The RWST is within the enveloped structural location. No consideration in LLNL FRS because of all the conservative assumption being made.

Following Topics and structural components of RWST have been checked.
1. Anchor bolt and bolt chair capacity
2. Overturning moment capacity
3. Integrity of rings and lugs
4. Integrity of bumper structure
5. Freeboard clearance vs. slosh height
6. Base shear capacity vs. demand
7. Buckling of tank wall.

Summary of Results
RWST has shown to keep its integrity during an seismic event of SSE (A-46 Outlier Resolution) and has the median fragility of 0.69 G *

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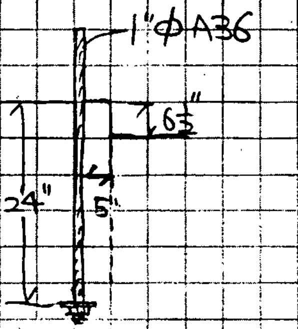
Tensile strength of 1" ϕ bolt

a) Based on bolt material

$P_u = 1.7 \times 19.1\ \text{ksi} \times \pi \cdot (1/2)^2 = 25.5$ kips

↖ AISC 9th Edition ASD, $F_t = 0.33 F_u$, $F_u = 58$ ksi



b) Based on concrete (shear cone)

Spacing $= 2 \times 13' \times \sin 22.5° = 10' > 2 \times L = 4'$

ACI 349 Appendix B, Only consider half of the shear cone

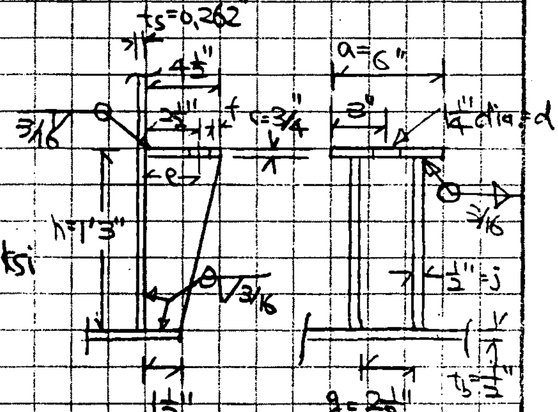
$P_u' = \frac{1}{2} \times 4 \phi \sqrt{f_c'}\ \pi (L+D) L$

$= 2 \times 0.65 \times \sqrt{3000}\ \pi \cdot (24+1) 24 = 134$ kips $> P_u$

Thus, $P_u = 25.5$ kips (bolt material in control)

Bolt Chair Capacity

(Ref. Kewaunee Drwg. # K-152-9, Rev.5)



a) Top plate

$\sigma = \dfrac{(0.375g - 0.22d) P_u}{f c^2}$

$= \dfrac{(0.375 \times 2.125 - 0.22 \cdot 1.25) \times 25.5}{(4.5 - 2.5 - 0.5 \times 1.25) \cdot 0.75^2} = 17.2\ \text{ksi} < f_y = 36$ ksi

o.k.

b) Tank Shell Stress

$Z = \dfrac{1.0}{\dfrac{0.177\, a\, t_b}{\sqrt{R t_s}} \left[\dfrac{t_b}{t_s}\right]^2 + 1.0} = \dfrac{1}{\dfrac{0.177 \cdot 6 \times 0.25}{\sqrt{13 \times 12 \times 0.262}} \left(\dfrac{0.25}{0.262}\right)^2 + 1}$

$= 0.964$

Material.
Chair: A36
Tank: A240-304

$\sigma = \dfrac{P_u \cdot e}{t_s^2} \left[\dfrac{1.32 Z}{\dfrac{1.43 a h^2}{R t_s} + (4 a h^2)^{0.333}} + \dfrac{0.031}{\sqrt{R t_s}} \right]$

$= \dfrac{25.5 \times 2.5}{0.262^2} \left[\dfrac{1.32 \times 0.964}{\dfrac{1.43 \times 6 \times 15^2}{13 \times 12 \times 0.262} + (4 \cdot 6 \cdot 15^2)^{0.333}} + \dfrac{0.031}{\sqrt{13 \times 12 \times 0.262}} \right] = 22.76\ \text{ksi} < f_{y_{tank}} = 30$ ksi o.k.

(Ref. ASME Table I-2.2)

c) Vertical stiff

shear stress

$k = \frac{1}{2}(4.5 + 1.5) = 3$"

$\dfrac{k}{j} = \dfrac{3}{0.5} = 6 < \dfrac{95}{\sqrt{f_y/1000}} = \dfrac{95}{\sqrt{36}} = 15.8$ ok.

compressive stress

$\dfrac{P_u}{2kj} = \dfrac{25.5\ \text{kips}}{2 \times 3 \times 0.5} = 8.5\ \text{ksi} < 21$ ksi ok.

Buckling

$j = 0.5$ in. but $j < 0.04(15" - 0.75") = 0.57"$

check buckling stress

Rectangular plate under uniform compression on opposite edges b,

Boundary condition: edges b simply supported, one edge a simply supported, other edge a free

(Ref. Formulas for Stress and Strain by Roark/Young, 5th ed, Table 35, case 1.d)

Buckling stress, $\sigma = 0.416 \frac{E}{1-\nu^2}\left(\frac{t}{b}\right)^2 = 0.416 \times \frac{29\times10^6 \text{ psi}}{1-0.3^2}\left(\frac{0.5}{3}\right)^2$

$= 368 \text{ ksi} >> \frac{P}{2Kj} = 8.5 \text{ ksi}$ o.k.

For tapered plate

$jK = 0.5\times3 = 1.5 > \frac{P}{25} = \frac{25.5}{25} = 1.02$ o.k.

d) Chair-to-tank Wall weld

$W_u = P_u\sqrt{\left[\frac{1}{a+2h}\right]^2 + \left[\frac{e}{ah + 0.667h^2}\right]^2} = 25.5\sqrt{\left[\frac{1}{6+2\times15}\right]^2 + \left[\frac{2.5}{6\times15 + 0.667\times15^2}\right]^2} = 0.76 \text{ kip/in}$

$W_u = 0.76 \frac{\text{kip}}{\text{in}} < \frac{30.6 \text{ ksi } t_w}{\sqrt{2}} = \frac{30.6\times 3/16}{\sqrt{2}} = 4 \text{ kip/in}$ o.k.

Stiffnesses of the Lugs & Rings (Ref. Kewaunee Dwg.# K-152-9, Rev 5 and S-313)

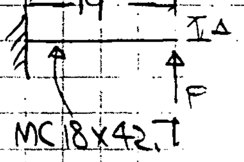
The lugs are assumed to be supported 3" from the wall where is the location of the stiffner of the bumper structure.

a) Top: Upper Bound: Assuming that the top ring is rigid because of the tank roof.

$\frac{1}{K_t} = \frac{L^3}{3EI} + \frac{L}{GA_s} = \frac{19^3}{3\times29\times10^3\times554} + \frac{19}{29\times10^3/2.6\times18\times0.45} = 3.526\times10^{-4} \frac{\text{in}}{\text{kip}}$

$K_t = 34031.5$ kip/ft

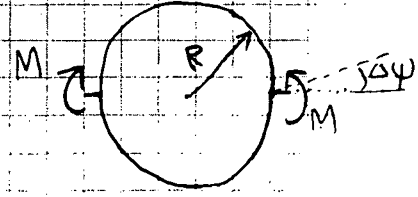
Lower Bound: same as the two middle rings



b) Two Middle

$\Delta\psi = \frac{MR}{EI}\left(\frac{\pi}{4} - \frac{2}{\pi}\right)$

(Ref. Roark/Young Table 17, case 3)



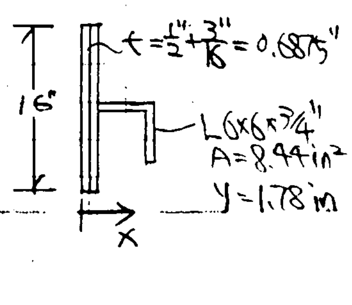
$K_\psi = \frac{M}{\Delta\psi} = \frac{6.7214EI}{R}$

$A = 16\times0.6875 + 8.44 = 19.44 \text{ in}^2$

$X_{cg} = \frac{16\times0.6875^2/2 + 8.44(6-1.78+0.6875)}{A} = 2.325"$

$I = \frac{16\times0.6875^3}{12} + 0.6875\times16\left(2.325 - \frac{0.6875}{2}\right)^2 + 28.2 + 8.44(6.6875 - 1.78 - 2.325)^2$

$= 128.1 \text{ in}^4$



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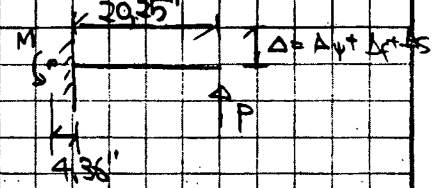
$$K_\psi = \frac{6.7214 \times 29\times10^3 \times 128.1}{13\times12} = 1.6\times10^5 \ \text{kip·in/rad}$$



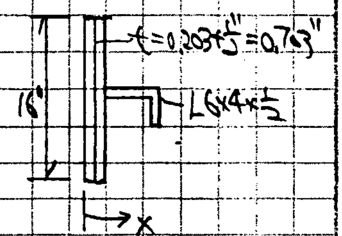
$$K_{\Delta\psi} = \frac{K_\psi}{\ell^2} = \frac{1.6\times10^5}{(20.25+4.36)^2} = 264 \ \frac{kip}{in} = 3170 \ \frac{kip}{ft}$$

$$K_{\Delta f} = \frac{3EI}{L^3} = \frac{3\times29\times10^3\times554}{20.25^3} = 5804 \ \frac{kip}{in} = 69652 \ \frac{kip}{ft}$$

$$K_{\Delta s} = \frac{GA_s}{L} = \frac{29\times10^3/2.6 \times 18\times0.45}{20.25} = 4461 \ \frac{kip}{in} = 53538 \ \frac{kip}{ft}$$

$$\frac{1}{K_M} = \frac{1}{K_{\Delta\psi}} + \frac{1}{K_{\Delta f}} + \frac{1}{K_{\Delta s}} \Rightarrow K_M = 2870 \ \frac{kip}{ft}$$

c) Bottom



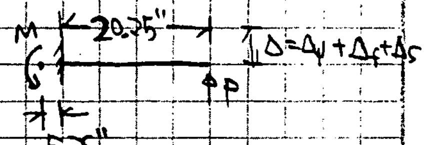
$$A = 16\times0.703 + 4.75 = 16 \ in^2$$

$$X_{cg} = \frac{11.248\times.703/2 + 4.75(6-1.99)}{A} = 1.44 \ in.$$

$$I = \frac{16\times0.703^3}{12} + 11.248\left(1.44 - \frac{0.703}{2}\right)^2 + 17.4 + 4.75\,(0.703+6-1.99-1.44)^2$$

$$= 82.1 \ in^4$$

$$K_\psi = \frac{6.7214\times29\times10^3\times82.1}{13\times12} = 1.026\times10^5 \ \frac{kip\cdot in}{rad}$$



$$K_{\Delta\psi} = \frac{K_\psi}{\ell^2} = \frac{1.026\times10^5}{(20.25+5.28)^2} = 157.66 \ \frac{kip}{in} = 1892 \ \frac{kip}{ft}$$

$$K_{\Delta f} = 69652 \ kip/ft, \quad K_{\Delta s} = 53538 \ kip/ft$$

$$\frac{1}{K_B} = \frac{1}{K_{\Delta\psi}} + \frac{1}{K_{\Delta f}} + \frac{1}{K_{\Delta s}} \Rightarrow K_B = 1781 \ \frac{kip}{ft}$$

Overturning Moment Capacity
(Ref. Kewaunee Drwg. # K-152-1, Rev. 9B and GIP, Section 7, Rev. 2A)
Kewaunee Operator Aid No. 89-1

$R = 13$ ft, $H = 69.5$ ft

$$t_{avg} = \frac{0.262+0.232+0.203+6\times3/16}{9} = 0.203"$$

$$t_{eff} = \frac{0.203 + 3/16}{2} = 0.1953"$$

$$\frac{t_{eff}}{R} = 0.00125$$

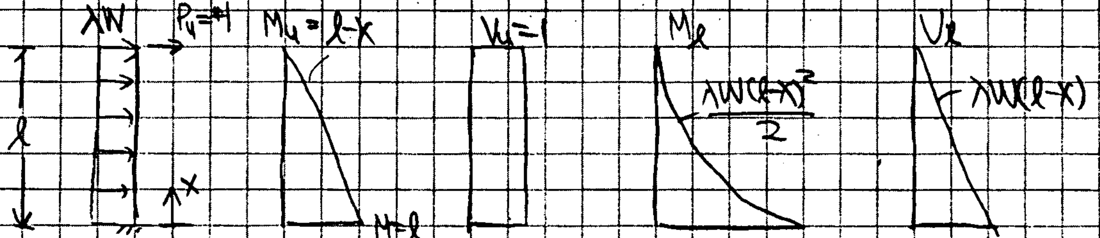
$$\frac{H}{R} = \frac{69.5'}{13'} = 5.35$$

