Lake Whatcom Water and Sewer District
Reservoir Seismic Vulnerability Assessment
Technical Report

December 2016

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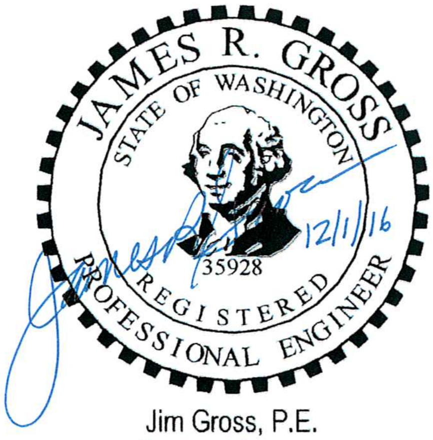

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1. Executive Summary

A structural analysis was performed on five District water storage reservoirs to determine their sufficiency to withstand existing earthquake code requirements. The shells of all five tanks except the Division 7 and 22 tanks were found to be adequate; however, the foundations and/or anchorage were inadequate in all five tanks. The Division 7 Reservoir is the largest in the system, has the most serious deficiencies, and would have the worst adverse impact if removed from service by an earthquake. It is recommended as the highest priority for retrofit. The recommended priority for further investigation of retrofit options are:

- Division 7 Reservoir

  A supplemental, external ringwall is the recommended retrofit option at an estimated approximate project cost of $721,000. Project costs include general conditions (10%), sales tax (8.7%), contingency (20%), and engineering, permitting, legal and admin (15%). This retrofit also includes supplemental shell plates to resolve issues with overstress.

- SVWTP Reservoir

  An attached, below ground ringwall addition to the existing ringwall foundation is the recommended retrofit option at an estimated approximate project cost of $156,000.

- Division 22 Reservoir

  The addition of an external gravity ringwall collar, is the least expensive and recommended retrofit option at an approximate estimated project cost of $367,000. This retrofit also includes a small amount of supplemental shell plate to resolve issues with overstress.

- Geneva Reservoir

  An anchored external ringwall is the least expensive and intrusive retrofit alternative, and is the recommended retrofit approach for the Geneva Reservoir at an estimated approximate project cost of $505,000.

- Division 30 Reservoir

  The recommended retrofit option for this reservoir is an anchored supplemental ringwall. Although a gravity collar may appear less expensive at first glance, the unit price for concrete could be substantially higher than assumed generally due to the remoteness and elevation of the site. A gravity collar would also involve very poor shell manway access. The estimated approximate project cost for this retrofit option is $541,000.

2. Introduction

This report is prepared pursuant to a contract between the Lake Whatcom Water and Sewer District and BHC Consultants LLC dated November 30, 2015. The purpose of the contract is to obtain a seismic and structural evaluation of five existing water storage reservoirs within the District boundaries and provide a report discussing the planning level opinion of probability and consequence of failure, specific structural deficiencies, and estimated costs and methods to retrofit these structures to bring them to current standards.

The five welded steel, ground storage reservoirs which are the subject of this report were constructed in the 1970's and 1990's. Their names, dimensions, and maximum capacities are provided in Table 1.

Table 1 – Reservoir Data

Reservoir Name	Nominal Capacity (gal)	Maximum Capacity (gal)*	Year Constructed	Diameter (ft.)	Height of Shell (ft.)
Geneva	500,000	519,206	1979	53'-0"	32'-8"
Division 22	500,000	520,088	1971	50'-0"	35'-0"
Division 7	1,000,000	997,939	1971	70'-0"	35'-0"
Division 30	150,000	151,390	1973	25'-5"	40'-4 ½"
Sudden Valley Water Treatment Plant (SVWTP)**	235,000	225,591	1992	40'-0"	25'-0"

Notes:
- * Maximum capacity is the gross storage volume with the tank filled to the overflow level, with no reductions for internal piping or appurtenances.
- ** The Sudden Valley WTP reservoir also functions as a chlorine contact tank and has an internal baffle system. The nominal capacity of the tank is per the shop drawings.

The evaluation did not include tank roofs or vents, corrosion or coatings, or geotechnical evaluation of site stability.

3. Summary of Observations

BHC visited each tank site on September 1, 2015 and again on December 15, 2015, when the tanks were examined and certain dimensional measurements made. In addition, BHC reviewed available District record information for the tanks, which included limited design or shop drawings, soils reports, and external and underwater inspections. Tank nameplate data or record drawings indicate that the welded steel ground storage tanks were designed in accordance with earlier editions of AWWA D100 *Welded Carbon Steel Tanks for Water Storage.*

The District obtained estimated thickness measurements for ringwall thickness at Reservoirs 7, 22, and 30 using both ground penetrating radar (GPR) and an Olsen concrete thickness gauge (CTG). These tests were performed on January 7, 2016 by Geotest of Bellingham, WA and are described in their report dated January 13, 2016, which is attached as Appendix A.2. Unlike the Geneva and SVWTP Reservoirs, these three reservoirs had no surviving records related to ringwall foundation depth or thickness.

The District excavated near the above ringwalls on December 15, 2015 and January 7, 2016, at which time depth measurements were made at three locations on the perimeter of each tank.

The condition of interior and exterior coatings was not evaluated. Visually, conditions appeared consistent with tank inspection reports prepared in 2012 by H2O Solutions.

4. Summary of Analysis Methodology

Each reservoir was analyzed for conformance to AWWA Standard D100-11, *Welded Carbon Steel Tanks for Water Storage*, supplemented by requirements of the 2012 International Building Code and ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*. Only seismic load combinations were considered, but partial snow mass was included with the roof weight when required by code. Wind and roof live load combinations were ignored.

Analysis was limited to shell, anchorage and foundation elements. Roof framing evaluation was not included, since it does not perform a significant role in lateral resistance to seismic loads. The weights of appurtenances and floor or roof plate overlaps were ignored, except for the weight of internal baffles on the SVWTP Reservoir.

The assumed ground motion applicable for all tanks was the Maximum Considered Earthquake (MCE) which is a maximum ground motion considered to have a risk of occurrence not greater than 2 percent in 50 years (a “2,500 year” earthquake). Ground motions were derived using latitude and longitude for each tank and interpolation software available on-line from the U.S. Geological Service. It should be stressed that the MCE is a “risk adjusted” value and not necessarily the worst possible earthquake that might be expected at less frequent intervals. The MCE is the worst case earthquake considered by the building codes. Design meeting code requirements does not mean there will be no damage, but that an acceptable level of performance will be achieved for the risk category assumed.

All the reservoirs are used for fire protection and are classified in Risk Category IV in the Building Code and as Category III in AWWA D100. These are equivalent categories and refer to essential facilities. The addition of the new Division 22 Reservoir would not change the classifications of the existing reservoirs.

Ground motions were adjusted for soil type using factors in the Building Code. Site Class B has been assumed for the Division 30 reservoir, based on rock encountered during the test pit excavation to expose the ringwall. The Division 22 Reservoir site, where recent soil investigations for a future tank are available, is assumed to be Site Class C. All foundation soils for the other three reservoirs are assumed to be Site Class D.

Analysis methodology in AWWA D100 is based on an assumption of “rigid” shells and an open surface at the top of the tank, in other words, no contact with the roof by sloshing waves induced by earthquake ground motions. When sloshing involves roof contact, the horizontal forces on the tank are magnified and result in increased forces on the tank superstructure and foundation. To account for this effect, methodology in the literature was used to adjust the apparent seismic mass. Reference details are provided in the calculations attached in Appendix B of this report.

Forces computed for design purposes by AWWA D100 methods adjust the predicted forces downward to account for some ductility and deformation in the tank and what is considered an acceptable amount of damage short of failure. Seismic forces due to impulsive mass (structure weight and most of the water mass) are divided by the factor R_w which is 2.5 for unanchored tanks and 3.0 for anchored tanks. Convective loads associated with convective mass (sloshing portion of the contents) are divided by a factor R_i which is 1.5 for both anchored and unanchored tanks. Vertical acceleration concurrent with horizontal ground motion is included.

Unanchored tanks were checked for stability, and anchored tanks were checked for stability in case of anchorage failure. Anchored tanks were checked for uplift of the foundation and for overturning stability about a pivot point at the toe of the shell.

Finally, because the SVWTP tank has internal baffles, the effect of ground motions parallel to the baffles is not the same as for ground motions perpendicular to the baffles. The behavior in the first case would be similar to an un-baffled tank. For ground motions perpendicular to the baffles, the sloshing would be reduced, resulting in less of the water mass counted as convective and more as impulsive, increasing the base shear and overturning moment. The mass of the baffles and the mass of an equivalent volume of displaced water was included in the analysis as an approximation for these effects. However, determining their full effect on the relative amount of impulsive water mass is beyond the scope of this evaluation, and would require a much more complicated analysis.

Table 2 is a summary of analysis assumptions.

Table 2 – Analysis Assumption Summary

	Geneva Reservoir	Division 22 Reservoir	Division 7 Reservoir	Division 30 Reservoir	SVWTP Reservoir
Physical Data Summary					
Diameter, D	53'-0"	50'-0"	70—0"	25'-5"	40'-0"
Shell height, H_s	32'-8"	35'0"	35'-0"	40'4.5"	25"
Roof type	Cone with rafters and center column	Cone with rafters and center column	Cone with rafters and center column	Simply supported dome	Cone with rafters and center column
Roof pitch (varies, number shown used for analysis	1:12	1:12	1:12	N/A	1:12
Ringwall height	36" (record)	40"	40"	40"	72" (record)
Ringwall width	18" (record)	28"	30"	18" min	18" (record)
Anchors (approximately equal spaces where provided)	12 each strap type	None	None	12 each strap type	13 each anchor bolt and chair type
Floor elevation (per District)	662.0 ft.	800.0 ft.	669.0 ft.	1025.5 ft.	344.5 ft.
Maximum operating depth, H (per District)	31.5 ft.	33.5 ft.	33.5 ft.	39.3 ft.	22.0 ft.
Latitude, degrees (Google Earth)	48.7392	48.7272	48.7111	48.7028	48.7169
Longitude, degrees (Google Earth)	-122.4056	-122.3556	-122.3189	-122.3333	-122.3172
Ground elevation (Google Earth)	661 ft.	805 ft.	673 ft.	1030 ft.	335 ft.
Ground snow load, p_g (from greater of Google elevation or District floor elevation times .075 coefficient from SEAW Snow Load Analysis for Washington, 2nd ed.)	50 psf	60 psf	50 psf	77 psf	26 psf
Site Class	D	C	D	B	D

Table 2 – Analysis Assumption Summary					
	Geneva Reservoir	Division 22 Reservoir	Division 7 Reservoir	Division 30 Reservoir	SVWTP Reservoir
IBC/ASCE Analysis Parameters					
S_S, 0.2 second spectral acceleration at MCE_R, normalized for Site Class B, 5% damping. (Source USGS)	.948g	.943g	.940g	.944g	.939g
S1, 1 second spectral acceleration at MCE_R, normalized for Site Class B, 5% damping (Source USGS)	.371g	.368g	.367g	.369g	.366g
Site Coefficient F_a (from 2012 IBC and ASCE 7-10)	1.12	1.02	1.12	1.00	1.12
Site Coefficient F_v (from 2012 IBC and ASCE 7-10)	1.66	1.43	1.67	1.00	1.67
SM_S (S_S x F_a)	1.062g	.962g	1.053g	.944g	1.052g
SM_1 (S_1 x F_v)	.616g	.526g	.613g	.369g	.611g
S_{DS} (2/3 x S_{MS})	.708g	.641g	.702g	.629g	.701g
S_{D1} (2/3 x S_{M1})	.411g	.351g	.409g	.246g	.407
Seismic Design Category (ASCE 7-10)	D	D	D	D	D
Risk Category (2012 IBC and ASCE 7-10)	IV				
Snow load importance factor I_S	1.20				
Seismic importance factor I_E	1.50				
AWWA Analysis Parameters					
Material Class	2	1			2
Alternative Design Basis Applicable (Chapter 14, AWWA D100-11 for higher strength steel)	No				
Minimum Design Roof Snow Load	25 psf				

Table 2 – Analysis Assumption Summary					
	Geneva Reservoir	Division 22 Reservoir	Division 7 Reservoir	Division 30 Reservoir	SVWTP Reservoir
Minimum Roof Live Load	15 psf				
Seismic Use Group	III				
Seismic Importance Factor	1.5				
R_i (response modification factor for impulsive loads)	3.0	2.5	2.5	3.0	3.0
R_c (response modification factor for convective loads)	1.5				
Transition period for longer period ground motion, T_L (mapped)	16 sec				
Minimum required freeboard as a fraction of computed sloshing wave amplitude, d	1.0				
Other Analysis Assumptions					
Year of construction	1979	1971	1971	1973	1992
Foundation concrete 28 day compressive strength, f'_c	3000 psi (record)	Assume default value of 3000 psi			
Foundation reinforcement F_y, ksi	Assume default value of 60 ksi				Grade 60 (record)
Allowable foundation soil pressure, static. Increase by 1/3 for seismic loads	2500 psf (original soils report)	4000 psf (soils report for proposed second tank)	Use 2500 psf default value for Class D site class based on comparison to Geneva site	10,000 psf	Use 2500 psf default value for Class D site class based on comparison to Geneva site

5. Summary of Findings – Structural

5.1 Geneva Reservoir

5.1.1 Record Information

The Geneva Reservoir was constructed by Reliable Steel Fabricators (no longer in business) of Olympia, WA in 1979. Original design and shop drawings were provided by the District, along with a December 13, 2012 investigation report by Wilson Engineering of Bellingham, WA and a cleaning and inspection report and video by H2O Solutions dated July 9, 2012. In addition, a soils report by Anvil Corporation dated March 1979 was available. Design drawings and specifications dated May 1979 by Yoshida, Inc. of Seattle, WA were available, as well as shop drawings by Reliable Steel dated May 24, 1979 (see Figures 1 and 2). The shop drawings indicate design in accordance with AWWA D100-84, Seismic Zone 3.

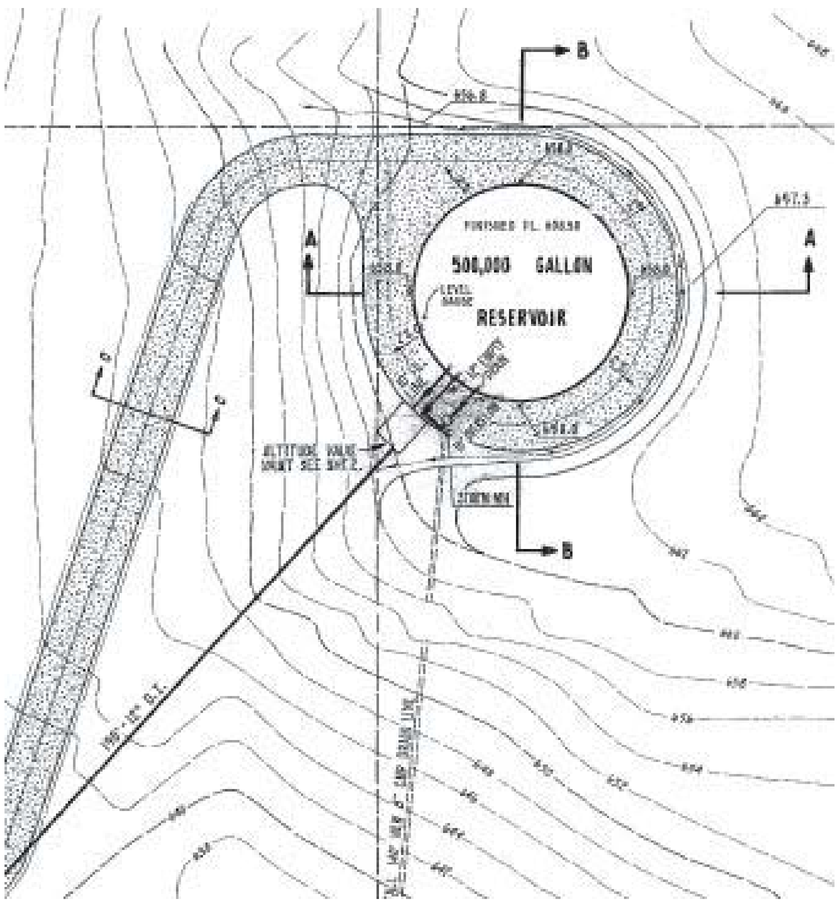


Figure 1 Site Plan from Original Design Drawings

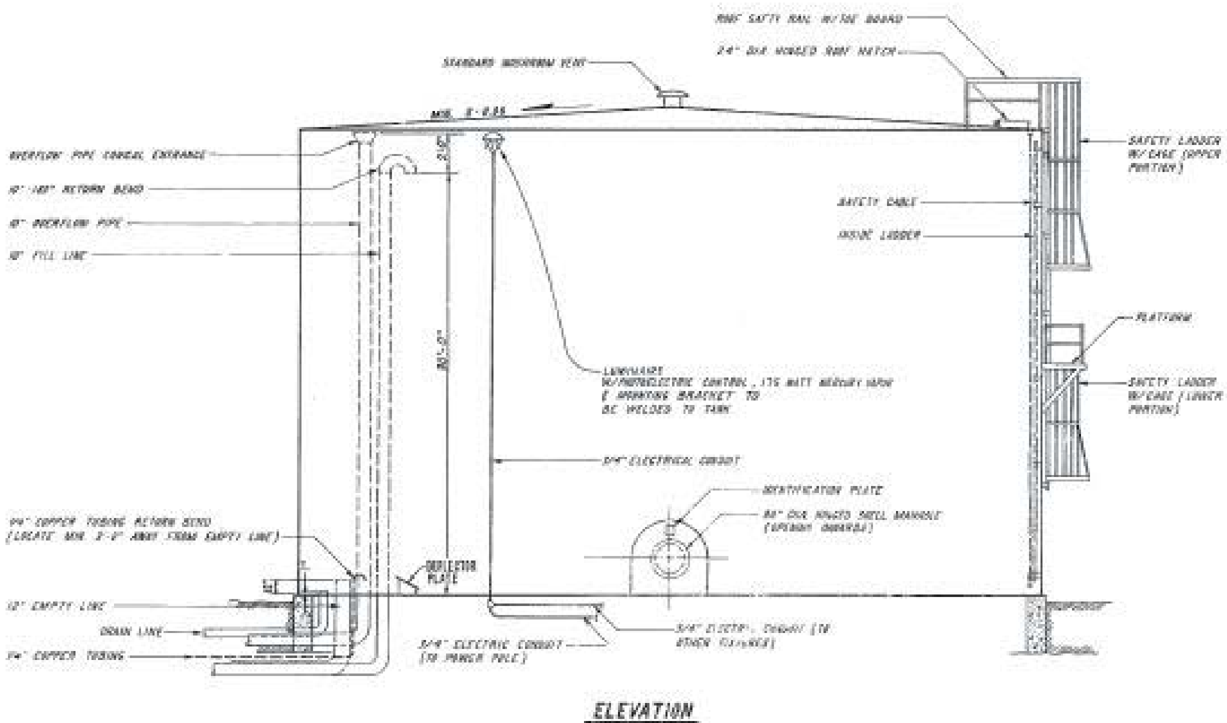


Figure 2 Elevation View from Original Design Drawings

The reservoir has a mildly sloped cone roof, supported by 27 channel-shaped rafters which span from the shell to a steel center column. The Wilson report noted a few bolts were missing at rafter connections, but the missing bolts did not appear to be critical. A site location map for the Geneva Reservoir is provided in Figure 3.

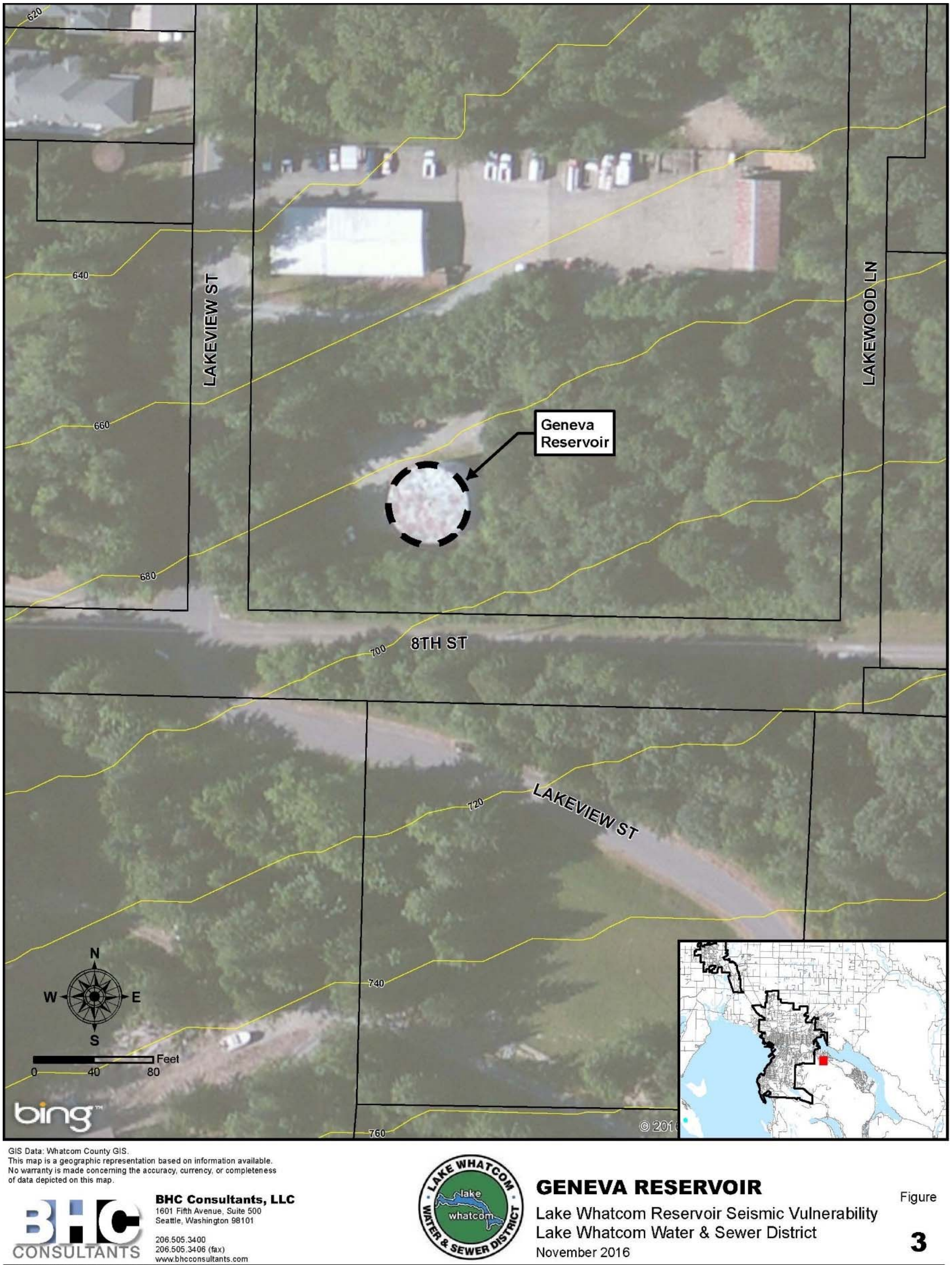


Figure 3 Geneva Reservoir

5.1.2 BHC Field Observations

General condition, appurtenances, and site conditions appeared consistent with record information. The tank has a single 30 inch diameter shell manhole, and a 2 feet square roof hatch with partial roof railing. The roof is accessed by caged exterior ladder and an un-caged interior ladder (see Figure 4).

Figure 4 Geneva Reservoir, September 1, 2015

Water level at the time of examination on July 15, 2015 was 31.3 feet. BHC measured the tank diameter and height, and the height and metal thickness of shell courses, floor, and roof plates. Metal thickness for the shell and roof was measured using a Cygnus 6 Plus Ultrasonic Thickness Gauge. Other dimensions were measured using a steel tape. Record metal thicknesses and measurements are shown in Table 3 below. For analysis, thicknesses were rounded to the nearest 1/32 inch.

Table 3 – Metal Thicknesses – Geneva Reservoir							
Item	Distance from Top of Floor Plate to Top of Shell Course (ft)		Metal Thickness (in)				
	Record	Measured By Tape	Record	Measured By Tape	Measured Using UT Gauge	Average	Used for Analysis
Roof Plate	N/A		3/16	N/A	0.120, 0.120. 0.120	0.120	3/16
Shell Course 4 (highest)	32.67	32.67	1/4	N/A	0.245,0.245	0.245	1/4
Shell Course 3	24.52	24.52	1/4	N/A	0.230, 0.230	0.230	1/4
Shell Course 2	16.34	16.34	9/32	N/A	0.265,0.265	0.265	9/32
Shell Course 1 (lowest)	8.17	8.17	11/32	N/A	0.35, 0.345,0.345	0.35	11/32
Floor Plate	N/A		1/4	1/4	N/A	1/4	1/4

The measured diameter of the tank is 52 feet, and the shell height is 32 feet 8 inches. The overflow elevation (record) is 32 feet above the floor, for a top capacity of 519,206 gallons compared to a nominal capacity of 500,000 gallons. The tank is held down by 12 steel plate anchors embedded in a concrete ringwall foundation. The ringwall record dimensions are 18 inches wide by 36 inches high. The observed configuration and spacing of the anchors was consistent with the record drawings. Grade was approximately 7 inches below the top of the ringwall. Photos from the site visit are shown in Figures 5 and 6.

Anchored tanks are required by AWWA D100 to have a grout layer between the floor plate and the ringwall at the shell; however, no grout was observed.

Figure 5 Geneva Reservoir at Shell to Foundation Interface

Figure 6 Roof at Entry Hatch

The interior was observed from the roof hatch and photographed without entering. Framing conditions appeared consistent with record information.

5.1.3 Summary of Findings – Structural

Table 4 compares the results of the seismic analysis to standards in AWWA D100-11. Supporting calculations for these ratios are provided in Appendix B.1. The recommended allowable forces do not represent failure loads, but have a liberal safety factor. Anytime the ratio of actual to allowable exceeds about two, however, the demand is approaching ultimate capacity and should be a cause for concern. When comparisons are made on an ultimate strength basis, the safety limit has been reached when the ratio of factored loads to allowable strength is less than 1.0.

Because the predicted sloshing wave will contact the tank roof, the seismic load is considerably increased compared to a tank with adequate freeboard.

Table 4 – Seismic Load vs AWWA D100 Allowable – Geneva Reservoir

	Analysis	AWWA Requirement	Result
Sloshing Wave			
First Mode Amplitude	3.60 ft.	N/A	
Freeboard at Maximum Operating Level (MOL)	1.17 ft.	N/A	
Wave contacts roof	Yes	No	
Ratio of Wave Height to Freeboard	3.13	≤1.00	No Good

Table 4 – Seismic Load vs AWWA D100 Allowable – Geneva Reservoir			
	Analysis	AWWA Requirement	Result
Seismic Load Increase Due to Sloshing Wave Roof Contact			
Base Shear Without Roof Contact	727 kip	N/A	
Base Shear With Roof Contact	913 kip	N/A	
Increase Due to Roof Contact	+26%	N/A	
Overturning Without Roof Contact	9,207 kip-ft.	N/A	
Overturning With Roof Contact	11,229 kip-ft.	N/A	
Increase Due to Roof Contact	+22%	N/A	
Sloshing Force on Roof-Shell Joint	1,201 plf	N/A	
Shell Static Stress			
Maximum hoop tensile stress/allowable ratio	1.05 at base. 1.02 at bottom of second course	1.0	Say OK See Item 1 in Seismic Evaluation Summary below
Shell Seismic Stress			
Maximum hoop tensile stress/allowable Ratio	1.36	≤ 1.33	Say OK See Item 1 in Seismic Evaluation Summary below
Maximum longitudinal compressive stress/allowable Ratio	0.67	≤ 1.33	OK
Maximum longitudinal tensile stress/allowable ratio	0.15	≤ 1.33	OK
Maximum shear stress/allowable at shell to floor connection	0.24	≤ 1.33	OK
Anchors			
Anchor spacing	12.5 ft.	≤ 10 ft.	No Good
Predicted/Allowable Stress Ratio (anchor top plate)	9.61	≤ 1.33	No Good
Predicted/Allowable Stress Ratio (anchor embedded plate)	6.40	≤ 1.33	No Good
Predicted/Allowable Stress Ratio (anchor weld at shell)	7.15	≤ 1.33	No Good
Predicted/Allowable Stress Ratio (anchor splice weld))	5.36	≤ 1.33	No Good
Bond Stress/Allowable Stress (embedded plate)	7.27	≤ 1.33	No Good
Foundation			
Overturning safety factor	0.92	≥ 1.67	No Good
Uplift safety factor	0.24	≥ 1.0	No Good, Uplift occurs

Table 4 – Seismic Load vs AWWA D100 Allowable – Geneva Reservoir			
	Analysis	AWWA Requirement	Result
Base shear/friction resistance at floor level	0.24	≤ 1.33	OK
Bearing pressure/allowable	2.44	≤1.33	No Good
Check Stability As Unanchored Tank			
Stability ratio, J	9.45	≤1.54	Unstable

5.1.4 Seismic Evaluation Summary

1. The static hoop stress at the base of the shell is overstated because the calculations typically ignore the restraint provided by the floor plate. The static hoop stress at the base of the second shell course is within 2 percent of allowable. Consider all shell plates adequate for static as well as seismic hoop and compression stresses.
2. Anchors are inadequate. If anchors fail, the tank would behave as if unanchored but the tank does not have the required stability without anchors and could fail catastrophically.
3. The existing ringwall does not provide enough weight to prevent uplift by a wide margin, even assuming it could be adequately anchored. This means that much of the ringwall will be subject to bending and torsional forces for which it was not designed, and the bottom of the tank could pull apart from the shell, with catastrophic failure.
4. The safety factor against overturning is insufficient.

5.2 Division 22 Reservoir

5.2.1 Record Information

The Division 22 Reservoir was constructed by Union Tank Company (no longer in business) of Seattle, WA in 1971. The nameplate indicates the use of the AWWA D100 standard. Original design drawings were prepared by Horton Dennis Engineers and were provided to BHC by the District, along with a cleaning and inspection report and video by H2O Solutions dated July 12, 2012 (see Figures 7 through 9). The Division 22 Reservoir design drawing provided basic dimensional data for the Division 7 and 30 Reservoirs on the same sheet. An original soils report by Dames and Moore was referenced but the report was unavailable.

A new reservoir near the existing one has been proposed with a capacity of 630,000 gallons. A recent soils report for this companion reservoir was prepared by PanGeo in December 2014 and recommended the use of Site Class C for design purposes. A site location map for the Division 22 Reservoir is provided in Figure 10.

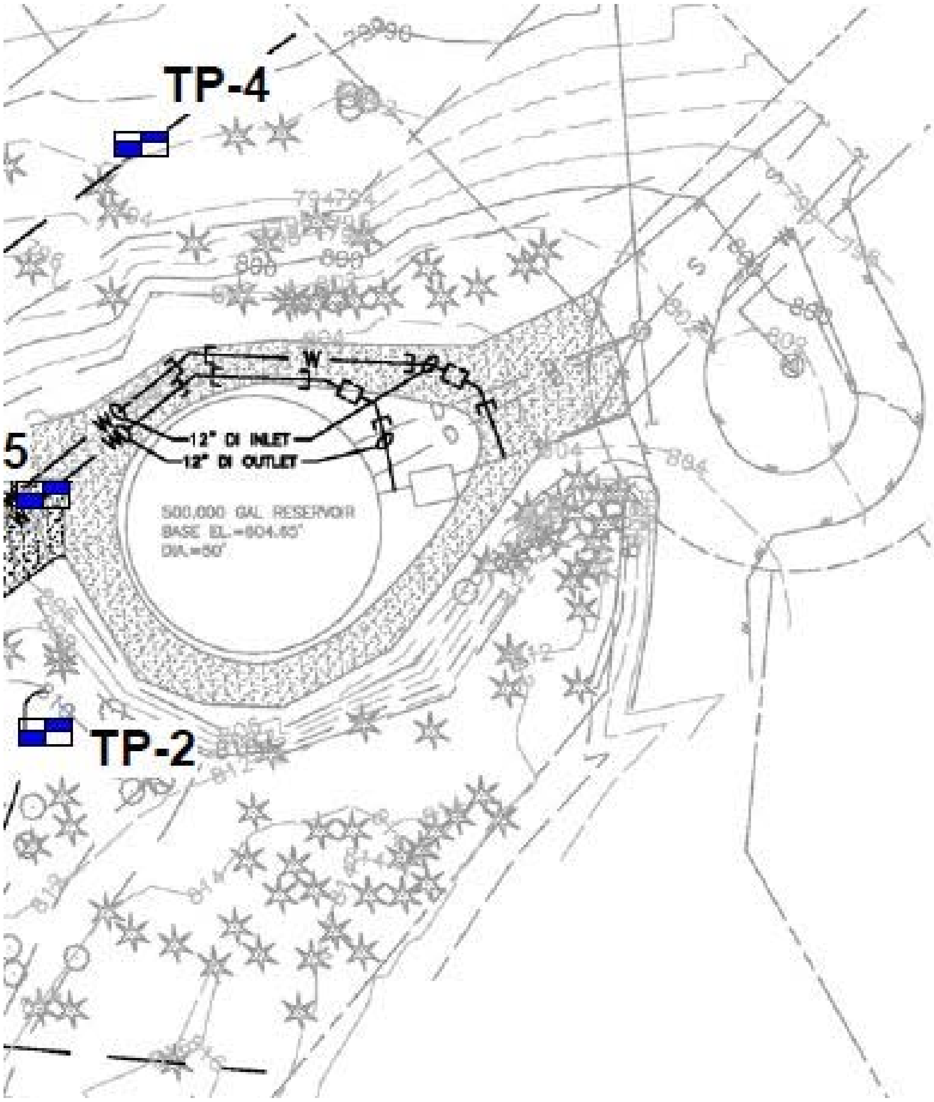


Figure 7 Division 22 Reservoir Site Plan from PanGeo Report

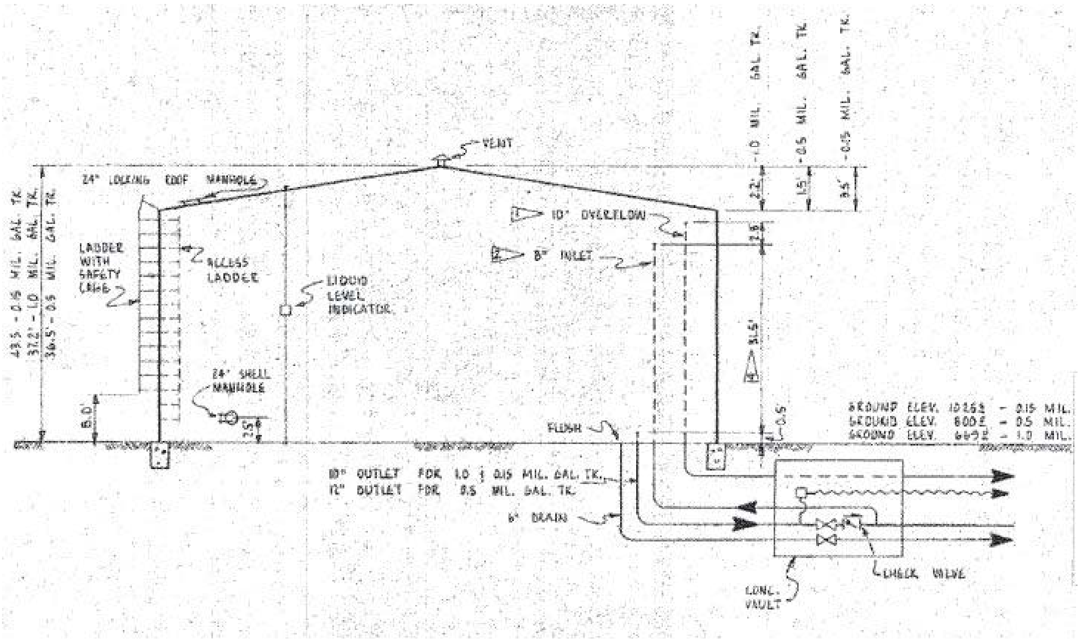


Figure 8 Division 22 Reservoir Elevation View from Original Design Drawings

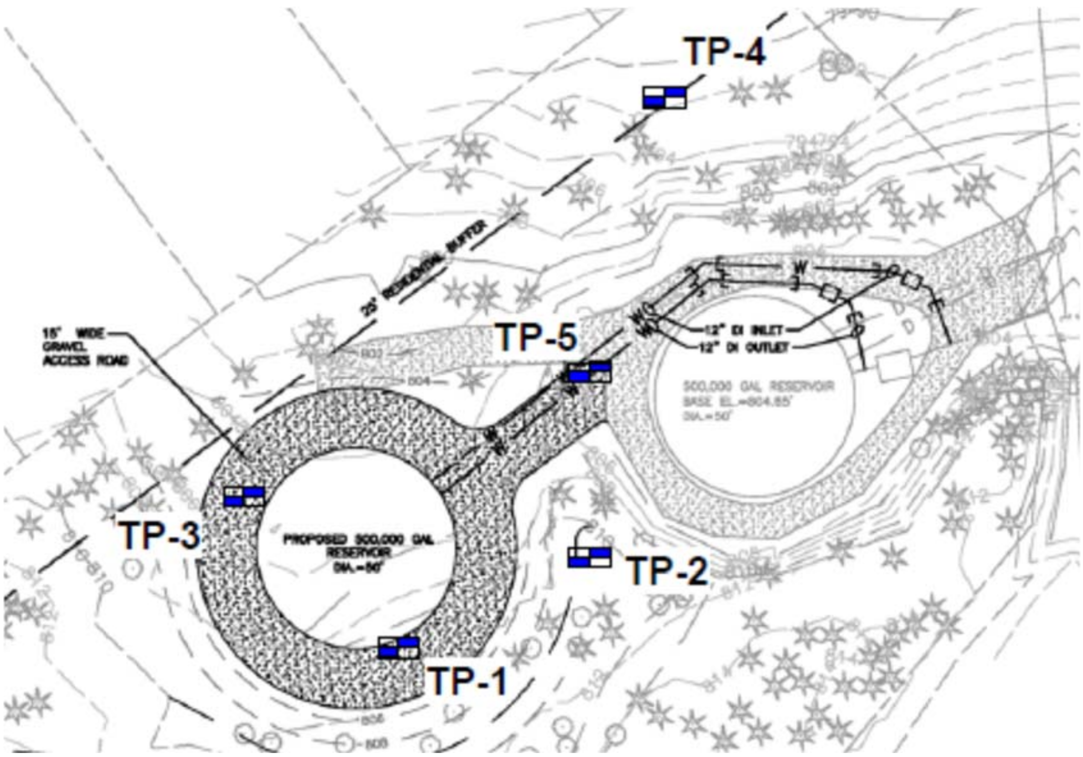


Figure 9 Division 22 Reservoir – Proposed Second Tank, from Pan Geo Report

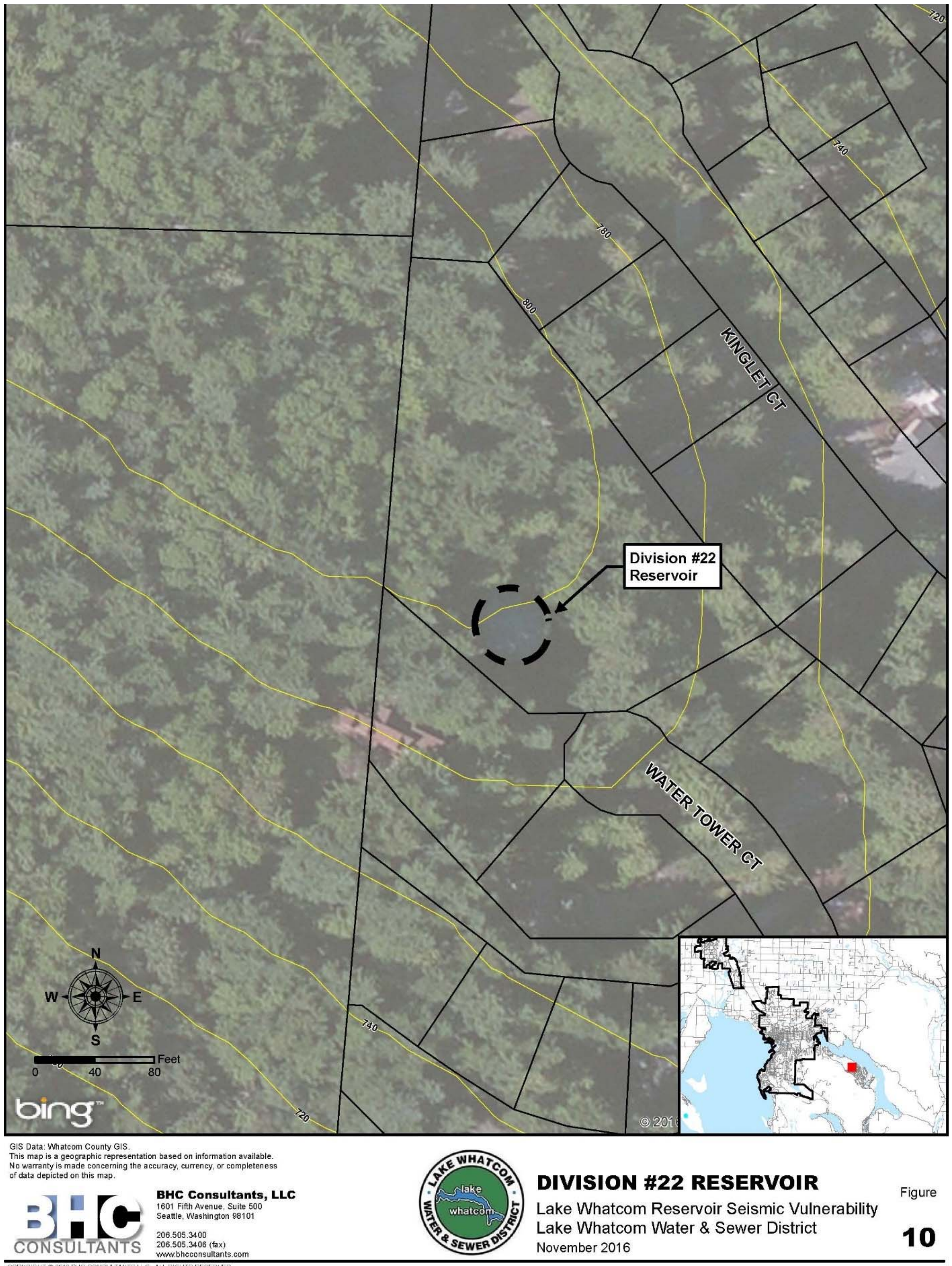


Figure 10 Division 22 Reservoir

The reservoir has a mildly sloped cone roof, supported by 25 channel-shaped rafters which span from the shell to a steel center column.

5.2.2 BHC Field Observations

General condition, appurtenances, and site conditions appeared consistent with record information. The tank has a single 24 inch by 18 inch elliptical shell manhole, and a 24 inch diameter roof hatch with no roof railing. The roof is accessed by a caged exterior ladder and an un-caged interior ladder (see Figure 11).

Figure 11 Division 22 Reservoir, September 1, 2015

BHC measured the tank diameter and height, and the height and metal thickness of shell courses, floor, and roof plates Other dimensions were measured using a steel tape. A measurement summary is provided in Table 5. Based on measured thicknesses, it appears that shell courses 3 and 5 were installed in reverse order.

The measured diameter of the tank is 50 feet, and the shell height is 35 feet. The overflow elevation (record) is 34 feet 8 inches above the floor, for a gross top capacity of 520,088 gallons compared to a nominal capacity of 500,000 gallons. The tank is unanchored. Grade was at or within a few inches below the top of the ringwall.

Table 5 – Metal Thicknesses – Division 22 Reservoir

Item	Distance from Top of Floor Plate to Top of Shell Course (ft)		Metal Thickness (in)				
	Record	Measured By Tape	Record	Measured By Tape	Measured Using UT Gauge	Average	Used for Analysis
Roof Plate	N/A		N/A	N/A	0.18, 0.18	0.18	3/16
Shell Course 5 (highest)	36.5	35	N/A	N/A	0.270, 0.270	0.270	9/32
Shell Course 4	N/A	28.05	N/A	N/A	0.255,0.255	0.255	1/4
Shell Course 3	N/A	21.02	N/A	N/A	0.265, 0.265	0.265	1/4
Shell Course 2	N/A	14.02	N/A	N/A	0.295, 0.295	0.295	9/32
Shell Course 1 (lowest)	N/A	7.02	N/A	N/A	0.395, 0,398, 0.400	.398	13/32
Floor Plate	N/A		N/A	1/4	N/A	1/4	1/4

A test pit excavated along the side of the ringwall by the District allowed measurement of a ringwall height of 60 inches at perimeter station 1+25.83 feet measured clockwise from the center of the shell manhole. Additional measurements by the District on January 7, 2016 measured heights of 40 inches and 37 inches, respectively, at stations 1+25 and 0+63. This variability in ringwall height was also observed at Reservoirs 7 and 30. An depth of 40 inches was used for analysis.

Geotest measured ringwall thicknesses at two locations. The Geotest thickness averaged 25.7 inches at station 1+25 using the CTG method and 27 to 33 inches using the GPR method. Readings at station 0+63 averaged 28 inches using the CTG method and 24 to 30 inches using the GPR method. A width of 28 inches was used for analysis.

The tank has a grout layer between the floor plate and the ringwall at the shell. The grout layer is in poor condition, with gaps several feet long where the grout has fallen out. The thickness of the grout layer varies from about 1 inch to virtually nothing. The ringwall circumference is irregular and the tank floor plate barely sits on the ringwall in some locations. Photos from the site visit are shown in Figures 12 through 15.

Figures 12, 13, and 14 Division 22 Reservoir at Foundation
Note minimal or missing grout and irregular ringwall.

Figure 15 Division 22 Reservoir at Roof Hatch

The interior was observed from the roof hatch and photographed without entering. Conditions appeared consistent with previous video by H2O Solutions.

5.2.3 Summary of Findings – Structural

For purposes of analysis, an average ringwall thickness of 28 inches has been assumed. Wall thicknesses are generally designed in 2 inch multiples. An average ringwall height of 40 inches was assumed.