$F_f = 2.31$ Hz (GIP Table 7-3)

The maximum spectral acceleration, $S_a > 1g$ over the frequency range of 20% broadening from 5% damping SSE & LLNL FRS (40 pg 23)

Compute the spectral acceleration to displace one eighth of an inch at the top of the tank

Assume uniform thickness $t_{ave} = 0.203''$ and use unit load method



$$\Delta = \int_0^l \frac{M_u M_l}{EI} dx + \int_0^l \frac{V_u V_l}{KGA} = \frac{l}{4} \frac{l \, \lambda W l^2/2}{EI} + \frac{1}{KGA} \frac{l}{2} \lambda W l = \frac{\lambda W l^4}{8EI} + \frac{\lambda W l^2}{2KGA}$$

$$K = \frac{2(1+\nu)}{4+3\nu} = 0.5306, \quad I = \pi R^3 t_{ave} = \pi \cdot 13^3 \cdot \frac{0.203}{12} = 116.76 \, ft^4, \quad A = 2\pi R t_{ave} = 2\pi \cdot 13 \cdot \frac{0.203}{12} = 1.382 \, ft^2$$

$$E = 28.3 \times 10^6 \, psi = 4.075 \times 10^6 \, kip/ft^2, \quad G = E/2.6, \quad W = \pi R^2 \, 62.4 = 33.13 \, kip/ft$$

$$\Delta = \frac{0.125''}{12} = \lambda \cdot 33.13 \cdot \left(\frac{69.5^4}{8 \times 4.075 \times 10^6 \times 116.76} + \frac{69.5^2 \cdot 2.6}{2 \times 0.5306 \times 4.075 \times 10^6 \times 1.382} \right) \Rightarrow \lambda = 0.04 \, (G)$$

Fluid pressure for elephant-foot buckling (P_e)

From EP Figure 7-7 with $S_{ae} = 0.048$, $H/R = 5.35$, we get $P_e' = 5.4$

$$P_e = P_e' \gamma_f R = 5.4 \times \frac{62.4}{1728} \times 13 \times 12 = 31 \, psi$$

Thus, the elephant-foot buckling stress capacity becomes

$$\sigma_{Pe} = \frac{0.6 E_s}{(R/t_s)} \left[1 - \left(\frac{P_e R}{\sigma_y t_s} \right)^2 \right] \left[1 - \frac{1}{1.12 + S_1^{1.5}} \right] \left[\frac{S_1 + \sigma_y/36000}{S_1 + 1} \right]$$

$$S_1 = \frac{R}{400 t_s} = \frac{13 \times 12}{400 \times 0.262} = 1.49$$

$$\sigma_{Pe} = \frac{0.6 \times 28.3 \times 10^6}{(13 \times 12 / 0.262)} \left[1 - \left(\frac{31 \times 13 \times 12}{30 \times 10^3 \times 0.262} \right)^2 \right] \left[1 - \frac{1}{1.12 + 1.49^{1.5}} \right] \left[\frac{1.49 + 30/36}{1.49 + 1} \right] = 11 \, ksi$$

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0 — MLi 5/20/94 — TM7 5/24/94
1 — MSLi 12/12/94 — [illegible] 12/12/94

Fluid pressure for diamond-shape buckling (P_d)

From GIP Figure 7-9 with $S_{af}=0.04g$, $H/R=5.35$, we get $P_d'=5.35$

$$P_d = P_d' \gamma_f R = 5.35 \times \frac{62.4}{123} \times 13 \times 12 = 30.14\ psi$$

Thus, the diamond-shape buckling stress capacity

$$\sigma_{Pd} = (0.6\gamma + \Delta\gamma)\frac{E_s}{R/t_s}$$

where $\gamma = 1 - 0.73\left[1 - e^{-\left(\frac{1}{16}\sqrt{R/t_s}\right)}\right] = 1 - 0.73\left[1 - e^{-\frac{1}{16}\sqrt{\frac{13\times12}{0.262}}}\right] = 0.43$

~~$\Delta\gamma = 0.05$ from GIP Figure 7-11~~ & $\frac{P_d}{E_s}\left(\frac{R}{t_s}\right)^2 = \frac{30.14}{28.3\times10^6}\left(\frac{13\times12}{0.262}\right)^2 = 0.38$

$$\sigma_{Pd} = (0.6\times0.43+0.05)\frac{28.3\times10^6}{13\times12/0.262} = 14.64\ ksi$$

Allowable Buckling Stress σ_c

$$\sigma_c = 0.72 \times \min[\sigma_{Pe}, \sigma_{Pd}]$$
$$= 0.72 \times \min[11\ ksi, 14.64\ ksi]$$
$$= 7.92\ ksi$$

Overturning Moment coefficient M'_{cap}

From GIP Fig. 7-12 with $t' = \frac{NA_b}{2\pi R}\left(\frac{E_b}{E_s}\right) = \frac{8\times\pi(0.5)^2}{2\pi\cdot13\times12}\times\frac{29}{28.3} = 0.0068\ in.$

or $C' = \left(\frac{t'}{t_s}\right)\left(\frac{h_s}{h_b}\right) = \frac{0.0068}{0.262}\cdot\frac{15}{39} = 0.01$

and $\sigma_c = 7.92\ ksi$ or $\frac{\sigma_c}{F_b}\frac{h_s}{h_b} = \frac{7.92}{25.5/(\pi0.5^2)}\frac{15}{39} = 0.1$

We get $M'_{cap} = 0.03$

Thus, the overturning moment capacity, M_{cap}

$$M_{cap} = M'_{cap}\times 2F_b\cdot R^2 t_s\cdot\left(\frac{h_b}{h_c}\right) = 0.03\times2\times\frac{25.5}{\pi0.5^2}\times(13\times12)^2\times0.262\times\frac{39}{15}$$
$$= 32300\ Kip\cdot in = 2691\ kip\cdot ft$$

Overturning moment demand, M

$$M = S_{af}\frac{W}{g}\frac{H^2}{2} = 0.04\times3313\times\frac{69.5^2}{2} = 3200\ kip\cdot ft > M_{cap} = 2691\ kip\cdot ft$$

$\Rightarrow$ Bottom of the tank will buckle, see page 31 for discussion of tank buckling.

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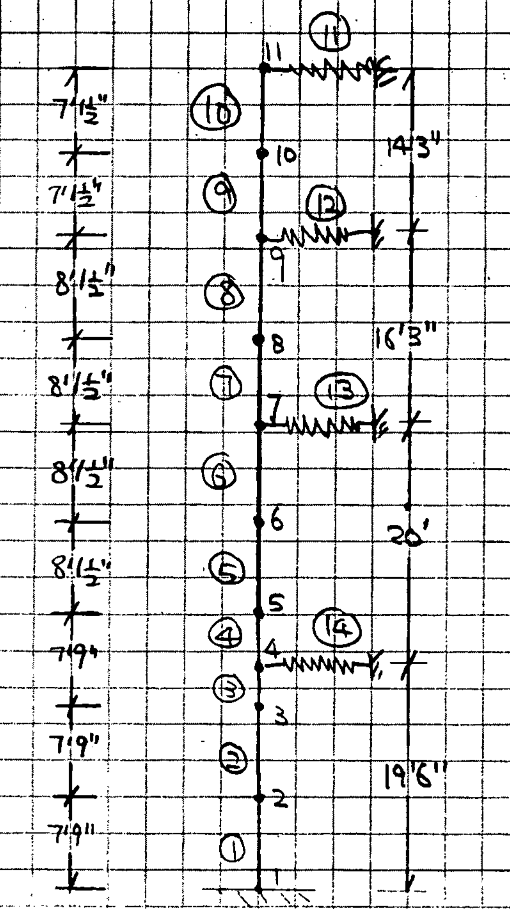
1 MSL 8/6/97 TMT 8/6/97

Finite Element Model of the tank (Ref. Kewaunee Dwg #K-152-1, Rev. 9B, 4/12/72)



element number	thickness (in)	I (ft⁴)	A (ft²)	Mass Density (kips·sec²/ft⁴)
1	0.262	150.695	1.783	0.5927
2	0.232	133.440	1.579	0.6674
3-4	0.203	116.760	1.382	0.7605
5-10	3/16	107.845	1.276	0.8221

where $I = \pi R^3 \cdot t$

$A = 2\pi R t$

$$\rho = \frac{\pi R^2 \cdot 62.4 + A \cdot 490}{A \cdot 32170} \ (\text{kip} \cdot \text{sec}^2/\text{ft}^4)$$

Maximum water level 69'6"
(Kewaunee Operator Aid No. 89-1, See page 27)
Assume water level 70' and ignore the weight of tank roof in computing the mass density

Stiffnesses of element # 11 - 14

$K_{11} = 2 \times 3403.15 = 6806.3$ kip/ft (upper bound), $K_{11} = K_{12}$ (lower bound)

$K_{12} = K_{13} = 2 \times 2870 = 5740$ kip/ft

$K_{14} = 2 \times 1781 = 3562$ kip/ft

The impulsive and sloshing modes are not significant in this case because the top and other lugs will make contact with the bumper structure which are embedded into the concrete wall. Therefore, the tank is no longer to be a free standing tank. However, the impact amplification factor at lugs is set to one since the gap is only one eighth of an inch and assume that no significant momentum will be developed in this short distance.

Pressure $= \pi R^2 \cdot 62.4 + A_{ave} \cdot 490 = 33.31$ kip/ft (Use for Stress Analysis in following COSMOS/M Runs)

COSMOS/M Run – RWST with Fixed Base and no spring supports

Analysis	Input file*	Output file*
Frequency	fxrwst.mod	fxrwst.out
Stress	fxdprwst.mod	fxdprwst.out

* All COSMOS/M run files in this calculation are enclosed in the attached diskette.

First mode of structural frequency, $F_f = 2.13$ Hz ($\approx F_f = 2.31$ Hz by E/P) ok.

Displacement at top with 1g, $d = 0.2632'$

$$S_{af}\left(d = \frac{1}{8}''\right) = \frac{0.125}{0.2632 \times 12} = 0.04g \quad \text{as before} \quad \text{ok.}$$

Thus, the finite element model of tank is adequate.

Moment at base with 1g, M = 82830 kip-ft

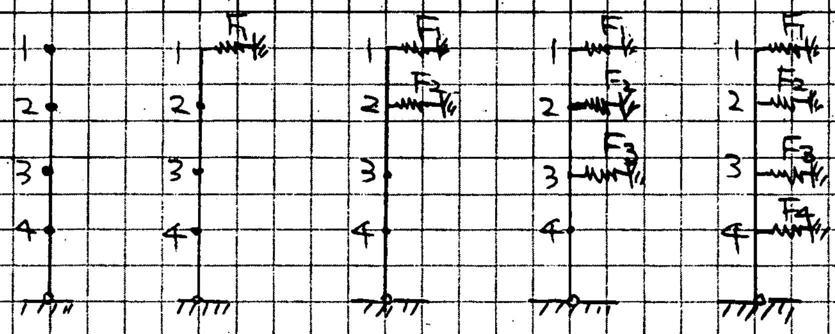
$$S_{af} = \frac{M_{cap\ with\ 0.04g}}{M} = \frac{2691}{82830} = 0.033(g)$$

Base moment capacity exceeds the demand until the spectral acceleration reaches 0.033g.

The following COSMOS/M analysis is assumed that the boundary condition of the base is hinge. As we know it is a conservative assumption since we ignore the fluid holddown force. Therefore, the results of frequency analysis from COSMOS/M runs are considered to be the lower bound.

$S_{af} > 1g$ when the tank is free standing. Therefore, it is assumed that once the lugs make the contact with the bumper structures, they will maintain the contact until the cycle is ended. Based on this assumption, the problem becomes a linear problem at each stage of lugs' contacts.

The lugs and their bumper structure will be subjected to the smaller value of the accumulated inertia force until other lugs make contact and the inertia force under the spectra acceleration.



case 0 case 1 case 2 case 3 case 4

Input & output files of COSMOS/M Runs

Case #	K1 bound	Analysis	Input file	Output file
1	upper	Frequency	s1urwst.mod	s1urwst.out
		Stress	s1durwst.mod	s1durwst.out
	lower	Frequency	s1lrwst.mod	s1lrwst.out
		Stress	s1dlrwst.mod	s1dlrwst.out
2	upper	Frequency	s2urwst.mod	s2urwst.out
		Stress	s2durwst.mod	s2durwst.out
	lower	Frequency	s2lrwst.mod	s2lrwst.out
		Stress	s2dlrwst.mod	s2dlrwst.out
3	upper	Frequency	s3urwst.mod	s3urwst.out
		Stress	s3durwst.mod	s3durwst.out
	lower	Frequency	s3lrwst.mod	s3lrwst.out
		Stress	s3dlrwst.mod	s3dlrwst.out
4	lower	Frequency	s4lrwst.mod	s4lrwst.out
		Stress	s4dlrwst.mod	s4dlrwst.out

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Summary Results of Upper bound stiffness of top lugs (assuming that the top ring is rigid) from COSMOS/M Runs

Incr. Accel. (g)	case 0			case 1			case 2			case 3	
	0	0.033		1	0.052		1	0.090		1	0.145
Displ. (in)	initial gap	deform.	remain. gap	deform.	deform.	remain. gap	deform	deform	remain. gap	deform	deform
pt. #1	0.125	0.125	0	—	—	0	—	—	0	—	—
pt. #2	0.125	0.100	0.025	0.484	0.025	0	—	—	0	—	—
pt. #3	0.125	0.071	0.054	0.612	0.032	0.022	0.555	0.022	0	—	—
pt. #4	0.125	0.035	0.090	0.435	0.023	0.067	0.409	0.016	0.051	0.353	0.051

case #	1	2	3	4
lower bound of frequency (Hz) from COSMOS/M Run	4.45	4.68	5.05	—
Max. Saf (g) from 5% damping SSE & LLNL FRS (page 23)	0.27	0.268	0.26	—
Max. Accumulated Saf (g) before other lugs make contact	0.085	0.125	0.27	—
controlling Saf (g)	0.085	0.125	0.26*	—

Case 4 does not develop because the max. Saf from 5% damping SSE & LLNL FRS is less than the max. accumulated Saf making all the lugs contact.

case #	0		1		2		3		4	
Total Accel. (g)	1	0.033	1	0.085	1	0.125	1	0.26	—	—
F_1 (kips)	—	—	1183	101	1024	128	918	228	—	—
F_2 (kips)	—	—	—	—	201	25	175	45	—	—
F_3 (kips)	—	—	—	—	—	—	223	58	—	—
F_4 (kips)	—	—	—	—	—	—	—	—	—	—
base shear (kips)	2367	78	1183	101	1143	143	1051	273	—	—

Summary Results of lower bound stiffness of top lugs (assuming that the stiffness of top lugs is the same as the stiffness of the two middle lugs) from COSMOS/M Runs

Incr. Accel. (g)	case 0			case 1			case 2			case 3	
	0	0.033		1	0.011		1	0.028		1	0.103
Displ. (in.)	initial gap	deform.	remain. gap	deform.	deform.	remain. gap	deform.	deform.	remain. gap	deform.	deform.
pl. #1	0.125	0.125	0	—	—	0	—	—	0	—	—
pl. #2	0.125	0.100	0.025	2.288	0.025	0	—	—	0	—	—
pl. #3	0.125	0.071	0.054	1.890	0.021	0.033	1.170	0.033	0	—	—
pl. #4	0.125	0.035	0.090	1.066	0.012	0.078	0.701	0.020	0.058	0.564	0.058

case #	1	2	3	4
lower bound of frequency (Hz) from COSMOS/M Run	2.25	2.92	3.30	3.36
Max. Saf (g) from 5% damping SSE & LLNL FRS (page 23)	0.625	0.41	0.323	0.315
Max. Accumulated Saf (g) before other lugs make contact	0.044	0.072	0.175	—
controlling Saf (g)	0.044	0.072	0.175	0.315

case #	0		1		2		3		4	
Total Accel. (g)	1	0.033	1	0.044	1	0.072	1	0.175	1	0.315
F_1 (kips)	—	—	1183	52	681	49	549	96	532	168
F_2 (kips)	—	—	—	—	631	45	498	87	480	151
F_3 (kips)	—	—	—	—	—	—	422	74	402	127
F_4 (kips)	—	—	—	—	—	—	—	—	150	47
Base Shear (kips)	2367	78	1183	52	1055	76	898	157	802	252

Maximum force resultants at various supported structures

Top ring : 238 kips (upper bound case)

Two Middle rings : 151 kips (lower bound case)

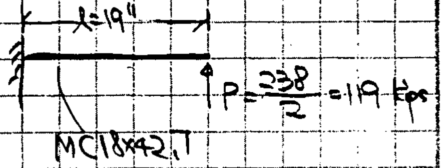
bottom rings : 47 kips (lower bound case)

Bumper structures : 238/2 = 119 kips (upper bound)

Base shear : 273 kips

Top Ring and Lugs Check (Ref. Kewaunee Drwg # E-152-9, Rev 5)

(a) Channel MC18x42.7



$M = P \times 19 = 2261$ kip-in

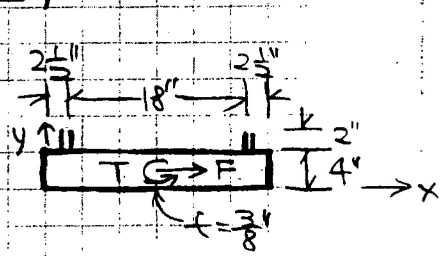
$\frac{b}{t} = \frac{3.95}{0.625} = 6.32 < \frac{65}{\sqrt{F_y}} = 10.8$

$l = 19" <$ smaller of $\frac{76\, b_f}{\sqrt{F_y}} = \frac{76 \times 3.95}{\sqrt{36}} = 50"$ or $\frac{20000}{(d/A_f) F_y} = \frac{20000}{7.29 \times 36} = 76"$

$f_b = \frac{M}{S_x} = \frac{2261}{61.6} = 36.70$ ksi $< F_b = 1.7 \times 0.66 F_y = 40$ ksi ok.

$f_v = \frac{F}{d \cdot t_w} = \frac{119}{18 \times 0.45} = 15$ ksi $< F_v = 1.7 \times 0.4 F_y = 24.48$ ksi ok.

(b) Channel-to-Ring Weld
(Ref. Field sketch by G. Ridder, 12/2/94, page 22)



$A = 2 \times (18 + 2 \times 2.5 + 4 + 2 \times 2) \times 0.265 = 16.43$ in^2

$X_c = 9 + 2.5 = 11.5"$

$Y = \frac{(2 \times 4 \times 2 + 4 \times 23 + 4 \times 2 \times 5)\, 0.265}{16.43} = 2.39$ in

$I_{xc} = 23 \times 0.265 \times [2.39^2 + (4 - 2.39)^2] + \frac{0.265 \times 4^3}{12} \times 2 + 2 \times 4 \times 0.265 \times (2 - 2.39)^2$
$+ 4 \times \left[\frac{0.265 \times 2^3}{12} + 2 \times 0.265 \times (5 - 2.39)^2\right] = 68.91$ in^4

$I_{yc} = 2 \times \frac{0.265 \times 23^3}{12} + 2 \times 4 \times 0.265 \times 11.5^2 + 4 \times 2 \times 0.265 \times (11.5 - 2.5)^2 = 989.47$ in^4

$I_p = I_{xc} + I_{yc} = 68.91 + 989.47 = 1058.38$ in^4

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$F = 119$ kips

$T = 119 \times (19 + 2.39) = 2545$ kip-in

$f_x' = \frac{F}{A} = \frac{119}{16.43} = 7.24$ ksi, $f_y' = 0$

$f_x'' = \frac{T \cdot (6 - 2.39)}{I_p} = \frac{2545 \times 3.61}{1058.38} = 8.68$ ksi

$f_y'' = \frac{T \times 11.5}{I_p} = \frac{2545 \times 11.5}{1058.38} = 27.65$ ksi

$f = \sqrt{(f_x' + f_x'')^2 + (f_y' + f_y'')^2} = \sqrt{27.65^2 + (7.24 + 8.68)^2} = 31.9\text{ ksi} < F_w = 0.56 \times 60\text{ ksi} = 33.6\text{ ksi}$

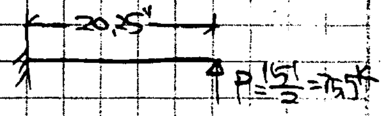
(Ref. EPRI NP-6041-SL, Rev. 1, Appendix P)

(c) Top Ring

Top Ring is judged ok because ring is supported by tank roof.

Two Middle Rings and Lugs Check (Ref. Kewaunee Drwg. # K-152-9, Rev. 5)

(a) Channel MC 18×42.7



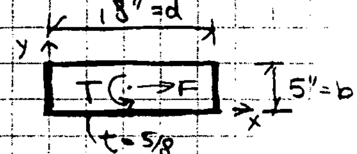
$M = 75.5 \times 20.25 = 1528.9$ kip-in

$f_b = \frac{M}{S_x} = \frac{1528.9}{61.6} = 24.82\text{ ksi} < F_b = 40\text{ ksi}$ ok.

$f_v = \frac{F}{d \times t_w} = \frac{75.5}{18 \times 0.45} = 9.32\text{ ksi} < F_v = 24.48\text{ ksi}$ o.k.

(b) Channel-to Ring Weld

(Ref. Kewaunee Drwg. # K-152-9, Rev. 5)



$t_e = 0.707 \cdot \frac{5}{8} = 0.442"$

$A = 0.442 \times 2 \times (18 + 5) = 20\text{ in}^2$

$I_p = \frac{(b+d)^3}{6} t = \frac{(5+18)^3}{6} \times 0.442$

$= 896.3\text{ in}^4$

$F = 75.5$ kips

$T = F \cdot (20.25 + 5/3) = 1717.63$ kip-in

$f_x' = \frac{F}{A} = \frac{75.5}{20} = 3.80$ ksi, $f_y' = 0$

$f_x'' = \frac{T \cdot b/2}{I_p} = \frac{1717.63 \times 2.5}{896.3} = 4.80$ ksi

$f_y'' = \frac{T \cdot d/2}{I_p} = \frac{1717.63 \times 9}{896.3} = 17.25$ ksi

$f = \sqrt{(f_x' + f_x'')^2 + (f_y' + f_y'')^2} = \sqrt{(3.8 + 4.8)^2 + 17.25^2} = 19.28\text{ ksi} < F_w = 30.6\text{ ksi}$ ok.

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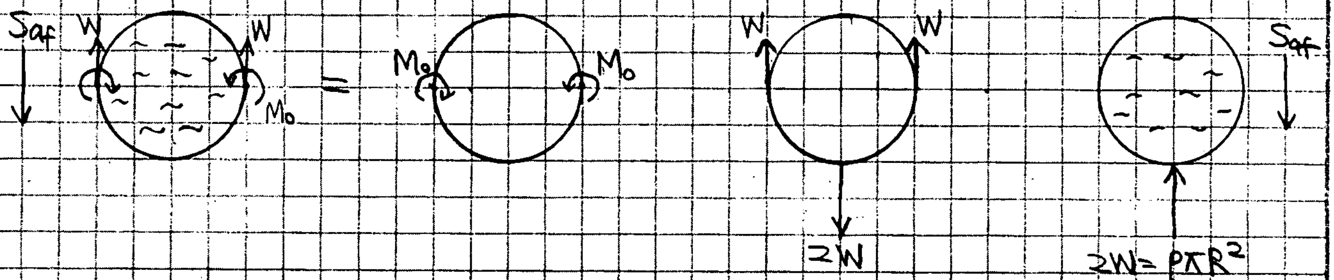
(c) Two Middle Rings

The inertia forces of the ring can be considered to be the combination of the following cases in Roark/Young, 5th edition, Table 17.