Table 6 compares the results of the seismic analysis to standards in AWWA D100-11. Supporting calculations for these ratios are provided in Appendix B.2. The recommended allowable forces do not represent failure loads, but have a liberal safety factor. Anytime the ratio of actual to allowable exceeds about two, however, the demand is approaching ultimate capacity and should be a cause for concern.

Table 6 – Seismic Load vs AWWA D100 Allowable – Division 22 Reservoir

	Analysis	AWWA Requirement	Result
Sloshing Wave			
First Mode Amplitude	3.11 ft.	N/A	
Freeboard at Maximum Operating Level (MOL)	1.5 ft.	N/A	
Wave contacts roof	Yes	No	
Ratio of Wave Height to Freeboard	2.07	≤1.00	No Good
Seismic Load Increase Due to Sloshing Wave Roof Contact			
Base Shear Without Roof Contact	799 kip	N/A	
Base Shear With Roof Contact	908 kip	N/A	
Increase Due to Roof Contact	+14%	N/A	

Table 6 – Seismic Load vs AWWA D100 Allowable – Division 22 Reservoir			
	Analysis	AWWA Requirement	Result
Overturning Without Roof Contact	10,619 kip-ft.	N/A	
Overturning With Roof Contact	11,908 kip-ft.	N/A	
Increase Due to Roof Contact	+12%	N/A	
Sloshing Force on Roof-Shell Joint	415 plf	N/A	
Shell Static Stress			
Maximum hoop tensile stress/allowable ratio	0.96	≤ 1.00	OK
Shell Seismic Stress			
Maximum hoop tensile stress/allowable Ratio	1.40	≤ 1.33	No Good
Maximum longitudinal compressive stress/allowable Ratio	0.75	≤ 1.00	OK
Maximum longitudinal tensile stress/allowable ratio	0.13	≤ 1.33	OK
Maximum shear stress/allowable at shell to floor connection	0.26	≤ 1.33	OK
Foundation			
Overturning ratio	1.68	≥ 1.67	OK
Unit resistance/unit uplift	0.79	≥ 1.00	No Good
Base shear/friction resistance at floor level	0.41	≤ 1.33	OK
Bearing pressure/allowable	1.12	≤1.33	OK
Check Stability As Unanchored Tank			
Stability ratio, J	9.91	≤1.54	Unstable
Note: 1) Foundation resistance against uplift is an indication of the resistance that would be provided by the foundation if it were adequately anchored to the foundation. If the ratio is less than 1.0, it means that even if anchored, the existing ringwall would be inadequate to keep the tank from lifting.			

5.2.4 Seismic Evaluation Summary

1. Under seismic loading, the bottom of the second shell course is slightly overstressed in hoop tension.
2. The existing ringwall does not provide enough weight to prevent uplift, even assuming it could be adequately anchored. This means that some of the ringwall will be subject to bending and torsional forces for which it was not designed, and the bottom of the tank could pull apart from the shell, with catastrophic failure.
3. Because the tank is unanchored, the tank will not be stable and could fail catastrophically under the assumed earthquake loading.
4. Without anchors, tank uplift may be on the order of 50 times the bottom plate thickness, or roughly 12 inches. AWWA D100 limits upward vertical displacements in unanchored tanks to 1 inch for piping attachments, so piping connections are at risk of failure in an earthquake.

5.3 Division 7 Reservoir

5.3.1 Record Information

The Division 7 Reservoir was constructed by Union Tank Company (no longer in business) of Seattle, WA in 1971. The nameplate indicates the use of the AWWA D100 standard. Original design drawings were prepared by Horton Dennis and were provided to BHC by the District (see Figure 16). The reservoir was included in the previously mentioned structural evaluation report by Wilson Engineering dated December 13, 2012 along with a cleaning and inspection report and video by H2O Solutions dated July 10, 2012. The Division 7 design drawing provided basic dimensional data for the Division 22 and 30 Reservoirs on the same sheet. An original soils report by Dames and Moore was referenced but the report was unavailable. A site location map for the Division 7 Reservoir is provided in Figure 17.

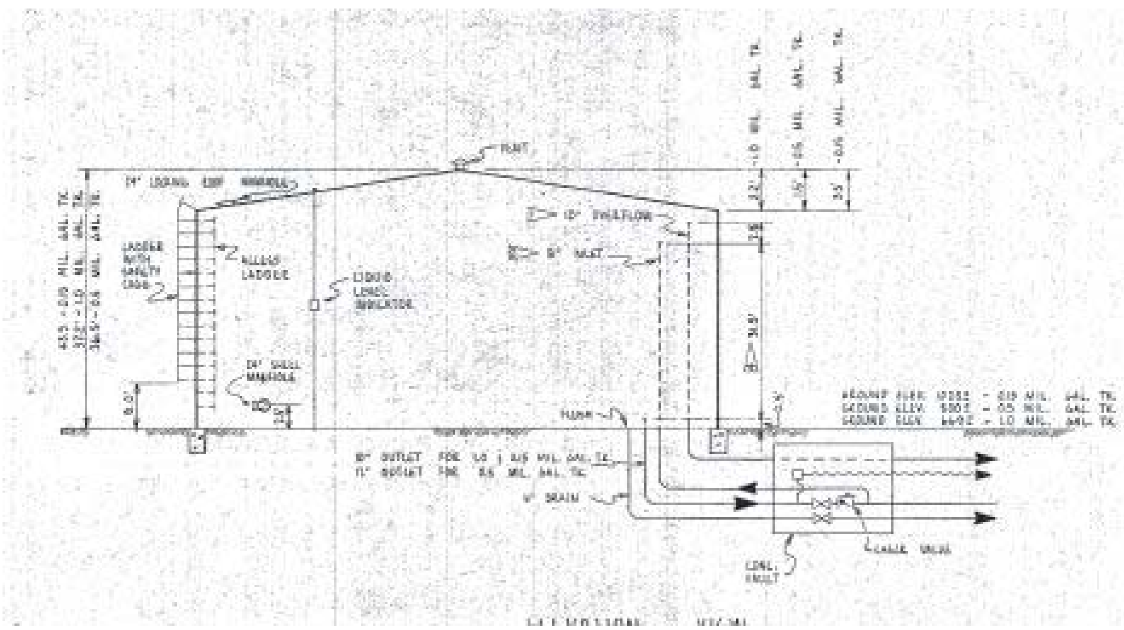


Figure 16 Elevation View from Original Design Drawings

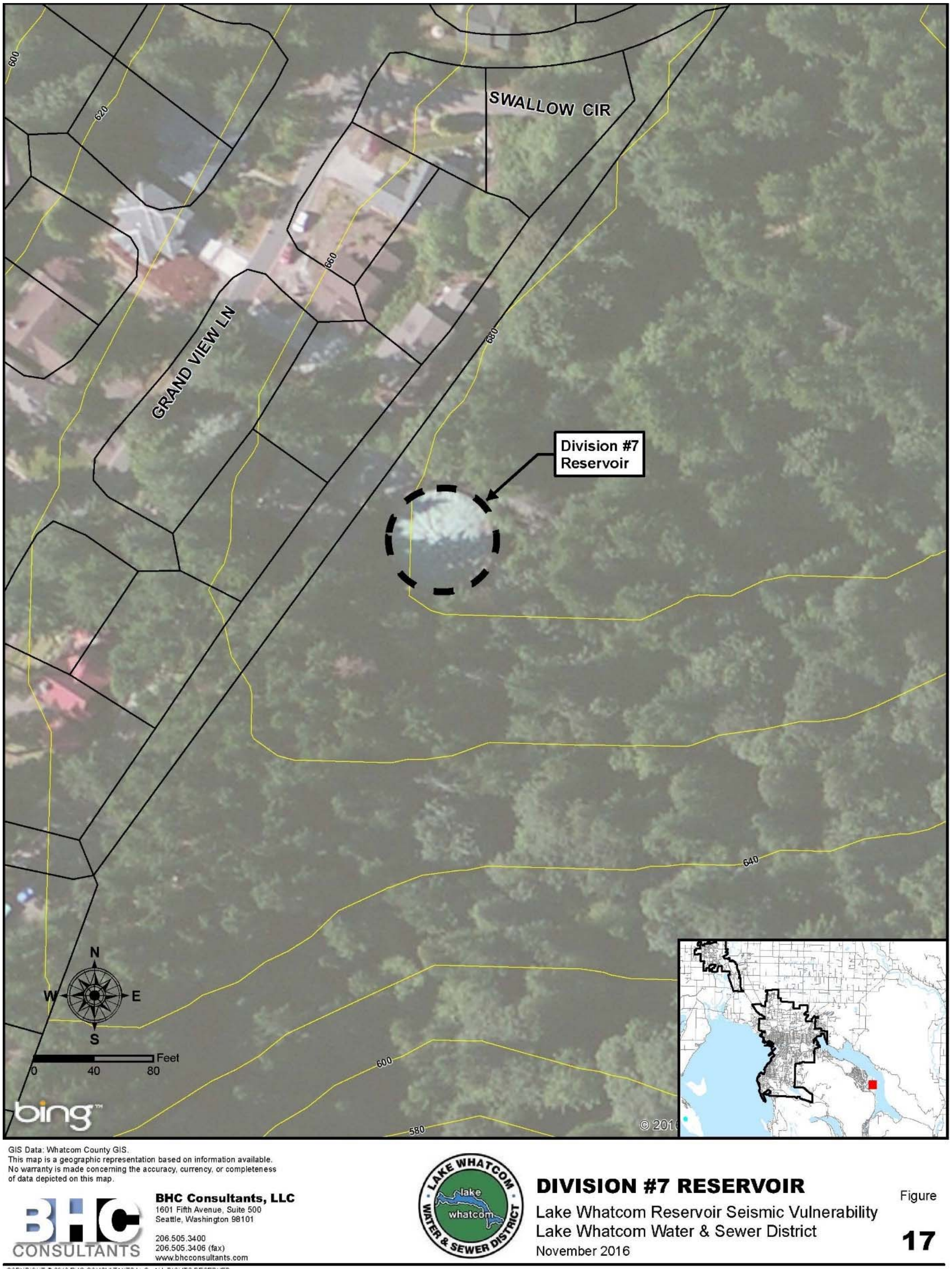


Figure 17 Division 7 Reservoir

The reservoir has a mildly sloped cone roof, supported by 18 equally spaced W8 primary rafters which span from the shell to a steel center column. Partial C6 secondary rafters span from the shell to C6 headers which transfer the load to the primary rafters. The headers are located roughly a quarter of the distance from the shell to the center. Member sizes were estimated from visual observation and approximate capacity calculations. Wilson Engineering noted a partial failure of one of the C6 header connections to a W8 primary rafter in its report. No remedial repair was documented or observed.

5.3.2 BHC Field Observations

General condition, appurtenances, and site conditions appeared consistent with record information. The tank has a single 24 inch by 18 inch elliptical shell manhole, and a 24 inch diameter roof hatch with no roof railing. The roof is accessed by a caged exterior ladder and an un-caged interior ladder (see Figure 18).

Figure 18 Division 7 Reservoir, September 1, 2015

BHC measured the tank diameter and height, and the height and metal thickness of shell courses, floor, and roof plates. Metal thickness for the shell and roof was measured using a Cygnus 6 Plus Ultrasonic Thickness Gauge. Other dimensions were measured using a steel tape. Measurements are summarized in Table 7.

Table 7 – Metal Thicknesses – Division 7 Reservoir							
Item	Distance from Top of Floor Plate to Top of Shell Course (ft)		Metal Thickness (in)				
	Record	Measured By Tape	Record	Measured By Tape	Measured Using UT Gauge	Average	Used for Analysis
Roof Plate	N/A		N/A	N/A	0.175, 0.175	0.175	5/16
Shell Course 5 (highest)	37.2	35.0	N/A	N/A	0.26, 0.26	0.26	1/4
Shell Course 4	N/A	28.04	N/A	N/A	0.255, 0.255	0.255	1/4
Shell Course 3	N/A	21.03	N/A	N/A	0.255, 0.255	0.255	1/4
Shell Course 2	N/A	14.02	N/A	N/A	0.275, 0.275	0.275	9/32
Shell Course 1 (lowest)	N/A	7.02	N/A	N/A	0.335, 0.34	0.34	11/32
Floor Plate	N/A		N/A		0.32, 0.31	0.315	5/16

The measured diameter of the tank is 70 feet, and the shell height is 35 feet. The overflow elevation (record) is 34 feet 8 inches above the floor, for a gross top capacity of 997,939 gallons compared to a nominal capacity of 1,000,000 gallons. The tank is unanchored. Grade varied from zero to 8 inches below the top of the ringwall.

A test pit excavated along the side of the ringwall by the District allowed measurement of a ringwall height of 59.5 inches at perimeter station 1+25.83 feet measured clockwise from the center of the shell manway. Additional measurements by the District on January 7, 2016 measured heights of 37 inches and 43 inches, respectively, at stations 1+00 and 1+90, also measured clockwise from the center of the shell manhole. A representative depth of 40 inches was assumed for analysis.

Geotest measured ringwall thicknesses at two locations. The Geotest thickness averaged 28.1 inches at station 1+00 using the CTG method (impact-echo theory) and 32 to 33 inches using the GPR (ground penetrating radar) method. Readings at station 1+90 averaged 29.4 inches using the CTG method and 30 to 36 inches using the GPR method. There was considerable scatter in the results. A thickness of 30 inches was assumed for analysis as a reasonable and conservative thickness based on the low end of the range from the GPR method.

The tank has a grout layer between the floor plate and the ringwall at the shell which is in very poor condition, with gaps several feet long where the grout has fallen out. The thickness of the grout layer varies from about 2 inches to virtually nothing. Photos from the site visit are shown in Figures 19 through 21.

Figure 19 Division 7 Reservoir at Foundation
Note missing grout.

Figures 20 and 21 Division 7 Reservoir at Roof Hatch and Vent

The interior was observed from the roof hatch and photographed without entering. Conditions appeared consistent with previous video by H2O Solutions. The roof was approximately 25 percent covered by branch and needle debris from nearby trees.

5.3.3 Summary of Findings – Structural

Table 8 compares the results of the seismic analysis to standards in AWWA D100-11. Supporting calculations for these ratios are provided in Appendix B.3. The recommended allowable forces do not represent failure loads, but have a liberal safety factor. Anytime the ratio of actual to allowable exceeds about two, however, the demand is approaching ultimate capacity and should be a cause for concern.

Table 8 – Seismic Load vs AWWA D100 Allowable – Division 7 Reservoir

	Analysis	AWWA Requirement	Result
Sloshing Wave			
First Mode Amplitude	3.47 ft.	N/A	
Freeboard at Maximum Operating Level (MOL)	1.5 ft.	N/A	
Wave contacts roof	Yes	No	
Ratio of Wave Height to Freeboard	2.33	≤1.00	No Good
Seismic Load Increase Due to Sloshing Wave Roof Contact			
Base Shear Without Roof Contact	1,365 kip	N/A	
Base Shear With Roof Contact	1,750 kip	N/A	
Increase Due to Roof Contact	+28%	N/A	
Overturning Without Roof Contact	18,227 kip*ft.	N/A	
Overturning With Roof Contact	22,978 kip-ft.	N/A	
Increase Due to Roof Contact	+26%	N/A	
Sloshing Force on Roof-Shell Joint	939 plf	N/A	
Shell Static Stress			
Maximum hoop tensile stress/allowable ratio	1.39	≤ 1.00	No Good

Table 8 – Seismic Load vs AWWA D100 Allowable – Division 7 Reservoir			
	Analysis	AWWA Requirement	Result
Shell Seismic Stresses			
Maximum hoop tensile stress/allowable Ratio	2.18	≤ 1.33	No Good
Maximum longitudinal compressive stress/allowable Ratio	0.35	≤ 1.00	OK
Maximum longitudinal tensile stress/allowable ratio	0.17	≤ 1.33	OK
Maximum shear stress/allowable at shell to floor connection	0.28	≤ 1.33	OK
Foundation			
Safety Factor against overturning	1.77	≥ 1.67	OK
Unit resistance/unit uplift	0.74	≥ 1.00	No Good
Base shear/friction resistance at floor level	0.32	≤ 1.33	OK
Bearing pressure/allowable	2.14	≤1.33	No Good
Check Stability As Unanchored Tank			
Stability ratio, J	8.01	≤1.54	Unstable

Note:

1) Foundation resistance against uplift is an indication of the resistance that would be provided by the foundation if it were adequately anchored to the foundation. If the ratio is less than 1.0, it means that even if anchored, the existing ringwall would be inadequate to keep the tank from lifting.

5.3.4 Seismic Evaluation Summary

1. The bottom half of the tank shell has excessive hoop tensile stress under both ordinary hydrostatic load as well as seismic conditions.
2. The tank has acceptable longitudinal compressive stress under seismic load, but this is only because AWWA allows consideration of shell stiffening from water pressure for unanchored tanks under earthquake loading (AWWA D100 section 13.5.4.2.4). If the tank is anchored, the allowable compressive stress will be reduced and the margin of safety reduced.
3. Without anchors, tank uplift may be on the order of 50 times the bottom plate thickness, or roughly 16 inches. AWWA D100 limits upward vertical displacements in unanchored tanks to 1 inch for piping attachments, so piping connections are at risk of failure in an earthquake.
4. The failing header connection cited in the Wilson Engineering report in 2012 should be repaired before it fails, resulting in roof damage.
5. The anchorage and foundation are inadequate. As a result, the tank will not be stable under the earthquake loads assumed and could fail catastrophically.

5.4 Division 30 Reservoir

5.4.1 Record Information

The Division 30 Reservoir was constructed by Union Tank Company (no longer in business) of Seattle, WA in 1973. The nameplate indicates the use of the AWWA D100 standard. Original design drawings were prepared by Horton Dennis and were provided to BHC by the District (see Figure 22). The reservoir was the subject of a cleaning and inspection report and video by H2O Solutions dated July 10, 2012. The Division 30 Reservoir design drawing provided basic dimensional data for the Division 22 and 7 Reservoirs on the same sheet. An original soils report by Dames and Moore was referenced but the report was unavailable.

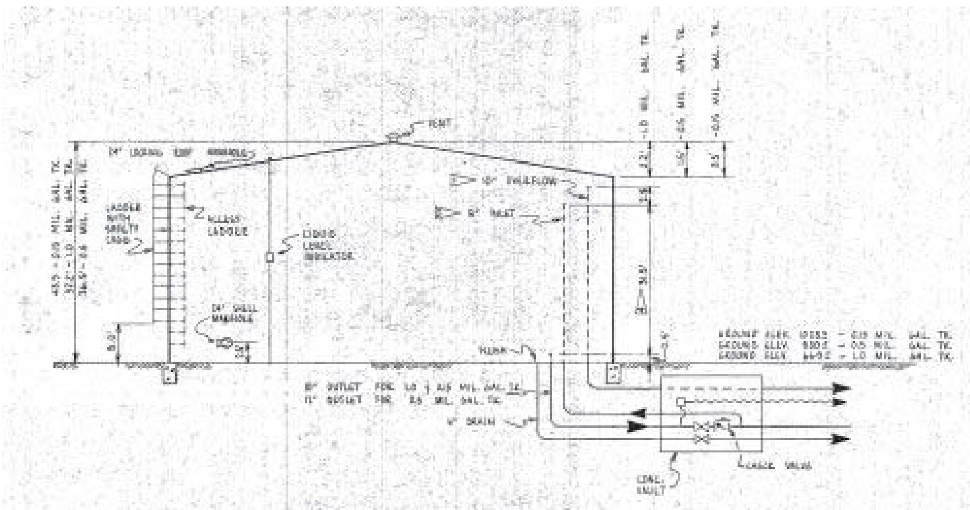


Figure 22 Elevation View from Original Design Drawings

Reservoir 30 has a spherical segment, self-supporting dome roof with no stiffener plates or knuckle transitions. This is different than the cone roof profile shown in Figure 22. A site location map for the Division 30 Reservoir is provided in Figure 23.

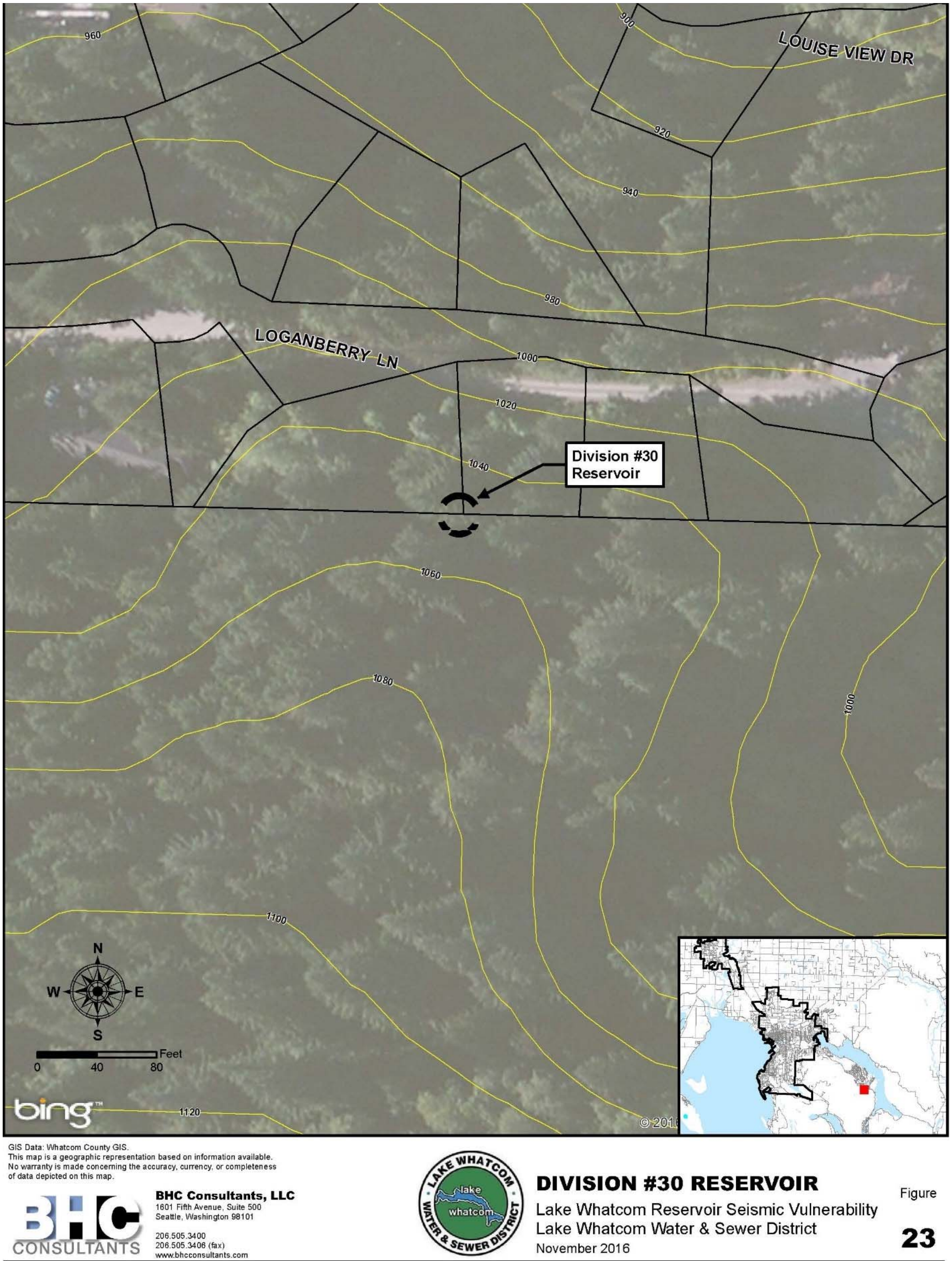


Figure 23 Division 30 Reservoir

5.4.2 BHC Field Observations

General condition, appurtenances, and site conditions appeared consistent with record information. The tank has a single 24 inch diameter shell manhole, and a 24 inch square roof hatch with partial roof railing. The roof is accessed by caged exterior and interior ladders. The exterior ladder has an intermediate landing platform (see Figure 24).

Figure 24 Division 30 Reservoir, September 1, 2015

BHC measured the tank diameter and height, and the height and metal thickness of shell courses, floor, and roof plates. Metal thickness for the shell and roof was measured using a Cygnus 6 Plus Ultrasonic Thickness Gauge. Other dimensions were measured using a steel tape. A measurement summary is provided in Table 9.

Table 9 – Metal Thicknesses – Division 30 Reservoir

Item	Distance from Top of Floor Plate to Top of Shell Course (ft)		Metal Thickness (in)				
	Record	Measured By Tape	Record	Measured By Tape	Measured Using UT Gauge	Average	Used for Analysis
Roof Plate	N/A		N/A	N/A	0.15,0.145	.148	5/32
Shell Course 5 (highest)	43.5	40.36	N/A	N/A	0.245,0.25	0.25	1/4
Shell Course 4	N/A	32.04	N/A	N/A	0.25,0.245	0.25	1/4
Shell Course 3	N/A	24.01	N/A	N/A	0.235,0.245	0.24	1/4
Shell Course 2	N/A	16.02	N/A	N/A	0.245,0.24	0.24	1/4
Shell Course 1 (lowest)	N/A	8.02	N/A	N/A	0.235,0.245,0.245	0.24	1/4
Floor Plate	N/A		N/A	1/4	N/A	1/4	1/4

The measured diameter of the tank is 25 feet 5 inches, and the shell height is 40 feet 4.5 inches. The overflow elevation (record) is 6 inches below the top of shell, for a gross top capacity of 151,390 gallons compared to a nominal capacity of 150,000 gallons. The tank is anchored with 12 strap anchors at about 6 feet 8 inch spacing. Grade varied from zero to 8 inches below the top of the ringwall.

A test pit excavated along the side of the ringwall by the District allowed measurement of a ringwall height of 58.5 inches. Additional measurements by the District on January 7, 2016 measured variations from 24 to 36 inches and 43 inches at excavations near station 0+10. The District test pits also indicated rock at the bottom of the ringwall. Given the wide variation, an average height of 40 inches has been used for computations.

Geotest measured ringwall thickness at one location at the west end of the tank. The Geotest thickness averaged 17.2 inches using the CTG method and 15 to 21 inches using the GPR method. Given the wide variation, 18 inches has been used for computations.

The tank has no grout layer between the floor plate and the ringwall at the shell. The current AWWA D100 standard requires that all anchored tanks be grouted at the base of the shell. Photos from the site visit are shown in Figures 25 through 28.

Figure 25 Division 30 Reservoir at Foundation
Note typical strap anchor and no grout under the shell.



Figures 26, 27, and 28 Division 30 Reservoir at Roof Hatch and Vent

The interior was observed from the roof hatch and photographed without entering. Conditions appeared consistent with previous video by H2O Solutions.

5.4.3 Summary of Findings – Structural

Based on field observations, the ringwall thickness varies over its depth. For purposes of analysis, an average ringwall thickness of 18 inches was assumed for the portion of the ringwall that extended from the top of the ringwall to a depth of 32 inches. For the portion of the ringwall that was located from a depth of 32 inches to the bottom of the ringwall, a thickness of 20.5 inches was assumed. An overall ringwall height of 40 inches was assumed for analysis purposes.

Table 10 compares the results of the seismic analysis to standards in AWWA D100-11. Supporting calculations for these ratios are provided in Appendix B.4. The recommended allowable forces do not represent failure loads, but have a liberal safety factor. Anytime the ratio of actual to allowable exceeds about two, however, the demand is approaching ultimate capacity and should be a cause for concern.

Table 10 – Seismic Load vs AWWA D100 Allowable – Division 30 Reservoir

	Analysis	AWWA Requirement	Result
Sloshing Wave			
First Mode Amplitude	1.61 ft.	N/A	
Freeboard at Maximum Operating Level (MOL)	1.08 ft.	N/A	
Wave contacts roof	Yes	No	
Ratio of Wave Height to Freeboard	1.49	≤1.00	No Good
Seismic Load Increase Due to Sloshing Wave Roof Contact			
Base Shear Without Roof Contact	251 kip	N/A	

Table 10 – Seismic Load vs AWWA D100 Allowable – Division 30 Reservoir			
	Analysis	AWWA Requirement	Result
Base Shear With Roof Contact	251 kip	N/A	
Increase Due to Roof Contact	Negligible	N/A	
Overturning Without Roof Contact	4,449 kip-ft.	N/A	
Overturning With Roof Contact	4,447 kip-ft.	N/A	
Increase Due to Roof Contact	Negligible	N/A	
Sloshing Force on Roof-Shell Joint	46 plf	N/A	
Shell			
Maximum hoop tensile stress/allowable Ratio	0.934	≤ 1.33	OK
Maximum longitudinal compressive stress/allowable Ratio	1.03	≤ 1.33	OK
Maximum longitudinal tensile stress/allowable ratio	0.15	≤ 1.33	OK
Maximum shear stress/allowable at shell to floor connection	0.28	≤ 1.33	OK
Anchors			
Anchor spacing	6.67 ft.	≤ 10 ft.	OK
Predicted/Allowable Stress Ratio (anchor plate)	3.94	≤ 1.33	No Good
Predicted/Allowable Stress Ratio (anchor weld at shell)	2.81	≤ 1.33	No Good
Bond Stress/Allowable Stress (embedded plate)	3.33	≤ 1.33	No Good
Foundation			
Overturning safety factor	0.74	≥ 1.67	No Good
Unit resistance/unit uplift	0.38	≥ 1.00	No Good
Base shear/friction resistance at floor level	0.37	≤ 1.33	OK
Bearing pressure/allowable	0.71	≤1.33	OK
Check Stability As Unanchored Tank			
Stability ratio, J	18.56	≤1.54	Unstable

5.4.4 Seismic Evaluation Summary

1. The tank shell appears adequate.
2. The anchorage and foundation are inadequate. In the absence of adequate anchorage and foundation, the tank will not be stable and could fail catastrophically.

5.5 SVWTP Reservoir

5.5.1 Record Information

The Sudden Valley Water Treatment Plant (SVWTP) Reservoir was constructed by Reliable Steel Fabricators (no longer in business) of Olympia, WA in 1992. Limited design drawings and shop drawings were provided by the District. The available design drawing, dated 1992, consisted of a site plan only and was prepared by Wilson Engineering (see Figure 29). An additional design drawing for an inlet diffuser, prepared by Wilson Engineering in 1994, was also provided (see Figure 30). Also included were a cleaning and inspection report and video dated July 9, 2012 and an as-built of the inlet diffuser dated August 6, 2012 by H2O Solutions.

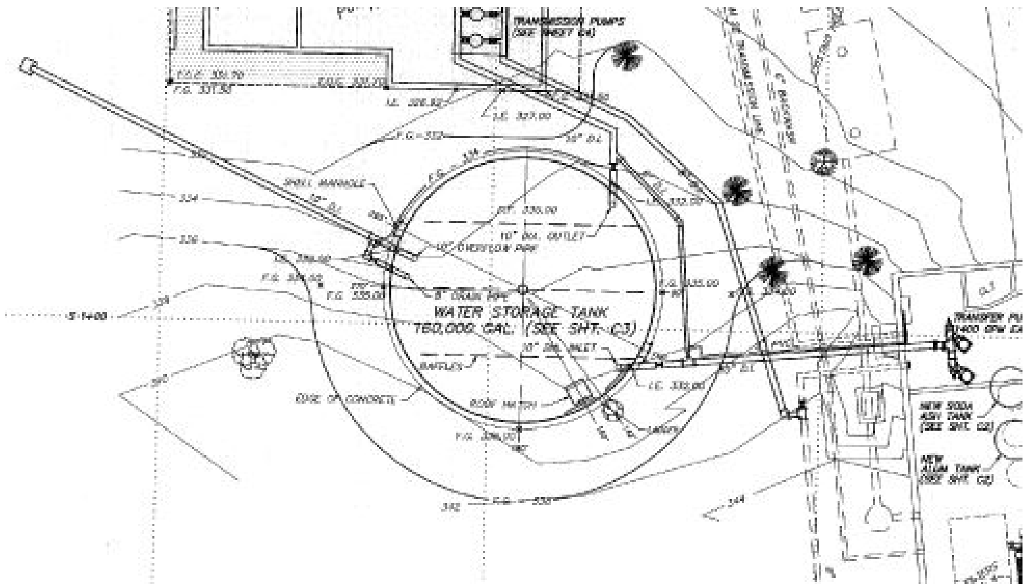


Figure 29 Site Plan from Original Design Drawings

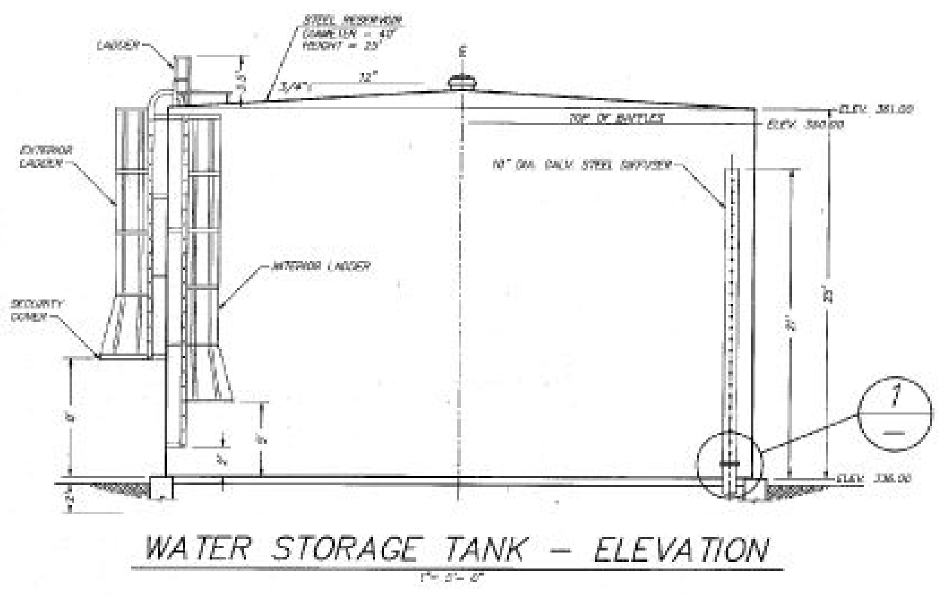


Figure 30 Elevation View with Inlet Diffuser, Wilson Engineering, 1992

No soils report was available. Shop drawings indicate design in accordance with AWWA D100-84, Seismic Zone 3. A site location map for the Sudden Valley Water Treatment Plant Reservoir is provided in Figure 31.

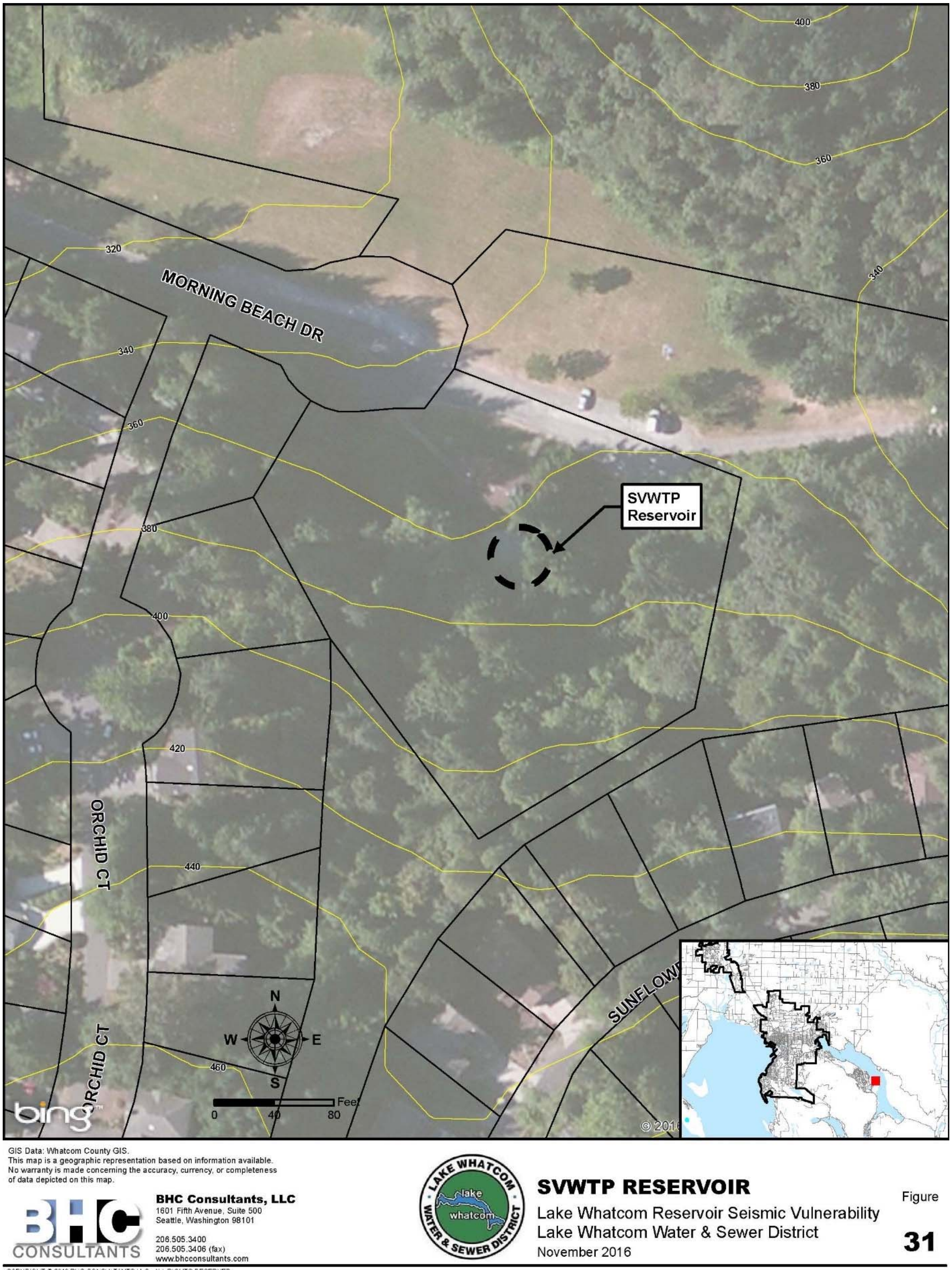
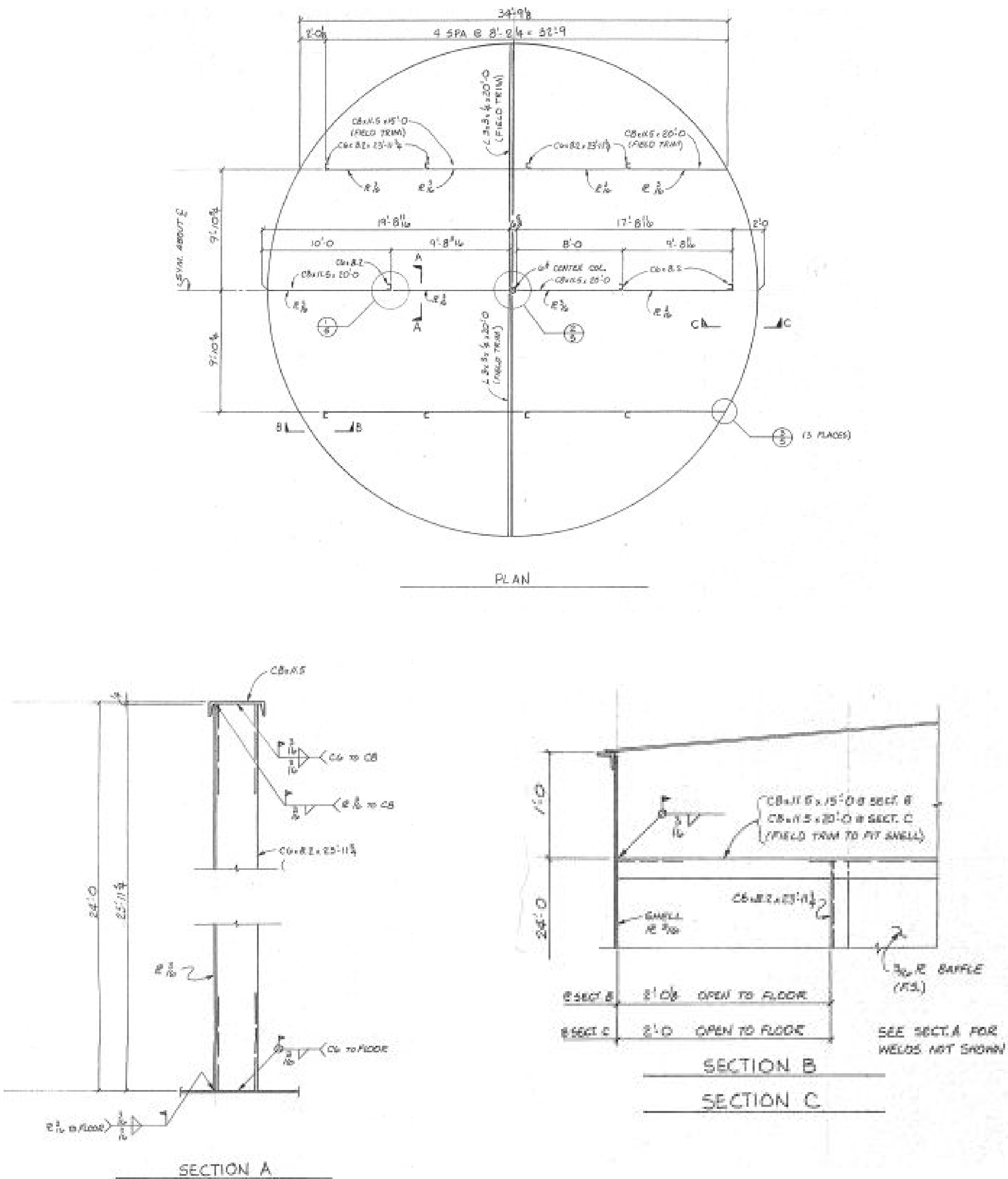


Figure 31 SVWTP Reservoir

The reservoir has a mildly sloped cone roof, supported by 20 C6X8.2 rafters which span from the shell to a steel center column. In addition, since the tank also provides chlorine contact, steel baffles are provided on the interior to promote mixing. The baffles consist of three runs of steel plate with vertical channel stiffeners and horizontal bracing (see Figures 32, 33, and 34).



Figures 32, 33, and 34 Details of Internal Baffle System for the SVWTP Reservoir

5.5.2 BHC Field Observations

General condition, appurtenances, and site conditions appeared consistent with record information. The tank has two 36 inch diameter shell manholes, and a 3 feet square roof hatch with partial roof railing. The roof is accessed by caged exterior ladder and an un-caged interior ladder (see Figure 35).

Figure 35 SVWTP Reservoir, September 1, 2015

BHC measured the tank diameter and metal thickness of shell courses, floor, and roof plates, which were all consistent with the shop drawings. Metal thickness for the shell and roof was measured using a Cygnus 6 Plus Ultrasonic Thickness Gauge. Other dimensions were measured using a steel tape. A measurement summary is provided in Table 11.

Table 11 – Metal Thicknesses – SVWTP Reservoir							
Item	Distance from Top of Floor Plate to Top of Shell Course (ft)		Metal Thickness (in)				
	Record	Measured By Tape	Record	Measured By Tape	Measured Using UT Gauge	Average	Used for Analysis
Roof Plate	N/A		3/16	N/A	0.18, 0.18	0.18	3/16
Shell Course 3 (highest)	25.0	N/A	3/16	N/A	0.18,0.19,0.17,0.19,0.135	0.173	3/16
Shell Course 2	16.67	N/A	3/16	N/A	0.185,0.18	0.18	3/16
Shell Course 1 (lowest)	8.33	N/A	3/16	N/A	0.18, 0.185	0.18	3/16
Floor Plate	N/A		1/4	N/A	N/A	N/A	1/4

Note:
1) Only verification measurements were taken at select locations. Complete shop drawing records were available for this tank.

The inside diameter of the tank is 40 feet, and the shell height is 25 feet. The overflow and top of baffle elevation is 24 feet above the floor, for a gross top capacity of 225,591 gallons compared to a nominal capacity of 235,000 gallons. The tank is held down by 13 1.5-inch diameter steel anchor bolts embedded in a concrete ringwall foundation with record dimensions of 18 inches wide by 72 inches high. The observed configuration and spacing of the anchors and anchor chairs was consistent with the record drawings. Grade varies considerably around the perimeter, up to nearly 24 inches below the top of the ringwall at the maximum.

A grout layer about 2 inches thick was observed beneath the shell plate and appeared to be in good condition. The ringwall appears to have had its outside face formed with straight rather than curved forms, so the distance from the shell to the outside face varies. Photos from the site visit are shown in Figures 36 and 37.


Figure 36 SVWTP Reservoir at Shell to Foundation Interface


Figure 37 Variable Diameter Ringwall

The interior was not observed since consistency of measurements with the shop drawings indicated that the drawings provided sufficient information for analysis.

5.5.3 *Summary of Findings – Structural*

Table 12 compares the results of the seismic analysis to standards in AWWA D100-11. Supporting calculations for these ratios are provided in Appendix B.5. The recommended allowable forces do not represent failure loads, but have a liberal safety factor. Anytime the ratio of actual to allowable exceeds about two, however, the demand is approaching ultimate capacity and should be a cause for concern.