Case 3 in Roark/Young with $\theta = \pi/2$

Case 4 in Roark/Young with $\theta = \pi/2$

Case 14 in Roark/Young

$W = 75.5$ Kips

$M_o = 75.5(20.25 + 6.685 - 2.325)$

$= 1858$ Kip-in

$\rho = \frac{2W}{\pi R^2} = \frac{151 \text{ Kip}}{\pi R^2}$

Summary of ring section properties (Ref. Page 3)

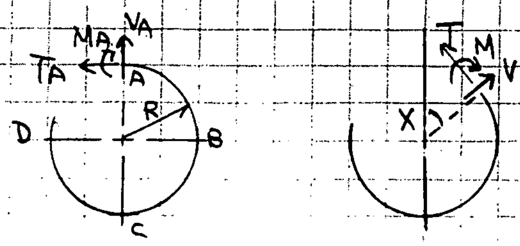
$A = 19.44 \text{ in}^2$

$I = 128.1 \text{ in}^4$

X_{cg} from the inside of tank wall $= 2.325''$

Summary of Formula for circular rings in "Roark/Young", 5th edition

Notation: θ and x are angles (radians) $s = \sin\theta$, $c = \cos\theta$, $z = \sin x$, $u = \cos x$



$M = M_A - T_A R(1-u) + V_A R z + LT_M$

$T = T_A u + V_A z + LT_T$

$V = -T_A z + V_A u + LT_V$

where LT_M, LT_T, and LT_V are load terms given below for the type of load cases above

with step-function $\langle x - \theta \rangle^0 \begin{cases} = 0 & \text{if } x < \theta \\ = 1 & \text{if } x \geq \theta \end{cases}$

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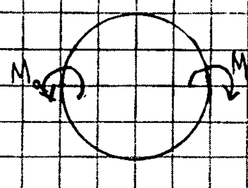
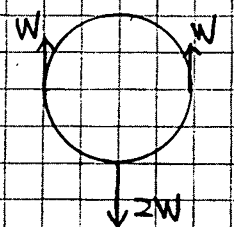
Case 3 with $\theta = \pi/2$



$LT_M = M_o\langle x - \frac{\pi}{2}\rangle^0$, $LT_T = 0$, $LT_V = 0$

$M_A = -M_o\left(\frac{1}{2} - \frac{2}{\pi}\right)$, $T_A = \frac{2M_o}{\pi R}$, $V_A = 0$

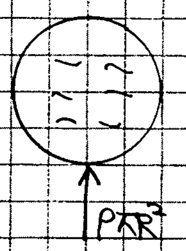
Case 4 with $\theta = \pi/2$



$LT_M = WR(z-1)\langle x - \frac{\pi}{2}\rangle^0$, $LT_T = Wz\langle x - \frac{\pi}{2}\rangle^0$, $LT_V = Wu\langle x - \frac{\pi}{2}\rangle^0$

$M_A = -WR\left(\frac{2}{\pi} - \frac{1}{2}\right)$, $T_A = -\frac{W}{\pi}$, $V_A = 0$

Case 18 with $P = \frac{2W}{\pi R^2}$



$LT_M = PR^3\left(1 - u - \frac{xz}{2}\right) = \frac{2WR}{\pi}\left(1 - u - \frac{xz}{2}\right)$

$LT_T = PR^2\left(1 - u - \frac{xz}{2}\right) = \frac{2W}{\pi}\left(1 - u - \frac{xz}{2}\right)$

$LT_V = PR^2\left(\frac{z}{2} - \frac{xu}{2}\right) = \frac{2W}{\pi}\left(\frac{z}{2} - \frac{xu}{2}\right)$

$M_A = \frac{PR^3}{4} = \frac{WR}{2\pi}$, $T_A = \frac{3PR^2}{4} = \frac{3W}{2\pi}$, $V_A = 0$

A spreadsheet program, ring.xls, written in EXCEL 5.0 is used to compute the internal forces, M, T and V of the three cases above and the combination. Then the shear and normal stress are computed. The printouts are enclosed at the end of this calculation and the plots of internal moment, axial and shear force in page 17.

Hydrostatic internal stress at the middle ring near the top.

$$\sigma_P = \frac{\gamma h \times 16'' \cdot R}{A} = \frac{62.4\ lb/ft^3 \cdot (69.5' - 55.75') \times 16'' \cdot 13'}{19.44\ in^2} = 0.765\ ksi$$

Maximum Bending tensile stress at the ring outside of the ring at 90° (Ref. page 25)

$$f_{bo} = \sigma_P + \frac{T}{A} + \frac{M \cdot (6.875 - 2.325)}{I} = 0.765 + \frac{86}{19.44} + \frac{934 \times 4.325}{128.1} = 37\ ksi \simeq F_y = 36\ ksi \quad o.k.$$

(Stress is overestimated since the internal moment acting on the ring distribute over the width of channel.)

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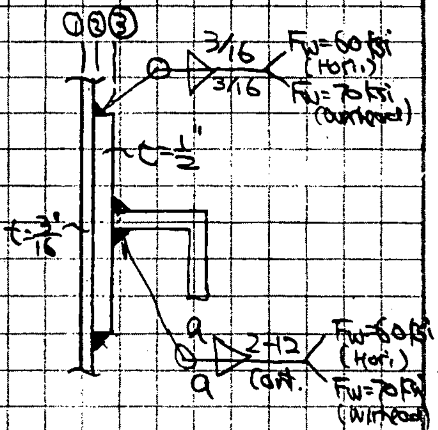
Maximum Bending compressive stress at the ring
Outside of the ring at 85° (Ref. page 25)

$$f_{bo} = f_\phi + \frac{T}{A} - \frac{M \cdot 4.3525}{I} = -0.765 + \frac{9}{9.84} - \frac{679\times4.3525}{128.1} = -22 \text{ ksi} \quad \text{o.k. } (F_y > |f_{bo}|)$$

Weld stress check
Bending stress at section ①, ② & ③
without the hydrostatic stress (Ref. page 25)

Location of Ring	Section ①	Section ②	Section ③
X=85°	12.78 ksi	11.78 ksi	9.13 ksi
X=90°	-12.54 ksi	+11.18 ksi	-7.53 ksi



Middle ring near the top
a = 5/16
Middle ring near the bottom
a = 1/4
(Ref. Kewaunee Drwg. # E-1529, Rev 5)

Ref. Designing steel structures by Sule Cooper P.326
Shear on weld at section ②

$$q_2 = \frac{A_f \Delta f_b}{\Delta L} = \frac{16\times 3/16 \times (12.78+11.78+12.54+11.18)/4}{156\times 5/180 \times \pi} = 2.1 \text{ kip/in}$$

Capacity on weld at section ②

$$F_w = \frac{3}{16}\times0.707\times0.56\times(60+70) = 9.65 \text{ kip/in} > q_2 \text{ (ok)}$$

Shear on weld at section ③

$$q_3 = \frac{A_f \Delta f_b}{\Delta L} = \frac{16(\frac{3}{16}+5)\times(12.78+9.13+12.54+7.53)/4}{156\times 5/180\times\pi} = 8.48 \text{ kip/in}$$

Capacity on weld at section ③

$$F_w = \frac{5}{16}\times0.707\times0.56\times(60\times\frac{2}{12}+70) = 9.9 \text{ k/in} > q_3 \text{ (o.k.)}$$

The Middle ring near the bottom is identical to the middle ring near the top except the welding size between the angle and the 1/2" plate as shown above. Only the weld stress at section ③ is needed to be check.

Shear on weld at section ③

$$q_3 = 8.48 \frac{\text{kip}}{\text{in}} \times \frac{\text{Lugs load at Middle ring near the bottom}}{\text{Lugs load at Middle ring near the top}} = 8.48\times\frac{127}{151} = 7.13 \text{ kip/in}$$

Capacity on weld at section ③

$$F_w = 9.9 \text{ kip/in} \times \frac{1/4}{5/16} = 7.92 \text{ kip/in} > q_3 \text{ (o.k.)}$$

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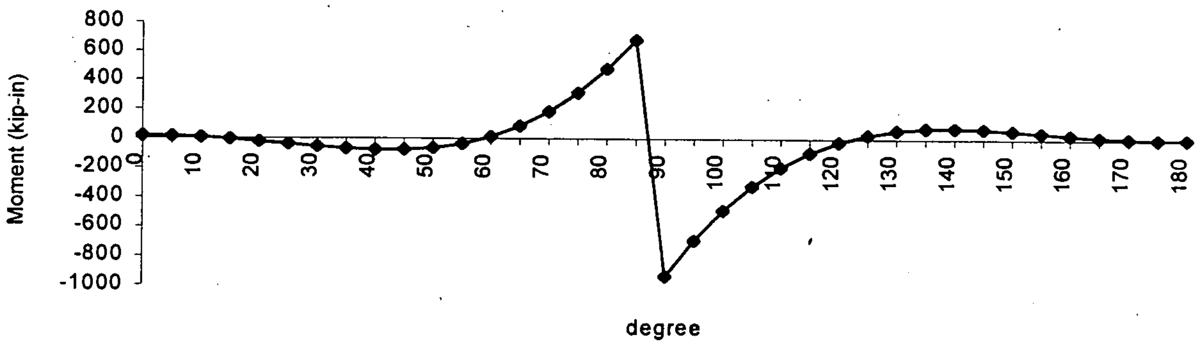
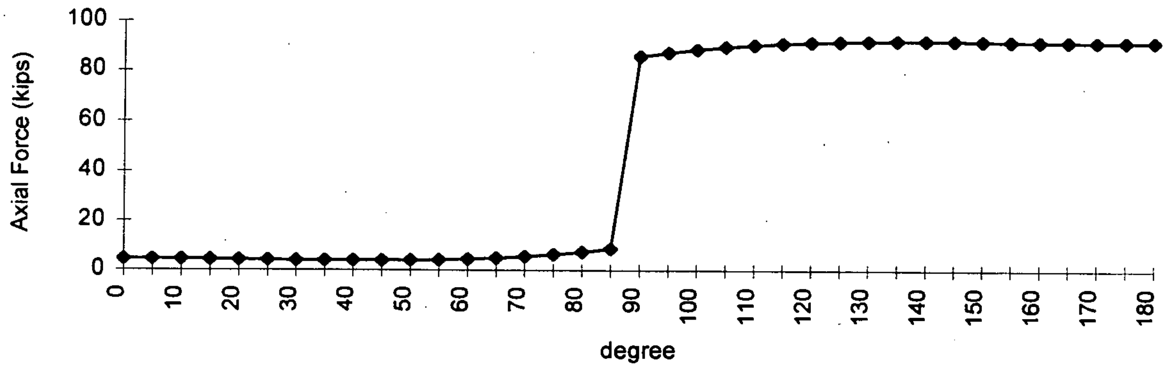
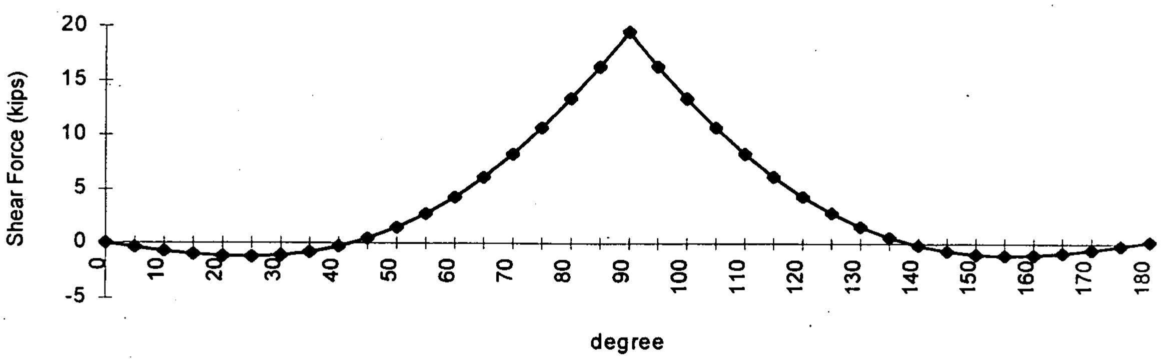




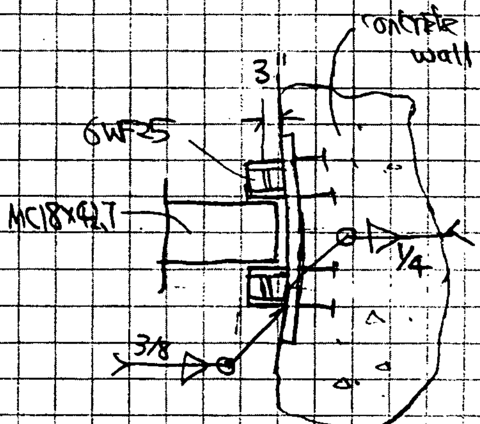


Bottom Ring and Lugs Check (Ref. Kewaunee Drwg. # E-152-9, Rev.5)

Bottom ring and lugs has the same geometry as those in the two middle rings and lugs except that the L6x6x3/4 in the middle ring is replaced by L6x4x1/2 in the bottom ring. However, the bottom ring is subjected to one-third of the loading acting on the middle ring. Thus, it is judged ok.

Bumper Structure (Ref. Kewaunee Drwg. # S-313)

(a) Base-plate-to-bumper structure weld



$F = 119$ Kips

$A = 6.38 \times 0.354 + 2 \times 6.08 \times 0.53 = 14.43 \text{ in}^2$

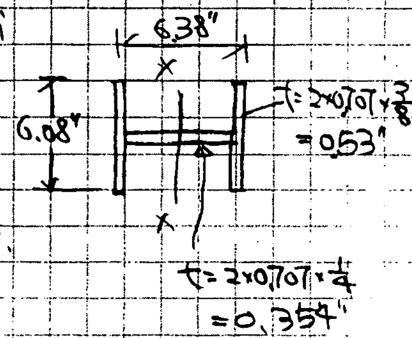
$I_x = 2 \times 6.08 \times 0.53 \times \left(\frac{6.38}{2}\right)^2 + \frac{1}{12} \times 0.354 \times 6.38^3$

$= 73.24 \text{ in}^4$

$f_v = \frac{F}{A} = \frac{119}{14.43} = 8.25 \text{ ksi}$

$f_b = \frac{F \cdot 3 \times 6.38/2}{I_x} = \frac{119 \times 3 \times 6.38/2}{73.24} = 15.55 \text{ ksi}$

$f = \sqrt{f_v^2 + f_b^2} = \sqrt{8.25^2 + 15.55^2} = 17.60 \text{ ksi} < F_w = 30.6 \text{ ksi}$ (o.k.)



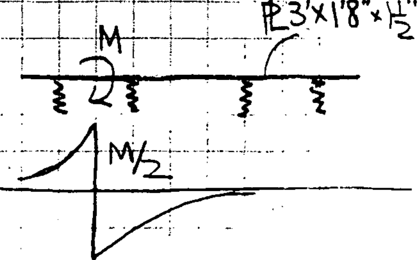
(b) WF 6x25

$f_b = \frac{F \cdot 3}{S} = \frac{119 \times 3}{16.7} = 21.4 \text{ ksi} < F_b = 1.7 \times 0.66 F_y = 40.4 \text{ ksi}$ o.k.

$f_v = \frac{119}{7.34} = 16.2 \text{ ksi} < F_v = 1.7 \times 0.4 F_y = 24.5 \text{ ksi}$ o.k.

(c) Base plate

Although the beam is not infinite long, the moment distribution is considered to be conservative since the acting moment is distribution over 6", the width of the beam



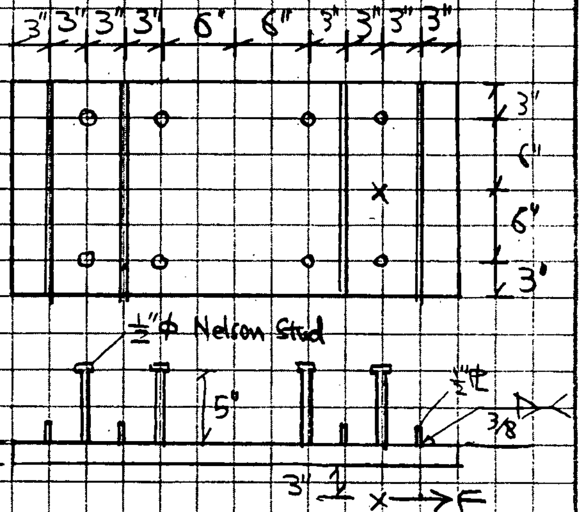
Moment distribution of an infinite beam on elastic foundation

$f_b = \frac{\frac{M}{2}}{\frac{bh^2}{6}} = \frac{\frac{119 \times 3}{2}}{\frac{18 \times 1.5^2}{6}} = 26.4 \text{ ksi} < F_y = 36 \text{ ksi}$

o.k.

(d) Base-plate Anchorage

$F = 19$ kips



Shear in Welding

$A_w = 4 \times 2 \times 0.707 \times 3/8 \times 18 = 38.18\ in^2$

$f_v = \frac{F}{A_w} = \frac{19}{38.18} = 3.12\ ksi < F_w = 30.6\ ksi$

ok.

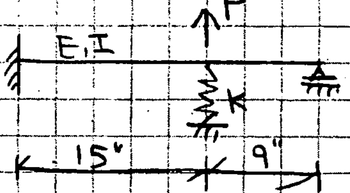
Shear in 4 - $\frac{1}{2}$" PL

$A_s = 4 \times 18 \times 0.5 \times 2/3 = 24\ in^2$

$f_v = \frac{F}{A_s} = \frac{119}{24} = 4.96\ ksi < F_v = 1.7 \times 0.4 F_y = 24.48\ ksi$

ok.

Following model is used to estimate the percentage of pull-out force resisted by the four far-side Nelson studs.



$I = \frac{18 \times 1.5^3}{12} = 5.0625\ in^4$

$K = \frac{EA}{L} = \frac{E \cdot (0.25)^2 \pi \times 4}{5} = 0.157E$

Without the spring K, the deflection at point of load P is (Ref. AISC 9th edition, ASD, page 2-300)

$\Delta a = \frac{Pa^2b^3}{12EI\ell^3}(3\ell + a) = \frac{P \cdot 9^2 \times 15^3}{12E \cdot 5.0625 \cdot 24^3}(3 \times 24 + 9) = 26.367 P/E$

Stiffness of the beam

$K_b = \frac{P}{\Delta a} = \frac{P}{26.367 P/E} = 0.038E$

Thus, the percentage of pull-out force resisted by the four-far-side Nelson studs is

$\%\ \text{of pull-out force} = \frac{K_b}{K + K_b} = \frac{0.038E}{0.157E + 0.038E} = 20\%$

The demand of the four Nelson studs near the external moment is estimated to be

$$T_p = \frac{F \times 3}{4 \times 9} \times (1-0.2) = \frac{119 \times 3}{4 \times 9} \times 0.8 = 7.33 \text{ kip per stud.}$$

Nelson Studs pull-out capacity
Material A-108, $F_u = 60$ ksi, $F_y = 50$ ksi (Ref. "Embedment properties of headed studs" by TRW Nelson Division)
Testing showed that the failure mode of $\frac{1}{2}$" Nelson studs with 5" embedment length and concrete strength equal to 3000 psi was shank of stud broke rather than weld at head broke. ⇒ Ductile failure
(Ref." "Embedment properties of headed studs" by TRW Nelson Division)

Design Tension of bolt (AISC, 1st edition, LRFD)
$P_u = 0.75 \times$ Nominal strength

Nominal strength $= \phi A_b \cdot F_u$ (Equation in AISC, LRFD, C-J3-1)
ϕ is taken to be 0.75 because tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts.

However, the Nelson studs in our case are not subjected to any bending stress, the ϕ is set to be one. The Nelson stud is considered to be an tension member

$$P_u = \min.(0.75 A_b F_u,\ 0.9 A_b F_y) = 8.84 \text{ kips}$$

Nelson studs capacity based on ACI shear cone theory
$P_c' = 4\phi\sqrt{f_c'}\,\pi(L+D)L$ ACI 349 Appendix B, $\phi = 0.65$
$= 4 \times 0.65 \times \sqrt{4000}\,\pi(5+0.5)5$
$= 14$ kips

Studs spacing is less than $2L+D = 10.5$" at one side
$A_{s,red} = \pi r^2 - \frac{1}{2}\left[r^2\theta - rS\sin\left(\frac{\theta}{2}\right)\right]$ where $r = \frac{2L+D}{2} = \frac{2 \times 5 + 0.5}{2} = 5.25$"
$= \pi\, 5.25^2 - \frac{1}{2}\left[5.25^2 \times 1.925 - 5.25 \times 6 \times \sin\left(\frac{1.925}{2}\right)\right]$ $\theta = 2\cos^{-1}\left[\frac{S}{2L+D}\right] = 2\cos^{-1}\left[\frac{6}{10+0.5}\right] = 1.925$ rad.
$= 72.99 \text{ in}^2$

Thus, $P_c' = 14 \text{ kips} \times \frac{A_{s,red}}{\pi r^2} = 14 \times \frac{72.99}{\pi \cdot 5.25^2} = 11.8 \text{ kip} > P_u = 8.84$ kips (Stud in control)

$$P_u = 8.84 \text{ kips} > T_p = 7.33 \text{ kip (o.k.)}$$

Slosh Height vs. Freeboard Clearance

Freeboard clearance = 6"

Slosh Mode Frequency

$$F_s = \frac{1}{2\pi}\sqrt{\frac{1.84g}{R}\tanh\left(\frac{1.84H}{R}\right)} = \frac{1}{2\pi}\sqrt{\frac{1.84\times32.2}{13}\tanh\left(\frac{1.84\times69.5}{13}\right)} = 0.34\ Hz$$

The maximum spectral Acceleration

$S_a = 0.0238\ g$ at $f = 1.2\times0.34 Hz = 0.41 Hz$ from 0.5% damping SSE & LLNL FRS

(Ref. 91C2683 Calculation Nos C-04 & C-05)

Slosh Height

$h_s = 0.837 R S_a = 0.837\times156\times0.0238 = 3.1$ in < 6 in freeboard

factor of safety ≐ 2 ok

Buckling of tank Wall

When the tank base reaches its moment capacity (Assuming the tank wall buckles), no additional force will be generated at the compression side of tank wall from the view point of vertical equilibrium condition. Additional horizontal seismic force will be transferred through its rings and lugs and bumper structures to its support structure.

Base Shear

$$Q_{cap} = 0.55(1-0.21 S_{af})W \quad \text{(Ref. EdP Section 7)}$$
$$= 0.55\cdot(1-0.21\times0.315)\times\pi\cdot13^2\times\frac{62.4}{1000}\times69.5$$
$$= 1183\ \text{kips}$$

$Q = 273$ kips $< Q_{cap} = 1183$ kips o.k.

Stevenson and Associates A Structural-Mechanical Consulting Engineering Firm	CLIENT WPSC	JOB No. 91C2683	SHEET 22 OF 28
	SUBJECT Kewaunee A46/IPEEE 153-021	REVISIONS 0	MSL 5/20/94 TMT 5/24/94

Seismic Fragilities

$$CDFM = F_\mu \cdot \frac{\sigma_u}{\sigma_{c,RLE}} S_{G,RLE}$$

where F_μ = ductility factor (1.25 if failure mode is ductile, 1.0 if brittle)
σ_u = Ultimate capacity applicable to the HCLPF calculation
$\sigma_{c,RLE}$ = Calculated stress due to IPEEE Review Level Earthquake (RLE)
$S_{G,RLE}$ = PGA of RLE

Previous stress calculation is considered to be conservative because the structural components are checked against their highest possible loading.

Following is only listed the most critical CDFM at each structural component.

a) Top Ring and lugs
Channel-to-Ring weld
$$CDFM = 1.0 \times \frac{33.6}{31.9} \times 0.306g = 0.322g \leftarrow$$

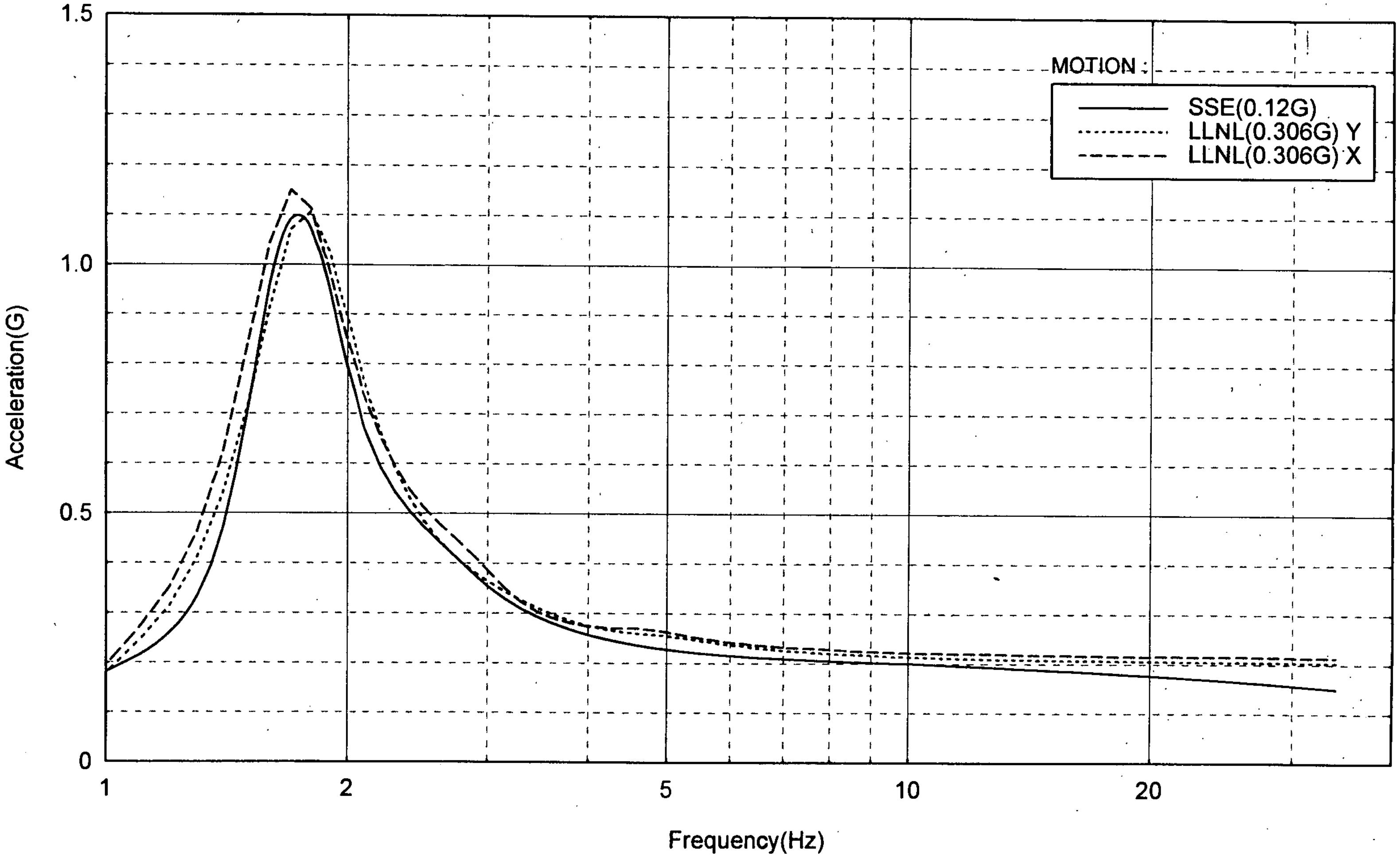
b) Two Middle Rings and lugs
Angle-to-$\frac{1}{2}$" plate weld at the Middle ring near the bottom
$$CDFM = 1.0 \times \frac{7.92}{7.13} \times 0.306g = 0.34g$$

c) Bumper Structure
Nelson Studs pull-out
$$CDFM = 1.25 \times \frac{8.84}{7.33} \times 0.306g = 0.46g$$