Table 12 – Seismic Load vs AWWA D100 Allowable – SVWTP Reservoir			
Effect of Baffles Ignored			
	Analysis	AWWA Requirement	Result
Sloshing Wave			
First Mode Amplitude	3.27 ft.	N/A	
Freeboard at Maximum Operating Level (MOL)	3.00 ft.	N/A	
Wave contacts roof	Yes	No	
Ratio of Wave Height to Freeboard	1.09	≤1.00	Say OK, See Item 2 in Seismic Evaluation Summary below
Seismic Load Increase Due to Sloshing Wave Roof Contact			
Base Shear Without Roof Contact	285 kip	N/A	
Base Shear With Roof Contact	285 kip	N/A	
Increase Due to Roof Contact	Negligible	N/A	
Overturning Without Roof Contact	2,543 kip-ft.	N/A	
Overturning With Roof Contact	2,543 kip*ft.	N/A	
Increase Due to Roof Contact	Negligible	N/A	
Sloshing Force on Roof-Shell Joint	7 plf	N/A	
Shell			
Maximum hoop tensile stress/allowable Ratio	0.96	≤ 1.33	OK
Maximum longitudinal compressive stress/allowable Ratio	0.97	≤ 1.33	OK
Maximum longitudinal tensile stress/allowable ratio	0.11	≤ 1.33	OK
Maximum shear stress/allowable at shell to floor connection	0.13	≤ 1.33	OK

Table 12 – Seismic Load vs AWWA D100 Allowable – SVWTP Reservoir			
Effect of Baffles Ignored			
	Analysis	AWWA Requirement	Result
Anchors			
Anchor spacing	9 ft. 8 in	≤ 10 ft.	OK
Predicted/Allowable Stress Ratio (anchor bolt)	1.05	≤ 1.0	Say OK, See Item 3 in Seismic Evaluation Summary below
Predicted/Ultimate Strength Ratio (anchor bolt)*	1.05	≤ 1.0	Say OK, See Item 3 in Seismic Evaluation Summary below
Predicted/Allowable Strength Ratio (anchor chair welds)*	0.74	≤ 1.33	OK
Predicted/Ultimate Strength Ratio (concrete breakout strength)*	0.49	≤ 1.00	OK
Predicted/Ultimate Strength Ratio (anchor pullout strength)*	0.05	≤ 1.0	OK
Predicted/Ultimate Strength (side face blowout)*	0.16	≤ 1.0	OK
Foundation			
Overturning Safety Factor	1.73	≥ 1.67	OK
Unit resistance/unit uplift	0.90	≥ 1.00	No Good
Base shear/friction resistance at floor level	0.30	≤ 1.33	OK
Bearing pressure/allowable	1.15	≤1.33	OK
Check Stability As Unanchored Tank			
Stability ratio, J	7.29	≤1.54	Unstable
Note: *Strength ratios per ACI 318 Appendix D. Other ratios per AWWA D100/ASCE 7.			

The effect of ground motions acting perpendicular to the baffles would not yield the same results, but would probably increase base shear and overturning moment to some degree by increasing the relative amount of impulsive water mass. Evaluating the magnitude of this effect is beyond the scope of the present analysis.

5.5.4 Seismic Evaluation Summary

1. The tank shell appears adequate for ground motions parallel to the tank baffles.

2. Although the sloshing wave impinges slightly on the roof, the resulting forces are negligible and the slight shortage of freeboard is acceptable.
3. Although the anchor bolts are stressed slightly above allowable levels, these are only overstressed by about 5 percent and can be regarded as acceptable.

5.6 Relative Predicted Overload

5.6.1 Shell Hoop Stresses

In terms of hoop stress, all tanks except Division 7 and 22 are within limits for both static and seismic loads. The relative maximum stress ratios are shown below in Figures 38 and 39.

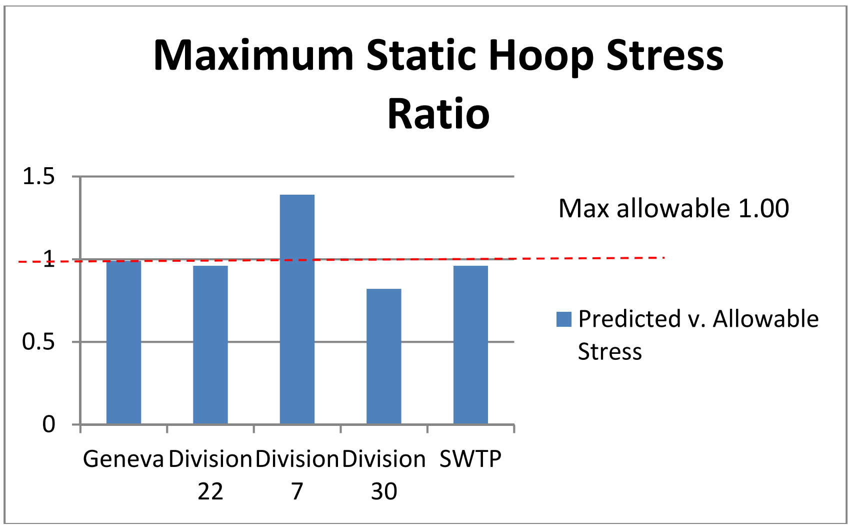


Figure 38 Maximum Static Hoop Stress Ratio

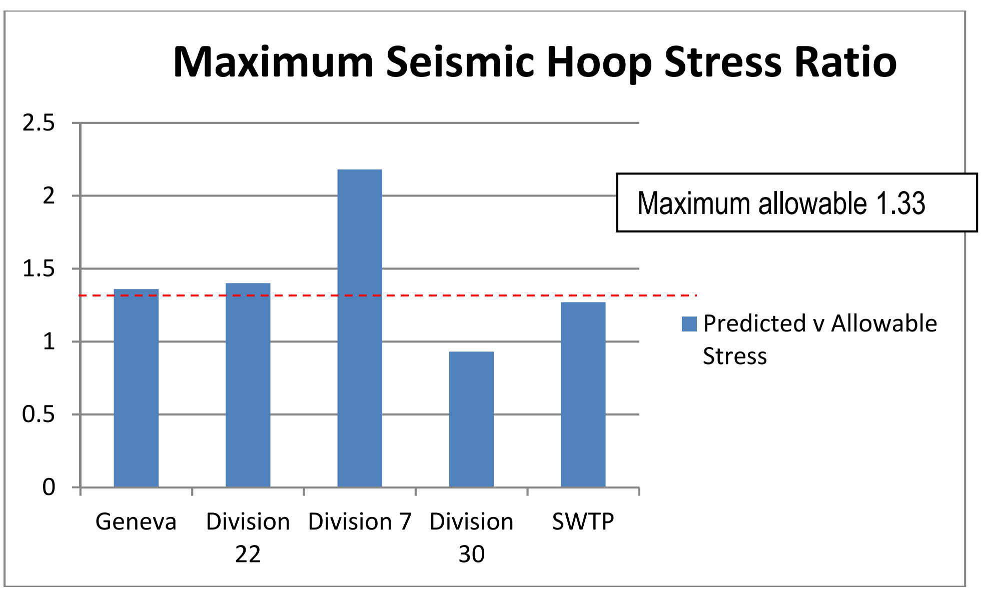


Figure 39 Maximum Seismic Hoop Stress

5.6.2 Longitudinal Shell Compressive Stress

In terms of maximum allowable longitudinal compressive stress in the shell under seismic loading, all the tanks except Division 7 are within allowable limits. In Figure 40, the ratios shown in previous tables have been normalized for the Division 7 and 22 tanks for an allowable ratio of 1.33, due to the slight difference in the way allowable stresses for unanchored tanks are computed compared to anchored tanks. Excessive longitudinal stress increases the likelihood of tank buckling.

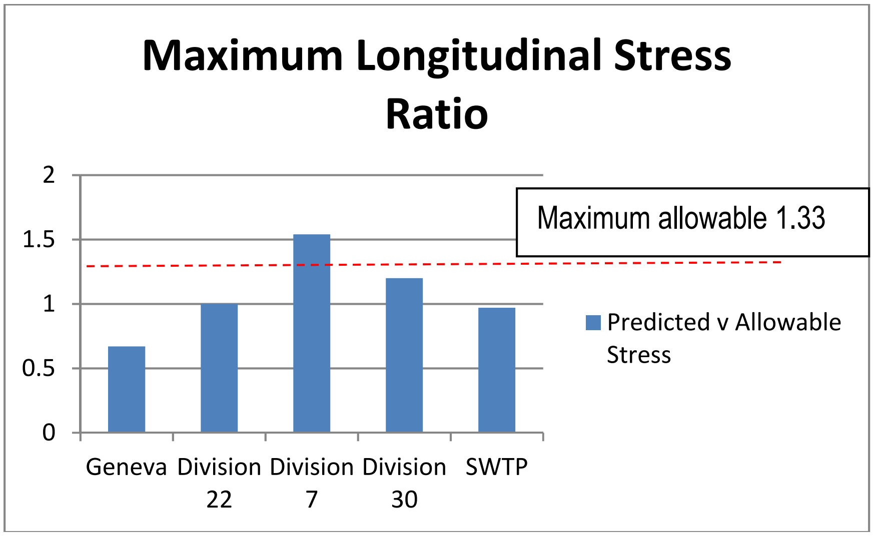


Figure 40 Maximum Longitudinal Stress Ratio

5.6.3 Stability as an Unanchored Tank

The stability ratio indicates whether or not an unanchored tank will be stable under seismic loading. This would apply to the currently unanchored Division 22 and 7 tanks, and to the anchored tanks in case of anchor failure. As shown in Figure 41, the limiting stability ratio of 1.54 is already exceeded in the case of Division 22 and 7 tanks, and would also be exceeded in the case of the others if the anchors failed. All of the tanks need to be anchored to avoid potential rollover and rupture of the shell to bottom plate joint.

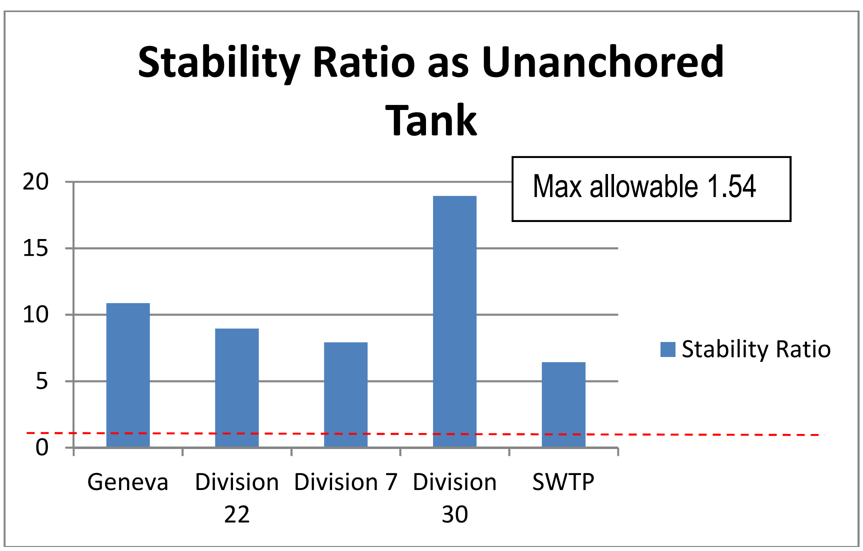


Figure 41 Stability Ratio as Unanchored Tank

5.6.4 Sloshing Wave Force on Roof to Shell Joint

The predicted sloshing wave uplift forces on the roof to shell joint are all approximately 100 lbs per foot or less, which is well within the allowable load on a 3/16 inch fillet weld, which is about 1,300 lbs per inch.

5.6.5 Foundation and Anchorage

In the case of the anchored tanks, maximum anchor spacing is within limits for the Division 30 and SVWTP tanks, but not for the Geneva tank. Anchor plate and anchor bolt stresses exceed allowable for all the anchored tanks. Anchorage failure for the embedded portion due to pullout or concrete failure is an issue for the Geneva and Division 30 tanks, but is adequate for the SVWTP tank.

None of the ringwall foundations, including soil resistance and the weight of water over the interior, are sufficient to prevent uplift, assuming anchorage were provided and adequately designed. Bearing pressure under seismic loading conditions appears to exceed the assumed limits; however, it is probably acceptable for the Division 30 tank if the ringwall is assumed to bear on rock.

Figure 42 below indicates the ratio of load to capacity for various foundation elements. All ratios have been normalized for comparison on an ultimate load to strength basis. All the reservoirs have inadequate foundations, but the SVWTP reservoir is the least problematic and most easily fixed.

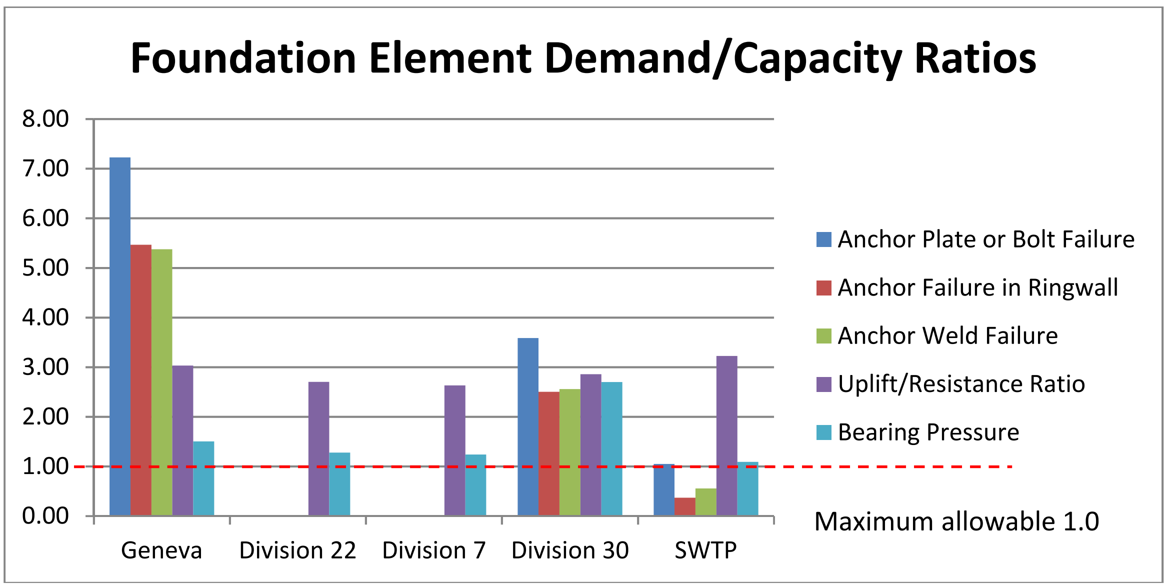


Figure 42 Foundation Element Demand/Capacity Ratios

6. Summary of Findings – Impact of Failure

The District's water system is tightly connected and redundant, with many tanks serving other zones where necessary with interties, PRVs and pump stations. The impact to nearby residences was determined by reviewing location map figures of the reservoirs and determining how many, if any, residences would be impacted should the reservoir fail. Impact to the water system was determined by evaluating the number of ERUs served, and by understanding how the reservoirs are inter-related with one another and provide storage and flow to other reservoirs within the system. Total impact, as shown in Table 13, was determined based on tank condition, impact to nearby residences, and water system impact.

Table 13 – Lake Whatcom Water and Sewer District - Reservoir Seismic Evaluation Impact Table

Reservoir	Capacity	Population (Per Section 2.1, WSP)	Flow Between Zones	Location	Tank Condition	Impact: Nearby Residences	Impact: Water System	Overall Impact
Sudden Valley Study Area								
Division 30	0.15MG	6,595	Fed by Div 7	Residential	Deficient	High	Medium (1,158 ERUs served; feeds high elevation homes)	Medium
Division 22	0.5MG		Linked with Geneva	Residential	Deficient	High	Medium (1,782 ERUs served; feeds Geneva)	Medium
Division 7	1.0MG		Fed by SVWTP; Feeds Div 30	Residential	Highly Deficient	High	High (2,153 ERUs served; largest size, feeds Div 30)	High
SVWTP	0.235MG		Feeds Div 7, Div 22, and Div 30	At WTP; no downstream residences	Somewhat Deficient	Low	High (3,935 ERUs served; feeds Div 7, Div 22, and Div 30)	Medium
Geneva Study Area								
Geneva	0.5MG	3,231	Div 22 also serves Geneva Area due to intertie	At District shops. Some residences nearby	Deficient	Medium	Medium (646 ERUs served; can be served by SV tanks, but could impact nearby District shops)	Medium

Notes:
1) Individual zone populations were not included within the current Water System Plan. Therefore, study area population was given as reference.
2) Fire flow considerations: Per the WSP, the fire flows within the system are adequate for all tanks.

7. Recommended Priorities for Retrofit

Due to both the nearby residence and water system impact, the Division 7 reservoir will have the most impact should failure occur. The SVWTP reservoir has a very high impact on the water system, as it feeds the entire water system, and its storage reservoir is part of the treatment process. SVWTP feeds both the Division 7 and the Division 22 reservoirs; Division 7 in turn feeds Division 30, and Division 22 connects to the Geneva reservoir through the existing intertie.

One way to determine the priority of tank retrofits is to evaluate risk. Risk is typically determined as the probability of occurrence times the consequence of the event. The District uses Business Risk Exposure (BRE) as the term for risk and BRE is defined as:

BRE = Probability of Failure (PoF) x Consequence of Failure (CoF).

Probability of Failure is the probability that the reservoir will fail during the design earthquake and is defined by the ratings in Table 14.

Table 14 – Probability of Failure (PoF)

PoF Rating	Probability that facility will fail during design earthquake
1	0%
2	10%
3	20%
4	30%
5	40%
6	50%
7	60%
8	70%
9	80%
10	90%

Consequence of Failure is a rating that is defined by the item that failed (a component, facility, or system), the level of failure (minor, major, intermediate, significant, or total), and the percentage of the system that is affected. Table 15 provides the ratings for CoF.

Table 15 – Consequence of Failure (CoF)

CoF Rating	Description	Level Affected	Percent Affected
1	Minor Component Failure	Asset	0 - 25%
2	Major Component Failure	Asset	25 – 50%
3	Major Asset Failure	Asset	0 – 25%

Table 15 – Consequence of Failure (CoF)			
CoF Rating	Description	Level Affected	Percent Affected
4	Multiple Asset Failure	Facility / Sub-System	25 – 50%
5	Major Facility Failure	Facility	50 – 100%
6	Minor System Failure	Total System	20 – 40%
7	Medium System Failure	Total System	40 – 60%
8	Intermediate System Failure	Total System	60 – 80%
9	Significant System Failure	Total System	80 – 90%
10	Total System Failure	Total System	90 – 100%

ERUs can be used to define the percentage of the District affected and provide a rating for CoF. The PoF rating is estimated based on the seismic evaluation calculations and professional judgement. The resulting BRE values are shown in Table 16.

Table 16 – Business Risk Exposure (BRE)					
Reservoir	ERUs	Percentage of Total South Shore System	CoF Rating	PoF Rating	BRE
Division 30	1,158	29%	6	10	60
Division 22	1,782	45%	7	10	70
Division 7	2,153	55%	7	10	70
SVWTP	3,935	100%	10	7	70
Geneva	646	16%	5	10	50

Based on Tables 13 and 16, recommended retrofits in order of priority are:

Division 7 Reservoir. Given its importance, the fact it is unanchored, it has the highest probability of failure, and it has one of the highest consequences of failure, the Division 7 Reservoir is recommended as the highest priority for retrofit or replacement.

SVWTP Reservoir. This reservoir is less of a hazard than the Division 7 Reservoir, but is critical as the source for other reservoirs and as part of the treatment process. The SVWTP Reservoir also has the highest consequences of failure since it serves the greatest number of ERUs in the South Shore System. The SVWTP has a lower probability of failure than the Division 7 reservoir.

Division 22 Reservoir. This reservoir is recommended next in priority because it is unanchored and liable to failure, has a large storage volume, and would result in high neighborhood impact in case of failure.

Division 30 Reservoir. This is the smallest reservoir and its failure would remove service from higher elevation customers and cause damage to nearby residences in the event of collapse. It is not that this tank is unimportant, but the risks and consequences of failure are greater at the other sites.

Geneva Reservoir. The Geneva Reservoir serves the fewest customers and, in the event of failure, service could be provided from other tanks. Based on ERUs, the Geneva Reservoir has the lowest consequences of failure. Given its size and proximity to the District's maintenance facility, failure of this tank could seriously disrupt the District's ability to respond to other problems in the system in the event of an earthquake.

8. Retrofit Options and Costs

Following are descriptions and estimated costs for various alternative retrofit schemes. These are very preliminary and are based on approximate sizing of major elements, with allowances for miscellaneous associated work. Detailed estimate spreadsheets are provided in Appendix A.1. Cost estimates are planning level and include sales tax, an allowance for design, permitting, inspection, and construction administration, plus a contingency.

The opinion of probable construction cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design matures. BHC Consultants has no control over: variances in the cost of labor, materials, equipment; cost for services provided by others; contractor's means and methods of executing the work or of determining prices; nor, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

8.1 Geneva Reservoir

Table 17 summarizes problems and possible solutions at the Geneva Reservoir, followed by discussion and estimated cost.

Table 17 – Geneva Reservoir Retrofit Options

Problem	Possible Solution	Positives	Negatives
Excessive seismic forces	Reduce water level	Least cost	May be operationally unacceptable
Inadequate anchorage and foundation capacity	Alternate A Provide supplementary external ringwall attached to shell with studs	• Less expensive than anchor chairs and bolts • Less excavation than other ringwall enlargements since most of new foundation is above grade	• May require relocation of shell manhole • Reduces access around tank more than other alternatives • May be aesthetically objectionable • Requires relocation of valves/piping
	Alternate B Provide supplementary external ringwall with	Supplemental ringwall can be constructed with minimal encroachment	• More excavation than previous alternative if

Table 17 – Geneva Reservoir Retrofit Options			
Problem	Possible Solution	Positives	Negatives
	new anchor bolts and chairs	above grade. Manway access not impacted.	part of new ringwall is above grade • Requires relocation of valves/piping
	Alternate C Provide supplementary external ringwall with new anchor bolts and chairs, ground anchors or micropiles	Minimum width and volume required for added ringwall. Minimal encroachment above and below grade	• Requires relocation of valves/piping • More expensive than previous alternatives • Requires geotechnical input to confirm feasibility
	Alternate D Provide supplemental internal bottom mat attached to shell with studs	No external encroachment or excavation required	• Reduces total storage • Requires partial shell removal and replacement for efficient construction access
Lack of piping flexibility	Provide force balanced Flex-tend couplings	Proven technology	Costly

8.1.1 Reducing Water Level

By reducing the maximum operating level from the existing 28 to 31.5 feet to a maximum of 14 feet, the tank would be stable even if the anchors fail; however, the piping connections would still be at risk. The maximum operating pressure would drop by around 8 psi and the storage volume would be reduced to 44 percent of existing. One of the consequences of the tank becoming unanchored is an increase in base shear and overturning moment.

8.1.2 Anchorage and Foundation Enhancements

Alternate A – External Ringwall Above and Below Grade

This alternate includes construction of an external reinforced concrete ring about 13 feet high and 11 feet wide at the base, with the base founded at the same elevation as the existing ringwall, connected to the existing shell with welded stud anchors and to the existing ringwall with dowels (see Figure 43).

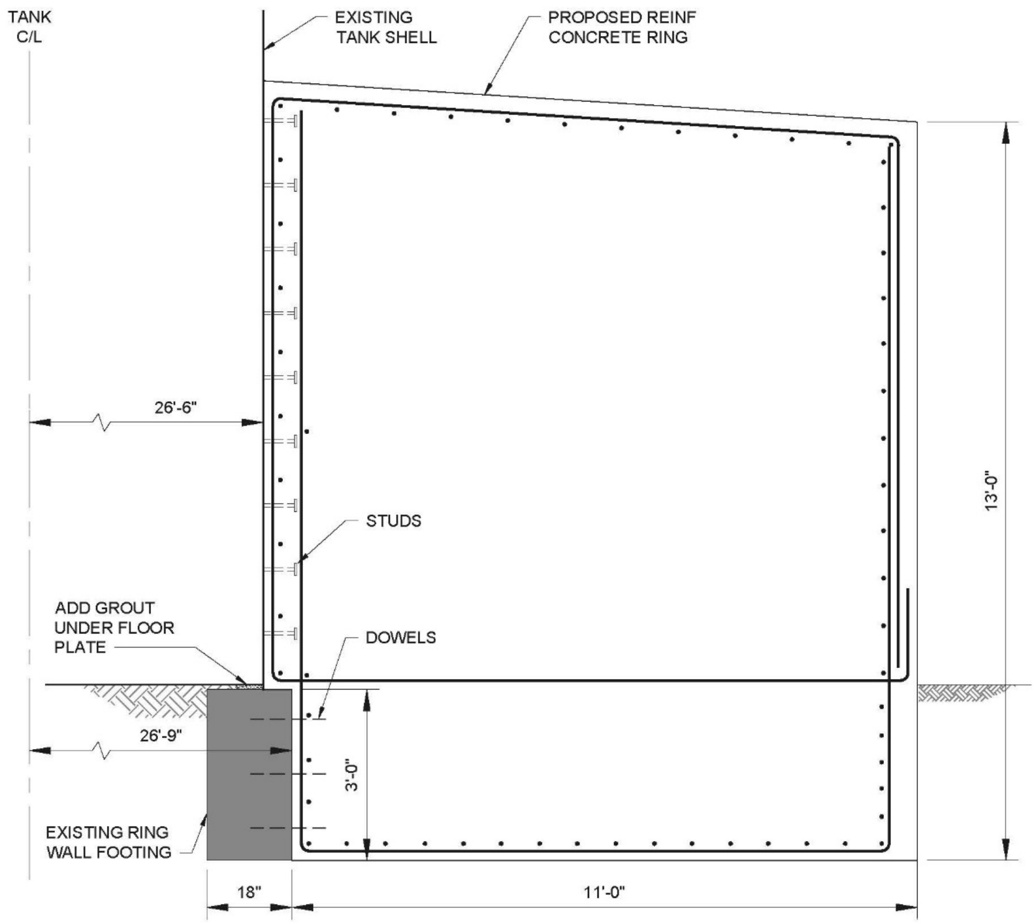


Figure 43 External Ringwall Above and Below Grade

The new ring would cover the existing shell manway, requiring relocation of the manway above the new ring and construction of internal ladder and handholds at the manway. This option would involve 185 cubic yards of concrete and 451 cubic yards of excavation and would cost approximately $664,000.

Alternate B – Below Grade External Ringwall with New Anchor Bolts and Chairs

This alternate would require no less concrete than Alternate A, and would require much wider excavation to avoid undermining the existing ringwall. Because of added excavation costs and the added costs of anchor bolts and chairs, it is not considered a practical option.

Alternate C - Supplementary External Ringwall with Anchor Bolts and Ground Anchors

This alternate would require only 49 cubic yards of concrete and 250 cubic yards of excavation. It would require 36 new anchor chairs and bolts, and 18 ground anchors. The exact details of the ground anchors will depend on recommendations of the geotechnical engineer at the time of design. For estimating purposes, post-tensioned thread bars have been assumed. The estimated cost would be approximately $505,000. Figure 44 shows the general configuration.

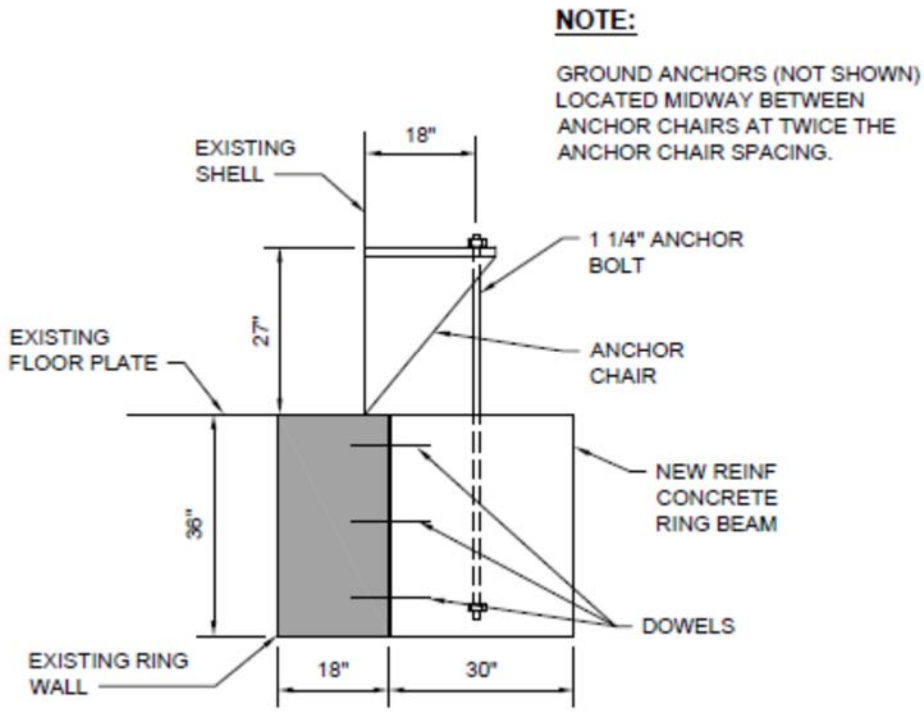


Figure 44 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

Alternate D - Supplemental Internal Bottom Mat Attached to Shell with Studs

This approach constructs a reinforced concrete mat foundation above the floor of the tank which is anchored to the shell wall with steel studs. The mat foundation results in somewhat greater overturning moment but mobilizes all of the weight of water in the tank to help resist overturning. It is simple to construct. Interior work typically requires temporary removal of a portion of the bottom course(s) to facilitate construction. A new steel floor plate is usually installed over the concrete mat. This alternative would require a mat about 24 inches thick, with a ¼ inch cover plate, concrete volume of 163 cubic yards, 13,200 lbs. of rebar, and 22,500 lbs. of steel plate, as shown in Figure 45. It would not require any exterior excavation except for new pipe fittings, if new flexible joints are to be installed. About 33,000 gallons of storage volume would be lost at the base of the tank. It would cost approximately $712,000.

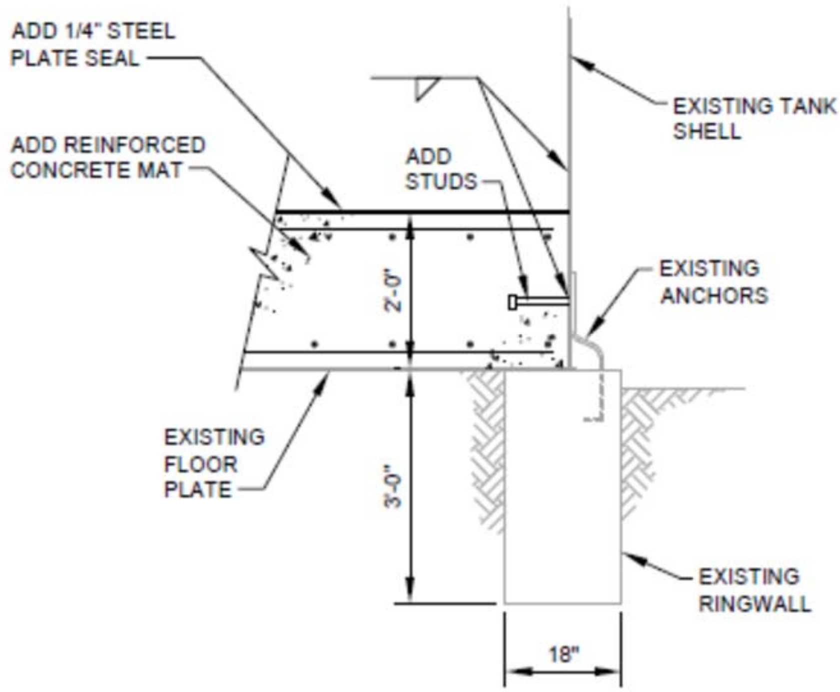


Figure 45 Supplemental Internal Bottom Mat Attached to Shell with Studs

8.1.3 Recommended Retrofit Option

Option C, an anchored external ringwall, is the least expensive and intrusive alternative, and is the recommended retrofit approach for the Geneva Reservoir at an estimated approximate project cost of $505,000.

8.2 Division 22 Reservoir

Table 18 summarizes problems and possible solutions at the Division 22 Reservoir, followed by discussion and estimated cost.

Table 18 – Division 22 Reservoir Retrofit Options			
Problem	Possible Solution	Positives	Negatives
Excessive seismic forces	Reduce water level	• Least cost	• Operationally unacceptable

Table 18 – Division 22 Reservoir Retrofit Options

Problem	Possible Solution	Positives	Negatives
No anchorage and limited foundation capacity	Alternate A Provide supplementary external ringwall attached to shell with studs	• Less expensive than anchor chairs and bolts • Less excavation than other ringwall enlargements since most of new foundation is above grade	• May require relocation of shell manhole • Reduces access around tank more than other alternatives • May be aesthetically objectionable • Requires relocation of valves/piping
	Alternate B Provide supplementary external ringwall with new anchor bolts and chairs	Supplemental ringwall can be constructed with minimal encroachment above grade	• More excavation than previous alternative if part of new ringwall is above grade • Requires relocation of valves/piping
	Alternate C Provide supplementary external ringwall with new anchor bolts and chairs, ground anchors or micropiles	Minimum width and volume required for added ringwall. Minimal encroachment above and below grade	• Requires relocation of valves/piping • More expensive than previous alternatives • Requires geotechnical input to confirm feasibility
	Alternate D Provide supplemental internal bottom mat attached to shell with studs	No external encroachment or excavation required	• Reduces total storage • Requires partial shell removal and replacement for efficient construction access
Excessive retrofit cost	Demolish tank and increase size of proposed companion tank to include existing tank volume	Avoids spending money on an aging facility Makes space available for other purposes	• Delays in risk reduction • Removes the flexibility of having two adjacent tanks
Lack of piping flexibility	Provide force balanced Flex-tend couplings	Proven technology	Costly

8.2.1 Reducing Water Level

By reducing the maximum operating level from the existing 33.5 feet to a maximum of 15 feet, the tank would be stable as an unanchored tank; however, the piping connections would still be at risk. The maximum operating pressure would drop by around 8 psi and the storage volume would be reduced to 45 percent of existing.

8.2.2 Anchorage and Foundation Enhancements

Alternate A – External Ringwall Above and Below Grade

This alternate includes construction of an external reinforced concrete ring about 10 feet high and 2 feet wide at the base, with the base founded at the same elevation as the existing ringwall, connected to the existing shell with welded stud anchors and to the existing ringwall with dowels (see Figure 46). The reason this ring configuration has such a high height to width ratio is to provide adequate contact area between the steel shell and ring for stud placement.

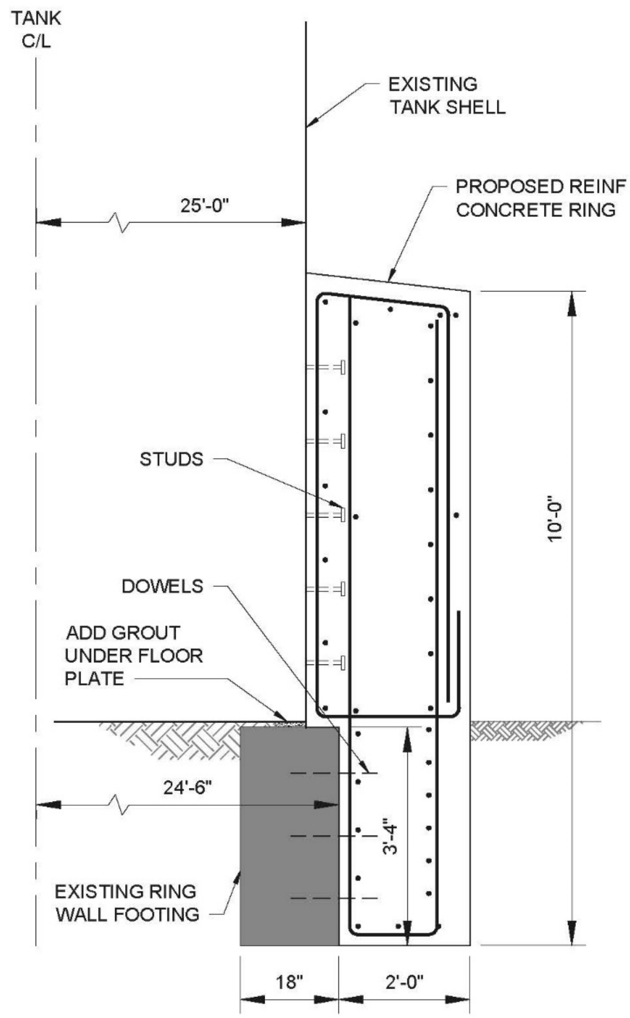


Figure 46 External Ringwall Above and Below Grade

The new ring would cover the existing shell manway, requiring either a "tunnel" through the new ring to the existing manway, or relocation of the manway above the new ring and construction of internal ladder and handholds at the manway. This option would involve 56 cubic yards of concrete and 249 cubic yards of excavation and would cost approximately $367,000.

Alternate B – Below Grade External Ringwall with New Anchor Bolts and Chairs

This alternate would require no less concrete than Alternate A, and would require much wider excavation to avoid undermining the existing ringwall. Because of added excavation costs and the added costs of anchor bolts and chairs, it is not considered a practical option.

Alternate C - Supplementary External Ringwall with Anchor Bolts and Ground Anchors

This alternate would require only 51 cubic yards of concrete and 274 cubic yards of excavation. It would require 36 new anchor chairs and bolts, and 18 ground anchors. The estimated cost would be approximately $478,000. Figure 47 shows the general configuration.

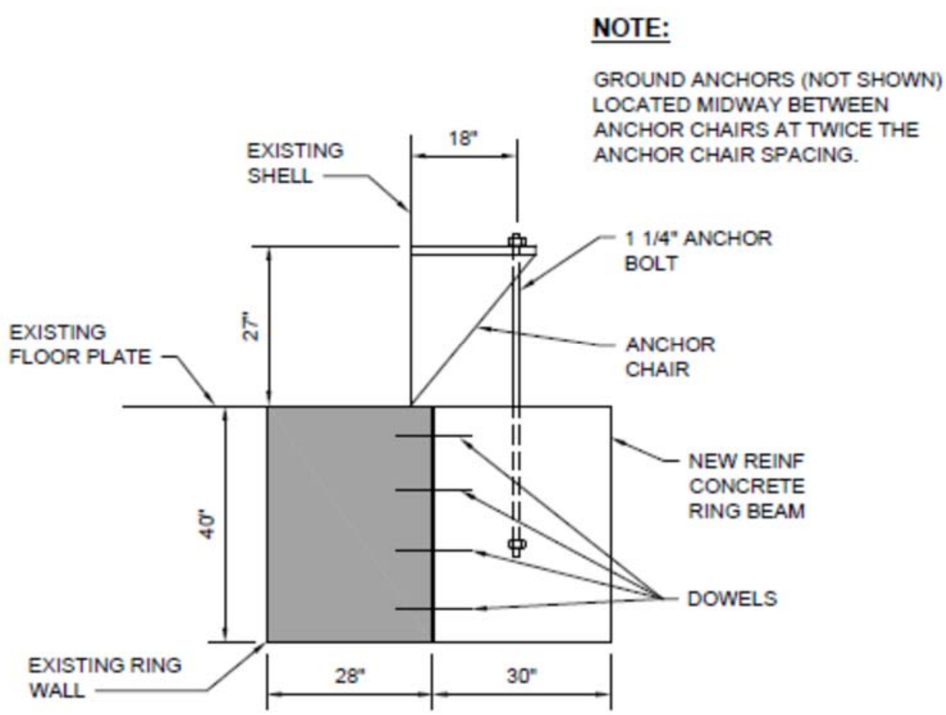


Figure 47 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

Alternate D - Supplemental Internal Bottom Mat Attached to Shell with Studs

This approach constructs a reinforced concrete mat foundation above the floor of the tank which is anchored to the shell wall with steel studs. The mat foundation results in somewhat greater overturning moment but mobilizes all of the weight of water in the tank to help resist overturning. It is simple to construct. Interior work typically requires temporary removal of a portion of the bottom course(s) to facilitate construction. A new steel floor plate is usually installed over the concrete mat (see Figure 48). This alternative would require a mat about 30 inches thick, with a ¼ inch cover plate, concrete volume of 182 cubic yards, 9,600 lbs. of rebar, and 20,000 lbs. of steel plate. It would not require any exterior excavation except for new pipe fittings, if new flexible joints are to be installed. About 36,800 gallons of storage volume would be lost at the base of the tank. It would cost approximately $710,000.

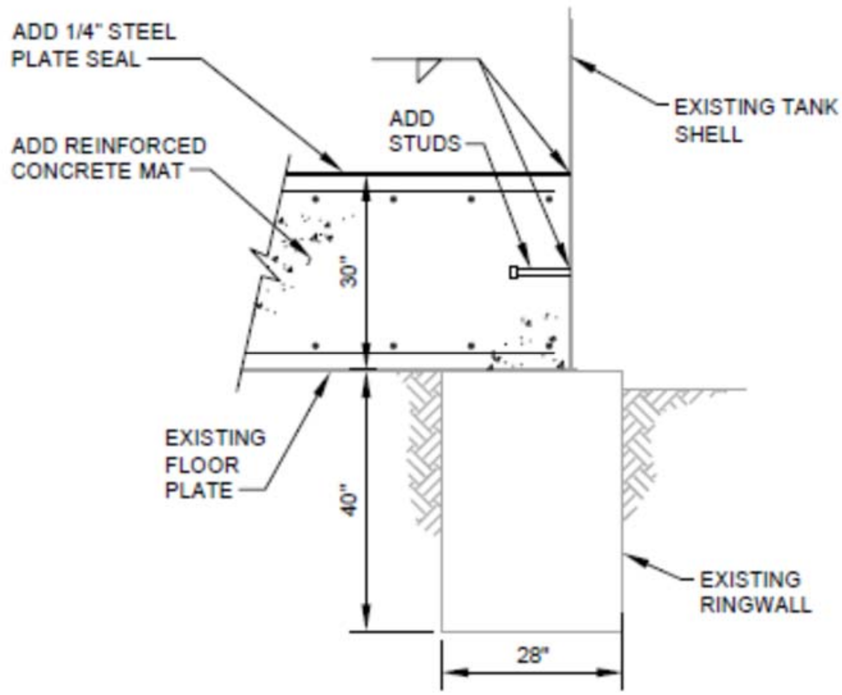


Figure 48 Supplemental Internal Bottom Mat Attached to Shell with Studs

8.2.3 Upsize Proposed Companion Tank and Demolish Existing

As previously discussed, a new reservoir near the existing one has been proposed with a capacity of 500,000 gallons and a diameter of approximately 50 feet. Doubling the capacity of the proposed tank to 1.0 MG would allow demolition of the existing tank without a reduction in total capacity once the new tank is built. The diameter of the tank would have to increase to 71 feet assuming the elevation of the floor and maximum operating levels match the existing. The additional cost to the new project, including demolition of the old reservoir would be approximately $661,000.

8.2.4 Recommended Retrofit Option

Alternate A, the addition of an external gravity ringwall collar, is the least expensive and recommended option at an approximate estimated project cost of $367,000.

8.3 Division 7 Reservoir

Table 19 summarizes problems and possible solutions at the Division 7 Reservoir, followed by discussion and estimated cost.

Table 19 – Division 7 Retrofit Options

Problem	Possible Solution	Positives	Negatives
Excessive seismic forces and shell stresses	Reduce water level	Least cost	Operationally unacceptable
Excessive shell hoop stress	Reinforce shell with new plate or ring girders	Allows continued use of tank	Expensive
Excessive shell longitudinal stress	Add vertical stiffeners or see if new plating solves the problem	Allows continued use of tank	Expensive
No anchorage and limited foundation capacity	Alternate A Provide supplementary external ringwall attached to shell with studs	• Less expensive than anchor chairs and bolts • Less excavation than other ringwall enlargements since most of new foundation is above grade	• May require relocation of shell manhole • Reduces access around tank more than other alternatives • Requires relocation of valves/piping
	Alternate B Provide supplementary external ringwall with new anchor bolts and chairs	Supplemental ringwall can be constructed with minimal encroachment above grade	• More excavation than previous alternative if part of new ringwall is above grade • Requires relocation of valves/piping
	Alternate C Provide supplementary external ringwall with new anchor bolts and chairs, ground anchors or micropiles	Minimum width and volume required for added ringwall. Minimal encroachment above and below grade	• Requires relocation of valves/piping • More expensive than previous alternatives. • Requires geotechnical input to confirm feasibility
	Alternate D Provide supplemental internal bottom mat attached to shell with studs	No external encroachment or excavation required	• Reduces total storage • Requires partial shell removal and replacement for efficient construction access

Table 19 – Division 7 Retrofit Options			
Problem	Possible Solution	Positives	Negatives
Excessive retrofit cost considering age of tank	Replace with new tank	Longer design life tank meeting current standards	• May not be feasible due to cost • Requires site acquisition, additional piping if existing tank must stay in service until tank is replaced
Lack of piping flexibility	Provide force balanced Flex-tend couplings	Proven technology	Costly

8.3.1 Reducing Water Level

By reducing the maximum operating level from the existing 33.5 feet to a maximum of 23.5 feet, overstresses in the shell would be eliminated, but the tank would still not be as stable as an unanchored tank. Maximum tank operating pressure would be reduced by 4.4 psi and the volume reduced to 70 percent of existing.

For the tank to be stable without anchorage, the maximum operating level would have to be further reduced to a maximum of 17.5 feet, for a total reduction in tank operating pressure of 7 psi and a volume reduction to 52 percent of existing. Piping connections would still be at risk.

8.3.2 Hoop and Longitudinal Overstress

Bringing the hoop stress down to acceptable levels would require reinforcing the existing shell with a 3/16" thick layer of steel plate or its equivalent from its base to about 20 feet above the base (bottom three shell courses.) The shell would not require vertical stiffeners if the shell plate is reinforced as described above. This work would be required as a prerequisite to anchorage and foundation improvements and is included in the three retrofit options examined.

8.3.3 Anchorage and Foundation Enhancements

Alternate A – External Ringwall Above and Below Grade

This alternate includes construction of an external reinforced concrete ring about 7 feet high and 3 feet wide at the base, with the base founded at the same elevation as the existing ringwall, connected to the existing shell with welded stud anchors and to the existing ringwall with dowels. (See Figure 49)

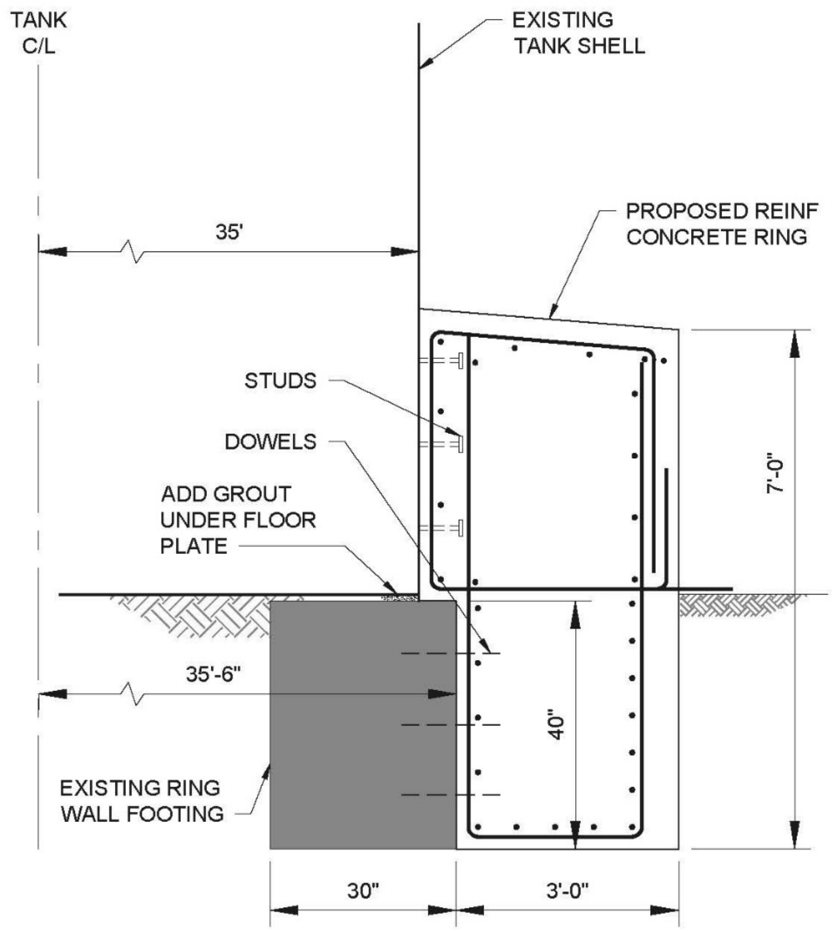


Figure 49 Alternate A – Division 7 Reservoir

The new ring would cover the existing shell manway, requiring either a "tunnel" through the new ring to the existing manway, or relocation of the manway above the new ring and construction of internal ladder and handholds at the manway. This option would involve 101 cubic yards of concrete and 402 cubic yards of excavation (see Figure 48) and would cost approximately $721,000.

Alternate B – Below Grade External Ringwall with New Anchor Bolts and Chairs

This alternate would require no less concrete than Alternate A, and would require much wider excavation to avoid undermining the existing ringwall. Because of added excavation costs and the added costs of anchor bolts and chairs, it is not considered a practical option.

Alternate C - Supplementary External Ringwall with Anchor Bolts and Ground Anchors

This alternate would require only 71 cubic yards of concrete and 370 cubic yards of excavation. It would require 40 new anchor chairs and bolts, and 20 ground anchors. The estimated cost is $803,000. Figure 50 shows the general configuration.

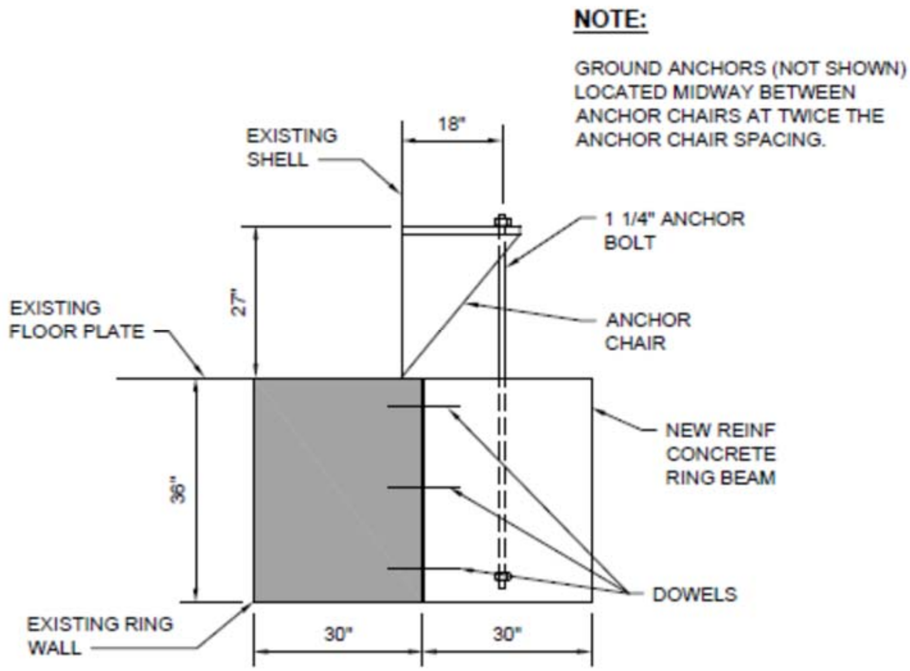


Figure 50 Alternate C – Division 7 Reservoir

Alternate D - Supplemental Internal Bottom Mat Attached to Shell with Studs

This approach constructs a reinforced concrete mat foundation above the floor of the tank which is anchored to the shell wall with steel studs. The mat foundation results in somewhat greater overturning moment but mobilizes all of the weight of water in the tank to help resist overturning. It is simple to construct. Interior work typically requires temporary removal of a portion of the bottom course(s) to facilitate construction. A new steel floor plate is usually installed over the concrete mat (see Figure 51). This alternative would require a mat about 2'-8" thick, with a ¼ inch cover plate, concrete volume of 381 cubic yards, 22,380 lbs. of rebar, and 39,286 lbs. of steel plate. It would not require any exterior excavation except for new pipe fittings, if new flexible joints are to be installed. About 76,865 gallons of storage volume would be lost at the base of the tank. It would cost approximately $1,496.000.

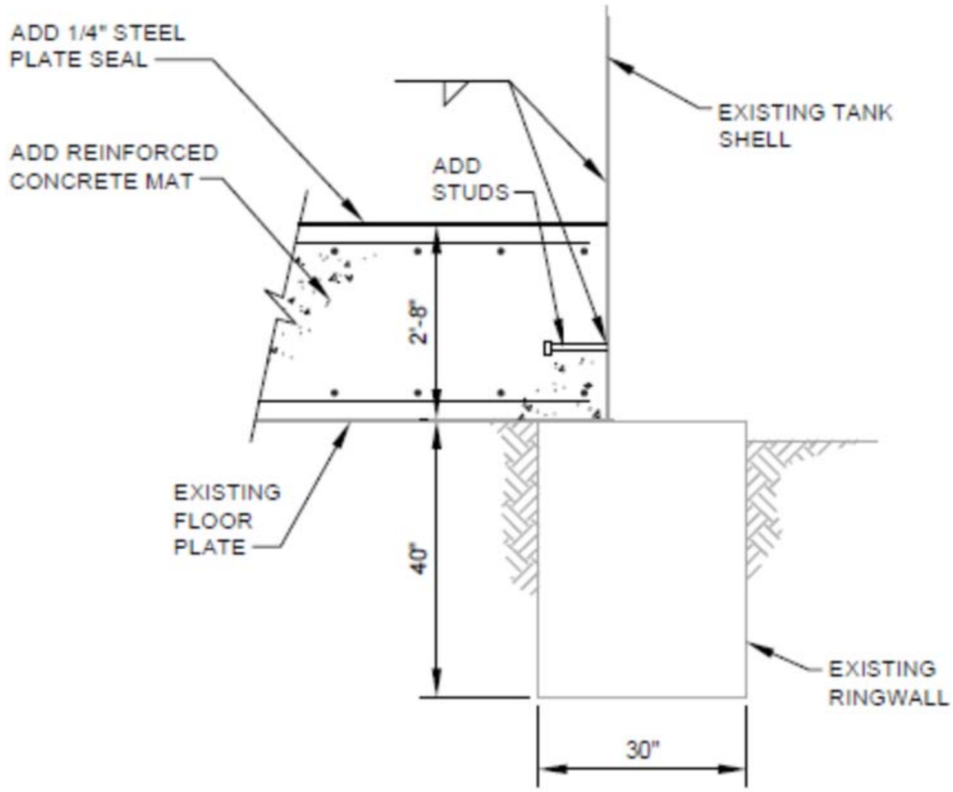


Figure 51 Supplemental Internal Bottom Mat Attached to Shell with Studs

8.3.4 Demolish and Replace Tank

As an alternate to retrofit, the existing tank could be demolished and replaced for a cost on the order of $1.8 million, not counting any temporary cost associated with providing water service with the tank off-line. Alternately, a new tank in the same pressure zone could be constructed at an adjacent site, but would involve additional permitting and property acquisition costs.

8.3.5 Recommended Retrofit Option

A supplemental, external ringwall is the recommended retrofit option at the Division 7 Reservoir at an estimated approximate project cost of $721,000. This retrofit also includes supplemental shell plates to resolve issues with overstress.

8.4 Division 30 Reservoir

Table 20 summarizes problems and possible solutions at the Division 30 Reservoir, followed by discussion and estimated cost.

Table 20 – Division 30 Retrofit Options

Problem	Possible Solution	Positives	Negatives
Excessive seismic forces	Reduce water level	Least cost	Operationally unacceptable
Inadequate anchorage and foundation capacity	Alternate A Provide supplementary external ringwall attached to shell with studs	• Less expensive than anchor chairs and bolts • Less excavation than other ringwall enlargements since most of new foundation is above grade	• May require relocation of shell manhole • Requires relocation of valves/piping
	Alternate B Provide supplementary external ringwall with new anchor bolts and chairs	Supplemental ringwall can be constructed with minimal encroachment above grade	• More excavation than previous alternative if part of new ringwall is above grade • Requires relocation of valves/piping
	Alternate C Provide supplementary external ringwall with new anchor bolts and chairs, ground anchors or micropiles	Minimum width and volume required for added ringwall. Minimal encroachment above and below grade	• Requires relocation of valves/piping • More expensive than previous alternatives • Requires geotechnical input to confirm feasibility
Lack of piping flexibility	Provide force balanced Flex-tend couplings	Proven technology	Costly

8.4.1 Reducing Water Level

By reducing the maximum operating level from the existing 39.3 feet to a maximum of 21 feet, overstresses in the anchorage would be eliminated, but the tank would still require modification to the foundation to prevent uplift. Maximum tank operating pressure would be reduced by 8 psi and the volume reduced to 53 percent of existing.

For the tank to be stable against uplift, the maximum operating level would have to be further reduced to a maximum of around 10 feet or less, for a total reduction in tank operating pressure of 13 psi and a volume reduction to 25 percent of existing. Piping connections would still be at risk.

For the tank to function without anchorage, the maximum operating level would have to drop to around 9.5 feet.

8.4.2 Anchorage and Foundation Enhancements

Alternate A – External Ringwall Above and Below Grade

This alternate includes construction of an external reinforced concrete ring about 10 feet high and 8 feet wide at the base, with the base founded at the same elevation as the existing ringwall, connected to the existing shell with welded stud anchors and to the existing ringwall with dowels (see Figure 52).

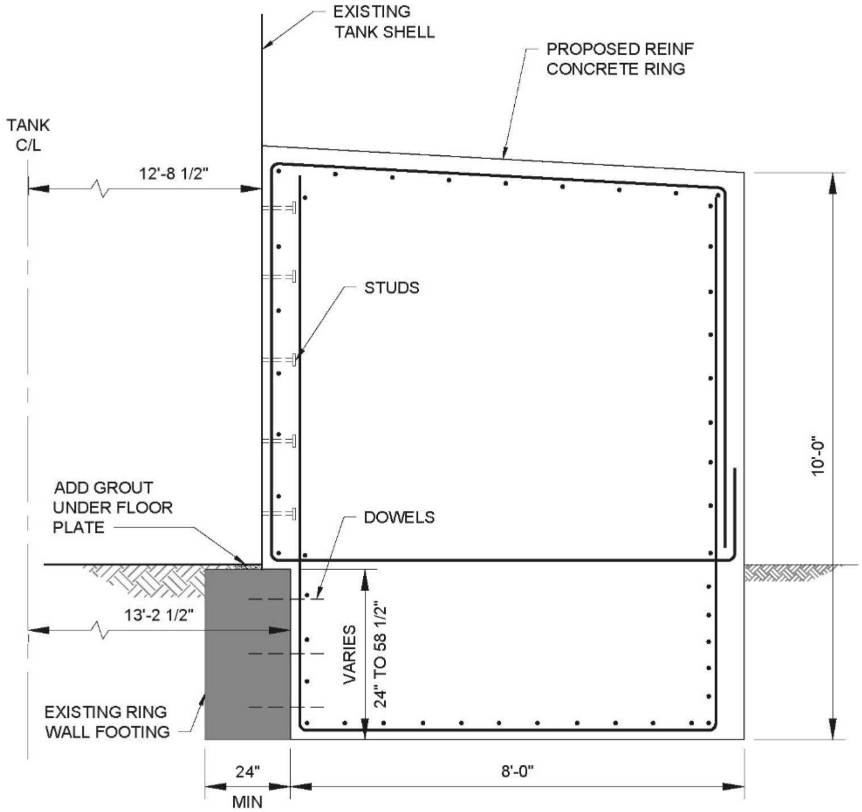


Figure 52 External Ringwall Above and Below Grade

The new ring would cover the existing shell manway, requiring either a "tunnel" through the new ring to the existing manway, or relocation of the manway above the new ring and construction of internal ladder and handholds at the manway. This option would involve 124 cubic yards of concrete and 353 cubic yards of excavation and would cost approximately $473,000.

Alternate B – Below Grade External Ringwall with New Anchor Bolts and Chairs

This alternate would require no less concrete than Alternate A, and would require much wider excavation to avoid undermining the existing ringwall. Because of added excavation costs and the added costs of anchor bolts and chairs, it is not considered a practical option.

Alternate C - Supplementary External Ringwall with Anchor Bolts and Ground Anchors

This alternate would require only 42 cubic yards of concrete and 273 cubic yards of excavation. It would require 36 new anchor chairs and bolts, and 18 ground anchors, probably drilled into rock. The estimated cost would be approximately $541,000. Figure 53 shows the general configuration.

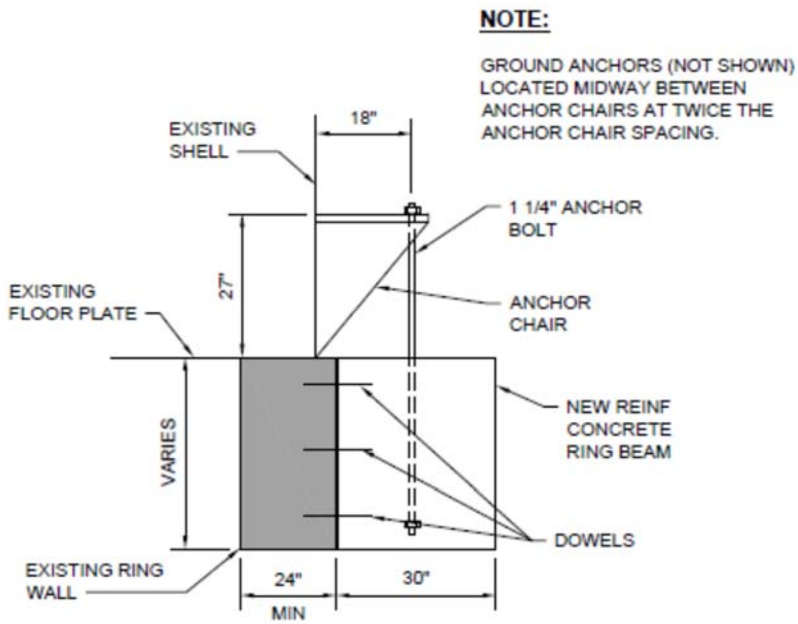


Figure 53 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

Alternate D - Supplemental Internal Bottom Mat Attached to Shell with Studs

Installing an interior concrete mat is not a feasible option. Although the mat provides a counterweight to tipping forces, as the mat thickness increases to provide more weight, the seismic forces on the mat increase faster than the counterbalancing weight (see Figure 54) and the tank uplifts. In this case, the minimum uplift would occur with mat about 12 feet thick, but there would still be uplift and the tank would rock, probably leading to tipping. Storage volume would be reduced to about 100,000 gallons at the optimum mat thickness; however, since uplift is not prevented, this alternative is not acceptable.

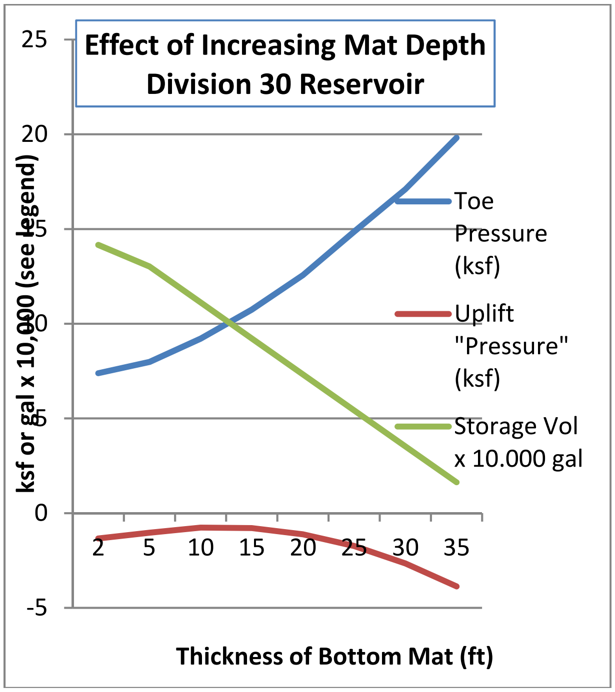


Figure 54 Effect of Increasing Mat Depth

8.4.3 *Recommended Retrofit Option*

The recommended retrofit option for this reservoir is Alternate C, the anchored supplemental ringwall. Although Alternate A may appear less expensive at first glance, the unit price for concrete could be substantially higher than assumed generally due to the remoteness and elevation of the site. Alternate A would also involve very poor shell manway access. The estimated approximate project cost for this retrofit option is $541,000.

8.5 *SVWTP Reservoir*

The shell, foundation, and anchorage appear to be adequate for predicted seismic loading except for insufficient uplift resistance of the foundation. The hold-down deficit can be matched by a widened ringwall without using ground anchors or mat concepts.

Table 21 summarizes problems and possible solutions at the SVWTP Reservoir, followed by discussion and estimated cost.

Table 21 – SVWTP Retrofit Options

Problem	Possible Solution	Positives	Negatives
Excessive seismic forces	Reduce water level	Least cost	Operationally unacceptable due to loss of storage and reduced chlorine detention time
Inadequate foundation uplift resistance	Provide supplementary external ringwall attached to existing ringwall with dowels	• Simple and relatively low cost • Tank can remain in service during construction • No reduction in storage volume or detention time	• Proximity to other structures limits access for construction and new foundation • Requires relocation of valves/piping.
Lack of piping flexibility	Provide force balanced Flex-tend couplings	Proven technology	Costly

8.5.1 Reducing Water Level

To prevent foundation uplift, the maximum operating level would have to be reduced from its current level of 22 feet to 18 feet or less. This would result in an operating pressure loss of nearly 2 psi, and a reduction in storage volume and chlorine contact time to 82 percent of existing.

8.5.2 Adding Ballast to Existing Ringwall

This alternate includes construction of an external reinforced concrete ringwall 6 feet high and 18 inches wide at the base, with the base founded at the same elevation as the existing ringwall, connected to the existing ringwall with dowels (see Figure 55).

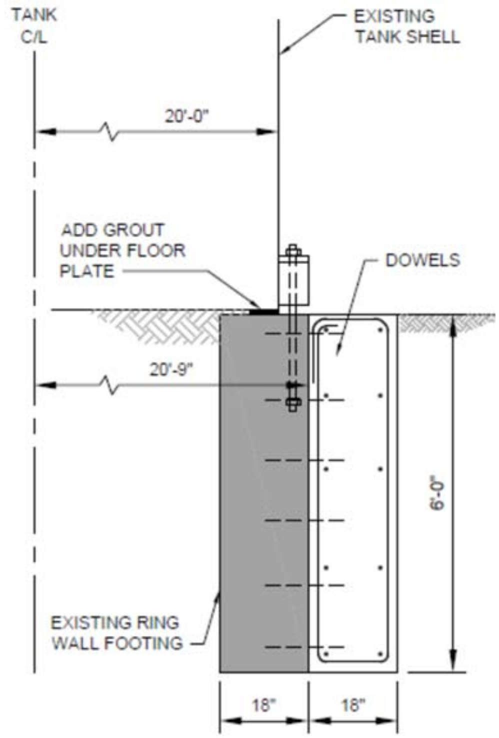


Figure 55 Added Ballast to Existing Ringwall

The new ring would not cover the existing shell manways or impact other appurtenances. This option would involve 58 cubic yards of concrete and 549 cubic yards of excavation and would cost approximately $156,000 and is the recommended retrofit approach.

APPENDIX A.1

COST ESTIMATES

Lake Whatcom Water and Sewer District
Reservoir Seismic Vulnerability Assessment
Preliminary Engineer's Opinion of Probable Project Costs

Prepared by: J. Lutz
Reviewed by: J. Gross
5 February 2016

Item	Unit	Unit Cost	Geneva Reservoir Option A Gravity Ring		Geneva Reservoir Option C Anchored Ring		Geneva Reservoir Option D Internal Mat	
			Qty	Cost	Qty	Cost	Qty	Cost
Reinforced concrete	CY	925	185	171125	49	45325	163	150775
Headed studs, 5/8"	EA	100	343	34300	0	0	343	34300
Dowels, #6	EA	50	167	8350	108	5400	0	0
Excavation	BCY	50	451	22550	230	11500	0	0
Backfill	BCY	50	312	15600	201	10050	0	0
Remove and seal shell manway	EA	5000	1	5000	0	0	1	5000
New 30" shell manway	EA	15000	1	15000	0	0	1	15000
New 24" shell manway	EA	10000	1	10000	0	0	1	10000
Replace depth gauge	LS	2000	1	2000	0	0	0	0
Relocate electrical panel & conduit	LS	10000	1	10000	1	10000	0	0
Replace/reconfigure exterior ladder	LS	20000	1	20000	0	0	0	0
Erosion control	LS	5000	1	5000	1	5000	0	0
Traffic control	LS	5000	1	5000	1	5000	1	5000
Site grading and seeding	SY	5	241	1205	83	415	0	0
Anchor Bolts	EA	400	0	0	36	14400	0	0
Anchor Chairs	LB	4	0	0	9576	38304	0	0
Ground anchors	EA	5000	0	0	18	90000	0	0
Welding for new anchors or seal plate	LF	20	0	0	204	4080	591	11820
Steel plate seal, 1/4"	LB	4	0	0	0	0	22521	90084
Cut out door sheet & reinforce	LS	15000	0	0	0	0	1	15000
Replace door sheet	LS	20000	0	0	0	0	1	20000
Add shell plate	LB	4	2870	11480	2870	11480	2870	11480
8" gate valve	EA	3100	0	0	0	0	0	0
10" gate valve	EA	5400	1	5400	1	5400	1	5400
12" gate va;ve	EA	7200	1	7200	1	7200	1	7200
8" double ball coupling	EA	5900	0	0	0	0	0	0
10" double ball coupling	EA	7400	1	7400	1	7400	1	7400
12" double ball coupling	EA	9900	1	9900	1	9900	1	9900
8" water line	EA	110	0	0	0	0	0	0
10" water line	EA	130	12	1560	8	1040	0	0
12" water line	EA	175	12	2100	8	1400	0	0
Field coating prep and repair exterior	SF	10	343	3430	360	3600	343	3430
Field coating prep and repair interior	SF	16	343	5488	108	1728	343	5488
Mobilization at percentage times previous items	PCT	6%	379088	22745	288622	17317	407277	24437
General Conditions at a percentage of previous items	PCT	10%	401833	40183	305939	30594	431714	43171
Pretax Construction Subtotal				442017		336533		474885
Sales Tax	PCT	8.70%		38455		29278		41315
Estimated Bid without Contingency				480472		365812		516200
Estimating Contingency	PCT	20%		96094		73162		103240
Estimated Construction Cost (nearest $1000)				**$577,000**		**$439,000**		**$619,000**
Engineering, Permitting, Legal and Admin	PCT	15%		86550		65850		92850
Estimated Project Cost (nearest $1000)				**$664,000**		**$505,000**		**$712,000**

Item	Division 22 Reservoir Option A Gravity Ring		Division 22 Reservoir Option C Anchored Ring		Division 22 Reservoir Option D Internal Mat	
	Qty	Cost	Qty	Cost	Qty	Cost
Reinforced concrete	56	51800	52	48100	182	168350
Headed studs, 5/8"	215	21500	0	0	215	21500
Dowels, #6	257	12850	108	5400	0	0
Excavation	249	12450	274	13700	0	0
Backfill	208	10400	222	11100	0	0
Remove and seal shell manway	1	5000	0	0	0	0
New 30" shell manway	1	15000	0	0	1	15000
New 24" shell manway	1	10000	0	0	1	10000
Replace depth gauge	1	2000	0	0	1	2000
Relocate electrical panel & conduit	1	10000	1	10000	0	0
Replace/reconfigure exterior ladder	1	20000	0	0	0	0
Erosion control	1	5000	1	5000	0	0
Traffic control	1	5000	1	5000	1	5000
Site grading and seeding	128	640	137	685	0	0
Anchor Bolts	0	0	36	14400	0	0
Anchor Chairs	0	0	9576	38304	0	0
Ground anchors	0	0	18	90000	0	0
Welding for new anchors or seal plate	0	0	204	4080	567	11340
Steel plate seal, 1/4"	0	0	0	0	20044	80176
Cut out door sheet & reinforce	0	0	0	0	1	15000
Replace door sheet	0	0	0	0	1	20000
Add shell plate	0	0	0	0	0	0
8" gate valve	1	3100	1	3100	1	3100
10" gate valve	0	0	0	0	0	0
12" gate va;ve	1	7200	1	7200	1	7200
8" double ball coupling	0	0	0	0	0	0
10" double ball coupling	0	0	0	0	0	0
12" double ball coupling	1	9900	1	9900	1	9900
8" water line	8	880	8	880	0	0
10" water line	0	0	0	0	0	0
12" water line	8	1400	8	1400	0	0
Field coating prep and repair exterior	215	2150	360	3600	215	2150
Field coating prep and repair interior	215	3440	108	1728	2178	34848
Mobilization at percentage times previous items	209710	12583	273577	16415	405564	24334
General Conditions at a percentage of previous items	222293	22229	289992	28999	429898	42990
Pretax Construction Subtotal		244522		318991		472888
Sales Tax		21273		27752		41141
Estimated Bid without Contingency		265795		346743		514029
Estimating Contingency		53159		69349		102806
Estimated Construction Cost (nearest $1000)		**$319,000**		**$416,000**		**$617,000**
Engineering, Permitting, Legal and Admin		47850		62400		92550
Estimated Project Cost (nearest $1000)		**$367,000**		**$478,000**		**$710,000**

Item	Division 7 Reservoir Option A Gravity Ring		Division 7 Reservoir Option C Anchored Ring		Division 7 Reservoir Option D Internal Mat	
	Qty	Cost	Qty	Cost	Qty	Cost
Reinforced concrete	101	93425	71	65675	381	352425
Headed studs, 5/8"	247	24700	0	0	247	24700
Dowels, #6	417	20850	120	6000	0	0
Excavation	402	20100	370	18500	0	0
Backfill	316	15800	299	14950	0	0
Remove and seal shell manway	1	5000	0	0	0	0
New 30" shell manway	1	15000	0	0	1	15000
New 24" shell manway	1	10000	0	0	1	10000
Replace depth gauge	1	2000	0	0	1	2000
Relocate electrical panel & conduit	1	10000	1	10000	0	0
Replace/reconfigure exterior ladder	1	20000	0	0	0	0
Erosion control	1	5000	1	5000	0	0
Traffic control	1	5000	1	5000	0	0
Site grading and seeding	0	0	0	0	0	0
Anchor Bolts	0	0	40	16000	0	0
Anchor Chairs	0	0	12080	48320	0	0
Ground anchors	0	0	20	100000	0	0
Welding for new anchors or seal plate	0	0	260	5200	0	0
Steel plate seal, 1/4"	0	0	0	0	39286	157144
Cut out door sheet & reinforce	0	0	0	0	1	15000
Replace door sheet	0	0	0	0	1	20000
Add shell plate	33690	134760	33690	134760	33690	134760
8" gate valve	1	3100	1	3100	1	3100
10" gate valve	1	5400	1	5400	1	5400
12" gate va;ve	0	0	0	0	0	0
8" double ball coupling	1	5900	1	5900	1	5900
10" double ball coupling	1	7400	1	7400	1	7400
12" double ball coupling	0	0	0	0	0	0
8" water line	9	990	8	880	0	0
10" water line	9	1170	8	1040	0	0
12" water line	0	0	0	0	0	0
Field coating prep and repair exterior	247	2470	400	4000	247	2470
Field coating prep and repair interior	247	3952	120	1920	6251	100016
Mobilization at percentage times previous items	412017	24721	459045	27543	855315	51319
General Conditions at a percentage of previous items	436738	43674	486588	48659	906634	90663
Pretax Construction Subtotal		480412		535246		997297
Sales Tax		41796		46566		86765
Estimated Bid without Contingency		522208		581813		1084062
Estimating Contingency		104442		116363		216812
Estimated Construction Cost (nearest $1000)		**$627,000**		**$698,000**		**$1,301,000**
Engineering, Permitting, Legal and Admin		94050		104700		195150
Estimated Project Cost (nearest $1000)		**$721,000**		**$803,000**		**$1,496,000**

Item	Division 30 Reservoir Option A Gravity Ring		Division 30 Reservoir Option C Anchored Ring		SVWTP Reservoir Option A Gravity Ring	
	Qty	Cost	Qty	Cost	Qty	Cost
Reinforced concrete	120	111000	42	38850	33	30525
Headed studs, 5/8"	192	19200	0	0	0	0
Dowels, #6	151	7550	108	5400	103	5150
Excavation	353	17650	273	13650	163	8150
Backfill	243	12150	232	11600	140	7000
Remove and seal shell manway	1	5000	0	0	0	0
New 30" shell manway	1	15000	0	0	0	0
New 24" shell manway	1	10000	0	0	0	0
Replace depth gauge	1	2000	0	0	0	0
Relocate electrical panel & conduit	1	10000	1	10000	0	0
Replace/reconfigure exterior ladder	1	20000	0	0	0	0
Erosion control	1	5000	1	5000	1	5000
Traffic control	1	5000	1	5000	1	5000
Site grading and seeding	114	570	85	425	91	455
Anchor Bolts	0	0	36	14400	0	0
Anchor Chairs	0	0	19512	78048	0	0
Ground anchors	0	0	18	90000	0	0
Welding for new anchors or seal plate	0	0	367	7340	0	0
Steel plate seal, 1/4"	0	0	0	0	0	0
Cut out door sheet & reinforce	0	0	0	0	0	0
Replace door sheet	0	0	0	0	0	0
Add shell plate	0	0	0	0	0	0
8" gate valve	1	3100	1	3100	0	0
10" gate valve	1	5400	1	5400	2	10800
12" gate va;ve	0	0	0	0	0	0
8" double ball coupling	1	5900	1	5900	0	0
10" double ball coupling	1	7400	1	7400	2	14800
12" double ball coupling	0	0	0	0	0	0
8" water line	13	1430	10	1100	0	0
10" water line	13	1690	10	1300	20	2600
12" water line	0	0	0	0	0	0
Field coating prep and repair exterior	192	1920	360	3600	0	0
Field coating prep and repair interior	192	3072	108	1728	0	0
Mobilization at percentage times previous items	270032	16202	309241	18554	89480	5369
General Conditions at a percentage of previous items	286234	28623	327795	32780	94849	9485
Pretax Construction Subtotal		314857		360575		104334
Sales Tax		27393		31370		9077
Estimated Bid without Contingency		342250		391945		113411
Estimating Contingency		68450		78389		22682
Estimated Construction Cost (nearest $1000)		**$411,000**		**$470,000**		**$136,000**
Engineering, Permitting, Legal and Admin		61650		70500		20400
Estimated Project Cost (nearest $1000)		**$473,000**		**$541,000**		**$156,000**

The opinion of probable construction cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design matures. BHC Consultants has no control over: variances in the cost of labor, materials, equipment; cost for services provided by others; contractor's means and methods of executing the work or of determining prices; nor, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

APPENDIX A.2

GEOTEST REPORT 15-0807 JANUARY 13, 2016

GEOTEST

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Bellingham, WA 98225

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January 13, 2016
Job No. 15-0807

Lake Whatcom Water & Sewer District
1220 Lakeway Dr.
Bellingham, WA 98229

Attn: Kristin Hemenway, P.E.

Re: Non Destructive Test Evaluation of Tank Foundations
Division 7, 22 and 30 Water Reservoir Tanks
Sudden Valley Community
Bellingham, WA 98229

Dear Mr. Hemenway,

This report highlights the findings of our Non-Destructive Test (NDT) evaluation for 3 water reservoirs tanks in the Sudden Valley Community in Bellingham, WA. Our evaluation was performed on January 7th and 8th, 2016 on the Division 7, 22 and 30 tanks as seen in Photo 1 below. The tanks are constructed from welded steel with each tank having variable diameters and heights. Per conversation with Kristen Hemenway with Lake Whatcom Water Sewer District (LWWSD), we understand the tanks are supported by a concrete "Ring Foundation" with structural fill in the interior. The primary goal of this evaluation was to determine the horizontal width of these concrete "Ring Foundations".

Photo1 – Overview of Tank Location

NDT testing equipment used consisted of a GSSI SIR-3000 Ground Penetrating Radar (GPR) with 270 MHz antenna and Olson Concrete Thickness Gauge (CTG). The vertical sides of each foundation was exposed prior to our evaluation. All excavations were provided by the LWWSD. Areas tested included two locations on the Division 7 and 22 tanks and one location on the Division 30 tank.

Our initial scope included performing GPR scanning to identify rebar patterns within the tank's foundations. Per conversation with Kristen Hemenway, PE with LWWSD, this part of the scope was not needed.

Olson Concrete Thickness Gauge

The Olson CTG uses Impact-Echo Theory to help estimate the thickness of a concrete slab or other plate-like structure. The Olson CTG measures the duration for a point impact wave to reflect off two opposite, parallel concrete surfaces. Thickness is estimated by assuming the compressional wave (P-wave) speed for the concrete and by measuring the duration of the wave.

Prior to performing our tests, the surface of the concrete was cleaned with a brush, water and a rag. For this project, GeoTest assumed a P-wave speed of 12,000 ft/s which correlates to concrete with an estimate compressional strength of 3,000 – 4500 psi. Olson recommends a wave speed of 12,000 ft/s for concrete of unknown thickness. Random tests were performed at each location until repeatable test results were obtained. Approximately 2 to 8 tests were performed at each location. Individual thickness readings varied between 25.7 to 31.0 inches for Tanks #7 and #22 and between 15.9 to 18.4 inches for Tank #30. Please refer to Table 1 below for average thickness readings at each specific locations. A station numbering system was utilized for Tank # 7 and 22. Thickness reading were performed in general accordance with ASTM C1383-04 (2010), *Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method.*

Estimate Thickness Using Ground Penetrating Radar (GPR)

GPR technology is used to identify changes in the dielectric properties of the materials being scanned. Conductive materials (rebar, metal, saturated soils, etc.) tend to have higher dielectric properties while insulated materials (concrete, sand, dry soils etc.) tend to have lower dielectric properties. For this project, GPR equipment was utilized to help identify changes in the dielectric properties of the objects scanned which could be interpreted as a potential rebar or a boundary for the concrete.

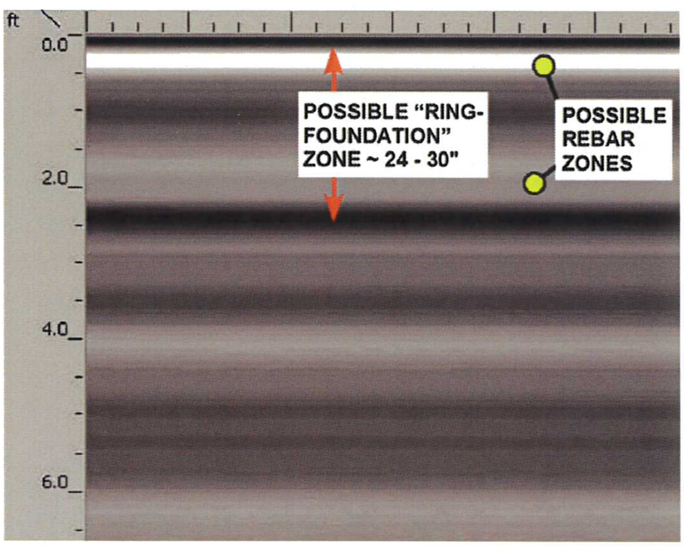

Photo 2 – GPR scan section of Tank 22, Sta # 0+63. Possible boundary for ring foundation seen at approximate 24 – 30 inch.

Within most areas scanned, GeoTest observed a distinct increase in dielectric properties near the surface of the foundation. This increase in dielectric properties was interpreted as the rebar zone near the outer edge of the foundation. GeoTest was able to observe a relatively consistent profile displayed on the monitor beneath the surface. This consistent profile was interpreted as uniform concrete. Typically, GeoTest observed an apparent increase, followed by an immediate decrease in dielectric properties.

GeoTest interpreted this increase in the dielectric properties as reinforcing near the inner portion of the foundation and the decrease in the dielectric properties as a potential boundary of the concrete and soil. This apparent boundary of the concrete ranged from 15 to 38 inches depending on the location. Interpreted depths according to the GPR can be seen within Table 1. Please refer to Photo 2 for examples of the interpreted GPR Data. GPR scanning was performed in general accordance with ASTM D6432 – 11, *Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation.*

Conclusions

According to the NDT methods described above, it appears that the results from the CTG and GPR are relatively consistent. Tanks #7 and 22 appeared to have similar thicknesses while Tank # 30 appeared to be thinner. A summary of the average CTG test results and estimated GPR data can be seen in Table 1.

Table 1
Estimated Thickness Per Olson CTG and GPR

Location		CTG Thickness Results (Inches)		Estimated Thickness per GPR (Inches)
		Individual Results	Average	
Tank # 7	Sta # 1+00	28.6, 26.7, 26.8, 29.0, 29.2	28.1	32 - 38
	Sta # 1+90	31.0, 30.6, 29.2, 26.8, 29.4	29.4	30 - 36
Tank # 22	Sta # 1+25	25.7, 25.7	25.7	27 - 33
	Sta # 0+63	28.6, 27.4, 28.0	28.0	24 - 30
Tank # 30	West End of Tank	16.9, 15.5, 15.9, 17.6, 18.4 17.6, 19.2, 16.1	17.2	15 - 21

Note: Accuracy of CTG is dependent on multiple variables. According to Olson, thickness should be ± 10% with no additional calibration. Accuracy of GPR is dependent on multiple variable including the material being scanned. The thickness listed above may not be accurate.

According to GPR scanning, GeoTest interpreted the boundary of the concrete (See Photo 2) near an apparent drop in dielectric constant at the estimated thicknesses listed in Table 2. GeoTest provided a 6 inch range for these estimated thicknesses due to the variable geophysical properties of the material being scanned and because the GPR imagery was not defined. These estimated thicknesses were slightly higher than the CTG readings.

The individual results of the CTG varied at each location. These thickness results are dependent on the quality of the echo and wave speed. GeoTest observed that the outer surface of the concrete was somewhat uneven and was originally poured with straight plywood form boards. The conditions and uniformity of the inner concrete surface is unknown. Uneven concrete and non-parallel surfaces can lead to poor wave echo quality. Variability with the CTG results may be attributed to the construction methods, variable wave speed and the quality of the echo.

Wave speed within concrete varies depending on the mix design, aggregate type and age of concrete and other factors. GeoTest utilized a wave speed of 12,000 ft/s which is typical for concrete with compressional strength ranging from 3,000 to 4,500 psi. Calibrating the unit to a known thickness is the preferred method of determining the actual wave speed within the concrete. Calibration was not possible due to the unknown thickness of the concrete. According to Olson, the thickness should be ± 10% with no additional calibration.

LIMITATIONS

GeoTest Services has prepared this report for the exclusive Lake Whatcom Water & Sewer District and their representatives for the specific application described in the beginning of this report. Use of this report by others is at the user's sole risk.

Concrete thicknesses according to the Olson CTG are dependent on assumed properties and dimensions of the material being tested. Interpretations are ultimately base on experience and judgement and may not represent the actual conditions of the concrete being tested. GeoTest Services Inc. does not extend any warranties or guaranties as to the accuracy or correctness of interpretations of the CTG tests and GeoTest Services Inc. will not accept liability or responsibility for any loss, damage, or expense that may be incurred or sustained by any services or interpretations performed by GeoTest Services, Inc. or others.

GPR interpretations were based on geophysical properties of the material and may not represent the actual conditions. Even though an apparent boundary was observed per GPR scanning, it should be noted that both concrete and the assumed structural fill inside the "ring foundation" are composed of similar material and react to radar energy similarly. Interpretations of the concrete and structural fill boundary is likely based on the apparent rebar zone at the inner edge of the ring.

Because depth of scanned objects with GPR is dependent upon the electrical properties of material(s) inspected and interpretations are opinions based on judgments made from those acquired radar signals and/or other data, GeoTest Services Inc. does not extend any warranties or guaranties as to the accuracy or correctness of interpretations and GeoTest Services Inc. will not accept liability or responsibility for any loss, damage, or expense that may be incurred or sustained by any services or interpretations performed by GeoTest Services, Inc. or others. GPR scanning cannot distinguish the difference between a single rebar, conduit, post tension cable, and/or subsurface target 100% of the time. It can only detect the center and approximate depth of targets. GeoTest Services, Inc. recognizes that other conditions may vary from those encountered at the location where geophysical or other explorations are made. The data interpretations and recommendations made by GeoTest Services, Inc. are based solely on the information available to them at the time of performance; and GeoTest Services, Inc. shall not be responsible for the interpretation, by others, of the information developed.

We appreciate the opportunity to be of service to you on this project. If any questions should arise regarding this report, please contact the undersigned.

Respectfully Submitted,
GeoTest Services, Inc.

DANIEL J. GOGER
STATE OF WASHINGTON
51078
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APPENDIX B.1

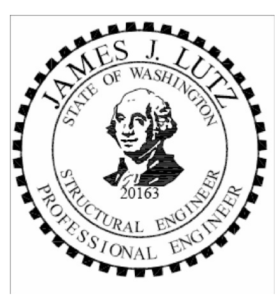
GENEVA RESERVOIR CALCULATIONS

Seismic Evaluation
for
Geneva Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Methodology Remarks

These calculations are limited to an assessment of the primary elements of the lateral force resisting system for the reservoir under seismic loading. Following is a summary of the methodology used:

1. All dimensions and weights are based on record drawings furnished by the client, supplemented by field measurements.In case of discrepancies, field measurements were used..

2. Water level assumed for seismic calculations is based on maximum current operating level provided by the District..

3. Methodology for determination of seismic loads for tanks with a free water surface is based on the 2012 International Building Code, ASCE 7-10, and AWWA Standard D100-11. These codes and standards post-date and are more stringent than codes and standards used at the time of original tank design.

4. For tanks where the free surface sloshing wave amplitude exceeds the roof elevation, the additional amplification of seismic load is based on an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave. The force is modeled by computing an increase in mass and adjusting the convective period of the water mass. The pressure distribution is assumed the same as for a tank with a free water surface.

5. For tanks where the static water surface level already contacts the roof, the free surface sloshing amplitude is based on a cylinder of the same height and radius with zero freeboard, however the actual water mass is assumed. The ratio of sloshing amplitude to roof height is computed using roof height measured from the free water surface. Adjustments in seismic load are otherwise the same as for the preceding step.

6. Ground motion spectral accelerations S_S and S_1 are those currently available from the USGS on their web site calculator for the latitude and longitude of the tank as taken from Google Earth.

7. Soil site class "D" is assumed as a default in the absence of a soils report for this reservoir..

8. Wind loads, hydrostatic loads at overflow elevation, and roof live loads were not considered in the analysis. However where calculated roof loads exceed 40 psf, a mass equal to .20 times the uniform roof snow load is added to the roof mass for seismic calculations. The gravity effects of snow load were considered whete applicable for determining loads on the shell, however no analysis of roof members was included.

Location and Site Data

Lat 48.7392, Long -122.4056
El 661
(Google Earth)

Superstructure Geometry

From record drawings

Tank diameter $D := 53 \cdot ft$

Tank radius $R := \frac{D}{2} = 26.5\,ft$

Shell height $H_s := 32.667 \cdot ft$

Floor elevation at shell
(Bottom capacity level)

$BCL := 451 \cdot ft$ (District)

Overflow height above floor

$h_{overflow} := 32.0 \cdot ft$

Overflow elevation
(Top capacity level)

$TCL := BCL + h_{overflow}$

$H := 31.5 \cdot ft$ Maximum operating level

$NOL := BCL + H = 482.5\,ft$

$BCL + H_s = 483.667\,ft$

This level is below the top of the shell.

Describe the roof geometry

$roof\_slope := \frac{1.0}{12} = 0.083$ (Actual varies between .72" and 1.25" per 12")

The roof height is $h_r := roof\_slope \cdot R = 2.208\,ft$

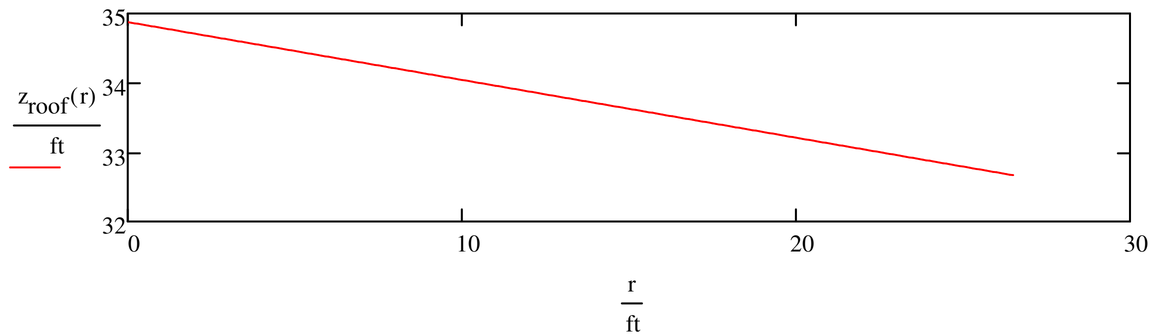
Let "z" be the distance measured vertically from the floor, and "r" the horizontal distance from the center

$z_{apex} := H_s + h_r = 34.875\,ft$

The expression for z for the roof for $0 < r < R$ is

$z_{roof}(r) := \left(\mathrm{if}\left(r > R, 0, z_{apex} - roof\_slope \cdot r\right)\right)$

Plot the roof elevation vs radius $r := 0, .1 \cdot ft .. R$



Enter shell and roof plate thickness.