Thus, the median fragility of the tank is
median Fragility = 2.15 × min CDFM = 2.15 × 0.322g = 0.69G *

Wisconsin Public Service Corp.
Kewaunee Nuclear Power Plant
Amplified Floor Response Spectra

BUILDING :Auxiliary
ELEVATION :586'
DAMPING :5%



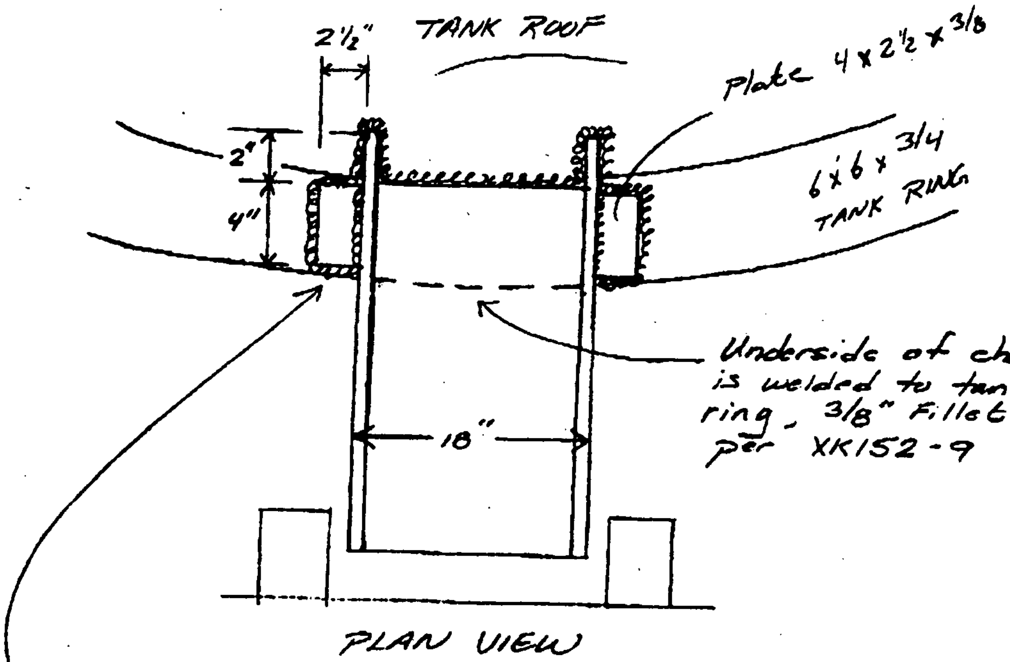
Job #91C2683 C-018
Sheet 24 of 28
By: MSLi 5/20/94
Chk: TMT 5/24/94

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Middle Rings (Lower Bound)													
3	Radius of Tank (in)		156											
5	Case # (Roark & Yo		3			4			14					
6	load, W, (kips)		0			75.5			0					
7	Moment, Mo, (kip-in)		-1,858			0			0					
8	density (kip/in^2)		0			0			0.00198					
9		Ma (kip-in)	-254			-1609.11			1879.23					
10		Ta (kip)	-8			-24.03			36.14					
11		Va (kip)	0			0.00			0.00					
13	x (degree)	x (radian)	LTm (kip-i	LTt (kip)	LTv (kip	LTm (kip-i	LTt (kip	LTv (ki	LTm (kip-i	LTt (ki	LTv (kip)			
14	0	0.0000	0	0	0	0	0	0	0	0	0			
15	5	0.0873	0	0	0	0	0	0	0	0	0			
16	10	0.1745	0	0	0	0	0	0	0	0	0			
17	15	0.2618	0	0	0	0	0	0	1	0	0			
18	20	0.3491	0	0	0	0	0	0	5	0	0			
19	25	0.4363	0	0	0	0	0	0	11	0	1			
20	30	0.5236	0	0	0	0	0	0	23	0	1			
21	35	0.6109	0	0	0	0	0	0	43	0	2			
22	40	0.6981	0	0	0	0	0	0	72	0	3			
23	45	0.7854	0	0	0	0	0	0	114	1	4			
24	50	0.8727	0	0	0	0	0	0	173	1	5			
25	55	0.9599	0	0	0	0	0	0	250	2	6			
26	60	1.0472	0	0	0	0	0	0	350	2	8			
27	65	1.1345	0	0	0	0	0	0	476	3	10			
28	70	1.2217	0	0	0	0	0	0	631	4	13			
29	75	1.3090	0	0	0	0	0	0	819	5	15			
30	80	1.3963	0	0	0	0	0	0	1,044	7	18			
31	85	1.4835	0	0	0	0	0	0	1,307	8	21			
32	90	1.5708	-1,858	0	0	0	76	0	1,613	10	24			
33	95	1.6581	-1,858	0	0	-45	75	-7	1,964	13	27			
34	100	1.7453	-1,858	0	0	-179	74	-13	2,362	15	31			
35	105	1.8326	-1,858	0	0	-401	73	-20	2,809	18	35			
36	110	1.9199	-1,858	0	0	-710	71	-26	3,307	21	38			
37	115	2.0071	-1,858	0	0	-1,104	68	-32	3,857	25	42			
38	120	2.0944	-1,858	0	0	-1,578	65	-38	4,458	29	46			
39	125	2.1817	-1,858	0	0	-2,130	62	-43	5,112	33	50			
40	130	2.2689	-1,858	0	0	-2,756	58	-49	5,818	37	54			
41	135	2.3562	-1,858	0	0	-3,450	53	-53	6,570	42	57			
42	140	2.4435	-1,858	0	0	-4,207	49	-58	7,372	47	61			
43	145	2.5307	-1,858	0	0	-5,022	43	-62	8,219	53	64			
44	150	2.6180	-1,858	0	0	-5,889	38	-65	9,107	58	67			
45	155	2.7053	-1,858	0	0	-6,800	32	-68	10,033	64	69			
46	160	2.7925	-1,858	0	0	-7,750	26	-71	10,991	70	71			
47	165	2.8798	-1,858	0	0	-6,730	20	-73	11,976	77	73			
48	170	2.9671	-1,858	0	0	-9,733	13	-74	12,983	83	75			
49	175	3.0543	-1,858	0	0	-10,751	7	-75	14,005	90	75			
50	180	3.1416	-1,858	0	0	-11,778	0	-76	15,034	96	76			

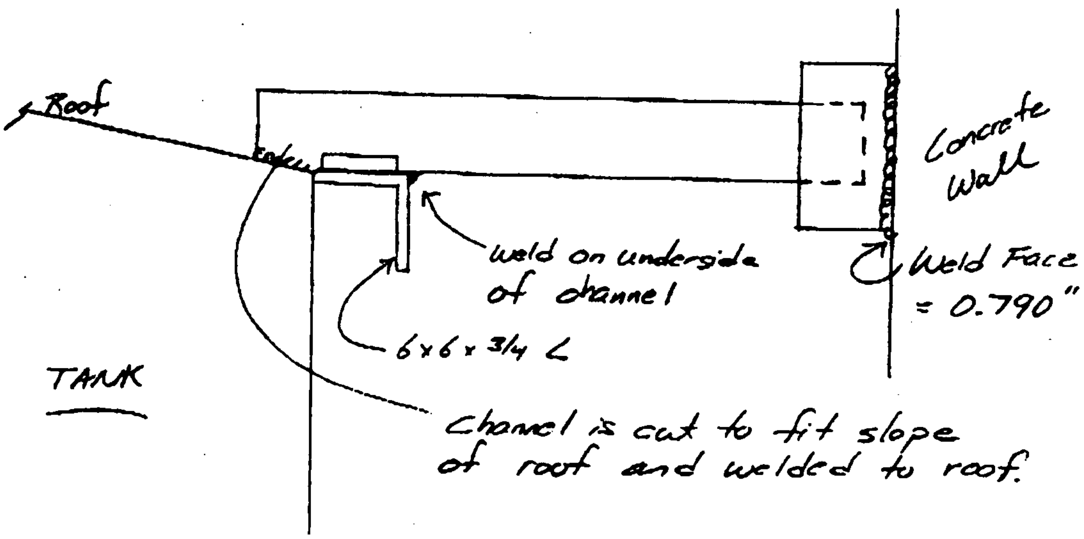
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
51												Sum of all three cases					
52	x (degree)	x (radian)	M (kip-in)	T (kip)	V (kip)	M (kip-in)	T (kip)	V (kip)	M (kip-in)	T (kip)	V (kip)	M (kip-in)	T (kip)	V (kip)	fbi (ksi)	fbo (ksi)	fv (ksi)
53	0	0.0000	-254	-8	0	-1,609	-24	0	1,879	36	0	16	5	0	0.53	-0.32	0
54	5	0.0873	-249	-8	1	-1,595	-24	2	1,858	36	-3	14	5	0	0.48	-0.23	-0.09
55	10	0.1745	-236	-7	1	-1,552	-24	4	1,794	36	-6	6	4	-1	0.34	0.03	-0.17
56	15	0.2618	-214	-7	2	-1,481	-23	6	1,689	35	-9	-6	4	-1	0.11	0.44	-0.23
57	20	0.3491	-183	-7	3	-1,383	-23	8	1,544	34	-12	-22	4	-1	-0.17	0.96	-0.27
58	25	0.4363	-143	-7	3	-1,258	-22	10	1,362	33	-15	-39	4	-1	-0.49	1.53	-0.28
59	30	0.5236	-95	-7	4	-1,107	-21	12	1,147	31	-17	-55	4	-1	-0.79	2.09	-0.25
60	35	0.6109	-40	-6	4	-931	-20	14	902	30	-19	-69	4	-1	-1.04	2.55	-0.18
61	40	0.6981	23	-6	5	-732	-18	15	632	28	-21	-77	4	0	-1.19	2.82	-0.07
62	45	0.7854	93	-5	5	-511	-17	17	342	26	-22	-76	4	0	-1.18	2.79	0.10
63	50	0.8727	169	-5	6	-270	-15	18	38	24	-23	-63	4	1	-0.94	2.36	0.33
64	55	0.9599	251	-4	6	-10	-14	20	-275	22	-23	-35	4	3	-0.41	1.40	0.61
65	60	1.0472	338	-4	7	265	-12	21	-590	20	-23	13	5	4	0.47	-0.22	0.96
66	65	1.1345	429	-3	7	556	-10	22	-900	18	-22	85	5	6	1.79	-2.62	1.37
67	70	1.2217	524	-3	7	858	-8	23	-1,199	16	-21	183	6	8	3.61	-5.94	1.85
68	75	1.3090	623	-2	7	1,170	-6	23	-1,480	15	-20	312	6	11	6.00	-10.31	2.39
69	80	1.3963	724	-1	7	1,489	-4	24	-1,736	13	-18	477	7	13	9.03	-15.85	2.98
70	85	1.4835	826	-1	8	1,813	-2	24	-1,960	12	-15	679	9	16	12.78	-22.68	3.64
71	90	1.5708	-929	0	8	2,140	76	24	-2,145	10	-12	-934	86	20	-12.54	36.24	4.35
72	95	1.6581	-826	1	8	2,422	77	17	-2,286	9	-9	-690	87	16	-8.02	27.99	3.64
73	100	1.7453	-724	1	7	2,612	79	11	-2,375	9	-5	-487	89	13	-4.27	21.14	2.99
74	105	1.8326	-623	2	7	2,709	79	4	-2,408	9	0	-322	90	11	-1.23	15.59	2.40
75	110	1.9199	-524	3	7	2,712	79	-3	-2,379	9	5	-192	91	8	1.18	11.20	1.86
76	115	2.0071	-429	3	7	2,621	79	-10	-2,264	9	10	-93	91	6	3.01	7.64	1.39
77	120	2.0944	-338	4	7	2,437	77	-17	-2,119	11	15	-20	92	4	4.35	5.40	0.98
78	125	2.1817	-251	4	6	2,160	76	-24	-1,880	12	20	29	92	3	5.27	3.74	0.64
79	130	2.2689	-169	5	6	1,704	73	-30	-1,566	14	26	59	92	2	5.82	2.72	0.35
80	135	2.3562	-93	5	5	1,341	70	-36	-1,175	17	32	74	92	1	6.09	2.23	0.13
81	140	2.4435	-23	6	5	805	67	-42	-705	20	37	77	92	0	6.14	2.14	-0.04
82	145	2.5307	40	6	4	189	63	-48	-158	23	43	71	92	-1	6.03	2.34	-0.15
83	150	2.6180	95	7	4	-502	59	-53	466	27	49	59	92	-1	5.82	2.73	-0.22
84	155	2.7053	143	7	3	-1,263	54	-58	1,165	32	54	45	92	-1	5.55	3.21	-0.24
85	160	2.7925	183	7	3	-2,087	48	-63	1,935	36	59	30	92	-1	5.29	3.70	-0.23
86	165	2.8798	214	7	2	-2,968	43	-67	2,772	42	64	17	92	-1	5.05	4.13	-0.19
87	170	2.9671	236	7	1	-3,901	37	-70	3,673	48	68	8	92	-1	4.87	4.46	-0.12
88	175	3.0543	249	8	1	-4,877	31	-73	4,630	54	72	3	92	0	4.77	4.64	-0.04
89	180	3.1416	254	8	0	-5,889	24	-76	5,638	60	76	3	92	0	4.77	4.64	0.04