Mathcad General Input - See Appendix for Mathcad nomenclature and symbols

$ORIGIN := 1$

Special unit definitions $each := 1$ $sf := ft^2$

number of shell plate courses, numbering starting with the base as course 1

$n_{course} := 4$ (the vertical leg of the top angle is included with the top shell plate course)

Calculate the elevation of the top of each shell course relative to the floor

$i := 1, 2 .. n_{course}$ i is the number of each shell course, starting from the bottom $\gamma_{steel} := 490 \cdot pcf$ unit weight of steel

zshell is the elevation of the top of each course relative to the top of the bottom plate

$$z_{shell} := \begin{pmatrix} 8.17 \\ 16.34 \\ 24.52 \\ 32.67 \end{pmatrix} \cdot ft \qquad t_{shell} := \begin{pmatrix} \frac{11}{32} \\ \frac{9}{32} \\ .25 \\ .25 \end{pmatrix} \cdot in \qquad w_{shell} := t_{shell} \cdot \gamma_{steel} = \begin{pmatrix} 14.036 \\ 11.484 \\ 10.208 \\ 10.208 \end{pmatrix} \cdot psf \qquad class_{shell} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Shell thickness is per nameplate data, which is consistent with thickness readings given instrument accuracy. Original specifications called for shell plate to be ASTM A283 Grades A (24 ksi yield stress), B (27 ksi), C (30 ksi) or D (33 ksi). Records do not indicate which was used. Assume at least Grade B.

Class 1 material has a yield stress 27 ksi < Fy < 34 ksi. Class 2 material has a yield stress Fy > 34 ksi

Roof thickness is 3/16" per nameplate, but thickness gauge measurements were .120". Use 3/16" to be conservative for roof weight calcualtions.

$t_{roof\_plate} := \frac{3}{16} \cdot in$ roof plate thickness

Compute weight of roof and shell

Define the roof slope at any point

$z'_{roof}(r) := \frac{d}{dr} z_{roof}(r)$

Compute the surface area of the roof plate tributary to the perimeter and the center column. . Ignore laps

For a surface of revolution, the general equation for the surface area is

$A := 2 \cdot \pi \cdot \int r\,ds$ where $ds := \sqrt{1 + \left(\frac{dz}{dr}\right)^2} \cdot dr$

$A_{roof\_plate} := 2 \cdot \pi \cdot \left(\int_0^R r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 2214\,ft^2$ (roof surface area)

$W_{roof\_plate} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate} = 16.95 \cdot kip$

$A_{roof\_plate\_center} := 2 \cdot \pi \cdot \left(\int_0^{\frac{R}{2}} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 553\,ft^2$

$W_{roof\_plate\_center} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_center} = 4.237 \cdot kip$ Portion of roof weight tributary to center column

$A_{roof\_plate\_edge} := 2 \cdot \pi \cdot \left(\int_{\frac{R}{2}}^{R} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 1660\,ft^2$

$W_{roof\_plate\_edge} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_edge} = 12.712 \cdot kip$ Portion of roof weight tributary to shell

Calculate the vertical center of gravity from the tank floor for the roof plate

$$x_{cg} := 2\cdot\pi\cdot\frac{\left(\int_0^R r^2\cdot\sqrt{1 + z'_{roof}(r)^2}\,dr\right)}{A_{roof\_plate}} = 18\,ft$$

$$X_{roof\_plate} := z_{roof}\left(x_{cg}\right) = 33.403\,ft$$

Define the number of the shell course for any value of $0 < z < Hs$ using a series of functions

$i_{course}(z) := n_{course}$ Default value

$$i_{course}(z) := if\left(z < z_{shell_{n_{course}}}, n_{course}, i_{course}(z)\right)$$

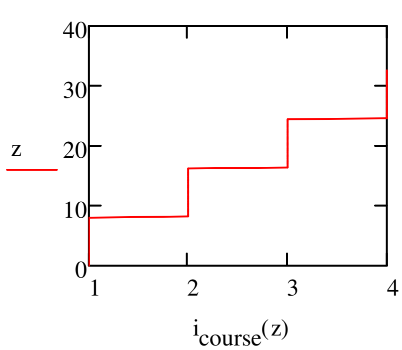
$$i_{course}(z) := if\left(z < z_{shell_4}, 4, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_3}, 3, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_2}, 2, i_{course}(z)\right)$$

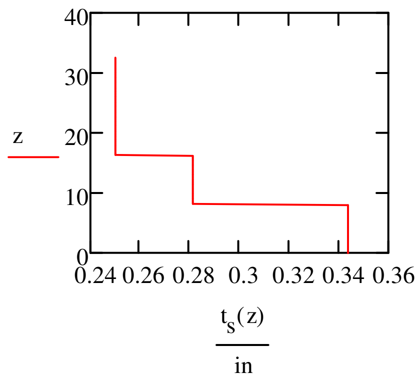
$$i_{course}(z) := if\left(z < z_{shell_1}, 1, i_{course}(z)\right)$$

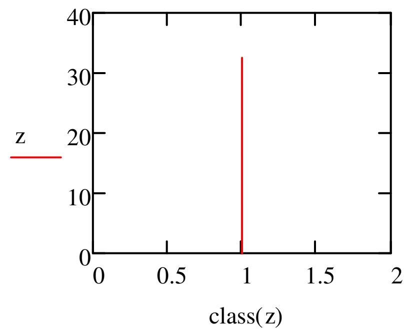
$z := 0\cdot ft, 0.2\cdot ft .. H_s$ Set plotting interval for graphs



write functions that return the shell plate thickness and class as a function of height above the base

$t_s(z) := t_{shell_{i_{course}(z)}}$ $class(z) := class_{shell_{i_{course}(z)}}$





Shell thickness vs elevation **Shell class vs elevation**

Floor plate thickness $t_{floor} := .25 \cdot in$

$floor\_flange := 1.75 \cdot in$ Bottom plate projection beyond shell plate $D_{floor} := D + 2 \cdot floor\_flange$

Compute floor weight

$$W_f := \gamma_{steel} \cdot t_{floor} \cdot \pi \cdot \frac{D_{floor}^{\;2}}{4} \qquad W_f = 22.8 \cdot kip$$

Compute the weight of the shell and establish its center of gravity from the base

$$W_s := \pi \cdot D \cdot \int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z)\, dz \qquad W_s = 62.466 \cdot kip$$

$$X_s := \pi \cdot D \cdot \frac{\int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \cdot z\, dz}{W_s} \qquad X_s = 15.199\, ft$$

Compute the weight of the roof and establish its center of gravity from the base

The total roof mass is a combination of the part tributary to the center column and the part tributary to the edge. The center portion includes part of the roof, half the weight of the rafters, the column cap, and half of the column. (The other half of the column and its base plate are assigned to the floor mass). The edge portion includes part of the roof, half the weight of the rafters, clips and the flange of the top angle. The weight of top angle and clips and top angle flange are ignored.

Based on record drawings, the rafters are C7X9.8 shapes, about 25.5 ft long. Column cap is .37" x 2 ft dia. Center pipe column is 6" diameter, Sch 40. Ignore weight of clips, bolts, laps, and appurtenances..

$$W_{rafters} := 27 \cdot 9.8 \cdot \frac{lbf}{ft} \cdot (25.5 \cdot ft) = 6.747 \cdot kip$$

$$W_{col\_cap} := \pi (12 \cdot in)^2 \cdot .375 \cdot in \cdot \gamma_{steel} = 0.048 \cdot kip$$

$$W_{col} := 33.6 \cdot ft \cdot 19.6 \cdot \frac{lbf}{ft} = 0.659 \cdot kip$$

$$W_{col\_base} := \gamma_{steel} \cdot \left[.5 \cdot in \cdot \pi \cdot (18 \cdot in)^2 + .375 \cdot in \cdot 2 \cdot 1 \cdot ft^2 \right] = 0.175 \cdot kip$$ base plate and gussets

$$W_{roof\_center} := W_{roof\_plate\_center} + \frac{W_{rafters}}{2} + W_{col\_cap} + \frac{W_{col}}{2} = 7.988 \cdot kip$$ Roof weight tributary to center column

$$W_{roof\_edge} := W_{roof\_plate\_edge} + \frac{W_{rafters}}{2} = 16.086 \cdot kip$$ Roof weight tributary to top of shell

$$\Delta W_f := W_{col\_base} + \frac{W_{col}}{2} = 0.504 \cdot kip$$ Column and base plate tributary to floor

Total roof structure mass for seismic calculation $W_r := W_{roof\_center} + W_{roof\_edge} = 24.074 \cdot kip$

Check to see if roof snow load mass must be included per ASCE 7-10

$p_g := 50 \cdot psf$ from "Snow Load Analysis for Washington", 2nd ed, SEAW

$I_s := 1.20$ Snow load importance factor for risk category IV, ASCE 7-10

$C_e := 1.2$ ASCE 7-10, Table 7-2. Exposure Factor, Terrain B, Sheltered

$C_t := 1.2$ ASCE 7-10, Table 7-3, Thermal Factor, Unheated

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 60.48 \cdot psf$ Flat roof snow load, ASCE 7-10 Eq 7.3-1. Since flat roof snow load exceeds 30 psf, add 20% of the design snow load to the roof mass per ASCE 7-10, section 12.7.2.

The roof slope is $\operatorname{atan}(roof\_slope) = 4.764 \cdot deg$

From ASCE 7-10 Fig 7-2c, the roof slope factor is

$C_s := 1.0$

$p_s := C_s \cdot p_f = 60.48 \cdot \text{psf}$

Snow weight to include with roof weight

$w_{snow} := .20 \cdot p_s = 12.096 \cdot \text{psf}$

$W_{snow} := w_{snow} \cdot \pi \cdot R^2 = 26.686 \cdot \text{kip}$

Snow weight tributary to edge

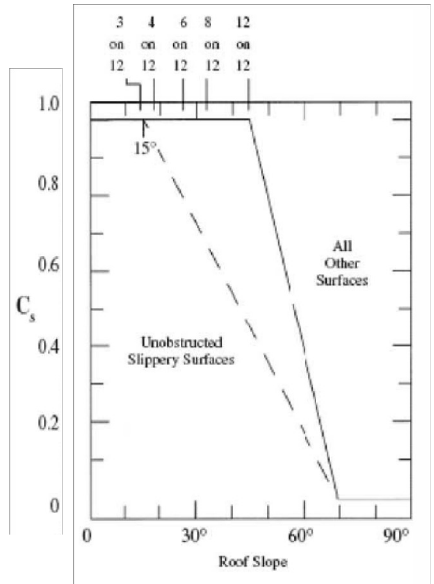
$$W_{snow\_shell} := W_{snow} \cdot \frac{A_{roof\_plate\_edge}}{A_{roof\_plate}} = 20.014 \cdot \text{kip}$$

$$P_{snow} := \frac{W_{snow\_shell}}{\pi \cdot D} = 120.204 \cdot \frac{\text{lbf}}{\text{ft}}$$

Snow load applied at top of shell concurrent with seismic

Snow weight tributary to floor

$W_{snow\_floor} := W_{snow} - W_{snow\_shell} = 6.671 \cdot \text{kip}$



7-2c: Cold roofs with C = 1.2 or larger

All the lateral resistance for the roof is assumed to be by the shell, except for the lower half of the column

Compute the center of gravity of the roof and column mass for seismic calculation

$$X_r := \frac{\left[W_{roof\_plate} \cdot X_{roof\_plate} \cdots + z_{apex} \cdot W_{col\_cap} + .75 \cdot z_{apex} \cdot \frac{W_{col}}{2} + W_{rafters} \cdot \left(H_s + \frac{h_r}{2} \right) \right]}{W_r} = 33.41\,\text{ft}$$

Compute the center of gravity of the roof snow load for seismic calculations

Snow density per ASCE 7-10 equation 7.7.1 is

$$\gamma_{snow} := \min\left(30 \cdot \text{pcf}, 0.13 \cdot \frac{p_g}{\text{ft}} + 30 \cdot \text{pcf} \right) = 30 \cdot \text{pcf} \qquad \text{snow depth} \qquad h_d := \frac{w_{snow}}{\gamma_{snow}} = 0.403\,\text{ft}$$

$$X_{snow} := X_{roof\_plate} + \frac{h_d}{2} = 33.605\,\text{ft}$$

centroid of snow mass

<u>Compute total water weight for seismic calculations</u>

$\gamma_{water} := 62.4 \cdot \text{pcf}$

$$W_T := \gamma_{water} \cdot H \cdot \pi \cdot \frac{D^2}{4} = 4336.47 \cdot kip$$

Calculate the impulsive and convective water weights and vertical centroids

$$\frac{D}{H} = 1.683$$

$$W_i := W_T \cdot \frac{\tanh\left(.866 \cdot \frac{D}{H}\right)}{.866 \cdot \frac{D}{H}}$$ if D/H > 1.333

$$W_i := \text{if}\left[\frac{D}{H} < 1.333, W_T \cdot \left(1.0 - 0.218 \cdot \frac{D}{H}\right), W_i\right]$$ if D/H < 1.33

$W_i = 2669.848 \cdot kip$ Impulsive water weight $\frac{W_i}{W_T} = 0.616$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated or the total moment on the foundation, shell plus floor.

$$X_i := H \cdot \text{if}\left[\left(\frac{D}{H}\right) > 1.333, 0.375, 0.50 - 0.094 \cdot \frac{D}{H}\right]$$ $X_i = 11.813\,ft$ centroid for calculation of just the shell moment

$$X_{imf} := 0.375 \cdot \left[1.0 + 1.333 \cdot \left(\frac{0.866 \cdot \frac{D}{H}}{\tanh\left(0.866 \cdot \frac{D}{H}\right)} - 1\right)\right] \cdot H$$ centroid for calculation of total bottom moment if D/H > 1.33

$$X_{imf} := \text{if}\left[\frac{D}{H} < 1.333, \left(0.50 + 0.06 \cdot \frac{D}{H}\right) \cdot H, X_{imf}\right]$$ centroid for calculation of total bottom moment if D/H < 1.33

$$X_{imf} = 21.642\,ft$$

Compute convective water weight and effective centroid above the base

$$W_c := W_T \cdot \left(.230 \cdot \frac{D}{H} \cdot \tanh\left(3.67 \cdot \frac{H}{D}\right)\right)$$ $W_c = 1635.9 \cdot kip$ $\frac{W_c}{W_T} = 0.377$ Ref 4, Eq 13-26

$$X_c := H \cdot \left[1 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1}{3.67 \cdot \left(\frac{H}{D}\right) \cdot \sinh\left(3.67 \frac{H}{D}\right)}\right]$$ $X_c = 19.989\,ft$ centroid for calculation of just the shell moment

$$X_{cmf} := H \cdot \left(1.0 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1.937}{3.67 \cdot \frac{H}{D} \cdot \sinh\left(3.67 \cdot \frac{H}{D}\right)} \right)$$

$X_{cmf} = 23.084\,ft$ centroid for calculation of total bottom moment

Seismic Design Criteria

Importance Factor: $I_E := 1.50$ Risk category IV

Ground Motion Parameters

Site Class D — Default Site Class in absence of a geotechnical report

$S_S := .948$ $S_1 := .371$ Mapped earthquake short period and long period spectral accelerations. For Site Class B, 5% damping, expressed as fraction of g.

$F_a := 1.12$ $F_v := 1.66$ Site coefficients from 2012 IBC Table 1613.3.3(2). Seismic Design Category "D"

Adjusted maximum considered earthquake for site class

$S_{MS} := F_a \cdot S_S$ $S_{MS} = 1.062$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.616$

Design spectral response parameters

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$ $S_{DS} = 0.708$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$ $S_{D1} = 0.411$

Compute points on the design response spectrum

$T_0 := 0.2 \cdot sec \cdot \frac{S_{D1}}{S_{DS}}$ $T_0 = 0.116 \cdot sec$

$T_S := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot sec$ $T_S = 0.58 \cdot sec$

$T_L := 6 \cdot sec$ Mapped value, ASCE 7-10, Figure 22-12

$T_L := \text{if}\left(T_L > 4.sec, 4.sec, T_L\right) = 4 \cdot sec$ Maximum required for tank sloshing wave calculations, ASCE 7-10, Section 15.7.6.1.d

$S_{ac}(T) := \text{if}\left(T > T_L, \frac{1.5 \cdot S_{D1} \cdot T_L \cdot sec}{T^2}, \min\left(\frac{1.5 \cdot S_{D1} \cdot sec}{T}, 1.5 \cdot S_{DS}\right)\right)$ Convective acceleration function

$S_{ac}(T) := \text{if}\left(S_{ac}(T) > 1.5S_{DS}, 1.5S_{DS}, S_{ac}(T)\right)$ Upper bound for S_{ac} for low values of T

$$S_{ai}(T) := \text{if}\left(T > T_L, \frac{S_{D1} \cdot T_L \cdot sec}{T^2}, \text{if}\left(T > T_S, \frac{S_{D1}}{T} \cdot sec, S_{DS}\right)\right)$$

Impulsive acceleration function

Calculate Free Surface Wave Height and Compare to Freeboard Requirements

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{D}{3.68\cdot g\cdot \tanh\left(3.68\cdot\frac{H}{D}\right)}} \qquad T_c = 4.257\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \mathrm{if}\left(SUG = 3, \mathrm{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\,2}}\right), \mathrm{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\,2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.136$

$d := 0.5\cdot D\cdot A_f = 3.602\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude.

freeboard $f := H_s - H = 1.167\,ft$

$\frac{d}{f} = 3.087$ > 1.0, therefore **freeboard is insufficient**

Compute Base Shear and Overturning Moments As If Free Surface

$S_{ai} := S_{DS}$ $R_i := 3.0$ $R_c := 1.5$ AWWA D100-11, Table 28 and section 13.2.9.2. Anchored tank

$$A_i := \max\left(\frac{S_{ai} \cdot I_E}{1.4 \cdot R_i}, \frac{0.36 \cdot S_1 \cdot I_E}{R_i}\right) \qquad A_i = 0.253$$

Impulsive design acceleration

$$A_c := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c} \qquad A_c = 0.097$$

Convective design acceleration

Calculate overturning moment at the base of the shell

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_i\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2} \qquad M_s = 9207 \cdot kip \cdot ft$$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_{imf}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2} \qquad M_{mf} = 15711 \cdot kip \cdot ft$$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_i\right]\right]^2 + \left(A_c \cdot W_c\right)^2} \qquad V_f = 727.01 \cdot kip$$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Adjust Effective Masses for Roof Contact

The methodology for roof contact effects is an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave.

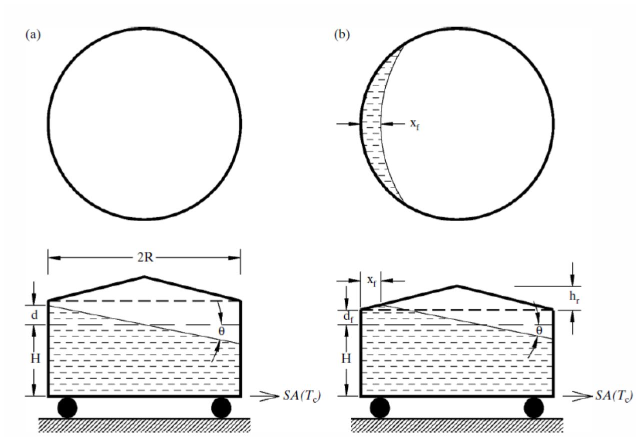
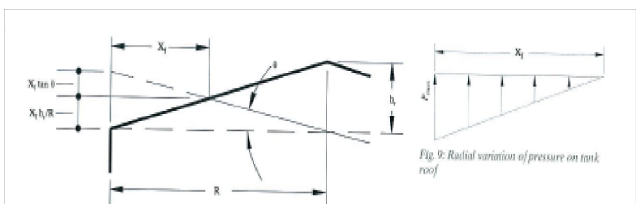


Fig. 5: Liquid-filled tank translating with an acceleration SA(T_c): (a) sufficient freeboard; and (b) insufficient freeboard



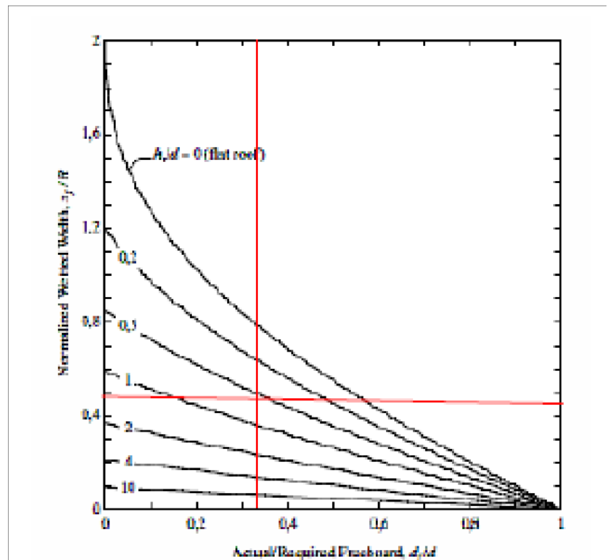


Fig. 6: Cone roof tank. Normalized wetted width of tank roof x_f/R as a function of actual/required freeboard d_f/d and normalized roof height h_r/d

Compute the angle θ

$$\theta := \operatorname{atan}\left(\frac{I_E \cdot S_{ac}(T_c) \cdot \frac{ft}{sec^2}}{g}\right) = 0.363 \cdot deg$$

Where $S_{ac}(T_c) = 0.136$ $\quad I_E = 1.5 \quad g = 32.174 \frac{ft}{s^2}$

$d_f := H_s - H = 1.167\,ft \quad d = 3.602\,ft \quad \frac{d_f}{d} = 0.324$ Compute input variables for graph above

$h_r = 2.208\,ft \quad \frac{h_r}{d} = 0.613$

From graph figure 6

$x_f := .5 \cdot R = 13.25\,ft$ horizontal extent of wetted dome surface from the shell $\frac{x_f}{R} = 0.5$ << 1.0 OK

$\rho := \frac{\gamma_{water}}{g} = 62.4 \cdot \frac{lbm}{ft^3}$ unit mass of water

$F_{max} := \frac{\rho}{2} \cdot g \cdot x_f^{\,2} \cdot \frac{(d + h_r)}{R}$ $F_{max} = 1201 \cdot \frac{lbf}{ft}$ Maximum uplift on shell due to hydrodynamic pressure caused by sloshing. Impact effects are considered minor and ignored

adjust mass for recalculation of seismic demand

$$\bar{m}_i = \begin{cases} m_i + m_c \cdot \left(1 - \dfrac{d_f + h_r/3}{d}\right) & \text{for } d_f + h_r/3 < d \\ m_i & \text{for } d_f + h_r/3 \ge d \end{cases}$$

$W_i = 2670 \cdot kip$

$W_T = 4336 \cdot kip$

$\dfrac{\left(d_f + \dfrac{h_r}{3}\right)}{d} = 0.528$ $W_{bar\_i} := W_i + W_c \cdot \left[1 - \dfrac{\left(d_f + \dfrac{h_r}{3}\right)}{d}\right] = 3441.5 \cdot kip$

$W_{bar\_i} := \text{if}\left[\dfrac{\left(d_f + \dfrac{h_r}{3}\right)}{d} < 1, W_{bar\_i}, W_i\right] = 3441 \cdot kip$

$$\bar{m}_c = m_l - \bar{m}_i$$

$W_c = 1635.9 \cdot kip$ $W_{bar\_c} := W_T - W_{bar\_i} = 895 \cdot kip$

$\dfrac{W_{bar\_i}}{W_i} = 1.289$ $\dfrac{W_{bar\_c}}{W_c} = 0.547$ Factors by which mass must be multiplied due to the slosh contact with the roof

Recaclulate convective period using adjusted mass. Maintain asssumption of T = 0 for impulsive mass

$$\bar{T}_i = T_i \cdot \sqrt{\frac{\bar{m}_i}{m_i}}$$

$$\bar{T}_c = T_c \cdot \sqrt{\frac{\bar{m}_c}{m_c}}$$

$T_c = 4.257\,s$ original convective period

$T_{c\_bar} := T_c \cdot \sqrt{\dfrac{W_{bar\_c}}{W_c}} = 3.149\,s$ modified convective period

$S_{ac}(T_c) = 0.136$ $A_c = 0.097$ original convective seismic factor

$S_{ac}(T_{c\_bar}) = 0.196$ $A_{c\_bar} := A_c \cdot \dfrac{S_{ac}(T_{c\_bar})}{S_{ac}(T_c)} = 0.140$ revised convective seismic factor

Recompute base shear and overturning moment

Change formula weights to adjusted values

$M_s = 9207 \cdot kip \cdot ft$ original overturning moment

$$M_{s\_rev} := \sqrt{\left[A_i \cdot \left[W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + \left(W_{bar\_i}\right) \cdot X_i\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_c\right)^2}$$

$M_{s\_rev} = 11229 \cdot kip \cdot ft$ revised moment

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$M_{mf} = 15711 \cdot kip \cdot ft$ original overturning moment

$$M_{mf\_rev} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{bar\_i} \cdot X_{imf}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_{cmf}\right)^2}$$

$M_{mf\_rev} = 19711 \cdot kip \cdot ft$ revised moment

Calculate base shear at top of foundation

$V_f = 727.01 \cdot kip$ original base shear

$$V_{f\_rev} := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_{bar\_i}\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c}\right)^2}$$

$V_{f\_rev} = 913.11 \cdot kip$ revised base shear

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Compute Shell Hoop Forces and Stresses

Impulsive and convective forces are distributed using Housner's distribution formulas

Define the following variables:

z	Height of a point above the tank floor
Y	Depth of a point below the water surface
n_I	Distributed hoop force, klf, due to impulsive load N_I
n_C	Distributed hoop force, klf, due to convective load N_C
n_V	Distributed hoop force, klf, due to vertical seismic force N_V
n_F	Distributed hoop force, klf, due to hydrostatic force at maximum normal operating level
n_{Fol}	Distributed hoop force, klf, due to hydrostatic force at overflow operating level

Define elevation, distribution, and force component functions

$Y(z) := H - z$ distance from MOL to z

Housner's distribution of impulsive load as a function of elevation above the base and, in the case of impulsive loads, depends on the ratio of D/H

For the case of D/H < 1.33 and Y(z) < 0.75 D (z > .75D, upper section)

$$\mathrm{Dist}_{ia}(z) := \frac{\left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2}{\left[\int_{.75\cdot D}^{H}\left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz + \int_0^{.75\cdot D} 0.5\,dz\right]}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H < 1.33 at lower elevations, the factor is a constant equal to

$$\mathrm{Dist}_{ib}(z) := \frac{0.5}{\left[\int_{.75\cdot D}^{H}\left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz\right] + \int_0^{.75\cdot D} 0.5\,dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H > 1.33

$$\mathrm{Dist}_{ic}(z) := \frac{\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)}{\int_{0\cdot ft}^{H}\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

$$\mathrm{Dist}_i(z) := \mathrm{if}\left[\left(\frac{D}{H}\right) \geq 1.333, \mathrm{Dist}_{ic}(z), \mathrm{if}\left(Y(z) < 0.75\cdot D, \mathrm{Dist}_{ia}(z), \mathrm{Dist}_{ib}(z)\right)\right]$$

select appropriate formula based on depth and diameter ratio

Housner's distribution of convective load as a function of elevation above the base

$$\mathrm{Dist}_c(z) := \frac{\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}}{\displaystyle\int_{0\cdot ft}^{H}\frac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}dz}$$

The above formula is the convective force per unit depth at elevation "z" expressed as a fraction of the total convective force.

$V_i := A_i\cdot W_{bar\_i}$ $\quad V_i = 870.002\cdot kip$ — Total base shear component due to impulsive fluid load

$N_i(z) := \left(\frac{V_i}{2}\right)\cdot \mathrm{Dist}_i(z)$ — Shell hoop force due to impulsive fluid load

$V_c := A_{c\_bar}\cdot W_{bar\_c}$ $\quad V_c = 125.036\cdot kip$ — Total base shear component due to convective fluid load

$N_c(z) := \frac{V_c}{2}\cdot \mathrm{Dist}_c(z)$ — Shell hoop force due to convective fluid load

$N_h(z) := \gamma_{water}\cdot\left(\frac{D}{2}\right)\cdot Y(z)$ — Shell hoop force due to hydrostatic load with water at MOL

$A_v := 0.14\cdot S_{DS}$ $\quad A_v = 0.099$ — Vertical seismic factor

$$\sigma_{static}(z) := \frac{N_h(z)}{t_s(z)}$$

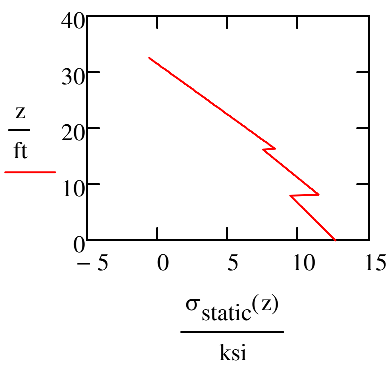
Hoop stress due to static fluid pressure at MOL

$$\sigma_s(z) := \frac{\sqrt{N_i(z)^2 + N_c(z)^2 + \left(N_h(z) \cdot A_v\right)^2}}{t_s(z)}$$

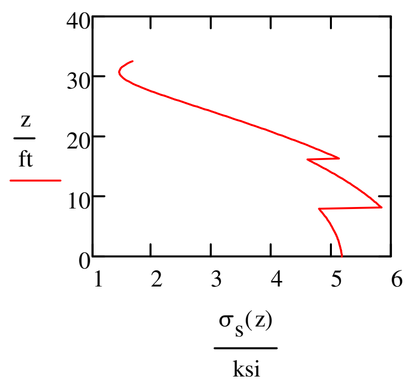
Hoop stress due to hydrodynamic pressure, Ref 4 Eq 13-42

$$\sigma_{total}(z) := \sigma_{static}(z) + \sigma_s(z)$$

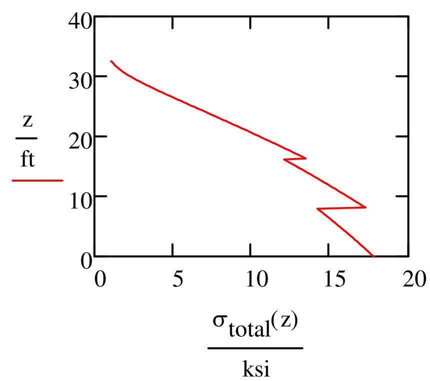
Combined static and seismic hoop stress at MOL



Hydrostatic Stress



Seismic Stress



Static + Seismic Stress

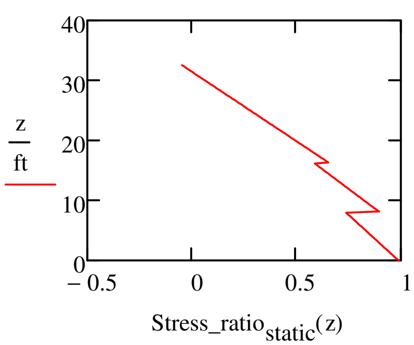
Note: the above plots are nominal based on treating each hoop course as acting independently. Actual stresses each side of girth joints are the same since strains are identical if the courses are attached, so the real stress near transition zones falls somewhere between the apparent discontinuous stress levels shown on the graphs. The actual maximum stress levels tend to occur about a foot above the joint and are not as high as predicted by the more simplified model. The simplified model is conservative and is the method reflected in the AWWA D-100 standard.

Check actual versus allowable stress based on the class of steel used.

Assumed joint efficiency and allowable stress $E_{joint} := 85\%$ $F_t(z) := E_{joint} \cdot 15 \cdot ksi$

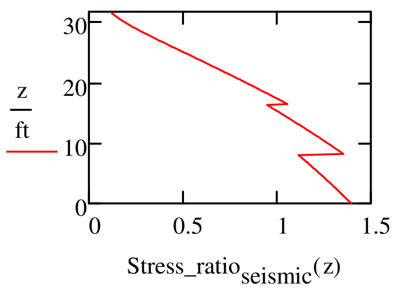
Chapter 14 of AWWA D100-11 does not apply

$$Stress\_ratio_{static}(z) := \left(\frac{\sigma_{static}(z)}{F_t(z)}\right)$$



Maximum static stress ratio is $Stress\_ratio_{static}(0) = 0.99$ < 1.0 OK

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{total}(z)}{F_t(z)}$$



The worst case stress ratio is at the bottom of the first shell couse, but also check th bottom of the second shell course

$Stress\_ratio_{seismic}(0) = 1.397$ at bottom of tank   > 1.33 $\frac{1.397}{1.33} = 1.05$

$Stress\_ratio_{seismic}\left(z_{shell_1}\right) = 1.355$ at bottom of second shell course   > 1.33 $\frac{1.355}{1.33} = 1.02$

The overstress for the second course is only 2%, so say ok. The bottom course is ok up to elevation

$z_{check} := 2.1 \cdot ft$ $Stress\_ratio_{seismic}\left(z_{check}\right) = 1.33$

Compute Shell Longitudinal Forces and Stresses

Define axial compressive force in the shell due to dead load for $0 < z < H_s$, in klf.

$$P_D(z) := \frac{W_{roof\_edge}}{\pi \cdot D} + \int_z^{H_s} \gamma_{steel} \cdot t_s(z)\, dz$$

Define overturning moment functions at elevation z, in kip-ft

$$M_{rs}(z) := A_i \cdot \left[W_r \cdot (X_r - z) + W_{snow} \cdot X_{snow} + \pi \cdot \gamma_{steel} \cdot D \cdot \int_z^H y \cdot t_s(y)\, dy \right]$$

Moment associated with roof, snow and shell mass

$$M_i(z) := 2 \cdot \int_z^H (y - z) \cdot N_i(y)\, dy$$

Moment associated with impulsive fluid mass, z < H

$$M_c(z) := 2 \cdot \int_z^H (y - z) \cdot N_c(y)\, dy$$

Moment associated with convective fluid mass, z < H

$$M_s(z) := M_{rs}(z) + M_i(z) + M_c(z)$$

Total moment at elevation z on the shell for z < H

Define functions for compressive stress under static or seismic load conditions

$$\sigma_{static}(z) := \frac{P_D(z) + P_{snow}}{t_s(z)}$$

$$\sigma_{comp}(z) := \frac{(1 + 0.4 \cdot A_v)(P_D(z) + P_{snow}) - F_{max} + \frac{4 M_s(z)}{\pi \cdot D^2}}{t_s(z)}$$

Includes deduction for roof uplift, $F_{max.}$

Check allowable stress for compression with local buckling and slenderness considered

Use Method 1. Yield stress of shell plate does not permit use of Method 2.

Local buckling stress formulas for Class 1 Materials

$$F_{L1a}(z) := \left[17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R} \right) \right] \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R} \right)^2 \right] \cdot psi$$

For Class 1 materials with 0 < t/R < t/Rc = .0031088, elastic buckling

$F_{L1b}(z) := 5775 \cdot psi + 738 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 1 materials with t/Rc = .0031088 < t/R < 0.0125, inelastic buckling

$F_{L1c}(z) := 15 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Local buckling stress formulas for Class 2 Materials

$$F_{L2a}(z) := \min\left[15 \cdot ksi, 17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R}\right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R}\right)^2\right] \cdot psi\right]$$

For Class 2 materials with 0 < t/R < t/Rc = .0035372, elastic buckling

$F_{L2b}(z) := 6925 \cdot psi + 886 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 2 materials with t/Rc = .0035372 < t/R < 0.0125, inelastic buckling

$F_{L2c}(z) := 18 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Write equation selection functions for F_L depending on t/R ratio and class

$ratio1 := .0031088 \quad ratio2 := .0035372$

$$F_{L1}(z) := \min\left(if\left(\frac{t_s(z)}{R} < ratio1, F_{L1a}(z), if\left(\frac{t_s(z)}{R} < 0.0125, F_{L1b}(z), F_{L1c}(z)\right)\right), 15 \cdot ksi\right)$$

$$F_{L2}(z) := \min\left(if\left(\frac{t_s(z)}{R} < ratio2, F_{L2a}(z), if\left(\frac{t_s(z)}{R} < 0.0125, F_{L2b}(z), F_{L2c}(z)\right)\right), 18 \cdot ksi\right)$$

$F_L(z) := if\left(class(z) = 1, F_{L1}(z), F_{L2}(z)\right)$

Slenderness reduction factor equations

$r := \frac{D \cdot \sqrt{2}}{4}$ radius of gyration of tank shell

$K := 1.0$ effective column length factor, pinned ends assumed

$E := 29 \cdot 10^6 \cdot psi$ modulus of elasticity for steel

Slenderness ratio at which overall elastic column buckling can occur (not local buckling)

$C'_c(z) := \sqrt{\pi^2 \cdot \frac{E}{F_L(z)}}$ $\quad L := H_s$

$K_{\varphi 1}(z) := 1 - \frac{1}{2}\cdot\left(\frac{\frac{K\cdot L}{r}}{C'_c(z)}\right)^2$ For 25 < KL/r < C'c

$K_{\varphi 2}(z) := \frac{1}{2}\cdot\left(\frac{C'_c(z)}{\frac{K\cdot L}{r}}\right)^2$ For KL/r > C'c

$K_{\varphi 3}(z) := 1.0$ For KL/r < 25

$ratio := K\cdot\frac{L}{r}$ $ratio = 1.743$

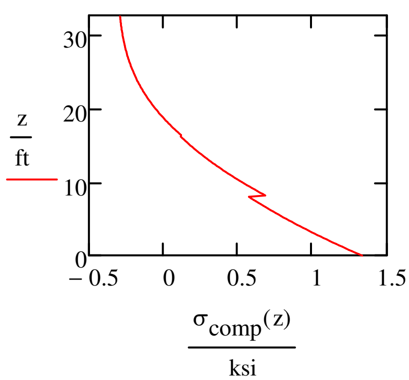
$K_{\varphi}(z) := \text{if}\left(ratio < 25, K_{\varphi 3}(z), \text{if}\left(ratio > C'_c(z), K_{\varphi 2}(z), K_{\varphi 1}(z)\right)\right)$

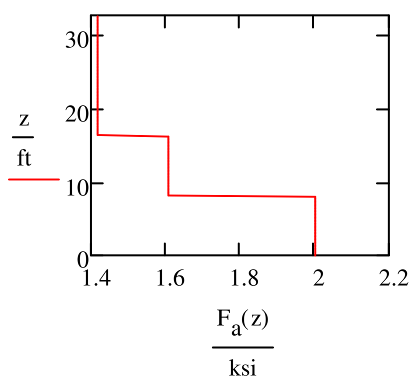
$F_a(z) := F_L(z)\cdot K_{\varphi}(z)$ allowable compressive stress due to axial load

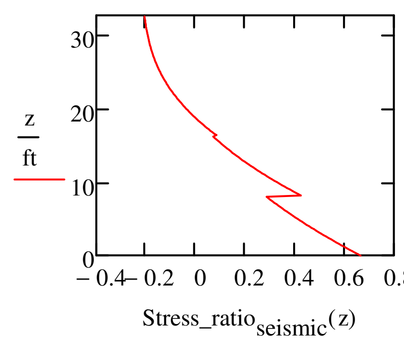
For shell longitudinal stress, treat all stress as axial

$Stress\_ratio_{seismic}(z) := \frac{\sigma_{comp}(z)}{F_a(z)}$

Plot static plus seismic compressive stress and compare to allowables



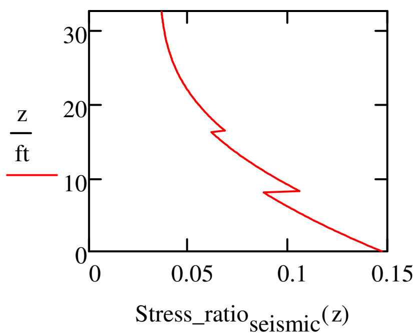




$Stress\_ratio_{seismic}(0) = 0.666$ << 1.33, **OK for static plus seismic longitudinal compression**

Check seismic longitudinal tensile stress

$$\sigma_{tens}(z) := \frac{(1 - .40 \cdot A_v) P_D(z) + F_{max} + \frac{4\,M_s(z)}{\pi \cdot D^2}}{t_s(z)} \qquad Stress\_ratio_{seismic}(z) := \frac{\sigma_{tens}(z)}{F_t(z)}$$



All stress ratios << 1.333 are **OK for static plus seismic stress in longitudinal tension**

$$Stress\_ratio_{seismic}(0) = 0.147$$

Horizontal Shear Transfer Capacity

The previously calculated base shear is $V_f = 727 \cdot kip$

From AWWA D100-11 Eq 13-57, the allowable resistance attributable to friction is (for the full tank, seismic condition)

$V_{ALLOW} := \tan(30 \cdot deg) \cdot (W_s + W_r + W_T + W_f) \cdot (1 - A_v) = 2312 \cdot kip$

>> V_f OK. No shear connection between the superstructure and base is required for shear. Shear resistance is provided by the bottom plate acting as a diaphragm kept in place by bottom friction. Check shell to bottom transfer capacity

The maximum shell to bottom plate shear load is $v := 2 \cdot \frac{V_f}{\pi \cdot D} = 8.733 \cdot klf$

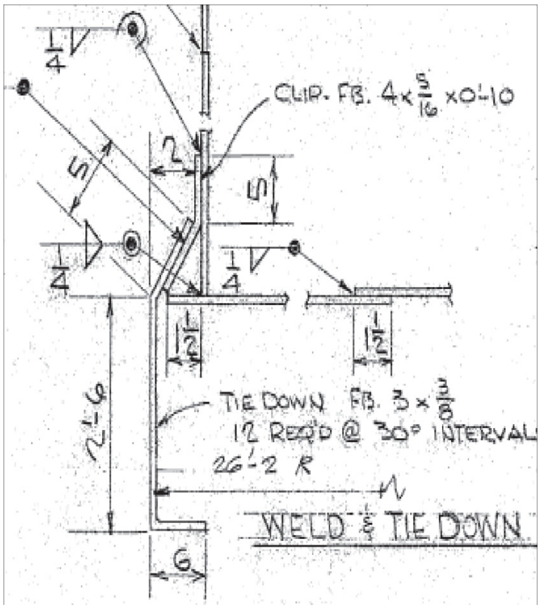
There is no annular plate, just the .25" floor plate

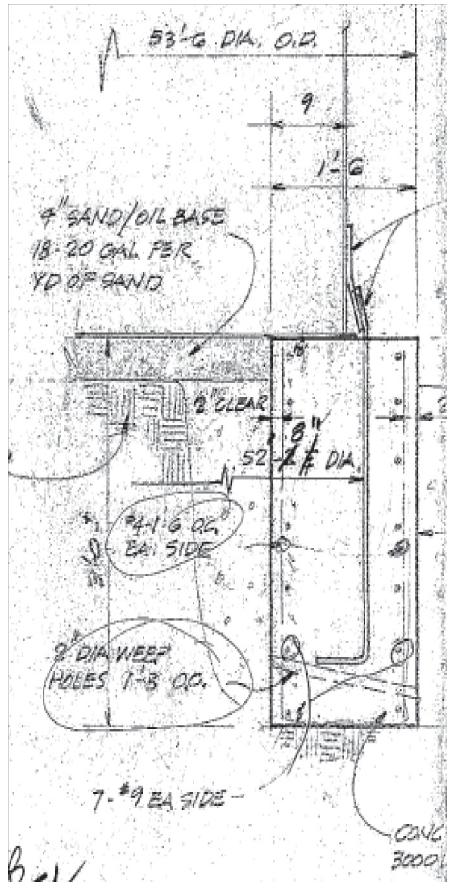
$t_f := .25 \cdot in$

And the maximum shear stress on the plate is $\tau := \frac{v}{t_f} = 3 \cdot ksi$ $\frac{\tau}{15 \cdot ksi} = 0.194$

AWWA D100 permits 12 ksi in shear, and this can be increased by 1.33 for seismic, so **floor plate should not tear in shear parallel to the floor plate**

Check Foundation





Check nominal anchor capacity

$\sigma_{tens}(0)\cdot t_s(0) = 7.743\cdot klf$

Compute existing anchor load

$n_{anchors} := 12 \qquad T_{anchor} := \left(\frac{\pi\cdot D}{n_{anchors}}\right)\cdot\left(\sigma_{tens}(0)\cdot t_s(0)\right) \qquad T_{anchor} = 107.4\cdot\frac{kip}{each}$

$A_{anchor} := \min\left(\frac{3}{8}\cdot in\cdot 3\cdot in, 4\cdot in\cdot\frac{3}{16}\cdot in\right) = 0.75\cdot in^2$ Underside plate controls.

Allowable stress $F_t := 15\cdot ksi = 15\cdot ksi$

$\sigma_{anchor} := \frac{\pi\cdot D\cdot\sigma_{tens}(0)\cdot t_s(0)}{n_{anchors}\cdot A_{anchor}} = 143.241\cdot ksi \qquad \frac{\sigma_{anchor}}{F_t} = 9.549$ >> 1.33 No Good for backing plate

Check stress in embedded plate $A_{anchor} := 3\cdot in\cdot\frac{3}{8}\cdot in = 1.125\cdot in^2$

$$\sigma_{anchor} := \frac{\pi \cdot D \cdot \sigma_{tens}(0) \cdot t_s(0)}{n_{anchors} \cdot A_{anchor}} = 95.494 \cdot ksi \qquad \frac{\sigma_{anchor}}{F_t} = 6.366$$ >> 1.33 No Good for backing plate

Anchors are overstressed

Compute anchor weld load vs allowable

$l_{weld\_longitudinal} := 10 \cdot in$ $l_{weld\_transverse} := 4 \cdot in$ Strap to strap

$t_{weld} := \frac{3}{16} \cdot in$ $F_t = 15000\,psi$ Note: record drawing says fillet weld of strap to shell is 1/4", but plate is only called out as 3/16"

$$T_{allowable} := .7071 \cdot t_{weld} \cdot F_t \cdot \left(.65 \cdot l_{weld\_transverse} + .50 \cdot l_{weld\_longitudinal}\right) = 15.114 \cdot kip$$

$\frac{T_{anchor}}{T_{allowable}} = 7.108$ >1.33 No good for strap to shell weld, even with offset ignored

$l_{weld\_longitudinal} := 10 \cdot in$ $l_{weld\_transverse} := 4 \cdot in$ Strap to strap

$t_{weld} := \frac{1}{4} \cdot in$ $F_t = 15000\,psi$ Note: record drawing says fillet weld of strap to shell is 1/4", but plate is only called out as 3/16"

$$T_{allowable} := .7071 \cdot t_{weld} \cdot F_t \cdot \left(.65 \cdot l_{weld\_transverse} + .50 \cdot l_{weld\_longitudinal}\right) = 20.152 \cdot kip$$

$\frac{T_{anchor}}{T_{allowable}} = 5.331$ >1.33 No good for strap to strap weld, even with offset ignored

Welds are overstressed

Compute embedded plate bond capacity

approximate method, use ACI 318-63 which allows the following allowable bond stress for plain bars

The perimeter of the embedded anchor is $P_{anchor} := (2 \cdot .375 + 2 \cdot 3) \cdot in = 6.75 \cdot in$

(this is for typical anchors only. anchors are shorter over pipe entrance, so capacity is less)

An equivalent round bar diameter would be $D_{equiv} := \frac{P_{anchor}}{\pi} = 2.149 \cdot in$

For deformed bars, the ACI 318-63 allowable bond stress is $F_{bond} := 4.8 \cdot \sqrt{3000} \cdot \frac{in \cdot psi}{D_{equiv}} = 122.362\,psi$

For plain bars $F_{bond} := \min\left(.5 \cdot F_{bond}, 160 \cdot psi\right) = 61.181\,psi$

The embedded length of the anchor, including the hook, is $l_{embed} := 36 \cdot in$

Allowable load based on bond $T_{allowable} := P_{anchor} \cdot l_{embed} \cdot F_{bond} = 14.867 \cdot kip$

$$\frac{T_{anchor}}{T_{allowable}} = 7.226$$

Check Foundation For Uplift and Overturning

$\gamma_{conc} := 150 \cdot pcf$

$b_{ftg} := 1.5 \cdot ft$ $h_{ftg} := 3 \cdot ft$ footing width and depth

$R_{ftg} := R + 9 \cdot in = 27.25\,ft$ $R_{in} := R_{ftg} - b_{ftg}$ footing outside and inside radii

$A_{ftg} := \pi \cdot \left(R_{ftg}^{\ 2} - R_{in}^{\ 2}\right) = 249.757\,ft^2$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 112.4 \cdot kip$ $w_{ftg} := \frac{W_{ftg}}{\pi \cdot D} = 0.675 \cdot klf$ total and unit footing weight

$W_{water} := H \cdot \gamma_{water} \cdot \pi \cdot \left(R^2 - R_{in}^{\ 2}\right) = 242.0 \cdot kip$ $w_{water} := \frac{W_{water}}{\pi \cdot D} = 1.453 \cdot klf$ total and unit weight of water over footing

$\gamma_{soil} := 125 \cdot pcf$ typical weight of compacted soil

$A_{soil} := 0$ area of soil over footing

$A_{wedge} := \frac{(29 \cdot in)^2}{2 \cdot 2} = 1.46\,ft^2$ area of soil resisting uplift in friction at 1H:2V, backfill to within 7" of top of footing. Skin friction assumed 0.4 between footing and soil

$w_{soil} := \gamma_{soil} \cdot \left(A_{soil} + 0.4A_{wedge}\right)$ $w_{soil} = 0.1 \cdot klf$ unit soil resistance

$W_s = 62.466 \cdot kip$ $w_{shell} := \frac{W_s}{\pi \cdot D} = 0.375 \cdot klf$ shell weight

$W_{roof\_edge} = 16.086 \cdot kip$ $w_{roof\_edge} := \frac{W_{roof\_edge}}{\pi \cdot D} = 0.097 \cdot klf$ roof edge weight

Compute overturning safety factor for pivoting about the toe of the shell

$M_{s\_rev} = 11229 \cdot kip \cdot ft$

$$SF_{overturning} := (1 - A_v)\cdot(W_{roof\_edge} + W_s + W_{ftg} + W_{water})\cdot\frac{R}{M_{s\_rev}} = 0.92$$

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Required safety factor based on ASCE 7 load combos is .7E/.6D where .7E is the earthquake load in allowable stress terms, an effective ratio of 1.67

Check ratio of resistance to uplift at the foundation

$$SF_{uplift} := \frac{\left[(1 - A_v)\cdot(w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water}) + w_{soil} - F_{max}\right]}{4\cdot\frac{M_{s\_rev}}{\pi\cdot D^2}} = 0.239$$

< 1.0 so there will be some foundation uplift

Check bearing pressure

The total load on the perimeter under static conditions is

$$w_{static} := w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water} = 2.6\cdot klf$$

$$q_{bearing\_static} := \frac{w_{static}}{b_{ftg}} = 1.733\cdot ksf$$

$$w_{seismic} := (1 + A_v)\cdot(w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water}) + F_{max} + 4\cdot\frac{M_{s\_rev}}{\pi\cdot D^2} = 9.148\cdot klf$$

$$q_{bearing\_seismic} := \frac{w_{seismic}}{b_{ftg}} = 6.099\cdot ksf$$

$q_{allow} := 2.5\cdot ksf$ Static allowable bearing pressure

$$\frac{q_{bearing\_static}}{q_{allow}} = 0.693$$ OK

$$\frac{q_{bearing\_seismic}}{q_{allow}} = 2.44$$ >1.33 NG

Check Stability As Self-Anchored Tank

Per AWWA D100 section 13.5.4.1

$w_t := w_{shell} + w_{roof\_edge} = 472 \cdot \frac{lbf}{ft}$ Weight of shell and roof supported by shell

$t_b := t_{floor} = 0.25 \cdot in$ $F_y := 27 \cdot ksi$ A283 Grade B steel assumed $G := 1.0$ specific gravity

$$w_L := \min\left(1.28 \cdot \frac{H}{ft} \cdot \frac{D}{ft} \cdot G, 7.29 \cdot \frac{t_b}{in}\sqrt{\frac{F_y}{ksi} \cdot \frac{H}{ft} \cdot G}\right) \cdot plf = 53 \cdot \frac{lbf}{ft}$$ Eq 13-37, normalized for units

$$J := \frac{M_s(0)}{D^2 \cdot \left[w_t \cdot \left(1 - 0.4 \cdot A_v\right) + w_L\right]} = 9.446$$

Above value was computed using Ri of 3.0, which is for anchored tanks. Using Ri of 2.5 for unanchored tanks, the corrected value, from a side calculation, is

J = 10.98 **>> 1.54 therefore the tank is not stable without anchorage**

References

1. 2012 *International Building Code*
2. Washington State Adoption of and Amendments to 2012 International Building Code (State Building Code)
3. ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
4. AWWA Standard D100-11 *Welded Carbon Steel Tanks for Water Storage*
5. Nuclear Reactors and Earthquakes, Chap. 6 and Appendix F. U.S. Nuclear Regulatory Commission publication, Division of Technical Information, TID-7024, National Technical Information Service (1963).
6. Not used
7. Not used
8. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" Praveen K. Malhotra, Structural Engineering International, March 2006
9. Not used
10. "Dynamic Pressures on Accelerated Fluid Containers," G.W. Housner, 1955, Bulletin of the Seismological Society of America.
11. "Snow Load Analysis for Washington, 2nd Ed." Structural Engineers Association of Washington,1995
12. Not used
13. Not used
14. ACI 318-11 Building Code Requirements for Structural Concrete
15. ANSI/AISC 360-10 Specification for Structural Steel Buildings
16. AWS D1.1 Structural Welding Code - Steel

Units and Mathcad Notation

All calculations are shown in U.S. customary units. Calculations have been performed using MathSoft's Mathcad Version 14.0 software, which automatically checks for unit consistency and applies any necessary unit conversion factors internally to the program. Where computations are imported from Excel, SAP2000, or other software, the source is identified. Input values are shaded. Others are computed.

Where equations are shown with a ":=" sign, the left hand side of the equation is being defined by the right hand side. Where equations are shown with a "=" sign, the current value of the expression on the left hand side is being displayed.

=	An ordinary "equals" sign indicates the value being shown is for the most current evaluation of the variable on the left hand side of the equation
:=	An "equals" sign with a colon indicates the value on the left hand side is being defined by the expression on the right. Variables may be redefined, the last definition taking precedence
=	A bold "equals" sign indicates the symbol is being used in a logical expression
if(a,b,c)	An "if" statement is evaluated as "b" if "a" is true, and as "c' is "a' is false. These expressions may be nested
(matrix$_{i,j}$)	In matrix expressions, the first subscript is the row, and the second is the column. Numbering starts with the value indicated as "ORIGIN" for the first row and column unless otherwise noted
submatrix (A,i1,i2,j1,j2)	Defines a vector or submatrix of matrix "A" from row i1 thru i2, and column j1 thru j2
-----------> ( )	An expression with a vector arrow over it indicates that the expression involves subscripted variables, and that the expression is being evaluated for each subscript in the range
|	A bold vertical line to the left of a series of expressions indicates that they are acting as a programming loop in the calculations
$ORIGIN := 1$	Sets initial subscript value for subscripted variables
M<j>	The vector in column "j" of matrix "M"
$sf := ft^2$	
$\Phi(x)$	Step function. Returns -1 for x < 0, +1 for x > 0 and .5 if x = 0

BHC CONSULTANTS

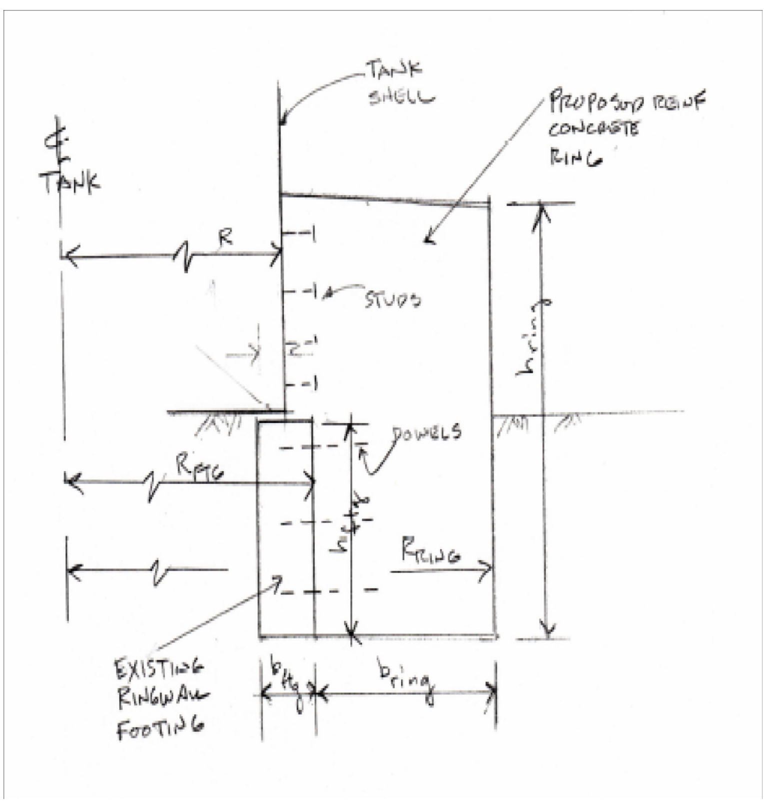
Job No.:15-10420.00 LWWSD
Geneva Reservoir
Sheet No.: 1 of 6
Calculated by: JJL   Date: 2/2/2016
Checked by: Date:_______

Seismic Retrofit
for
Geneva Reservoir-R ingwall Option A

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Existing ringwall and tank dimensions

Existing footing

$R_{ftg} := 26.75 \cdot ft$ outside radius, ex. ftg.

$b_{ftg} := 1.5 \cdot ft$

$h_{ftg} := 3 \cdot ft$

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi \cdot \left(R_{ftg}{}^{2} - R_{in}{}^{2}\right)$ footprint

Additional exterior ring

$h_{ring} := 13 \cdot ft$ Ring depth

$b_{ring} := 11 \cdot ft$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 37.75\,ft$

$A_{gross} := \pi \cdot R_{ring}{}^{2} = 4477\,ft^{2}$

$A_{ring} := A_{gross} - \pi \cdot R_{ftg}{}^{2}$

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0) \quad P_{static} = 520 \cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi) \cdot \frac{(4 \cdot M_s(0))}{\pi \cdot D^2}$$ Seismic load at base of shell from lateral ground motior

$P_{seismic}(0) = 6088 \cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -6088 \cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motic

$P_{seismic\_v} = 21 \cdot plf$

c. Existing footing Dead Load Component

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 110.3 \cdot kip$ Total weight of existing ringwall

$$w_{ftg} := \frac{W_{ftg}}{2 \cdot \pi \cdot R} = 662 \cdot plf$$ Ringwall weight per ft of shell

d. Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r \, dz \, dr \, d\phi \right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring} - h_{ftg}} r \, dz \, dr \, d\phi \right) = 263.152 \cdot cy$$ Ring volume

$W_{ring} := V_{ring} \cdot \gamma_{conc} \quad W_{ring} = 1066 \cdot kip$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 6401 \cdot plf$$ Anchor ring weight per ft of shell

e. Weight of water over footing

$p_{static} := \gamma_{water} \cdot H = 1966 \cdot psf$

$$w_{water} := p_{static} \cdot \frac{A_{ftg}}{2 \cdot \pi \cdot R} \qquad w_{water} = 2893 \cdot plf$$

f. Seismic pressure increase/decrease on footing

(base pressure functions hidden below for brevity)

$\Delta p := p_{base}(R, 0) = 498 \cdot psf$ Plus or minus water pressure at the toe or heel of the tank due to seismic effect

$$w_{seismic} := \int_{\frac{-\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R} p_{base}(r, \phi) \cdot \frac{r}{ft} \, dr \, d\phi$$

Calculate the required shear transfer capacity between footing and new anchor ring per foot of shell

$SF_{ot} := 1.67$ specified safety factor

$Uplift := P_{seismic}(0)$ $Uplift = 6.088 \cdot klf$ Transfer force at face of shell

The resistance of various components is

$D_{tank\_resist} := P_{static} \cdot (1 - .4 \cdot A_v) = 0.499 \cdot klf$

$w_{water\_resist} := (1 - .4 \cdot A_v) \cdot w_{water} - w_{seismic} = 2.685 \cdot klf$

$w_{ftg\_resist} := w_{ftg} \cdot (1 - .4 \cdot A_v) = 0.636 \cdot klf$

$w_{ring\_resist} := w_{ring} \cdot (1 - .4 \cdot A_v) = 6.147 \cdot klf$

Check uplift safety factor with added block

$Resistance := D_{tank\_resist} + w_{water} + w_{ftg} + w_{ring} = 10.455 \cdot klf$

$$SF_{check} := \frac{Resistance}{Uplift} = 1.717$$

$$\boxed{\frac{SF_{check}}{SF_{ot}} = 1.028}$$ $\boxed{> 1.0 \text{ OK}}$

The required shear transfer force between the shell and foundation is

$$Anchor\_load\_shell := \left(w_{water} + w_{ftg} + w_{ring}\right) = 9956 \cdot plf$$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Ring\_dowels := \left(w_{water} + w_{ftg}\right) = 3555 \cdot plf$$

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$s_{studs} := 35 \cdot in$ horizontal stud spacing

$s_{studs\_vert} := 20 \cdot in$

$$n_{studs\_per\_row} := \frac{\left(h_{ring} - h_{ftg}\right)}{s_{studs\_vert}} = 6$$

$$Load\_per\_stud := s_{studs} \cdot \frac{Anchor\_load\_shell}{n_{studs\_per\_row}} = 4840 \cdot lbf$$

$$V_u := \Omega_o \cdot 1.4 \cdot Load\_per\_stud = 13551\ lbf$$

Shear strength for a 5/8" Nelson stud is $stud\_capacity := 15113 \cdot lbf$ per AISC for f'c=4.5 ksi, Fu=65 ksi

$\phi_{shear} := .90$ $$\frac{V_u}{\phi_{shear} \cdot stud\_capacity} = 0.996$$ < 1.0 OK

From Ref 17, Table 3-21, for normal weight concrete, f'c = 3 ksi, Fu = 65*ksi, 1/2" studs, the nominal shear capacity is $Q_N := 9.35 \cdot kip$

$f'_c := 4 \cdot ksi$

$$\frac{Q_N}{.85 \cdot f'_c} = 2.75 \cdot in^2$$ concrete crushing not an issue

$DCR := \frac{V_u}{\phi_{shear} \cdot Q_N} = 1.61$ OK Assume 4 each 1/2" studs at 18" o.c. EW

Assume similar for deformed bar dowels into exist ringwall.

Quantities

$$N_{studs} := n_{studs\_per\_row} \cdot \pi \cdot \frac{D}{s_{studs}} = 343$$

Assume same for dowels @ 1/2"

$$N_{dowels} := N_{studs} \cdot \frac{h_{ftg}}{h_{ring} - h_{ftg}} = 102.757$$

$$V_{conc} := \pi \cdot \left(R_{ring}^2 - R_{ftg}^2\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}^2 - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 263 \cdot cy$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg}^2 = 2716\,ft^2$$

$$R_{ring} + 2 \cdot ft - R_{ftg} = 13\,ft$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^2 - \pi \cdot R_{ftg}^2 = 3493\,ft^2$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi \cdot R_{ftg}^2 = 3098\,ft^2$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 689 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}^2 - R_{ftg}^2\right) \cdot h_{ftg} = 441.219 \cdot cy$$

Add 3/16" shell plate to bottom 2.25 ft

$$2.25 \cdot ft \cdot \pi \cdot D \cdot 7.66 \cdot psf = 2870 \cdot lbf$$

Seismic Evaluation

for

Geneva Reservoir-Ringwall Option C

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Preliminary Design of Anchored Tank

General layout similar to Sumner Springs Reservoir shown below

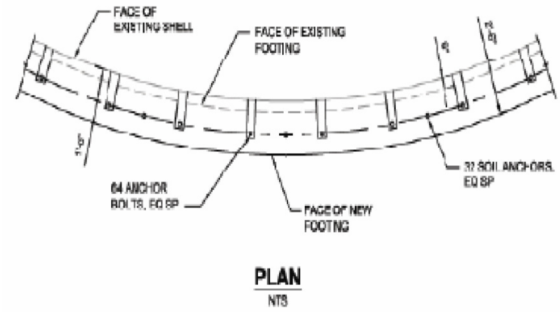


Figure 5. Enlarged Foundation Plan Detail

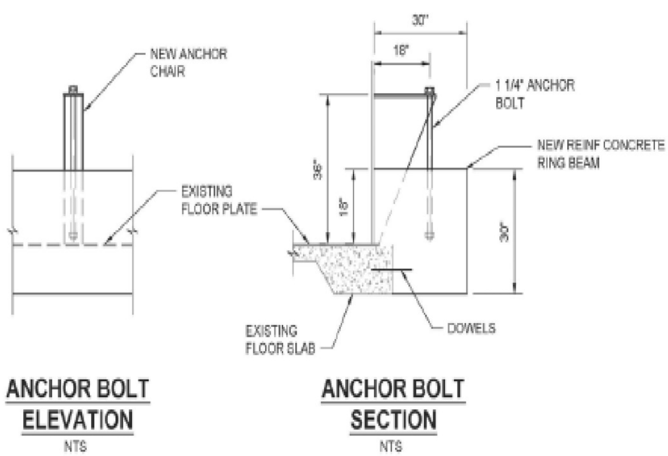
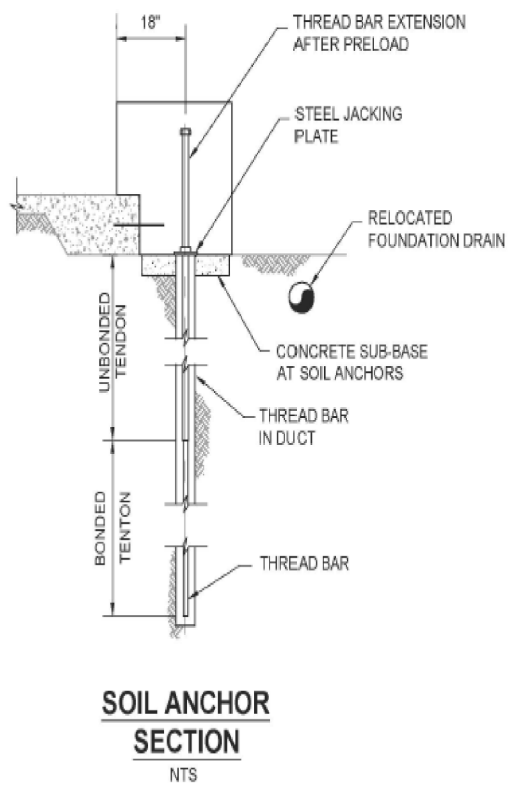
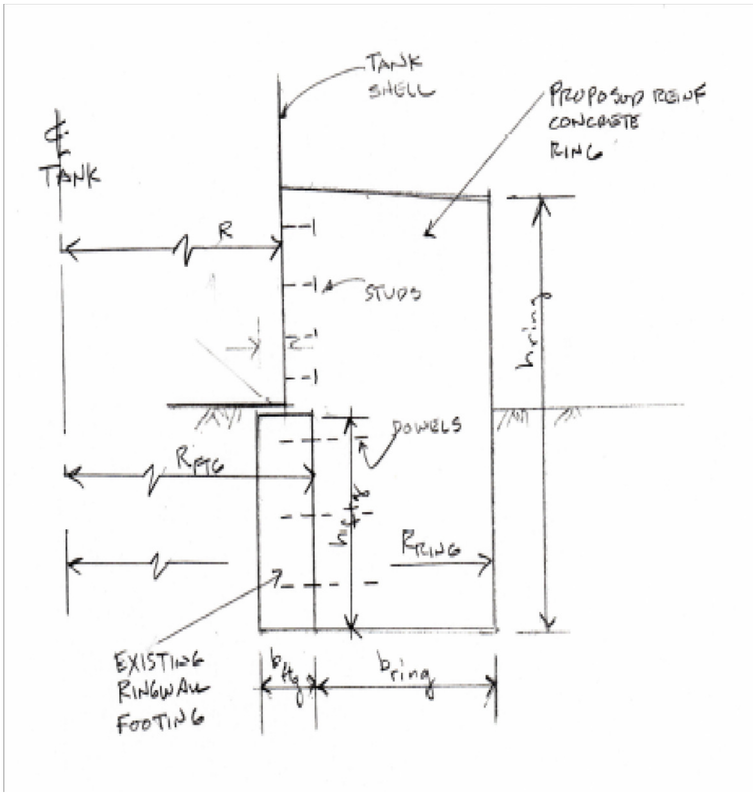


Figure 6. Anchor Bolt and Chair Detail



Supplemental units and unit weights

$$cy := yd^3$$



Existing ringwall and tank dimensions

Existing footing

$R_{ftg} := 26.75 \cdot ft$ outside radius, ex. ftg.

$b_{ftg} := 1.5 \cdot ft$

$h_{ftg} := 3 \cdot ft$

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi \cdot \left(R_{ftg}^{\ 2} - R_{in}^{\ 2} \right)$ footprint

Additional exterior ring

$h_{ring} := 3 \cdot ft$ Ring depth

$b_{ring} := 30 \cdot in$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 29.25\,ft$

$A_{gross} := \pi \cdot R_{ring}^{\ 2} = 2688\,ft^2$

$A_{ring} := A_{gross} - \pi \cdot R_{ftg}^{\ 2}$

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0)$ $P_{static} = 520 \cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi) \cdot \frac{\left(4 \cdot M_s(0)\right)}{\pi \cdot D^2}$$ Seismic load at base of shell from lateral ground motion

$P_{seismic}(0) = 6088 \cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -6088 \cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 21 \cdot plf$

c. Existing footing Dead Load Component

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 110.3 \cdot kip$ Total weight of existing ringwall

$w_{ftg} := \dfrac{W_{ftg}}{2 \cdot \pi \cdot R} = 662 \cdot plf$ Ringwall weight per ft of shell

d. Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\, dz\, dr\, d\phi\right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring}-h_{ftg}} r\, dz\, dr\, d\phi\right) = 48.869 \cdot cy$$ Ring volume

$W_{ring} := V_{ring} \cdot \gamma_{conc}$ $W_{ring} = 198 \cdot kip$

$w_{ring} := \dfrac{W_{ring}}{2 \cdot \pi \cdot R} = 1189 \cdot plf$ Anchor ring weight per ft of shell

e. Weight of water over footing

$p_{static} := \gamma_{water} \cdot H = 1966 \cdot psf$

$$w_{water} := p_{static} \cdot \frac{\pi \cdot \left(R^2 - R_{in}^{\;2}\right)}{2 \cdot \pi \cdot R}$$

f. Seismic pressure increase/decrease on footing

$w_{water} = 2399 \cdot plf$

(base pressure functions hidden below for brevity)

$\Delta p := p_{base}(R, 0) = 498 \cdot psf$ Plus or minus water pressure at the toe or heel of the tank due to seismic effects

$$w_{seismic} := \int_{\frac{-\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R} p_{base}(r, \phi) \cdot \frac{r}{ft}\, dr\, d\phi$$

$w_{seismic} = 93.289 \cdot plf$

Calculate the required anchor transfer capacity between tank and new anchor ring per foot of shell

$SF_{ot} := 1.67$ target safety factor

$Uplift := P_{seismic}(0)$ $Uplift = 6.088 \cdot klf$ Transfer force at face of shell

The resistance of various components is

$D_{tank\_resist} := P_{static} \cdot (1 - .4 \cdot A_v) = 0.499 \cdot klf$

$w_{water\_resist} := (1 - .4 \cdot A_v) \cdot w_{water} - w_{seismic} = 2.211 \cdot klf$

Set number of anchors and compute load. Assume three new anchors between each of the 12 existing

$n_{anchors} := 36$ $s_{anchor} := \pi \cdot \frac{D}{n_{anchors}} = 4.625\,ft$

$T_{anchor} := \frac{\left[\pi \cdot D \cdot \left(Uplift - D_{tank\_resist} - w_{water\_resist}\right)\right]}{n_{anchors}} = 15.627 \cdot kip$ measured at the shell

Resistance provided by ring $w_{ring} = 1.189 \cdot klf$

Resistance required by ground anchors

$Ground\_anchor\_resist := SF_{ot} \cdot (Uplift) - D_{tank\_resist} - w_{water\_resist} - w_{ring} = 6.269 \cdot klf$

$ground\_anchor\_capacity\_ASD := 75 \cdot kip$

$n_{ground\_anchors} := 18$ provide one ground anchor for every two anchors

$ground\_anchor\_load := Ground\_anchor\_resist \cdot \pi \cdot \frac{D}{n_{ground\_anchors}} = 57.992 \cdot kip$

$s_{ground\_anchor} := \pi \cdot \frac{D}{n_{ground\_anchors}} = 9.25\,ft$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$\text{Ring\_dowels} := \left(w_{water} + w_{ftg}\right) = 3061 \cdot \text{plf}$$

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$s_{dowels} := s_{anchor} = 4.625\,\text{ft}$ $\quad n_{dowels\_per\_row} := 3$

$$\text{Load\_per\_dowel} := \frac{s_{dowels}}{s_{anchor}} \cdot \frac{T_{anchor}}{n_{dowels\_per\_row}} = 5209 \cdot \text{lbf}$$

Half inch dowels should be more than enough $\quad n_{dowels} := n_{anchors} \cdot n_{dowels\_per\_row} = 108$

Quantities

$n_{dowels} = 108$ $\quad n_{anchors} = 36$ $\quad n_{ground\_anchors} = 18$

$$V_{conc} := \pi \cdot \left(R_{ring}{}^2 - R_{ftg}{}^2\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}{}^2 - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 49 \cdot \text{cy}$$

By compariison to Sumner Springs reservoir, assume reinforcement at $\quad \text{steel\_unit} := 210 \cdot \frac{\text{lbf}}{\text{cy}}$

$$\text{rebar} := V_{conc} \cdot \text{steel\_unit} = 10263\,\text{lbf}$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi\cdot\left(R_{ring} + 2\cdot ft\right)^2 - \pi\cdot R_{ftg}^{\ 2} = 820\,ft^2$$

$$A_{top} := \pi\cdot\left(R_{ring} + 2\cdot ft + h_{ftg}\right)^2 - \pi\cdot R_{ftg}^{\ 2} = 1437\,ft^2$$

$$A_{mid} := \pi\cdot\left(R_{ring} + 2\cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi\cdot R_{ftg}^{\ 2} = 1122\,ft^2$$

$$R_{exc} := R_{ring} + 2\cdot ft + h_{ftg} - R_{ftg} = 7.5\,ft$$

$$V_{exc} := \frac{h_{ftg}}{3}\cdot\left(A_{bot} + 4\cdot A_{mid} + A_{top}\right) = 250\cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi\cdot\left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right)\cdot h_{ftg} = 200.887\cdot cy$$

Anchor Bolt Sizing

Assume A36 anchor bolts $F_y := 36 \cdot ksi$ $F_u := 58 \cdot ksi$

$F_{anchor} := \min(.80 \cdot 36 \cdot ksi, .50 \cdot 58 \cdot ksi) = 28.8 \cdot ksi$ Allowable seismic load stress on anchors per Ref 5 section 3.3.3.2

$A_{root\_min} := \frac{T_{anchor}}{F_{anchor}} = 0.543 \cdot in^2$ $d_{root\_calc} := \sqrt{\frac{4}{\pi} \cdot A_{root\_min}} = 0.831 \cdot in$

Per Ref 5, 3.8.5.1, add a .25" corrosion allowance to the root diameter for bolts less than 1.25", and use not less than a 1" bolt. This makes an 1.25" bolt the practical minimum

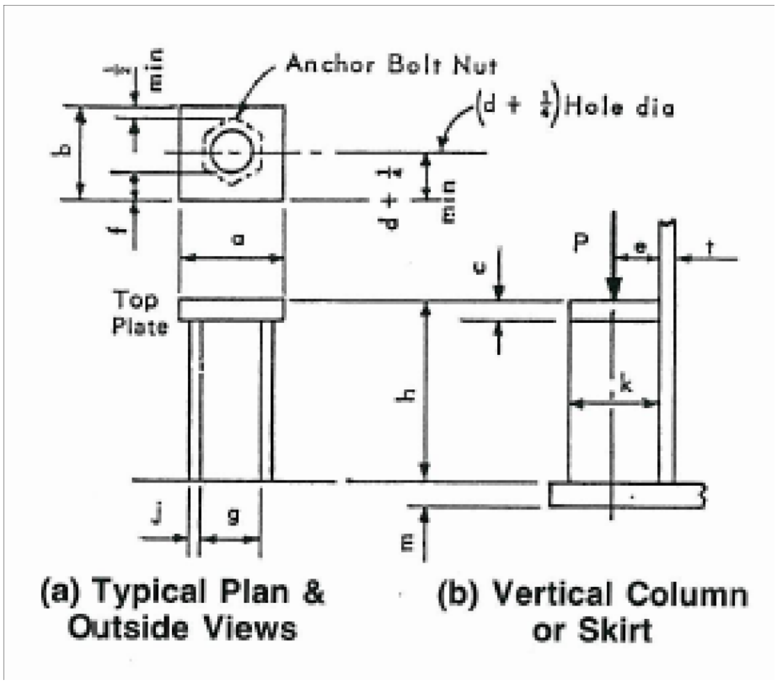
Bolt Dia (in)	Root Dia (in)	Root Area (in^2)	Gross Area (in^2)	Root Dia + .25" (in)	Min Bolt Dia (in)
1.000	0.865	0.587	0.785	1.115	1.375
1.125	0.970	0.74	0.994	1.220	1.500
1.250	1.100	0.942	1.23		1.250
1.375	1.190	1.12	1.49		1.375
1.500	1.320	1.37	1.77		1.500
1.750	1.530	1.85	2.41		1.750
2.000	1.760	2.43	3.14		2.000

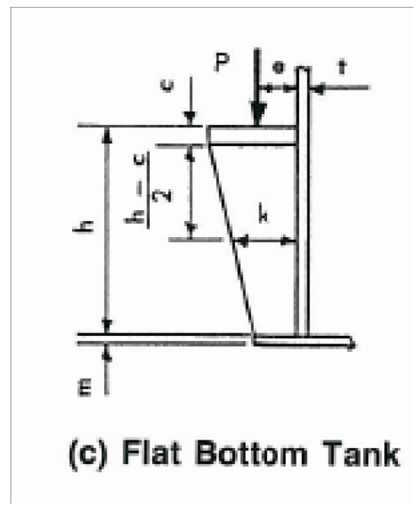
Ref 10, Table 7-18

$d := 1.25 \cdot in$ anchor diameter $A_{bolt} := \pi \cdot \frac{d^2}{4} = 1.227 \cdot in^2$ gross area of bolt

Anchor Chair Design

Methodology is from Ref 11, Part VII - Anchor Bolt Chairs





$e := 18 \cdot in$ bolt centerline distance from shell

Minimum bolt hole size per Ref 11 is

Oversized hole size per Ref 18 Table J.3.3 is $d + \frac{5}{16} \cdot in = 1.563 \cdot in$ for bolts >= 1.25 in. Use

$d_{hole} := d + \frac{5}{16} \cdot in$ $d_{hole} = 1.563 \cdot in$

Edge distance per Ref 10 Tables J.3.4 and J3.5 (from center of hole) is

$c_{edge} := 2.25 \cdot in + \frac{1}{8} \cdot in = 2.375 \cdot in$

$b := e + c_{edge} = 20.375 \cdot in$

$f := c_{edge} - \frac{d_{hole}}{2} = 1.594 \cdot in$

$g := d + 1 \cdot in = 2.25 \cdot in$ minimum side plate separation recommended by Ref 21, however this is very tight for seal welding on interior of plates. Increase this dimension to

$g := 8 \cdot in$

$t := t_s(0) \qquad t = 0.344 \cdot in$ Shell bottom course thickness

$P := T_{anchor} = 15.627 \cdot kip$

$S := 1.33 \cdot 15 \cdot ksi = 19.95 \cdot ksi$ Ref 4 allowable stress < 25 ksi recommended by Ref 11 OK

Compute top plate thickness

$$c_{min} := \left[\frac{P}{S \cdot f} \cdot (0.37 \cdot g - 0.22 \cdot d)\right]^{.5} = 1.149 \cdot in$$

use $c := 1.5 \cdot in$

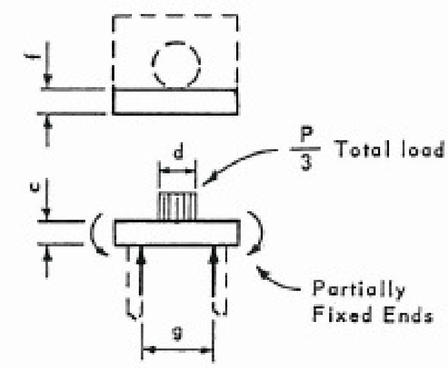


Figure 7-2. Assumed Top-Plate Beam.

top plate thickness

$h := 27 \cdot in$

$j_{min} := \max[.5 \cdot in, 0.04 \cdot (h - c)] = 1.02 \cdot in$ use $j := 1in$

$m := .25 \cdot in$ bottom plate thickness assumption $proj := 2 \cdot in - t$ bottom plate projection from shell face

$a := g + 2 \cdot j + .5 \cdot in = 10.5 \cdot in$ $>$ $2 \cdot c_{edge} = 4.75 \cdot in$ OK   Use $a := 12 \cdot in$

Recess the side plate not more than 1/2" from front edge of top plate per Ref 21. Use .25" to allow seal weld at front edge.

$plate\_top := b - .25 \cdot in \qquad k := \dfrac{(plate\_top + proj)}{2} = 10.891 \cdot in$ mean side plate width

$$\frac{j \cdot k}{\dfrac{P \cdot in^2}{25 \cdot kip}} = 17.423$$ > 1.0 OK per Ref 21

Compute reduction factor Z for local stress check

$$Z := \frac{1.0}{\dfrac{(.177 \cdot a \cdot m)}{in\sqrt{R \cdot t}} \cdot \left(\dfrac{m}{t}\right)^2 + 1.0} = 0.974$$

$$S := \frac{P \cdot e}{in\,t^2} \cdot \left[\frac{1.32 \cdot Z}{\left(\frac{1.43 \cdot a \cdot h^2}{R \cdot t \cdot in} + \left(4 \cdot \frac{a}{in^3} \cdot h^2 \right) \right)^{.333}} + \frac{.031 \cdot in}{\sqrt{R \cdot t}} \right] \qquad S = 27.869 \cdot ksi$$

localized vertical shell stress just above the chair. Ref 21 recommends 25ksi max.

Weld Design

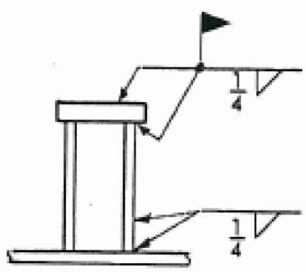


Figure 7-4. Typical Welding, Base Plate Shop Attached.

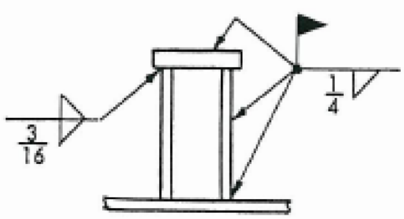


Figure 7-5. Typical Welding, Base or Bottom Field Attached.

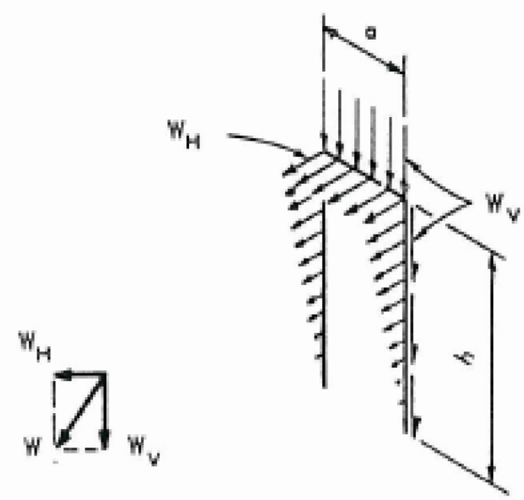


Figure 7-6. Loads on Welds.

$$W_v := \frac{P}{a + 2 \cdot h} = 237 \cdot \frac{lbf}{in} \qquad W_h := \frac{P \cdot e}{a \cdot h + 0.667 \cdot h^2} = 347 \cdot \frac{lbf}{in}$$

$$W := \sqrt{W_v^2 + W_h^2} = 420 \cdot \frac{lbf}{in}$$

By inspection, a .25" weld will be more than adequate.

Shell shear capacity per inch exceeds weld, OK

Anchor Quantities

$$V_{bp} := a \cdot b \cdot c \qquad V_{bp} = 366.75 \cdot in^3$$

$$V_{sp} := 2 \cdot \frac{(b + 2 \cdot in) \cdot (h - c) \cdot j}{2} \qquad V_{sp} = 570.563 \cdot in^3$$

$$W_{anchor} := \gamma_{steel} \cdot \left(V_{bp} + V_{sp}\right) = 265.789\,lbf$$

$$W_{anchor\_total} := W_{anchor} \cdot n_{anchors} = 9568\,lbf$$

$$L_{weld} := 2 \cdot h + a + (a - g - 2 \cdot j) = 68 \cdot in$$

$$L_{weld\_total} := n_{anchors} \cdot L_{weld} = 204 \cdot ft$$

Seismic Evaluation
for

Geneva Reservoir-Ringwall Option D

for

Lake Whatcom Water & Sewer District
Bellingham, WA

Compute mat weight and location of center of gravity above the base

$h_{mat} := 2 \cdot ft$ Mat thickness $BCL_{exist} := 0$ $cy := 27 \cdot ft^3$

Existing Bottom Capacity Level (elevation of base of tank)

$BCL := BCL_{exist} + h_{mat}$ $BCL = 2\,ft$ Bottom Capacity Level (water elevation at top of mat)

$MOL := H$ Assumed maximum operating level

$TCL := 655.5 \cdot ft$ Top Capacity Level (elevation at lip of overflow)

$D = 53\,ft$ Shell diameter

$A_{tank} := \pi \cdot \frac{D^2}{4}$ $A_{tank} = 2206\,ft^2$ Tank footprint

$V_{mat} := A_{tank} \cdot (BCL - BCL_{exist})$ $V_{mat} = 163.4 \cdot cy$

$\gamma_{conc} := 150 \cdot pcf$ Unit weight of concrete

$W_{mat} := V_{mat} \cdot \gamma_{conc}$ $W_{mat} = 662 \cdot kip$ $X_{mat} := \frac{h_{mat}}{2}$ $X_{mat} = 1\,ft$

Compute existing floor plate weight

$Floor\_flange := 2 \cdot in$ Bottom plate projection beyond shell plate

$D_{plate} := D + 2 \cdot Floor\_flange$ $D_{plate} = 53.333\,ft$

$t_{plate} := .25 \cdot in$ $W_f := \gamma_{steel} \cdot t_{plate} \cdot \pi \cdot \frac{D_{plate}^{\,2}}{4}$ $W_f = 23 \cdot kip$

Compute weight of assumed steel plate installed above mat to seal the bottom

$t_{seal} := .25 \cdot in$ $W_{seal} := \gamma_{steel} \cdot t_{seal} \cdot \pi \cdot \frac{D^2}{4}$ $W_{seal} = 23 \cdot kip$ $X_{seal} := h_{mat}$

Hydrodynamic Wall Pressure Functions

$MOL := H \qquad BCL_{exist} := 0$

$z_f := z$

$z(z_f) := z_f - h_{mat}$

$z(z_f) := \text{if}\left[\left[z_f > \left(MOL - BCL_{exist}\right)\right], H, z(z_f)\right]$ z cannot be greater than H when calculating water effects

Define fluid pressure functions

Hydrodynamic pressures due to impulsive and convective lateral loads vary around the shell as a function of the angle from the toe of the tank, ϕ. (See Ref 5)

The pressure distribution for impulsive forces is proportional to the function

$$\Psi_i(\phi) := \cos(\phi)$$

The pressure distribution for convective forces is proportional to the function

$$\Psi_c(\phi) := \cos(\phi)\cdot\left(1 - \frac{1}{3}\cdot\cos(\phi)^2\right)$$

Half of the impulsive and convective base shear, taken at the top of the mat, is represented by the region where $-\pi/2 < \phi < \pi/2$

$$\frac{V_i}{2} = 435.001\cdot kip \qquad \frac{V_c}{2} = 62.518\cdot kip$$

The maximum impulsive pressure distribution is

$$p_i(z_f) := \left(\frac{V_i}{2\cdot R}\right)\cdot\left(\frac{1}{\int_{\frac{-\pi}{2}}^{\frac{\pi}{2}} \Psi_i(\phi)\cdot\cos(\phi)\,d\phi}\right)\cdot Dist_i(z(z_f))$$

The maximum convective pressure distribution is

$$p_c(z_f) := \left(\frac{V_c}{2\cdot R}\right)\cdot\left(\frac{1}{\int_{\frac{-\pi}{2}}^{\frac{\pi}{2}} \Psi_c(\phi)\cdot\cos(\phi)\,d\phi}\right)\cdot Dist_c(z(z_f))$$

The static and vertical hydrodynamic wall pressures are

$$p_{static}(z_f) := \gamma_{water}\cdot Y(z(z_f))$$

$$p_z(z_f) := A_v\cdot p_{static}(z_f)$$

Set pressures equal to zero unless $h_{mat} < z_f < H + h_{mat}$

$$p_i(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_i(z_f)\right]\right]$$

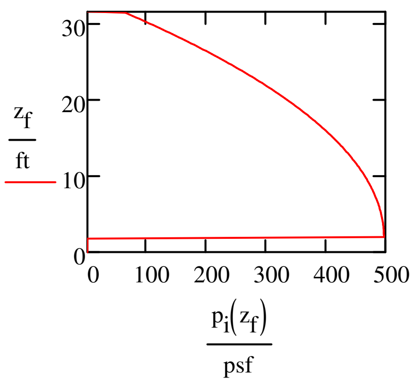
$$p_c(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_c(z_f)\right]\right]$$

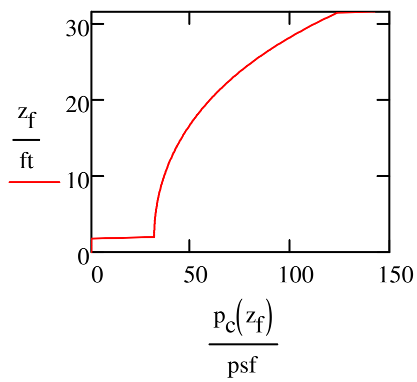
$$p_{static}(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_{static}(z_f)\right]\right]$$

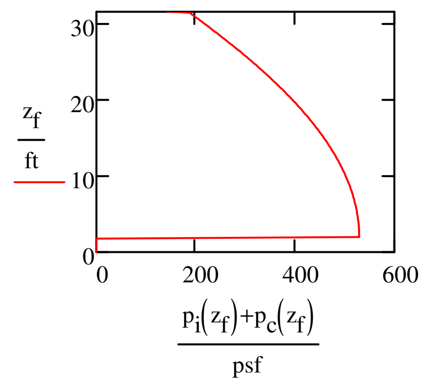
$$p_z(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_z(z_f)\right]\right]$$

$$p_i(5 \cdot ft) = 493.115 \cdot psf$$

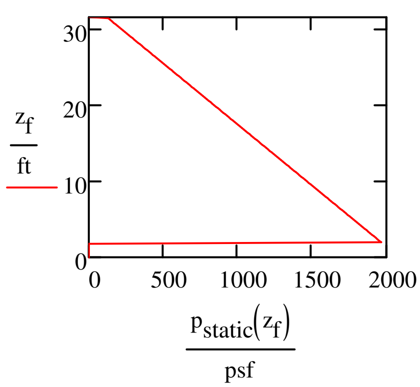
The maximum hydrodynamic impulsive, convective, and combined wall pressures are graphed below vs z_f at $\phi = 0$

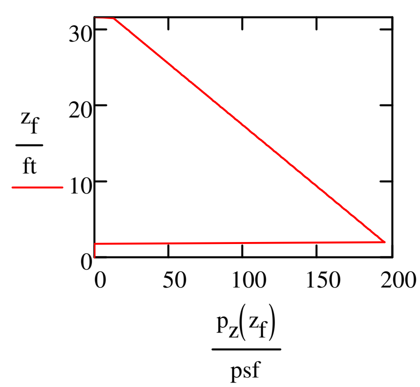






The static and vertical seismic wall pressures are graphed below for all φ
f





Hydrodynamic pressures are added (or subtracted) from hydrostatic pressure to obtain net water fluid pressures, along with the vertical seismic pressure (+ or -). Use the slightly higher straight addition values for the impulsive and convective components so the sign of the pressure will be correct when integrating over the mat surface. When using direct sum instead of SRSS (square root of the sum of the squares) Ref 4 allows the vertical acceleration component to be taken as .40Av. (See Ref 4 section 13.5.4.3)

The base pressure varies in a more complicated way and is computed in the following section

Calculate Loads to Foundation

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0) \quad P_{static} = 520 \cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi) \cdot \frac{(4 \cdot M_s(0))}{\pi \cdot D^2}$$
Seismic load at base of shell from lateral ground motion

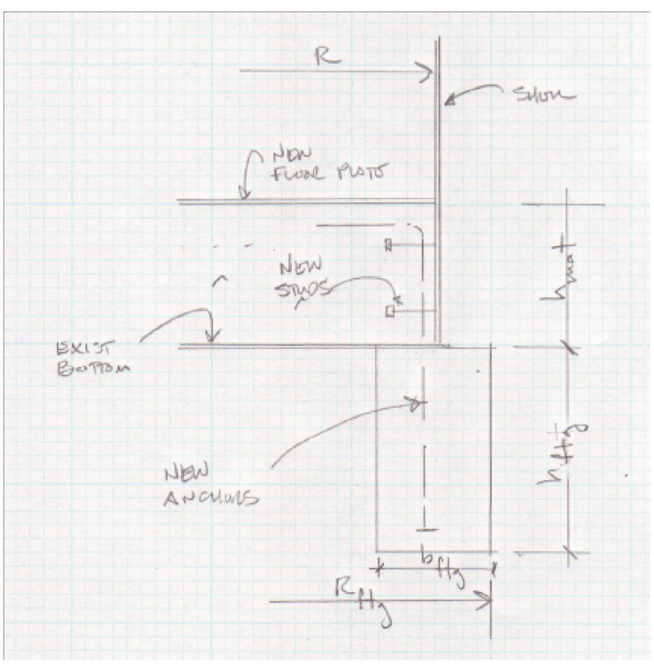
$P_{seismic}(0) = 6088 \cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -6088 \cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 21 \cdot plf$

c. Ringwall Dead Load Component



$R_{ftg} := 51.5 \cdot ft$ from as-built topo

$b_{ftg} := 2 \cdot ft$ from impact-echo measurement

$h_{ftg} := 4 \cdot ft$ field measurement

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$$A_{ftg} := \pi \cdot \left(R_{ftg}^{\;2} - R_{in}^{\;2}\right)$$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} \quad W_{ftg} = 380.761 \cdot kip$

Total weight of existing ringwall

$$w_{ftg} := \frac{W_{ftg}}{2 \cdot \pi \cdot R} = 2287 \cdot plf$$
Ringwall weight per ft of shell

Calculate the radial centroid for the ringwall area

$\theta_1 := \frac{1 \cdot ft}{R}$ tank angle subtended by one ft of shell length

$$A_{ringwall} := \int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r \, dr \, d\theta \qquad A_{ringwall} = 3.811\, ft^2$$

ringwall footprint per foot of shell

$$r_{ringwall} := \frac{\int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r^2 \, dr \, d\theta}{A_{ringwall}} \qquad r_{ringwall} = 50.507\, ft$$

Radial distance to ringwall center of gravity

d. Mat and New Floor Plate Unit Weight

$$w_{mat} := \frac{\left(W_{mat} + W_{seal}\right)}{\pi \cdot R^2} \qquad w_{mat} = 310 \cdot psf$$

The required safety factor is not stated directly in the design standards Ref 1 and Ref 3, nor for anchored tanks in Ref 4. It may be inferred from Ref 3 section 12.14.8.4 and the load combinations in Ref 3 section 2.4.