12-2-94 G. RIDDER

BRACE SUPPORT AT TOP OF RWST - 4 TOTAL



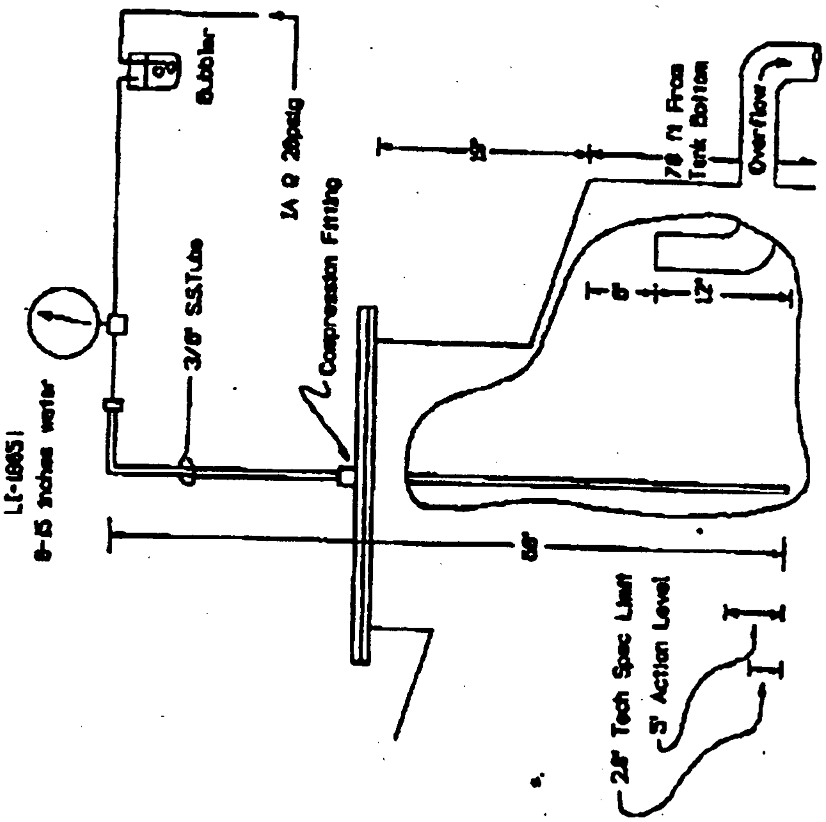
Weld face measures between 0.45" to 0.55", so a 3/8" Fillet weld seems reasonable as indicated on dwg XK152-9.



Stevenson & Associates a structural-mechanical consulting engineering firm	JOB NO. 91C2683 SUBJECT: Kewaunee A46/IPEEE CALCULATION NO. C-018	SHEET 27 of 28 Revision 0
	External Analysis and Outlier Resolution of Tanks and Heat Exchangers	By MLF 5/20/94 Chk. TMT 5/24/94

Operator Aid No. 89-1   Approved By: B 2-4-94

RWST LOCAL LEVEL INDICATOR



∴ Tank level @ 69'-6"
never below 68'-11" (Action level)

DESCRIPTION OF ANALYSIS: Eigenvalue Solution of Frequencies and Mode Shapes, Static Elastic Stress and Displacement Analysis of Kewaunee RWST

COMPUTER CODE: COSMOS/M VERSION: 1.61

RELEASE DATE: August 1990 AUTHOR/VENDOR: SRAC

COMPUTER TYPE/SYSTEM: IBM Compatible

PROGRAM STATUS: ☐ Project Specific ☒ General Use/QA Approved

VERIFICATION/VALIDATION DOCUMENTATION: ☐ Attached ☒ On File

RUN NUMBER:

	ORIGINATOR	DATE	CHECKER	DATE
INPUT REPRODUCED ON LISTING	MSLi	5/20/94	TMT	5/24/94
MODEL VALID AND ASSUMPTIONS DOCUMENTED	MSLi	5/20/94	TMT	5/24/94
PROGRAM APPROPRIATE AND ADEQUATE	MSLi	5/20/94	TMT	5/24/94
MODEL BEHAVES REASONABLE	MSLi	5/20/94	TMT	5/24/94
RESULTS PROPERLY INTERPRETED	MSLi	5/20/94	TMT	5/24/94

REMARKS:

S&A	COMPUTER PROGRAM COVER SHEET FIGURE 2.8	CONTRACT NO. 91C2683

S&A Stevenson and Associates — A Structural-Mechanical Consulting Engineering Firm	CLIENT WPSC JOB No. 91C2683 SHEET A1 OF A1
	SUBJECT Kewaunee A46/IPEEE
	CDFM Value of RWST, 153-021, in terms of SSE Ground Spectrum
	REVISIONS: 0 MSL 6/1/94 TMT 6/9/94

Objective: Compute CDFM Value of RWST in terms of SSE ground spectrum

Analysis:

a) Top Ring and lugs (Ref. pages 10, 12 & 13)

Max. Saf (g) from 5% damping SSE at f = 5.05 Hz (Ref. page 23)

$S_{af} = 0.227\,g$

Therefore, in Channel-to-Ring Weld

$F = 0.227 \times 918 \times \frac{1}{2} = 104$ kips

$T = 104 \times (19 + 2.39) = 2224.6$ kip-in

$f_x' = \frac{104}{16.43} = 6.33$ ksi $\qquad f_y' = 0$

$f_x'' = \frac{2224.6 \times 3.61}{1058.38} = 7.59$ ksi

$f_y'' = \frac{2224.6 \times 11.5}{1058.38} = 24.17$ ksi

$f = \sqrt{(f_x' + f_x'')^2 + (f_y' + f_y'')^2} = \sqrt{(6.33 + 7.59)^2 + 24.17^2} = 27.9$ ksi

$CDFM = 1.0 \times \frac{33.6}{27.9} \times 0.128 \approx 0.15\,g$

b) Two middle rings and lugs (Ref. pages 11, 14 to 17, 24, 25)

Max. Saf (g) from 5% damping SSE at f = 3.36 Hz (Ref. page 23)

$S_{af} = 0.303\,g$

In Angle-to-5/8" plate weld stress at the middle ring near the bottom, the seismic inertia contributes over 90% and the stress is over 30% less if one considers the external moment acting on the ring distributed over the width of channel, 18 inches.

Therefore, one can estimate

$CDFM = 1.0 \times \frac{7.92}{0.1 \times 7.13 + 0.9 \times 7.13 \times 0.303/0.315 \times 0.7} \times 0.129 = 0.19\,g$

Thus, the CDFM of RWST in terms of SSE ground spectrum is

$CDFM = 0.15\,g$ *

Stevenson and Associates
A Structural-Mechanical Consulting Engineering Firm

CLIENT WPSC JOB No. 91C2683 SHEET B1 OF B1

SUBJECT Kewaunee A46/IPEEE
153-021
Investigation of possibility of fracture at tank shell near base due to Shell buckling.

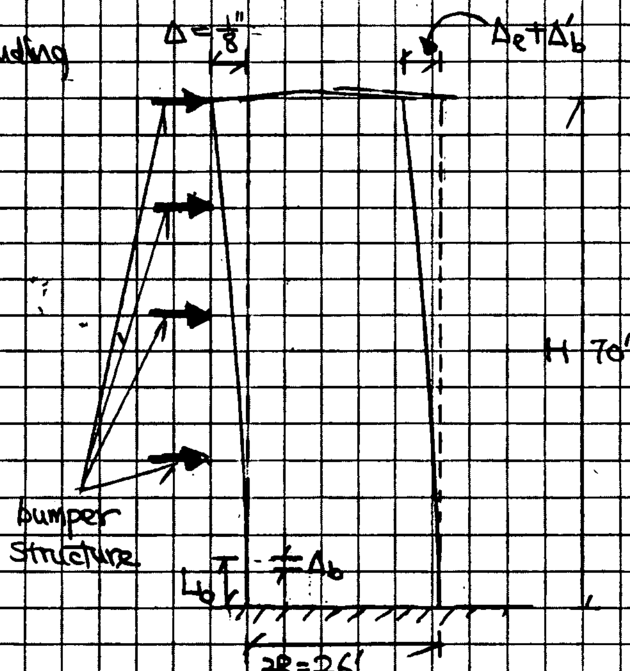
REVISIONS: 1 MSL 12/12/94; WD 12/12/94

As shown in page 8; for a free standing tank,

displacement at top with 1 g
$d = 0.2632'$

and moment at base
$M = 82830$ kip-ft

With nominal moment capacity of tank $= 2691$ kip-ft



$\Delta_e = \frac{2691}{82830} \times 0.2632 \times 12 = 0.103''$

$\Delta_b' = \frac{1}{8}'' - \Delta_e = 0.022''$

$\Delta_b \simeq \frac{2R}{H} \Delta_b' = \frac{26}{70} \times 0.022''$
$= 0.0082''$

Δ = spacing between tank and bumper structure

Δ_e = elastic deformation at top before tank shell buckles at base

Δ_b' = shorting at top after tank shell buckles at base

Δ_b = shorting at base shell after tank shell buckles at base

L_b = tank shell buckling length at base

Assuming $L_b = 18''$ ($L_b \sim 1'$ to $2'$ in general)

$\varepsilon_b = \frac{\Delta_b}{L_b} = \frac{0.0082}{18} = 4.5 \times 10^{-4}$

$\sigma_b = E_s \varepsilon_b = 30 \times 10^3 \times 4.5 \times 10^{-4}$
$= 13.5$ ksi *

Tank shell buckles when stress at tank shell near base reaches 7.92 ksi (page 6). Additional stress at tank shell near base due to shortening is 13.5 ksi. Therefore, the total stress at tank shell near base is

$\sigma_t = \sigma_c + \sigma_b = 7.92 + 13.5 = 21.42$ ksi $< F_y = 30$ ksi (A240-304 Steel)

As stated in page 21, additional horizontal seismic force will be transferred through the rings, lugs and bumper to the support structure when the tank and the bumper structure begin to contact. No additional force will be generated at the compression side of tank. With $\sigma_t = 22$ ksi, the tank shell will hold its integrity and will not fracture.