Safety factor >= 0.75 (from 12.14.8.4) * .98 (0.7 earthquake load factor x 1.4 scale up factor to convert Ref 4 earthquake loads to Ref 3 basis) / 0.6 (dead load factor, Ref 3 equation 8, section 3.2.4.1) = 1.23

e. Check Sliding Safety Factor

$V_f = 913 \cdot kip$ Base shear at base of mat

Weight of soil confined by ringwall $A_{soil} := \pi \cdot R_{in}^2$ $\gamma_{soil} := 125 \cdot pcf$ $W_{soil} := \gamma_{soil} \cdot A_{soil} \cdot h_{ftg}$

Ratio of base shear to total dead weight at the plane defined by the base of the footing

$$V_{allow} := \tan(30 \cdot deg) \cdot \left(W_s + W_r + W_i + W_c + W_f + W_{mat} + W_{seal} + W_{ftg} + W_{soil}\right) \cdot \left(1 - .40 \cdot A_v\right)$$

$V_{allow} = 5173 \cdot kip$ Ref 4 Eq 13-57

$SF_{sliding} := \frac{V_{allow}}{V_f}$ $SF_{sliding} = 5.665$ > 1.0 OK for sliding

f. Check Overturning Safety Factor about the Base of the Mat

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

Calculate overturning moment at the base of the mat

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2}$$ Ref 4 Eq 13-23

$M_s = 9162 \cdot kip \cdot ft$ $M_{ssave} := M_s$ placeholder for later calculation

$$M_{ssum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_c$$

$M_{shell} := M_{ssum}$ placeholder for later calculation

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2}$$ Ref 4 Eq 13-32

$M_{mf} = 15664 \cdot kip \cdot ft$ Result using SRSS method

Results using straight sum method (more conservative)

$$M_{mfsum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_{cmf}$$

$M_{mfsum} = 18895 \cdot kip \cdot ft$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left(W_s + W_r + W_f + W_i + W_{mat} + W_{seal}\right)\right]^2 + \left(A_c \cdot W_c\right)^2}$$ Ref 4 Eq 13-31

$V_f = 890 \cdot kip$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

$M_{mfsum} = 18895 \cdot kip \cdot ft$ Total overturning moment about the base of the mat, including base pressure effects

$$W_{resist} := \left(1 - .40 \cdot A_v\right) \cdot \left(W_s + W_r + W_i + W_c + W_{mat} + W_{seal} + W_{ftg}\right) \qquad W_{resist} = 5241 \cdot kip$$

$$M_{res} := W_{resist} \cdot R = 138889 \cdot kip \cdot ft$$

$$SF_{ot} := \frac{M_{res}}{M_{mfsum}} \qquad SF_{ot} = 7.35$$

Global safety factor against overturning without regard to uplift, soil pressure, or concrete capacity

g. Check Pressure at Base of Mat Floor Plate - Static - Rigid Mat Assumption

$$q_{static} := \frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2} + \left(H - h_{mat}\right) \cdot \gamma_{water} \qquad q_{static} = 4339 \cdot psf$$

Weight of structure and water at emergency operating level applied uniformly to the mat.

h. Check Soil Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Down

$$q1_{max} := \left(1 + .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q1_{max} = 3890 \cdot psf$$

$$q1_{min} := \left(1 + .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] - \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q1_{min} = 1304 \cdot psf$$

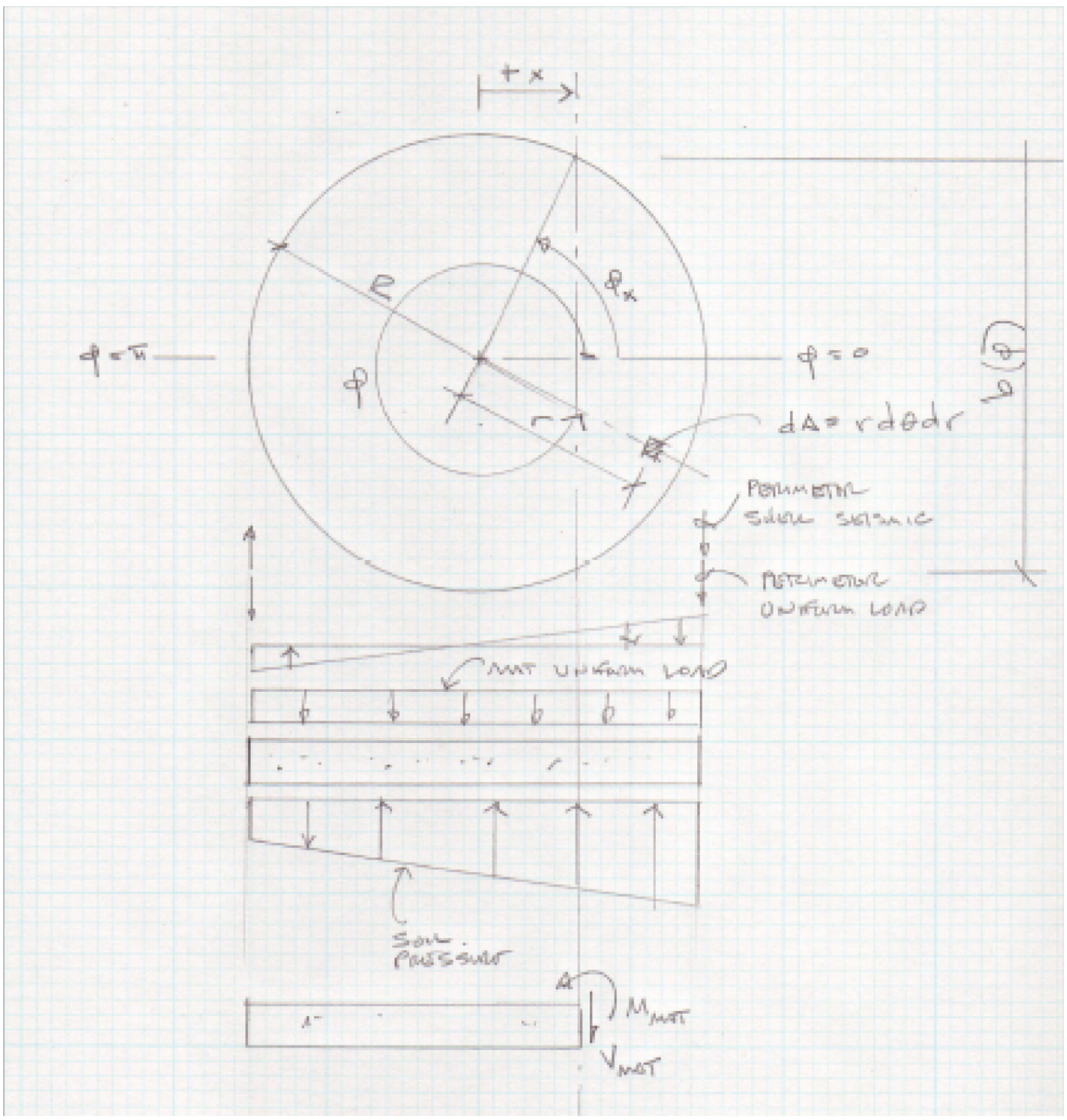
i. Check Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Up

$$q2_{max} := \left(1 - .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q2_{max} = 3692 \cdot psf$$

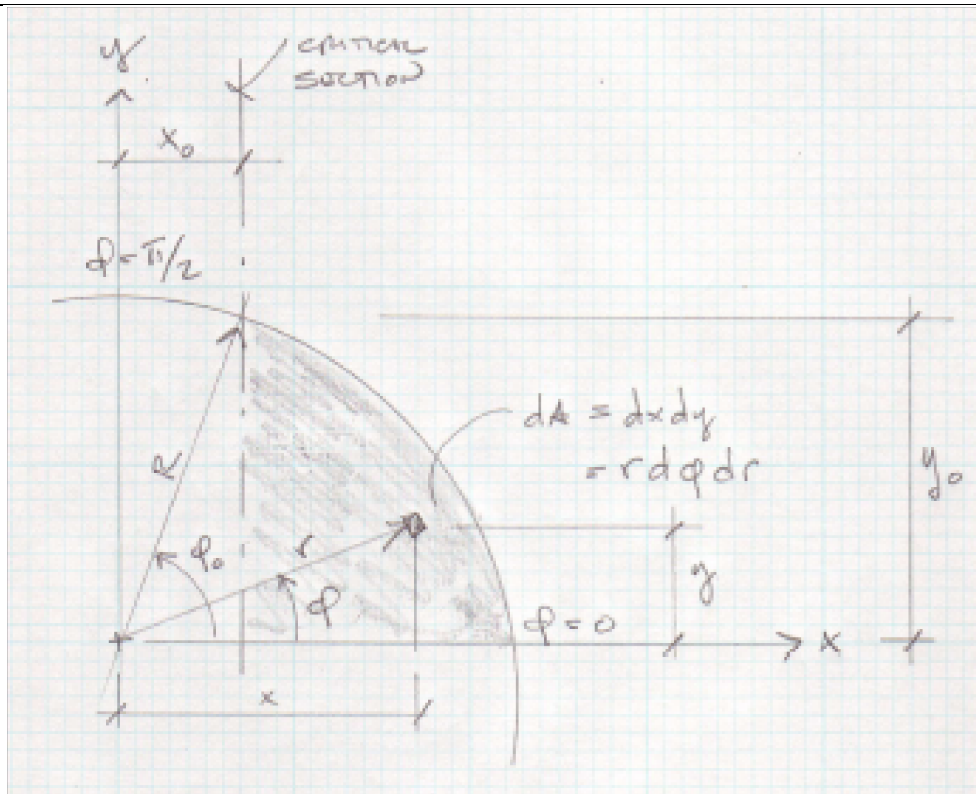
$$q2_{min} := (1 - .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2} \right] - \frac{4M_{mfsum}}{\pi \cdot R^3}$$

$q2_{min} = 1106 \cdot psf$

i. Compute the mat shear and moment under seismic load



(1) First define some basic geometric relationships for the range $0 < \varphi < \pi$



$x(r,\varphi) := r\cdot\cos(\varphi)$ $y(r,\varphi) := r\cdot\sin(\varphi)$ x,y coordinates as functions of polar coordinates r,φ

$r(x,y) := \sqrt{x^2+y^2}$ $\varphi(x,y) := \text{angle}(x,y)$ polar coordinates as funtions of x,y coordinates

$\varphi_o(x_o) := \text{acos}\left(\frac{x_o}{R}\right)$ $y_o(x_o) := R\cdot\sin(\varphi_o(x_o))$ coordinates of x_o intercept with shell

$x_p(\varphi) := x(R,\varphi)$ $y_p(\varphi) := y(R,\varphi)$ Coordinates of the shell perimeter vs angle from toe

$y_R(x_R) := \sqrt{R^2 - x_R^2}$ $y'_R(x_R) := \frac{d}{dx_R}y_R(x_R)$ Equation for the shell perimeter and its derivative

$L(y) := \sqrt{R^2 - y^2}$

(2) Define functions for soil pressure and for associated mat shear and moment

Write soil pressure functions vs x (soil pressure must be greater than zero at all locations)

$q1_{av} := \frac{(q1_{max} + q1_{min})}{2}$ $q1(x) := q1_{av} + \left(\frac{x}{R}\right)\cdot(q1_{max} - q1_{av})$

$q2_{av} := \frac{(q2_{max} + q2_{min})}{2}$ $q2(x) := q2_{av} + \left(\frac{x}{R}\right)\cdot(q2_{max} - q2_{av})$ Case of vertical seismic loads up

Write functions for shear and moment due to soil pressure at section cut xo due to total soil reaction to the right of the cut

$$Vq1(x_o) := 2\cdot\int_{x_o}^{R} q1(x)\sqrt{R^2 - x^2}\,dx \qquad Mq1(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q1(x)\sqrt{R^2 - x^2}\,dx$$

$$Vq2(x_o) := 2\cdot\int_{x_o}^{R} q2(x)\sqrt{R^2 - x^2}\,dx \qquad Mq2(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q2(x)\sqrt{R^2 - x^2}\,dx$$

(3) Define functions for mat shear and moment due to hydrostatic load and mat, floor, and seal plate loads

$$w_{unif} := \frac{(W_T + W_{mat} + W_{seal} + W_f)}{\pi\cdot R^2} \qquad w_{unif} = 2286\cdot psf$$

uniform load acting down on interior

$$V_{unif}(x_o) := -2\cdot\int_{x_o}^{R} w_{unif}\sqrt{R^2 - x^2}\,dx \qquad M_{unif}(x_o) := -2\cdot\int_{x_o}^{R} (x - x_o)\cdot w_{unif}\sqrt{R^2 - x^2}\,dx$$

(4) Define functions for mat shear and moment due to hydrodynamic base pressure (excluding Av effects)

Total moment due to impulsive and convective effects

$$\Delta M_{imp} := A_i\cdot W_i\cdot(X_{imf} - X_i) = 6634\cdot kip\cdot ft$$

$$\Delta M_{conv} := A_c\cdot W_c\cdot(X_{cmf} - X_c) = 492\cdot kip\cdot ft$$

The impulsive base pressure varies as

$$\frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

From Ref 5, Equation F80

Integration constant for impulsive base pressure is

$$Const_{imp} := \frac{\Delta M_{imp}}{2\int_{-R}^{R}\int_{0}^{y_o(x)} \frac{x\cdot\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}\,dy\,dx}$$

$$Const_{imp} = 525\cdot psf$$

And the pressure function can be written as

$$p_{base\_i}(x,y) := Const_{imp} \cdot \frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

The convective base pressure varies as $\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3$ From Ref 5, Equation F108

Integration constant for convective base pressure is

$$Const_{conv} := \frac{\Delta M_{conv}}{2\int_{-R}^{R}\int_{0}^{y_o(x)} x\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right] dy\, dx}$$

$$Const_{conv} = 40\cdot psf$$

And the pressure function can be written as

$$p_{base\_c}(x,y) := Const_{conv}\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right]$$

The combined base pressure associated with convective and impulsive effects is

$$p_{base}(x,y) := p_{base\_i}(x,y) + p_{base\_c}(x,y)$$

$p_{base}(R,0) = 498\cdot psf$ Maximum pressure at toe

As a check, compare maximum bottom pressure if an approximate linear distribution of base pressure is assumed by dividing the total moment by the section modulus of the foundation footprint

$$p_{toe\_linear} := 4\cdot\frac{(\Delta M_{imp} + \Delta M_{conv})}{\pi\cdot R^3} \qquad p_{toe\_linear} = 488\cdot psf \qquad \frac{p_{toe\_linear}}{p_{base}(R,0)} = 0.979 \quad \text{OK}$$

$$V_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} p_{base}(x,y)\, dy\, dx$$

$$M_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} (x - x_o)\, p_{base}(x,y)\, dy\, dx$$

<u>(5) Define functions for mat shear and moment due to Av only (up or down, not including loads at shell)</u>

$$V_{Av1}(x_o) := -2 \cdot \int_{x_o}^{R} .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx \qquad M_{Av1}(x_o) := -2 \cdot \int_{x_o}^{R} (x - x_o) \cdot .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx$$

$$V_{Av2}(x_o) := 2 \cdot \int_{x_o}^{R} .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx \qquad M_{Av2}(x_o) := 2 \cdot \int_{x_o}^{R} (x - x_o) \cdot .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx$$

(6) Define functions for mat shear and moment due to roof shell and footing dead load applied at the perimeter

$$V_{shell\_static}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg}) \cdot R\, d\varphi$$

$$M_{shell\_static}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg}) \cdot (R \cdot \cos(\varphi) - x_o) \cdot R\, d\varphi$$

(7) Define functions for mat shear and moment due to lateral seismic loads all applied at the perimeter

Write hydrodynamic force intensity at the shell as a function of φ

$$E_{shell}(\varphi) := \left(\frac{M_{shell}}{\pi \cdot R^2}\right) \cdot \cos(\varphi)$$

$$V_{E\_shell}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi) \cdot R\, d\varphi$$

$$M_{E\_shell}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi) \cdot (R \cdot \cos(\varphi) - x_o) \cdot R\, d\varphi$$

(8) Define functions for mat shear and moment due to Av loads applied at the perimeter

$$V_{shell\_Av1}(x_o) := .40 \cdot A_v \cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av1}(x_o) := .40 \cdot A_v \cdot M_{shell\_static}(x_o)$$

$$V_{shell\_Av2}(x_o) := -.40 \cdot A_v \cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av2}(x_o) := -.40 \cdot A_v \cdot M_{shell\_static}(x_o)$$

(9) Define functions for mat shear and moment due to center column force

$P_{D\_ctr} := W_{roof\_center} + W_{col\_base} + W_{col} = 8.8 \cdot kip$

$V_{ctr}(x_o) := \text{if}(x_o > 0, 0, -P_{D\_ctr})$

$M_{ctr}(x_o) := \text{if}(x_o > 0, 0, x_o \cdot P_{D\_ctr})$

(10) Define functions for total mat shear and moment due to combined loadins for the case of Av up or down

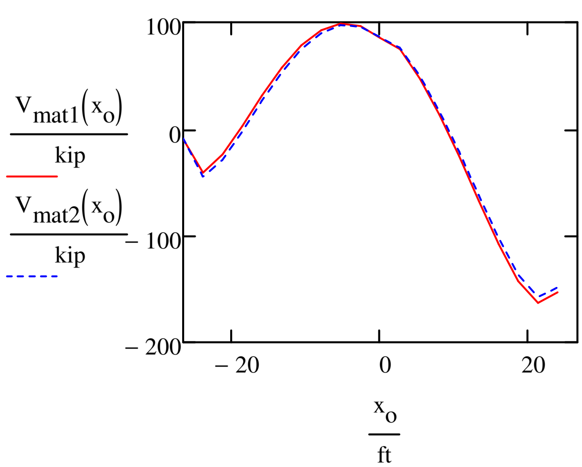
$$V_{mat1}(x_o) := Vq1(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av1}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av1}(x_o) + V_{ctr}(x_o) \cdot (1 + .40 \cdot A_v)$$

$$V_{mat2}(x_o) := Vq2(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av2}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av2}(x_o) + V_{ctr}(x_o) \cdot (1 - .40A_v)$$

$$M_{mat1}(x_o) := Mq1(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av1}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av1}(x_o) + M_{ctr}(x_o) \cdot (1 + .40 \cdot A_v)$$

$$M_{mat2}(x_o) := Mq2(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av2}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av2}(x_o) + M_{ctr}(x_o) \cdot (1 - .40 \cdot A_v)$$

$x_o := -R, -R + \frac{R}{10} .. R$ Set plot parameters



$V_{mat1}(R) = 0 \cdot kip$

$V_{mat2}(R) = 0 \cdot kip$

$V_{mat1}(-R) = -9.2 \cdot kip$

$V_{mat2}(-R) = -8.5 \cdot kip$ All values zero, check

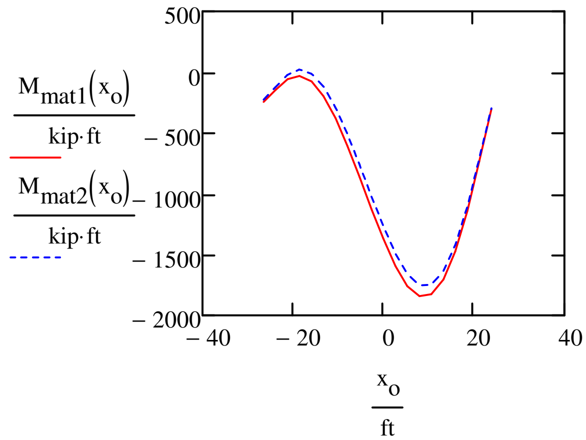
$M_{mat1}(R) = 0 \cdot kip \cdot ft$

$M_{mat2}(R) = 0 \cdot kip \cdot ft$

$M_{mat1}(-R) = -243 \cdot kip \cdot ft$

$M_{mat2}(-R) = -224.481 \cdot kip \cdot ft$

All values zero, check



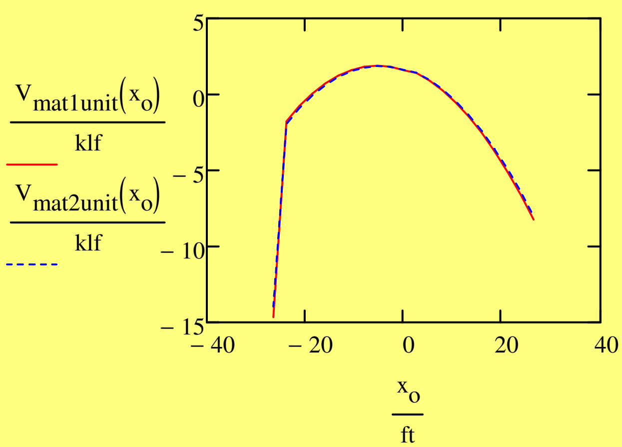
These forces are distributed over a variable mat width. Convert to average unit forces in the mat

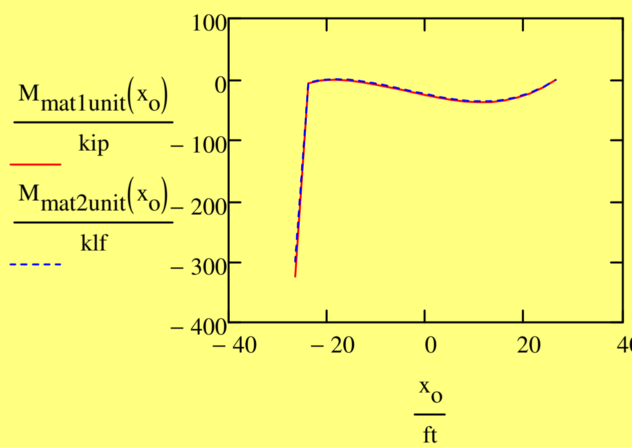
Note: These expressions cannot be evaluated at R or -R because the denominato
is zero at the limits. Evaluate at values of x close to +/- R

$$V_{mat1unit}(x_o) := \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat1unit}(x_o) := \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$$V_{mat2unit}(x_o) := \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat2unit}(x_o) := \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$x_o := -.9999R, -R + \frac{R}{10} .. .9999R$ Plot parameters





Average unit shear and moment in the mat, ASD basis

Compute maxima and minimima

$x_o := 0$

Given

$$V_{mat1unit}(x_o) = \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$V_{mat1unitmax} := V_{mat1unit}(\mathrm{Maximize}(V_{mat1unit}, x_o)) = 1.787 \cdot klf$

$V_{mat1unitmin} := \min(V_{mat1unit}(-.9999R), V_{mat1unit}(.9999R)) = -14.65 \cdot klf$

$Vu_{mat1} := 1.4 \max(|V_{mat1unitmax}|, |V_{mat1unitmin}$ $Vu_{mat1} = 20.51 \cdot klf$

Given

$$V_{mat2unit}(x_o) = \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$V_{mat2unitmax} := V_{mat2unit}(\mathrm{Maximize}(V_{mat2unit}, x_o)) = 1.787 \cdot klf$

$V_{mat2unitmin} := \min(V_{mat2unit}(-.9999R), V_{mat2unit}(.9999R)) = -13.94 \cdot klf$

$Vu_{mat2} := 1.4 \max(|V_{mat2unitmax}|, |V_{mat2unitmin}$ $Vu_{mat2} = 19.515 \cdot klf$

$Vu_{mat} := \max(Vu_{mat1}, Vu_{mat2})$ $Vu_{mat} = 20.51 \cdot klf$

$x_o := \frac{-R}{2}$

Given

$$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat1unitmax} := M_{mat1unit}(\mathrm{Maximize}(M_{mat1unit}, x_o))$ $M_{mat1unitmax} = -0.746 \cdot kip$

$x_o := \frac{R}{2}$

Given

$$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat1unitmin} := M_{mat1unit}(\mathrm{Minimize}(M_{mat1unit}, x_o))$ $M_{mat1unitmin} = -37.609 \cdot kip$

$x_o := \frac{-R}{2}$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmax} := M_{mat2unit}(\text{Maximize}(M_{mat2unit}, x_o))$ $M_{mat2unitmax} = 0.597 \cdot \text{kip}$

$x_o := \frac{R}{2}$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmin} := M_{mat2unit}(\text{Minimize}(M_{mat2unit}, x_o))$ $M_{mat2unitmin} = -36.004 \cdot \text{kip}$

$Mu_{mat\_pos} := 1.4 \max(M_{mat1unitmax}, M_{mat2unitmax}$ $Mu_{mat\_pos} = 0.836 \cdot \text{kip}$

$Mu_{mat\_neg} := 1.4 \min(M_{mat1unitmin}, M_{mat2unitmin})$ $Mu_{mat\_neg} = -52.652 \cdot \text{kip}$

Capacity Check and Preliminary Quantities

Material assumptions

$f'c := 4000 \cdot psi$ $f_y := 60 \cdot ksi$ $d := h_{mat} - 4 \cdot in$

Check shear capacity

$\varphi Vc := .75 \cdot 2 \cdot d \cdot \sqrt{\frac{f'c}{psi}} \cdot psi$ $\varphi Vc = 22.768 \cdot klf$ $\frac{Vu_{mat}}{\varphi Vc} = 0.901$ 1.0 OK

Compute approximate bottom steel requirement

$A_{s\_bot} := \frac{Mu_{mat\_pos}}{.90 \cdot .90 \cdot d \cdot f_y}$ $A_{s\_bot} = 0.01 \cdot \frac{in^2}{ft}$ Computed steel requirement

$A_{s\_bot} := \text{if}\left[\left(\frac{A_{s\_bot}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_bot}, A_{s\_bot}\right]$ $A_{s\_bot} = 0.014 \cdot \frac{in^2}{ft}$ Adjust steel requirement if computed steel ratio less than 200/fy

$A_{s\_top} := \frac{-Mu_{mat\_neg}}{.90 \cdot .90 \cdot d \cdot f_y}$ $A_{s\_top} = 0.65 \cdot \frac{in^2}{ft}$ Computed steel requirement

$A_{s\_top} := \text{if}\left[\left(\frac{A_{s\_top}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_top}, A_{s\_top}\right]$ $A_{s\_top} = 0.866 \cdot \frac{in^2}{ft}$ Adjust steel requirement if computed steel ratio less than 200/fy

Reinforcement requirement per unit area of mat

$w_{reinf} := \gamma_{steel} \cdot 2 \cdot \left(A_{s\_bot} + A_{s\_top}\right)$ $w_{reinf} = 5.99 \cdot psf$

$W_{reinf} := w_{reinf} \cdot \pi \cdot R^2$ $W_{reinf} = 13216\, lbf$

$cy := 27 \cdot ft^3$

Concrete and seal steel quantities

$V_{conc} := h_{mat} \cdot \pi \cdot R^2$ $V_{conc} = 163.421 \cdot cy$ $W_{seal} = 22521\, lbf$

Placeholder unit costs for concrete and steel $reinf\_cost := \frac{1}{lbf}$ $conc\_cost := \frac{500}{cy}$ $steel\_cost := \frac{2}{lbf}$

$Cost := W_{reinf} \cdot reinf\_cost + V_{conc} \cdot conc\_cost + W_{seal} \cdot steel\_cost$ $Cost = 139970$

APPENDIX B.2

DIVISION 22 RESERVOIR CALCULATIONS

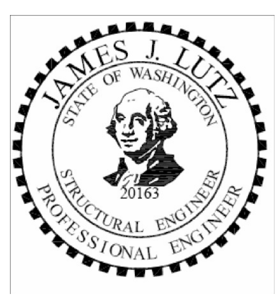
BHC CONSULTANTS

Seismic Evaluation
for
Division 22 Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Methodology Remarks

These calculations are limited to an assessment of the primary elements of the lateral force resisting system for the reservoir under seismic loading. Following is a summary of the methodology used:

1. All dimensions and weights are based on record drawings furnished by the client, supplemented by field measurements.In case of discrepancies, field measurements were used..

2. Water level assumed for seismic calculations is based on maximum current operating level provided by the District..

3. Methodology for determination of seismic loads for tanks with a free water surface is based on the 2012 International Building Code, ASCE 7-10, and AWWA Standard D100-11. These codes and standards post-date and are more stringent than codes and standards used at the time of original tank design.

4. For tanks where the free surface sloshing wave amplitude exceeds the roof elevation, the additional amplification of seismic load is based on an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave. The force is modeled by computing an increase in mass and adjusting the convective period of the water mass. The pressure distribution is assumed the same as for a tank with a free water surface.

5. For tanks where the static water surface level already contacts the roof, the free surface sloshing amplitude is based on a cylinder of the same height and radius with zero freeboard, however the actual water mass is assumed. The ratio of sloshing amplitude to roof height is computed using roof height measured from the free water surface. Adjustments in seismic load are otherwise the same as for the preceding step.

6. Ground motion spectral accelerations S_S and S_1 are those currently available from the USGS on their web site calculator for the latitude and longitude of the tank as taken from Google Earth.

7. Soil site class "D" is assumed as a default in the absence of a soils report for this reservoir..

8. Wind loads, hydrostatic loads at overflow elevation, and roof live loads were not considered in the analysis. However where calculated roof loads exceed 40 psf, a mass equal to .20 times the uniform roof snow load is added to the roof mass for seismic calculations. The gravity effects of snow load were considered whete applicable for determining loads on the shell, however no analysis of roof members was included.

Location and Site Data

Lat 48.7272, Long -122.3556
El 805
(Google Earth)

Superstructure Geometry

From record drawings

Tank diameter $D := 50 \cdot ft$

Tank radius $R := \frac{D}{2} = 25\,ft$

Shell height $H_s := 35 \cdot ft$

Floor elevation at shell
(Bottom capacity level)

$BCL := 800 \cdot ft$ (District)

Overflow height above floor

$h_{overflow} := H_s - 6 \cdot in = 34.5\,ft$ (Estimated)

Overflow elevation
(Top capacity level)

$TCL := BCL + h_{overflow}$

$H := 33.5 \cdot ft$ Maximum operating level

$NOL := BCL + H = 833.5\,ft$

$BCL + H_s = 835\,ft$

This level is below the top of the shell.

Describe the roof geometry

$roof\_slope := \frac{1.0}{12} = 0.083$ (Actual varies between .78" and .083" per 12")

The roof height is $h_r := roof\_slope \cdot R = 2.083\,ft$

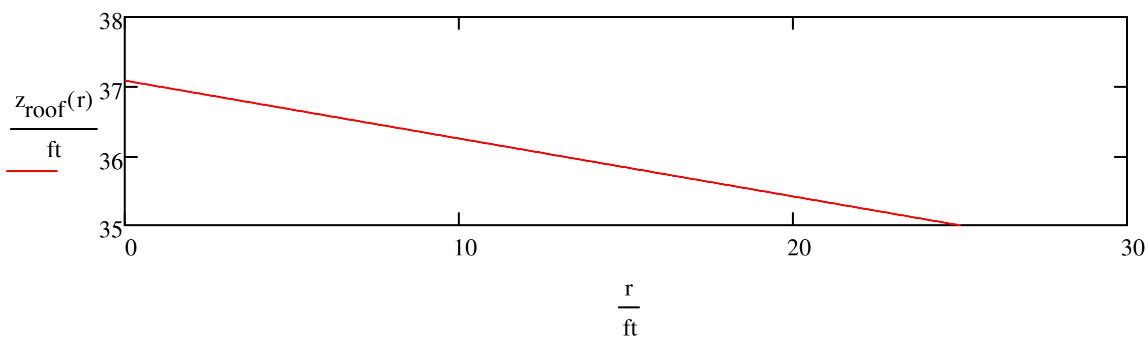
Let "z" be the distance measured vertically from the floor, and "r" the horizontal distance from the center

$z_{apex} := H_s + h_r = 37.083\,ft$

The expression for z for the roof for $0 < r < R$ is

$z_{roof}(r) := \left(\mathrm{if}\left(r > R, 0, z_{apex} - roof\_slope \cdot r\right)\right)$

Plot the roof elevation vs radius $r := 0, .1 \cdot ft .. R$



Enter shell and roof plate thickness.

Mathcad General Input - See Appendix for Mathcad nomenclature and symbols

$ORIGIN := 1$

Special unit definitions $each := 1$ $sf := ft^2$

number of shell plate courses, numbering starting with the base as course 1

$n_{course} := 5$ (the vertical leg of the top angle is included with the top shell plate course)

Calculate the elevation of the top of each shell course relative to the floor

$i := 1, 2 .. n_{course}$ i is the number of each shell course, starting from the bottom $\gamma_{steel} := 490 \cdot pcf$ unit weight of steel

zshell is the elevation of the top of each course relative to the top of the bottom plate

$$z_{shell} := \begin{pmatrix} 7.02 \\ 14.02 \\ 21.02 \\ 28.05 \\ 35 \end{pmatrix} \cdot ft \qquad t_{shell} := \begin{pmatrix} \frac{13}{32} \\ \frac{9}{32} \\ .25 \\ .25 \\ \frac{9}{32} \end{pmatrix} \cdot in \qquad w_{shell} := t_{shell} \cdot \gamma_{steel} = \begin{pmatrix} 16.589 \\ 11.484 \\ 10.208 \\ 10.208 \\ 11.484 \end{pmatrix} \cdot psf \qquad class_{shell} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Shell thickness is per field measurements, rounded to the nearest 1/32 inch. Original specifications not known. Assume minimum yield stress to qualify as AWWA D100 Class 1, Fy=27 ksi.

Class 1 material has a yield stress 27 ksi < Fy < 34 ksi. Class 2 material has a yield stress Fy > 34 ksi

Roof thickness is 3/16" per nameplate, but thickness gauge measurements were .120". Use 3/16" to be conservative for roof weight calcualtions.

$$t_{roof\_plate} := \frac{3}{16} \cdot in$$

roof plate thickness as measured, rounded to nearest 1/32 inch

Compute weight of roof and shell

Define the roof slope at any point

$$z'_{roof}(r) := \frac{d}{dr} z_{roof}(r)$$

Compute the surface area of the roof plate tributary to the perimeter and the center column. . Ignore laps

For a surface of revolution, the general equation for the surface area is

$$A := 2 \cdot \pi \cdot \int r\, ds \qquad \text{where} \qquad ds := \sqrt{1 + \left(\frac{dz}{dr}\right)^2} \cdot dr$$

$$A_{roof\_plate} := 2 \cdot \pi \cdot \left(\int_0^R r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 1970\, ft^2 \text{(roof surface area)}$$

$$W_{roof\_plate} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate} = 15.085 \cdot kip$$

$$A_{roof\_plate\_center} := 2 \cdot \pi \cdot \left(\int_0^{\frac{R}{2}} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 493\, ft^2$$

$$W_{roof\_plate\_center} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_center} = 3.771 \cdot kip$$

Portion of roof weight tributary to center column

$$A_{roof\_plate\_edge} := 2 \cdot \pi \cdot \left(\int_{\frac{R}{2}}^{R} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 1478\, ft^2$$

$$W_{roof\_plate\_edge} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_edge} = 11.314 \cdot kip$$

Portion of roof weight tributary to shell

Calculate the vertical center of gravity from the tank floor for the roof plate

$$x_{cg} := 2\cdot\pi\cdot\frac{\left(\int_0^R r^2\cdot\sqrt{1+z'_{roof}(r)^2}\,dr\right)}{A_{roof\_plate}} = 17\,ft$$

$$X_{roof\_plate} := z_{roof}\left(x_{cg}\right) = 35.694\,ft$$

Define the number of the shell course for any value of 0 < z < Hs using a series of functions

$i_{course}(z) := n_{course}$ Default value

$$i_{course}(z) := if\left(z < z_{shell_{n_{course}}}, n_{course}, i_{course}(z)\right)$$

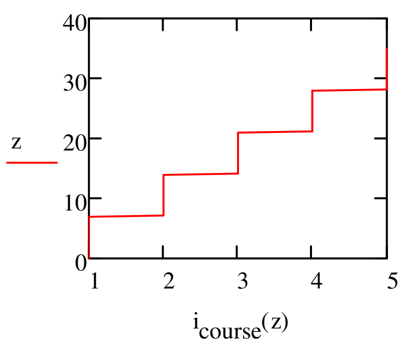
$$i_{course}(z) := if\left(z < z_{shell_4}, 4, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_3}, 3, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_2}, 2, i_{course}(z)\right)$$

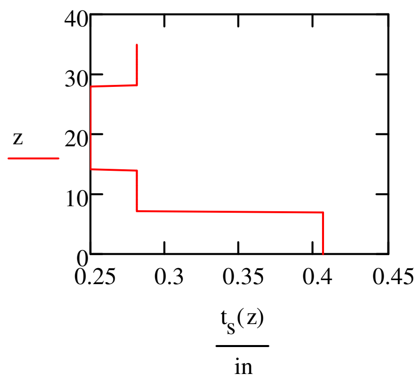
$$i_{course}(z) := if\left(z < z_{shell_1}, 1, i_{course}(z)\right)$$

$z := 0\cdot ft, 0.2\cdot ft .. H_s$ Set plotting interval for graphs

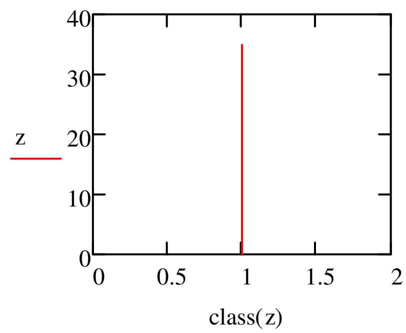


write functions that return the shell plate thickness and class as a function of height above the base

$t_s(z) := t_{shell_{i_{course}(z)}}$ $class(z) := class_{shell_{i_{course}(z)}}$



Shell thickness vs elevation



Shell class vs elevation

Floor plate thickness $t_{floor} := .25 \cdot in$

$floor\_flange := 2.0 \cdot in$ Bottom plate projection beyond shell plate $D_{floor} := D + 2 \cdot floor\_flange$

Compute floor weight

$$W_f := \gamma_{steel} \cdot t_{floor} \cdot \pi \cdot \frac{D_{floor}^{\ 2}}{4} \qquad W_f = 20.3 \cdot kip$$

Compute the weight of the shell and establish its center of gravity from the base

$$W_s := \pi \cdot D \cdot \int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z)\, dz \qquad W_s = 65.945 \cdot kip$$

$$X_s := \pi \cdot D \cdot \frac{\int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \cdot z\, dz}{W_s} \qquad X_s = 16.157\, ft$$

Compute the weight of the roof and establish its center of gravity from the base

The total roof mass is a combination of the part tributary to the center column and the part tributary to the edge. The center portion includes part of the roof, half the weight of the rafters, the column cap, and half of the column. (The other half of the column and its base plate are assigned to the floor mass). The edge portion includes part of the roof, half the weight of the rafters, clips and the flange of the top angle. The weight of top angle and clips and top angle flange are ignored.

Based on video, there are 25 rafters spanning from the shell to the center column to plate. Estimate of the web depth was 7.5 inches, which is not a standard channel size. Conservatively, use the weight for the largest 8" deep standard channel, C8X18.75. Assume column cap is .5" x 2 ft dia., center pipe column is 8" diameter, Sch 40. Ignore weight of clips, bolts, laps, and appurtenances..

$$W_{rafters} := 25 \cdot 18.75 \cdot \frac{lbf}{ft} \cdot (R - .5 \cdot ft) = 11.484 \cdot kip$$

$$W_{col\_cap} := \pi (12 \cdot in)^2 \cdot .5 \cdot in \cdot \gamma_{steel} = 0.064 \cdot kip$$

$$W_{col} := 33.6 \cdot ft \cdot 18.7 \cdot \frac{lbf}{ft} = 0.628 \cdot kip$$

$$W_{col\_base} := \gamma_{steel} \cdot \left[.5 \cdot in \cdot \pi \cdot (18 \cdot in)^2 + .375 \cdot in \cdot 2 \cdot 1 \cdot ft^2\right] = 0.175 \cdot kip$$ assumed base plate and gussets

$$W_{roof\_center} := W_{roof\_plate\_center} + \frac{W_{rafters}}{2} + W_{col\_cap} + \frac{W_{col}}{2} = 9.892 \cdot kip$$ Roof weight tributary to center column

$$W_{roof\_edge} := W_{roof\_plate\_edge} + \frac{W_{rafters}}{2} = 17.056 \cdot kip$$ Roof weight tributary to top of shell

$$\Delta W_f := W_{col\_base} + \frac{W_{col}}{2} = 0.489 \cdot kip$$ Column and base plate tributary to floor

Total roof structure mass for seismic calculation $W_r := W_{roof\_center} + W_{roof\_edge} = 26.948 \cdot kip$

Check to see if roof snow load mass must be included per ASCE 7-10

$p_g := 60 \cdot psf$ from "Snow Load Analysis for Washington", 2nd ed, SEAW

$I_s := 1.20$ Snow load importance factor for risk category IV, ASCE 7-10

$C_e := 1.2$ ASCE 7-10, Table 7-2. Exposure Factor, Terrain B, Sheltered

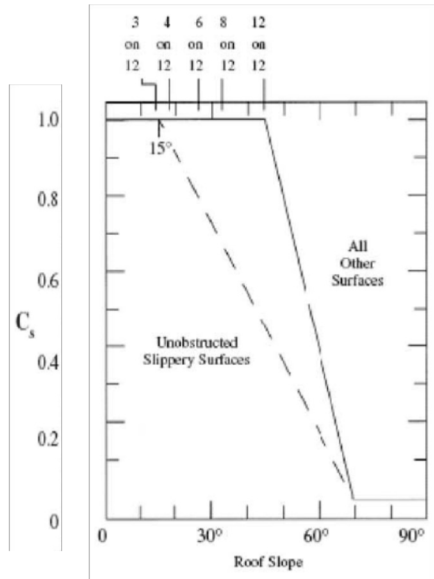
$C_t := 1.2$ ASCE 7-10, Table 7-3, Thermal Factor, Unheated

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 72.576 \cdot psf$ Flat roof snow load, ASCE 7-10 Eq 7.3-1. Since flat roof snow load exceeds 30 psf, add 20% of the design snow load to the roof mass per ASCE 7-10, section

12.7.2.