ATTACHMENT C

Letter from C.R. Steinhardt (WPSC)

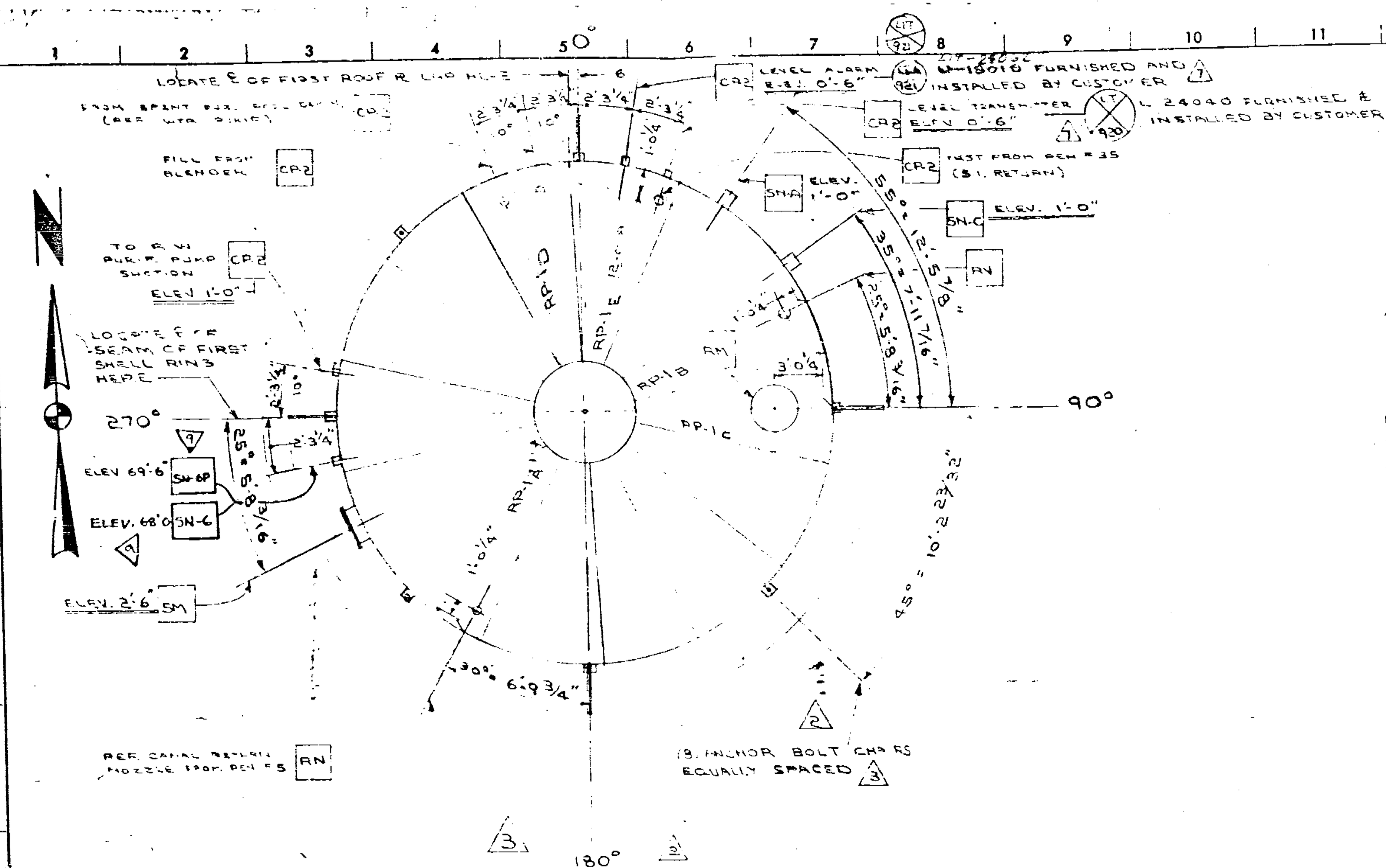
To

Document Control Desk (NRC)

Dated

August 15, 1997

Drawing XK-152-1, Rev. 9
and
Drawing XK-152-9, Rev. 5



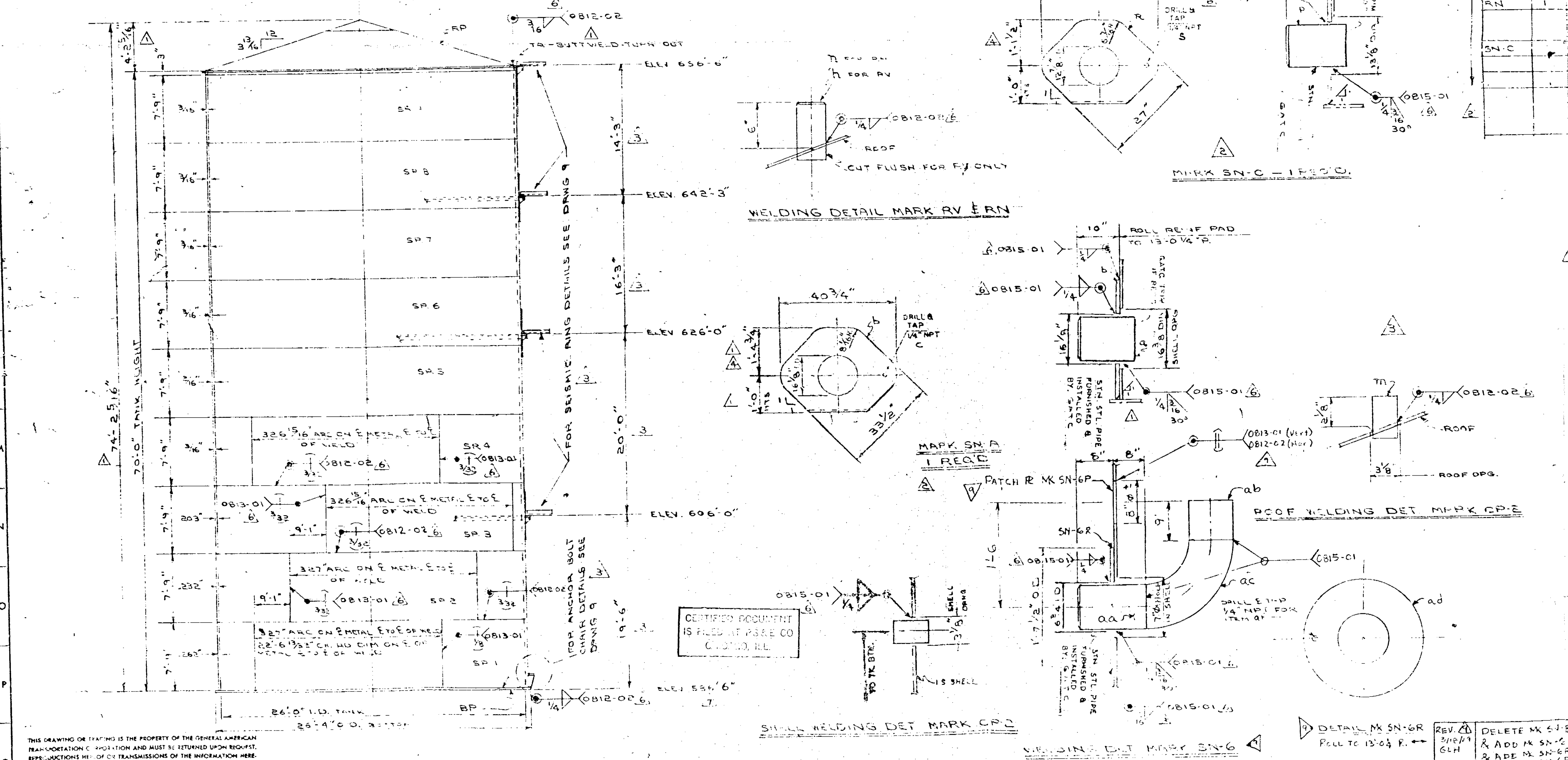
DRAWING & FITTING LIST FOR ONE TANK SUPPLIED & ATTACHED BY GAT.C.

MARK	ITEM NO	NO REQ'D	DESCRIPTION	DWG NO	NOZ EXT OUTSIDE TANK	NOZ EXT INSIDE TANK	ELEV	REMARKS
RP		1	LAP-WELDED SELF SUPP'T CONE ROOF	2				
BP		1	LAP-WELDED FLAT BOTTOM	3				
RM		1	20" ROOF MANWAY	4	SEE DRWG			
SM		1	20" SHELL MANWAY	5	SEE DRWG			
NP-1		1	NAME PLATE	6				
NP-2		1	BASE PLATE FOR NAME P.	7				
			SEISMIC RINGS & ANCHOR CHAIRS	9				
			FIELD NOTES	8 (7 PG)				
			ERECTION PROCEDURE	EP-1 (2 PG)				
			VACUUM TEST PROCEDURE	VT-1 (2 PG)				
			RADIOGRAPHIC TEST PROCEDURE	RT-1 (12 PG)				
			SHELL X-RAY LAYOUT	DC-8297XR				
			BOTTOM, SHELL & ROOF HEAT NO'S	DC-8297HI				

BILL OF MATERIAL FOR ONE UNIT

SHIP LIST MARK	NO. REQ'D	ITEM NO.	NO. PER UNIT	DESCRIPTION	SIZE	MATERIAL	TOTAL WEIGHT	REMARKS
			1	SHELL				
SR-1	3			PLATES	.262 x 93 x 327	A240-304	6861	CUT & SQ. ±1/16
SR-2	3				.232 x 93 x 327		6078	
SR-3	3				.203 x 93 x 325 15/16		5316	
SR-4 THRU 9	18				3/16 x 93 x 326 15/16		29456	
TA	1			ANGLE	3 x 3 x 3/8 x 82'-0 2EA/PC	STN STL 304	1214	
			1	16" SHELL OUTLET NOZZLE				
SN-A	1	a		PIPE	16 SCH #40S SMLS x 0'-11	A312 304	124	B.O.E
	1	b		PLATE	.262 x 34 x 34	A240 304	172	
		c		PIPE PLUG	1/4 SQ HD.	STN STL	–	
SN-6a	1			6" SHELL OVERFLOW NOZZLE				
		aa	1	PIPE	6 SCH 40S SMLS x 1'-4	A312-304	29	B.B.E.
		ab	1	PIPE	6 SCH 40S SMLS x 0'-9	A312-304	15	B.O.E.
		ac	1	90° WELDELL	6 SCH 40S LONG RAD	A403-304	24	
SN-6R	1	ad		PLATE	1/4 x 19 1/2 O.D. x 6 3/4 I.D.	A240 304	27	
		af		PIPE PLUG	1/4 SQ HD	STN STL	–	
SN-6P	1			PLATE	3/16 x 8" DIA	A240 304	3	
			1	3" ROOF VENT NOZZLE				
RV		h		PIPE	3 SCH #40S SMLS x 0-7 1/2	A312 304	5	B.O.E
				NOT REQ'D.				
			5	ROOF & SHELL SOCKET-WELD CPLGS.				
CP-2	5	m		COUPLINGS	2 3000# SOCKET-WELD	A182 GR F 304	12	
			1	REFUELING CANAL RETURN NOZZLE - 3"				
RN	1	n	1	PIPE	3 SCH #40 S. SMLS x 0-3	A312 304		B.O.E.
			1	12" SHELL OUTLET NOZZLE				
SN-C		P		PIPE	12 SCH #40S SMLS x 0'-10	A312 304	48	B.O.E
		R		PLATE	.262 x 27 1/2 x 27 1/2	A240 304	57	
		S		PLUG	1/4 SQ. HD.	STN STL	–	
				ALL MATERIAL — PICKLED & ANNEALED PLATE FINISH				
				PHYSICAL & CHEMICAL TEST REPORTS REQ'D ON ALL MAT'L				

New MAT'L



NOTE:
VERTICAL AND HORIZONTAL SHELL SEAMS WELDED WITH FULL PENETRATION AND FUSION. — PLATES AND TOP ANGLE TO BE FORMED TO TANK RADIUS.

SHOP NOTE FOR WELD MARKING
ALL WELDERS SHALL BE ASSIGNED AN I.D. NUMBER AND ALL WELDS MADE BY EACH WELDER SHALL BE IDENTIFIED BY HIS NUMBER.

FIELD NOTES:
ELEVATIONS ARE GIVEN FROM TOP OF TANK BOTTOM TO ℄ OF FITTING. ARC DIMENSIONS ARE GIVEN ON OUTSIDE OF SHELL. NOZZLE EXTENSIONS ARE GIVEN FROM THE OUTSIDE SURFACE OF THE SHELL OR ROOF ON ℄ OF NOZZLE TO FACE OF NOZZLE.

Docket No: 50-305
Rec'd w/ ltr dtd 8/15/97
Accession: 9708190228

ANSTEC APERTURE CARD

ERECTION DIAGRAM
TO BE ERECTED COMPLETE BY GATC. FOR
WISCONSIN PUBLIC SERVICE CORP.
LOCATION ERECTED: KEWAUNEE, WISC.
CUSTOMERS ORDER NO K-152
TANK SIZE: 26'-0" I.D. x 70'-0" HI. SELF SUPP'T CONE ROOF
CONTRACT NO C-8297
NO REQ'D 1

REV.	
REV. 8, DATE 2-25-69, BY KOL, CHK'D GFS	ADD DWG'S # XR & HI TO DWG LIST
REV. 7, BY GPS, CHK'D GLH	ELEV. 586'-6" WAS 586'-0", ADDED INSTRUMENT NOS TO LEVEL ALARM & LEVEL TRANSMITTER AT A-7, ADD DWGS VT-1, EP-1, RT-1
REV. 6, DATE 2-4-69, BY GFS, CHK'D KOL	ADDED WELD PROCEDURE NOS. TO WELD SYMBOLS
REV. 5, DATE 1-29-69, BY GPS, CHK'D GLH	ADDED "SHOP NOTE FOR WELD MARKING"
REV. 9, 3/18/69, GLH	DELETE MK SN-B & ADD MK SN-6 & ADD MK SN-6R & ADD MK SN-6P

GENERAL AMERICAN TRANSPORTATION CORPORATION
CHICAGO
DRAWN GPS
CHECKED KOL 11-21-68
DWG. NO. CONTR. C-8297 - DWG 1

4/12/72 K-152-1 (9)

9708190032-01