The roof slope is $\mathrm{atan}(roof\_slope) = 4.764 \cdot deg$

From ASCE 7-10 Fig 7-2c, the roof slope factor is



7-2c: Cold roofs with C = 1.2 or larger

$C_s := 1.0$

$p_s := C_s \cdot p_f = 72.576 \cdot psf$

Snow weight to include with roof weight

$w_{snow} := .20 \cdot p_s = 14.515 \cdot psf$

$W_{snow} := w_{snow} \cdot \pi \cdot R^2 = 28.501 \cdot kip$

Snow weight tributary to edge

$$W_{snow\_shell} := W_{snow} \cdot \frac{A_{roof\_plate\_edge}}{A_{roof\_plate}} = 21.375 \cdot kip$$

$$P_{snow} := \frac{W_{snow\_shell}}{\pi \cdot D} = 136.08 \cdot \frac{lbf}{ft}$$

Snow load applied at top of shell concurrent with seismic

Snow weight tributary to floor

$W_{snow\_floor} := W_{snow} - W_{snow\_shell} = 7.125 \cdot kip$

All the lateral resistance for the roof is assumed to be by the shell, except for the lower half of the column

Compute the center of gravity of the roof and column mass for seismic calculation

$$X_r := \frac{\left[W_{roof\_plate} \cdot X_{roof\_plate} \cdots + z_{apex} \cdot W_{col\_cap} + .75 \cdot z_{apex} \cdot \frac{W_{col}}{2} + W_{rafters} \cdot \left(H_s + \frac{h_r}{2} \right) \right]}{W_r} = 35.754\,ft$$

Compute the center of gravity of the roof snow load for seismic calculations

Snow density per ASCE 7-10 equation 7.7.1 is

$$\gamma_{snow} := \min\left(30 \cdot pcf , 0.13 \cdot \frac{p_g}{ft} + 30 \cdot pcf \right) = 30 \cdot pcf$$

snow depth $h_d := \frac{w_{snow}}{\gamma_{snow}} = 0.484\,ft$

$$X_{snow} := X_{roof\_plate} + \frac{h_d}{2} = 35.936\,ft$$ centroid of snow mass

Compute total water weight for seismic calculations

$$\gamma_{water} := 62.4\cdot pcf$$

$$W_T := \gamma_{water}\cdot H\cdot\pi\cdot\frac{D^2}{4} = 4104.49\cdot kip$$

Calculate the impulsive and convective water weights and vertical centroids

$$\frac{D}{H} = 1.493$$

$$W_i := W_T\cdot\frac{\tanh\left(.866\cdot\frac{D}{H}\right)}{.866\cdot\frac{D}{H}}$$ if D/H > 1.333

$$W_i := \mathrm{if}\left[\frac{D}{H} < 1.333, W_T\cdot\left(1.0 - 0.218\cdot\frac{D}{H}\right), W_i\right]$$ if D/H < 1.33

$$W_i = 2730.288\cdot kip$$ Impulsive water weight $$\frac{W_i}{W_T} = 0.665$$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated or the total moment on the foundation, shell plus floor.

$$X_i := H\cdot\mathrm{if}\left[\left(\frac{D}{H}\right) > 1.333, 0.375, 0.50 - 0.094\cdot\frac{D}{H}\right]$$ $$X_i = 12.563\,ft$$ centroid for calculation of just the shell moment

$$X_{imf} := 0.375\cdot\left[1.0 + 1.333\cdot\left(\frac{0.866\cdot\frac{D}{H}}{\tanh\left(0.866\cdot\frac{D}{H}\right)} - 1\right)\right]\cdot H$$ centroid for calculation of total bottom moment if D/H > 1.33

$$X_{imf} := \mathrm{if}\left[\frac{D}{H} < 1.333, \left(0.50 + 0.06\cdot\frac{D}{H}\right)\cdot H, X_{imf}\right]$$ centroid for calculation of total bottom moment if D/H < 1.33

$$X_{imf} = 20.991\,ft$$

Compute convective water weight and effective centroid above the base

$$W_c := W_T\cdot\left(.230\cdot\frac{D}{H}\cdot\tanh\left(3.67\cdot\frac{H}{D}\right)\right)$$ $$W_c = 1388.54\cdot kip$$ $$\frac{W_c}{W_T} = 0.338$$ Ref 4, Eq 13-26

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

$X_c = 22.023\,ft$ centroid for calculation of just the shell moment

$$X_{cmf} := H\cdot\left(1.0-\frac{\cosh\left(3.67\cdot\frac{H}{D}\right)-1.937}{3.67\cdot\frac{H}{D}\cdot\sinh\left(3.67\cdot\frac{H}{D}\right)}\right)$$

$X_{cmf} = 24.223\,ft$ centroid for calculation of total bottom moment

Seismic Design Criteria

Importance Factor: $I_E := 1.50$ Risk category IV

Ground Motion Parameters

Site Class C Site Class based on soils report for proposed adjacent reservoir

$S_S := .943$ $S_1 := .368$ Mapped earthquake short period and long period spectral accelerations. For Site Class B, 5% damping, expressed as fraction of g.

$F_a := 1.02$ $F_v := 1.43$ Site coefficients from 2012 IBC Table 1613.3.3(2). Seismic Design Category "C"

Adjusted maximum considered earthquake for site class

$S_{MS} := F_a \cdot S_S$ $S_{MS} = 0.962$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.526$

Design spectral response parameters

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$ $S_{DS} = 0.641$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$ $S_{D1} = 0.351$

Compute points on the design response spectrum

$T_0 := 0.2 \cdot sec \cdot \frac{S_{D1}}{S_{DS}}$ $T_0 = 0.109 \cdot sec$

$T_S := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot sec$ $T_S = 0.547 \cdot sec$

$T_L := 6 \cdot sec$ Mapped value, ASCE 7-10, Figure 22-12

$T_L := if(T_L > 4.sec, 4.sec, T_L) = 4 \cdot sec$ Maximum required for tank sloshing wave calculations, ASCE 7-10, Section 15.7.6.1.d

$S_{ac}(T) := if\left(T > T_L, \frac{1.5 \cdot S_{D1} \cdot T_L \cdot sec}{T^2}, min\left(\frac{1.5 \cdot S_{D1} \cdot sec}{T}, 1.5 \cdot S_{DS}\right)\right)$ Convective acceleration function

$S_{ac}(T) := \text{if}\left(S_{ac}(T) > 1.5S_{DS}, 1.5S_{DS}, S_{ac}(T)\right)$ Upper bound for S_{ac} for low values of T

$$S_{ai}(T) := \text{if}\left(T > T_L, \frac{S_{D1} \cdot T_L \cdot sec}{T^2}, \text{if}\left(T > T_S, \frac{S_{D1}}{T} \cdot sec, S_{DS}\right)\right)$$

Impulsive acceleration function

Calculate Free Surface Wave Height and Compare to Freeboard Requirements

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{D}{3.68\cdot g\cdot \tanh\left(3.68\cdot\frac{H}{D}\right)}} \qquad T_c = 4.113\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \text{if}\left(SUG = 3, \text{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\,2}}\right), \text{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\,2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.124$

$d := 0.5\cdot D\cdot A_f = 3.111\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude.

freeboard $f := H_s - H = 1.5\,ft$

$\frac{d}{f} = 2.074$ > 1.0, therefore **freeboard is insufficient** $\frac{f}{d} = 0.482$

CONSULTANTS

Compute Base Shear and Overturning Moments As If Free Surface

$S_{ai} := S_{DS}$ $R_i := 2.5$ $R_c := 1.5$ AWWA D100-11, Table 28 and section 13.2.9.2. Unnchored tank

$$A_i := \max\left(\frac{S_{ai} \cdot I_E}{1.4 \cdot R_i}, \frac{0.36 \cdot S_1 \cdot I_E}{R_i}\right) \quad A_i = 0.275$$

Impulsive design acceleration

$$A_c := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c} \quad A_c = 0.089$$

Convective design acceleration

Calculate overturning moment at the base of the shell

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_i\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2} \quad M_s = 10619 \cdot kip \cdot ft$$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_{imf}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2} \quad M_{mf} = 16856 \cdot kip \cdot ft$$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_i\right]\right]^2 + \left(A_c \cdot W_c\right)^2} \quad V_f = 799 \cdot kip$$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Adjust Effective Masses for Roof Contact

The methodology for roof contact effects is an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave.

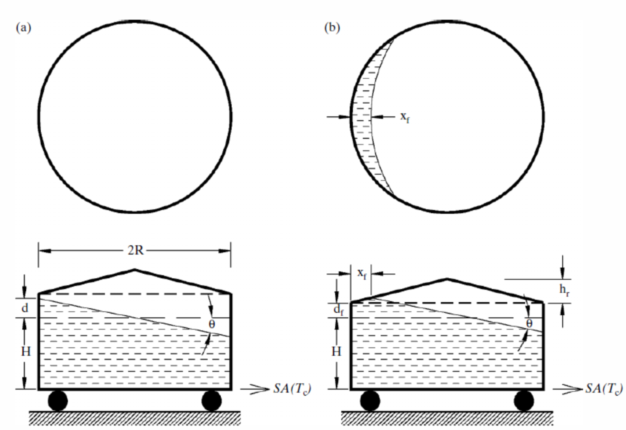
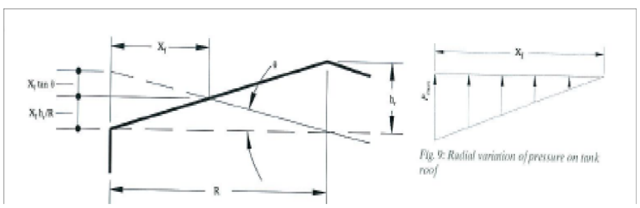


Fig. 5: Liquid-filled tank translating with an acceleration $SA(T_c)$: (a) sufficient freeboard; and (b) insufficient freeboard



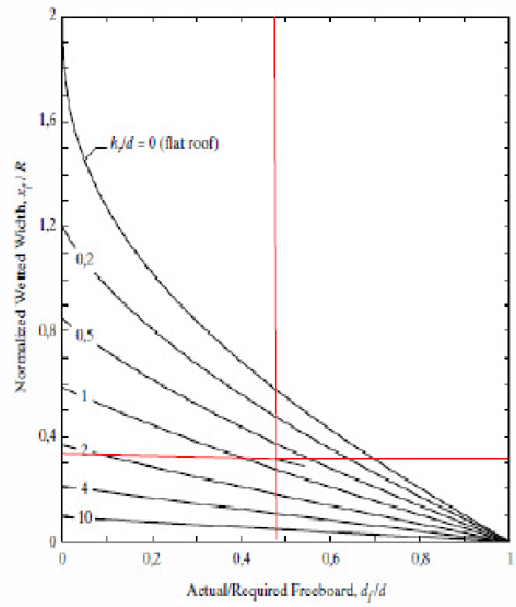


Fig. 6: Cone roof tank. Normalized wetted width of tank roof x_f/R as a function of actual/required freeboard d_f/d and normalized roof height h_r/d

Compute the angle θ

$$\theta := \operatorname{atan}\left(\frac{I_E \cdot S_{ac}(T_c) \cdot \frac{ft}{sec^2}}{g}\right) = 0.332 \cdot deg$$

Where $S_{ac}(T_c) = 0.124$ $I_E = 1.5$ $g = 32.174 \frac{ft}{s^2}$

$d_f := H_s - H = 1.5\,ft$ $d = 3.111\,ft$ $\frac{d_f}{d} = 0.482$ Compute input variables for graph above

$h_r = 2.083\,ft$ $\frac{h_r}{d} = 0.67$

From graph figure 6

$x_f := .32 \cdot R = 8\,ft$ horizontal extent of wetted dome surface from the shell $\frac{x_f}{R} = 0.32$ << 1.0 OK

$\rho := \frac{\gamma_{water}}{g} = 62.4 \cdot \frac{lbm}{ft^3}$ unit mass of water

$F_{max} := \frac{\rho}{2} \cdot g \cdot x_f^{\,2} \cdot \frac{(d + h_r)}{R}$ $\qquad F_{max} = 415 \cdot \frac{lbf}{ft}$

Maximum uplift on shell due to hydrodynamic pressure caused by sloshing. Impact effects are considered minor and ignored

adjust mass for recalculation of seismic demand

$$\bar{m}_i = \begin{cases} m_i + m_c \cdot \left(1 - \frac{d_f + h_r / 3}{d}\right) & \text{for } d_f + h_r / 3 < d \\ m_i & \text{for } d_f + h_r / 3 \geq d \end{cases}$$

$W_i = 2730 \cdot kip$

$W_T = 4104 \cdot kip$

$$\frac{\left(d_f + \frac{h_r}{3}\right)}{d} = 0.705 \qquad W_{bar\_i} := W_i + W_c \cdot \left(1 - \frac{d_f + \frac{h_r}{3}}{d}\right) = 3139.5 \cdot kip$$

$$W_{bar\_i} := \text{if}\left[\frac{\left(d_f + \frac{h_r}{3}\right)}{d} < 1, W_{bar\_i}, W_i\right] = 3139 \cdot kip$$

$$\bar{m}_c = m_l - \bar{m}_i$$

$W_c = 1388.5 \cdot kip$ $\qquad W_{bar\_c} := W_T - W_{bar\_i} = 965 \cdot kip$

$\frac{W_{bar\_i}}{W_i} = 1.15$ $\qquad \frac{W_{bar\_c}}{W_c} = 0.695$

Factors by which mass must be multiplied due to the slosh contact with the roof

Recaclulate convective period using adjusted mass. Maintain asssumption of T = 0 for impulsive mass

$$\bar{T}_i = T_i \cdot \sqrt{\frac{\bar{m}_i}{m_i}}$$

$$\bar{T}_c = T_c \cdot \sqrt{\frac{\bar{m}_c}{m_c}}$$

$T_c = 4.113\,s$ original convective period

$T_{c\_bar} := T_c \cdot \sqrt{\frac{W_{bar\_c}}{W_c}} = 3.429\,s$ modified convective period

$S_{ac}(T_c) = 0.124$ $\qquad A_c = 0.089$ original convective seismic factor

$S_{ac}(T_{c\_bar}) = 0.153$ $\qquad A_{c\_bar} := A_c \cdot \frac{S_{ac}(T_{c\_bar})}{S_{ac}(T_c)} = 0.110$ revised convective seismic factor

Recompute base shear and overturning moment

Change formula weights to adjusted values

$M_s = 10619 \cdot kip \cdot ft$ original overturning moment

$$M_{s\_rev} := \sqrt{\left[A_i \cdot \left[W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + \left(W_{bar\_i}\right) \cdot X_i\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_c\right)^2}$$

$M_{s\_rev} = 11908 \cdot kip \cdot ft$ revised moment $\dfrac{M_{s\_rev}}{M_s} = 1.121$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$M_{mf} = 16856 \cdot kip \cdot ft$ original overturning moment

$$M_{mf\_rev} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{bar\_i} \cdot X_{imf}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_{cmf}\right)^2}$$

$M_{mf\_rev} = 19122 \cdot kip \cdot ft$ revised moment

Calculate base shear at top of foundation

$V_f = 799 \cdot kip$ original base shear

$$V_{f\_rev} := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_{bar\_i}\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c}\right)^2}$$

$V_{f\_rev} = 908.04 \cdot kip$ revised base shear $\dfrac{V_{f\_rev}}{V_f} = 1.136$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Compute Shell Hoop Forces and Stresses

Impulsive and convective forces are distributed using Housner's distribution formulas

Define the following variables:

z	Height of a point above the tank floor
Y	Depth of a point below the water surface
n_I	Distributed hoop force, klf, due to impulsive load N_I
n_C	Distributed hoop force, klf, due to convective load N_C
n_V	Distributed hoop force, klf, due to vertical seismic force N_V
n_F	Distributed hoop force, klf, due to hydrostatic force at maximum normal operating level
n_{Fol}	Distributed hoop force, klf, due to hydrostatic force at overflow operating level

Define elevation, distribution, and force component functions

$Y(z) := H - z$ distance from MOL to z

Housner's distribution of impulsive load as a function of elevation above the base and, in the case of impulsive loads, depends on the ratio of D/H

For the case of D/H < 1.33 and Y(z) < 0.75 D (z > .75D, upper section)

$$Dist_{ia}(z) := \frac{\left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2}{\left[\int_{.75\cdot D}^{H} \left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz + \int_{0}^{.75\cdot D} 0.5\,dz\right]}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H < 1.33 at lower elevations, the factor is a constant equal to

$$Dist_{ib}(z) := \frac{0.5}{\left[\int_{.75\cdot D}^{H} \left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz\right] + \int_{0}^{.75\cdot D} 0.5\,dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H > 1.33

$$Dist_{ic}(z) := \frac{\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot \tanh\left(0.866\cdot\frac{D}{H}\right)}{\int_{0\cdot ft}^{H}\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot \tanh\left(0.866\cdot\frac{D}{H}\right) dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

$$Dist_i(z) := \text{if}\left[\left(\frac{D}{H}\right) \geq 1.333, Dist_{ic}(z), \text{if}\left(Y(z) < 0.75\cdot D, Dist_{ia}(z), Dist_{ib}(z)\right)\right]$$

select appropriate formula based on depth and diameter ratio

Housner's distribution of convective load as a function of elevation above the base

$$Dist_c(z) := \frac{\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}}{\int_{0\cdot ft}^{H}\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)} dz}$$

The above formula is the convective force per unit depth at elevation "z" expressed as a fraction of the total convective force.

$V_i := A_i\cdot W_{bar\_i}$ $\quad V_i = 862.778\cdot kip$ — Total base shear component due to impulsive fluid load

$N_i(z) := \left(\frac{V_i}{2}\right)\cdot Dist_i(z)$ — Shell hoop force due to impulsive fluid load

$V_c := A_{c\_bar}\cdot W_{bar\_c}$ $\quad V_c = 105.799\cdot kip$ — Total base shear component due to convective fluid load

$N_c(z) := \frac{V_c}{2}\cdot Dist_c(z)$ — Shell hoop force due to convective fluid load

$N_h(z) := \gamma_{water}\cdot\left(\frac{D}{2}\right)\cdot Y(z)$ — Shell hoop force due to hydrostatic load with water at MOL

$A_v := 0.14\cdot S_{DS}$ $\quad A_v = 0.09$ — Vertical seismic factor

$$\sigma_{static}(z) := \frac{N_h(z)}{t_s(z)}$$

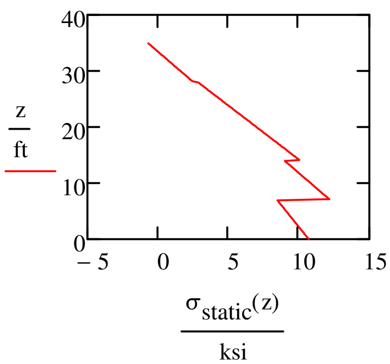
Hoop stress due to static fluid pressure at MOL

$$\sigma_s(z) := \frac{\sqrt{N_i(z)^2 + N_c(z)^2 + \left(N_h(z)\cdot A_v\right)^2}}{t_s(z)}$$

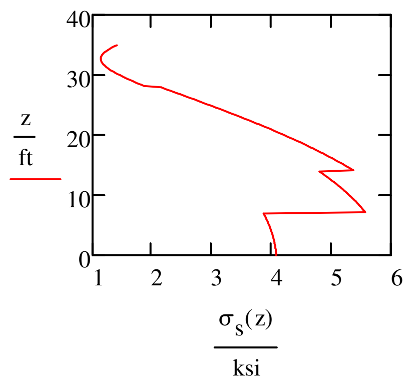
Hoop stress due to hydrodynamic pressure, Ref 4 Eq 13-42

$$\sigma_{total}(z) := \sigma_{static}(z) + \sigma_s(z)$$

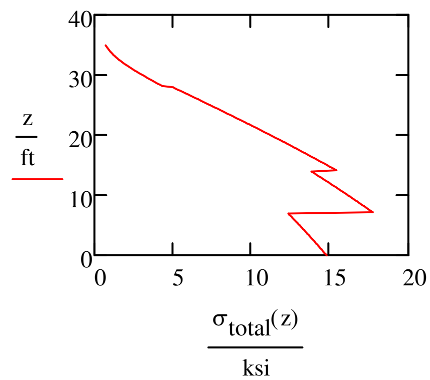
Combined static and seismic hoop stress at MOL



Hydrostatic Stress



Seismic Stress



Static + Seismic Stress

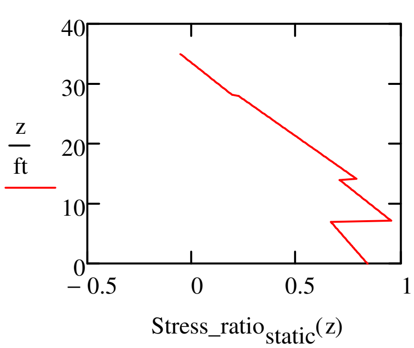
Note: the above plots are nominal based on treating each hoop course as acting independently. Actual stresses each side of girth joints are the same since strains are identical if the courses are attached, so the real stress near transition zones falls somewhere between the apparent discontinuous stress levels shown on the graphs. The actual maximum stress levels tend to occur about a foot above the joint and are not as high as predicted by the more simplified model. The simplified model is conservative and is the method reflected in the AWWA D-100 standard.

Check actual versus allowable stress based on the class of steel used.

Assumed joint efficiency and allowable stress $E_{joint} := 85\%$ $F_t(z) := E_{joint}\cdot 15\cdot ksi$

Chapter 14 of AWWA D100-11 does not apply

$$Stress\_ratio_{static}(z) := \left(\frac{\sigma_{static}(z)}{F_t(z)}\right)$$

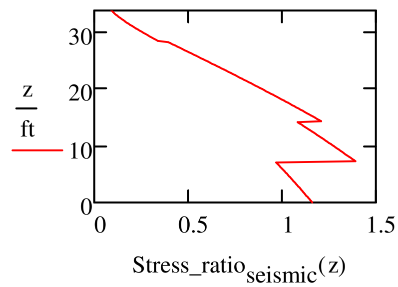


Maximum static stress ratio is $Stress\_ratio_{static}\left(z_{shell_1}\right) = 0.96$

< 1,0 OK

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{total}(z)}{F_t(z)}$$

The worst case stress ratio is at the top of the first shell course



$$Stress\_ratio\_max_{seismic} := Stress\_ratio_{seismic}\left(z_{shell_1}\right) = 1.398$$

> 1.33 NG

$$\frac{1.398}{1.33} = 1.051$$

Compute Shell Longitudinal Forces and Stresses

Define axial compressive force in the shell due to dead load for $0 < z < H_s$, in klf.

$$P_D(z) := \frac{W_r}{\pi \cdot D} + \int_z^{H_s} \gamma_{steel} \cdot t_s(z)\, dz$$

Define overturning moment functions at elevation z, in kip-ft

$$M_{rs}(z) := A_i \cdot \left[W_r \cdot \left(X_r - z\right) + W_{snow} \cdot X_{snow} + \pi \cdot \gamma_{steel} \cdot D \cdot \int_z^{H} y \cdot t_s(y)\, dy \right]$$

Moment associated with roof, snow and shell mass

$$M_i(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_i(y)\, dy$$

Moment associated with impulsive fluid mass, z < H

$$M_c(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_c(y)\, dy$$

Moment associated with convective fluid mass, z < H

$$M_s(z) := M_{rs}(z) + M_i(z) + M_c(z)$$

Total moment at elevation z on the shell for z < H

Define functions for compressive stress under static or seismic load conditions

$$\sigma_{static}(z) := \frac{P_D(z) + P_{snow}}{t_s(z)}$$

$$\sigma_{comp}(z) := \frac{\left(1 + 0.4 \cdot A_v\right)\left(P_D(z) + P_{snow}\right) - F_{max} + \dfrac{4\, M_s(z)}{\pi \cdot D^2}}{t_s(z)}$$

Includes deduction for roof uplift, $F_{max.}$

Check allowable stress for compression with local buckling and slenderness considered

Use Method 1. Yield stress of shell plate does not permit use of Method 2.

Local buckling stress formulas for Class 1 Materials

$$F_{L1a}(z) := \left[17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R} \right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R} \right)^2 \right] \cdot psi \right]$$

For Class 1 materials with 0 < t/R < t/Rc = .0031088, elastic buckling

$$F_{L1b}(z) := 5775 \cdot psi + 738 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$$

For Class 1 materials with t/Rc = .0031088 < t/R < 0.0125, inelastic buckling

$$F_{L1c}(z) := 15 \cdot ksi$$

For Class 1 materials with t/R > 0.0125, plastic buckling

Local buckling stress formulas for Class 2 Materials

$$F_{L2a}(z) := \min\left[15 \cdot ksi, 17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R}\right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R}\right)^2\right] \cdot psi\right]$$

For Class 2 materials with 0 < t/R < t/Rc = .0035372, elastic buckling

$$F_{L2b}(z) := 6925 \cdot psi + 886 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$$

For Class 2 materials with t/Rc = .0035372 < t/R < 0.0125, inelastic buckling

$$F_{L2c}(z) := 18 \cdot ksi$$

For Class 1 materials with t/R > 0.0125, plastic buckling

Write equation selection functions for F_L depending on t/R ratio and class

$ratio1 := .0031088 \quad ratio2 := .0035372$

$$F_{L1}(z) := \min\left(\text{if}\left(\frac{t_s(z)}{R} < ratio1, F_{L1a}(z), \text{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L1b}(z), F_{L1c}(z)\right)\right), 15 \cdot ksi\right)$$

$$F_{L2}(z) := \min\left(\text{if}\left(\frac{t_s(z)}{R} < ratio2, F_{L2a}(z), \text{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L2b}(z), F_{L2c}(z)\right)\right), 18 \cdot ksi\right)$$

$$F_L(z) := \text{if}\left(class(z) = 1, F_{L1}(z), F_{L2}(z)\right)$$

Slenderness reduction factor equations

$r := \frac{D \cdot \sqrt{2}}{4}$ radius of gyration of tank shell

$K := 1.0$ effective column length factor, pinned ends assumed

$E := 29 \cdot 10^6 \cdot psi$ modulus of elasticity for steel

Slenderness ratio at which overall elastic column buckling can occur (not local buckling)

$$C'_c(z) := \sqrt{\pi^2 \cdot \frac{E}{F_L(z)}} \qquad L := H_s$$

$$K_{\varphi 1}(z) := 1 - \frac{1}{2}\cdot\left(\frac{\frac{K\cdot L}{r}}{C'_c(z)}\right)^2$$ For 25 < KL/r < C'c

$$K_{\varphi 2}(z) := \frac{1}{2}\cdot\left(\frac{C'_c(z)}{\frac{K\cdot L}{r}}\right)^2$$ For KL/r > C'c

$$K_{\varphi 3}(z) := 1.0$$ For KL/r < 25

$$ratio := K\cdot\frac{L}{r} \qquad ratio = 1.98$$

$$K_{\varphi}(z) := \mathrm{if}\left(ratio < 25, K_{\varphi 3}(z), \mathrm{if}\left(ratio > C'_c(z), K_{\varphi 2}(z), K_{\varphi 1}(z)\right)\right)$$

$$F_a(z) := F_L(z)\cdot K_{\varphi}(z)$$ allowable compressive stress due to axial load

However, for unanchored tanks the allowable stress is permitted to be increased by accounting for the stability provided by hydrostatic pressure

Write a function for hydrostatic pressure for 0 < z < H $P(z) := \gamma_{water}\cdot Y(z)$ $E = 2.9\times 10^4\cdot ksi$

$$\Delta C_c(z) := \mathrm{if}\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2 \le .064, .072\cdot\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2\right]^{0.84}, .045\cdot\ln\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2 + .0018\right] + .194\right]$$

$$\Delta C_c(z) := \min\left(\Delta C_c(z), 0.22\right)$$ See AWWA D100 Eq 13-50 and 13-51

$$\Delta\sigma_{cr}(z) := \frac{\left(\Delta C_c(z)\cdot E\cdot t_s(z)\right)}{R} \qquad \Delta\sigma_{cr}(0) = 5.336\cdot ksi$$ Eq 13-49

$$\sigma_a(z) := F_a(z)$$

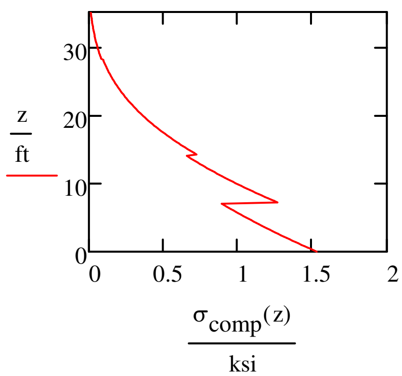
$$\sigma_e(z) := 1.33\cdot\left(\sigma_a(z) + \frac{\Delta\sigma_{cr}(z)}{2}\right)$$ Eq 13-47

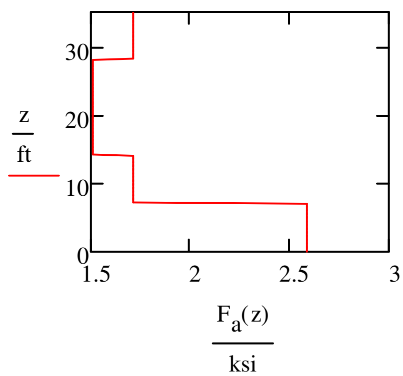
$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{comp}(z)}{\sigma_e(z)}$$

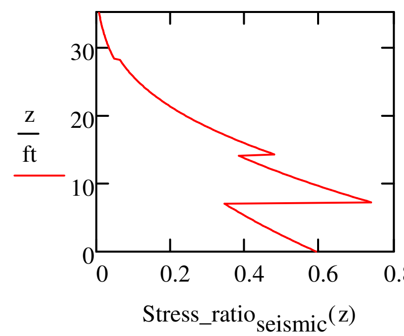
Plot static plus seismic compressive stress and compare to allowables

$$\text{Stress\_ratio}_{\text{seismic}}(z) := \frac{\sigma_{\text{comp}}(z)}{\sigma_a(z)}$$

Plot static plus seismic compressive stress and compare to allowables



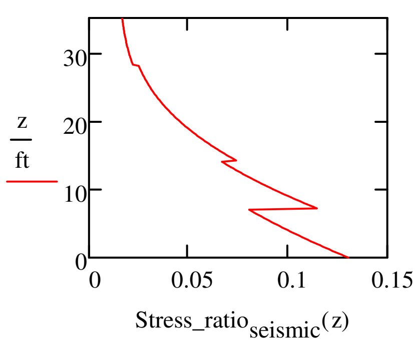




$\text{Stress\_ratio}_{\text{seismic}}\left(z_{\text{shell}_1}\right) = 0.751$ << 1.00, **OK for static plus seismic longitudinal compression**

Check seismic longitudinal tensile stress

$$\sigma_{\text{tens}}(z) := \frac{(1 - .40 \cdot A_v)P_D(z) + F_{max} + \dfrac{4\,M_s(z)}{\pi \cdot D^2}}{t_s(z)} \qquad \text{Stress\_ratio}_{\text{seismic}}(z) := \frac{\sigma_{\text{tens}}(z)}{F_t(z)}$$



All stress ratios << 1.333 are **OK for static plus seismic stress in longitudinal tension**

$\text{Stress\_ratio}_{\text{seismic}}(0) = 0.13$

Horizontal Shear Transfer Capacity

The previously calculated base shear is $V_f = 799 \cdot kip$

From AWWA D100-11 Eq 13-57, the allowable resistance attributable to friction is (for the full tank, seismic condition)

$$V_{ALLOW} := \tan(30 \cdot deg) \cdot (W_s + W_r + W_T + W_f) \cdot (1 - A_v) = 2216 \cdot kip$$

>> V_f OK. No shear connection between the superstructure and base is required for shear. Shear resistance is provided by the bottom plate acting as a diaphragm kept in place by bottom friction. Check shell to bottom transfer capacity

The maximum shell to bottom plate shear load is $v := 2 \cdot \frac{V_f}{\pi \cdot D} = 10.173 \cdot klf$ $\qquad \frac{V_f}{V_{ALLOW}} = 0.36$

There is no annular plate, just the .25" floor plate

$t_f := .25 \cdot in$

And the maximum shear stress on the plate is $\tau := \frac{v}{t_f} = 3 \cdot ksi$ $\qquad \frac{\tau}{15 \cdot ksi} = 0.226$

AWWA D100 permits 12 ksi in shear, and this can be increased by 1.33 for seismic, so **floor plate should not tear in shear parallel to the floor plate**

Check Foundation

No record drawings exist giving the dimensions of the foundation. The foundation provides no resistance to uplift since it is unanchored.

Calculate Foundation Dead Weight

$\gamma_{conc} := 150 \cdot pcf$

$h_{ftg} := 40 \cdot in$ average ringwall height interpreted from three depth measurements

$b_{ftg} := 28 \cdot in$ ringwall width based on average NDT measurements

$R_{ftg} := R + 4.5 \cdot in = 25.375\,ft$ $R_{in} := R_{ftg} - b_{ftg}$ footing outside and inside radii

$$A_{ftg} := \pi \cdot \left(R_{ftg}^{\ 2} - R_{in}^{\ 2}\right) = 354.913\,ft^2$$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 177.5 \cdot kip$ $w_{ftg} := \frac{W_{ftg}}{\pi \cdot D} = 1.13 \cdot klf$ total and unit footing weight

$W_{water} := H \cdot \gamma_{water} \cdot \pi \cdot \left(R^2 - R_{in}^{\ 2}\right) = 617.9 \cdot kip$ $w_{water} := \frac{W_{water}}{\pi \cdot D} = 3.933 \cdot klf$ total and unit weight of water over footing

$\gamma_{soil} := 125 \cdot pcf$ typical weight of compacted soil

$A_{soil} := 0$ area of soil over footing

$A_{wedge} := \frac{\left(h_{ftg} - 6 \cdot in\right)^2}{2 \cdot 2} = 2.007\,ft^2$ area of soil resisting uplift in friction at 1H:2V, backfill to within 6" of top of footing. Skin friction assumed 0.4 between footing and soil

$w_{soil} := \gamma_{soil} \cdot \left(A_{soil} + 0.4 A_{wedge}\right)$ $w_{soil} = 0.1 \cdot klf$ unit soil resistance

$W_s = 65.945 \cdot kip$ $w_{shell} := \frac{W_s}{\pi \cdot D} = 0.42 \cdot klf$ shell weight

$W_{roof\_edge} = 17.056 \cdot kip$ $w_{roof\_edge} := \frac{W_{roof\_edge}}{\pi \cdot D} = 0.109 \cdot klf$ roof edge weight

Compute overturning safety factor for pivoting about the toe of the shell

$M_{s\_rev} = 11908 \cdot kip \cdot ft$

$$SF_{overturning} := (1 - A_v) \cdot (W_{roof\_edge} + W_s + W_{ftg} + W_{water}) \cdot \frac{R}{M_{s\_rev}} = 1.678$$ OK

Required safety factor based on ASCE 7 load combos is .7E/.6D where .7E is the earthquake load in allowable stress terms, an effective ratio of 1.67

Check ratio of resistance to uplift at the foundation

$$SF_{uplift} := \frac{\left[(1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water}) + w_{soil} - F_{max}\right]}{4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2}} = 0.787$$

< 1.0 so there will be some foundation uplift

Check bearing pressure

$$\sigma_{comp}(0) = 1.53 \times 10^3 \, psi$$

$$w_{static} := w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water} = 5.591 \cdot klf$$

$$q_{bearing\_static} := \frac{w_{static}}{b_{ftg}} = 2.396 \cdot ksf$$

$$w_{seismic} := (1 + A_v) \cdot (w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water}) + F_{max} = 6.508 \cdot klf$$

$$q_{bearing\_seismic} := \frac{w_{seismic}}{b_{ftg}} = 2.789 \cdot ksf$$

$q_{allow} := 2.5 \cdot ksf$ Static allowable bearing pressure

$$\frac{q_{bearing\_static}}{q_{allow}} = 0.959$$ OK

$$\frac{q_{bearing\_seismic}}{q} = 1.116$$ < 1.33 OK

q_{allow}

Check As Self-Anchored Tank

Per AWWA D100 section 13.5.4.1

$w_t := w_{shell} + w_{roof\_edge} = 528 \cdot \frac{lbf}{ft}$ Weight of shell and roof supported by shell

$t_b := t_{floor} = 0.25 \cdot in$ $F_y := 27 \cdot ksi$ A283 Grade B steel assumed $G := 1.0$ specific gravity

$$w_L := \min\left(1.28 \cdot \frac{H}{ft} \cdot \frac{D}{ft} \cdot G, 7.29 \cdot \frac{t_b}{in} \sqrt{\frac{F_y}{ksi} \cdot \frac{H}{ft} \cdot G}\right) \cdot plf = 55 \cdot \frac{lbf}{ft}$$

Eq 13-37, normalized for units

$$J := \frac{M_s(0)}{D^2 \cdot \left[w_t \cdot \left(1 - 0.4 \cdot A_v\right) + w_L\right]} = 9.914$$

>> 1.54 therefore the tank is not stable without anchorage

References

1. 2012 *International Building Code*
2. Washington State Adoption of and Amendments to 2012 International Building Code (State Building Code)
3. ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
4. AWWA Standard D100-11 *Welded Carbon Steel Tanks for Water Storage*
5. Nuclear Reactors and Earthquakes, Chap. 6 and Appendix F. U.S. Nuclear Regulatory Commission publication, Division of Technical Information, TID-7024, National Technical Information Service (1963).
6. Not used
7. Not used
8. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" Praveen K. Malhotra, Structural Engineering International, March 2006
9. Not used
10. "Dynamic Pressures on Accelerated Fluid Containers," G.W. Housner, 1955, Bulletin of the Seismological Society of America.
11. "Snow Load Analysis for Washington, 2nd Ed." Structural Engineers Association of Washington,1995
12. Not used
13. Not used
14. ACI 318-11 Building Code Requirements for Structural Concrete
15. ANSI/AISC 360-10 Specification for Structural Steel Buildings
16. AWS D1.1 Structural Welding Code - Steel

Units and Mathcad Notation

All calculations are shown in U.S. customary units. Calculations have been performed using MathSoft's Mathcad Version 14.0 software, which automatically checks for unit consistency and applies any necessary unit conversion factors internally to the program. Where computations are imported from Excel, SAP2000, or other software, the source is identified. Input values are shaded. Others are computed.

Where equations are shown with a ":=" sign, the left hand side of the equation is being defined by the right hand side. Where equations are shown with a "=" sign, the current value of the expression on the left hand side is being displayed.

=	An ordinary "equals" sign indicates the value being shown is for the most current evaluation of the variable on the left hand side of the equation
:=	An "equals" sign with a colon indicates the value on the left hand side is being defined by the expression on the right. Variables may be redefined, the last definition taking precedence
=	A bold "equals" sign indicates the symbol is being used in a logical expression
if(a,b,c)	An "if" statement is evaluated as "b" if "a" is true, and as "c' is "a' is false. These expressions may be nested
(matrix$_{i,j}$)	In matrix expressions, the first subscript is the row, and the second is the column. Numbering starts with the value indicated as "ORIGIN" for the first row and column unless otherwise noted
submatrix (A,i1,i2,j1,j2)	Defines a vector or submatrix of matrix "A" from row i1 thru i2, and column j1 thru j2
-----------> ( )	An expression with a vector arrow over it indicates that the expression involves subscripted variables, and that the expression is being evaluated for each subscript in the range
| | | |	A bold vertical line to the left of a series of expressions indicates that they are acting as a programming loop in the calculations
ORIGIN := 1	Sets initial subscript value for subscripted variables
M<j>	The vector in column "j" of matrix "M"
$sf := ft^2$	
$\Phi(x)$	Step function. Returns -1 for x < 0, +1 for x > 0 and .5 if x = 0

Seismic Retrofit
for
Division 22-Ringwall Option A

for

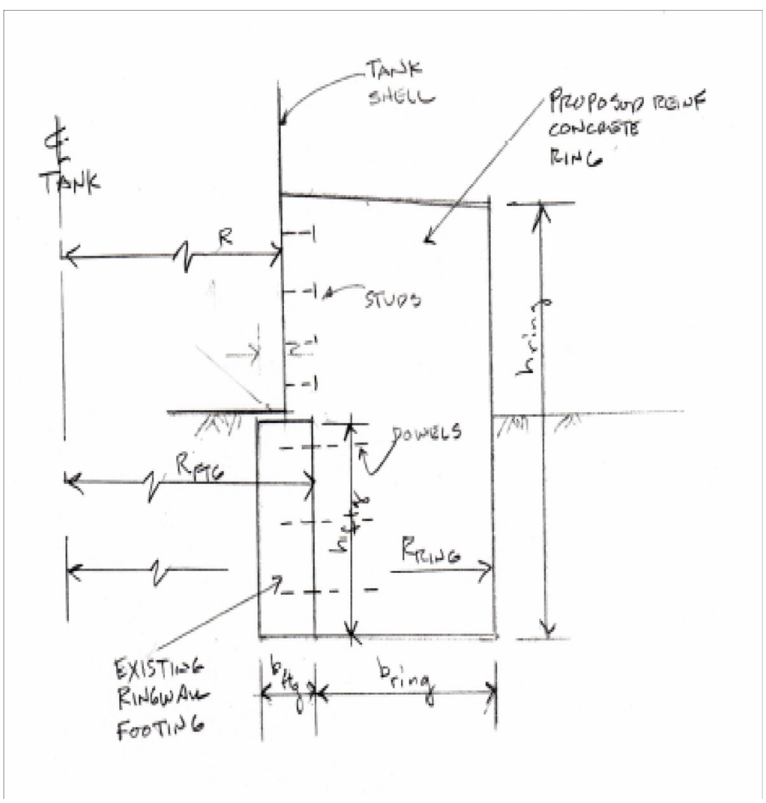
Lake Whatcom Water & Sewer District
Bellingham, Washington

These calculations are preliminary in nature for design approach analysis and are not to be used for construction

Incorporate calculations from existing tank analysis by reference.

Reference:S:\Projects\Lake Whatcom W&S District\Reservoir Seismic VA 2015\Structural Calculations\Division 22\Division 22 Rese

$cy := yd^3$



Existing ringwall and tank dimensions

Existing footing

$R_{ftg} = 25.375\,ft$ outside radius, ex. ftg.

$b_{ftg} = 2.333\,ft$

$h_{ftg} = 3.333\,ft$

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi \cdot \left(R_{ftg}^{\,2} - R_{in}^{\,2}\right)$ footprint

Additional exterior ring

$h_{ring} := 10 \cdot ft$ Ring depth

$b_{ring} := 2 \cdot ft$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 27.375\,ft$

$$A_{gross} := \pi \cdot R_{ring}^{\ 2} = 2354\, ft^2$$

$$A_{ring} := A_{gross} - \pi \cdot R_{ftg}^{\ 2}$$

Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\, dz\, dr\, d\phi\right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring}-h_{ftg}} r\, dz\, dr\, d\phi\right) = 55.572 \cdot cy$$ Ring volume

$$W_{ring} := V_{ring} \cdot \gamma_{conc} \quad W_{ring} = 225 \cdot kip$$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 1433 \cdot plf$$ Anchor ring weight per ft of shell

Check overturning stability safety factor

$$SF_{overturning} := (1 - A_v) \cdot (W_{roof\_edge} + W_s + W_{ftg} + W_{water} + W_{ring}) \cdot \frac{R}{M_{s\_rev}} = 2.109$$ > 1.67 OK

Calculate the required shear transfer capacity between footing and new anchor ring per foot of shell

$$Uplift := 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 6.062 \cdot klf$$ Transfer force at face of shell

The resistance available along the perimeter is

$$Resistance := (1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water} + w_{ring}) + w_{soil} - F_{max} = 6.079 \cdot klf$$

Check resistance/uplift safety factor with added block

$$Resistance\_ratio := \frac{Resistance}{Uplift} = 1.003$$ > 1.0 OK

The load to be transfered by the shell to the new ringwall is $Stud\_load := Uplift = 6.062 \cdot klf$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Dowel\_load := (w_{water} + w_{ftg} + F_{max}) = 5.478 \cdot klf$$

$\Omega_o := 2.0$

From Ref 3, Table 15.4-2, for tanks the overstrength factor

Stud design

$s_{studs} := 35 \cdot in$ horizontal stud spacing

$s_{studs\_vert} := 20 \cdot in$

$$n_{studs\_per\_row} := \frac{(h_{ring} - h_{ftg})}{s_{studs\_vert}} = 4$$

$$Load\_per\_stud := s_{studs} \cdot \frac{Stud\_load}{n_{studs\_per\_row}} = 4420 \cdot lbf$$

$V_u := \Omega_o \cdot 1.4 \cdot Load\_per\_stud = 12377\, lbf$

Shear strength for a 5/8" Nelson stud is $Q_N := 15113 \cdot lbf$ per AISC for f'c=4.5 ksi, Fu=65 ksi

$\phi_{shear} := .90$ $\quad \dfrac{V_u}{\phi_{shear} \cdot Q_N} = 0.91$ < 1.0 OK

$l_{stud} := 8 \cdot in$ $\quad d_{stud} := .625 \cdot in$

$f'_c := 4.5 \cdot ksi$ $\quad \dfrac{V_u}{l_{stud} \cdot d_{stud}} = 2.475 \cdot ksi$

$$DCR := \frac{V_u}{.85 \cdot f'_c\, l_{stud} \cdot d_{stud}} = 0.647$$ OK for crushing

Dowel Design

$s_{dowels} := 22 \cdot in$ horizontal stud spacing

$n_{dowels\_per\_row} := 3$

$Dowel\_load = 5.478 \cdot klf$

$$s_{dowels\_vert} := \frac{h_{ftg}}{n_{dowels\_per\_row} + 1} = 0.833\, ft$$

$$\text{Load\_per\_dowel} := s_{dowels} \cdot \frac{\text{Dowel\_load}}{n_{dowels\_per\_row}} = 3348 \cdot \text{lbf}$$

$$V_u := \Omega_o \cdot 1.4 \cdot \text{Load\_per\_dowel} = 9373\,\text{lbf}$$

for a #6 Grade 60 dowel, Hilti HIT-RE 500 adhesive in shear $V_{sa} := 15840 \cdot \text{lbf}$

$$DCR := \frac{V_u}{.60 \cdot V_{sa}} = 0.986$$ < 1 OK

Quantities

$$N_{studs} := n_{studs\_per\_row} \cdot \pi \cdot \frac{D}{s_{studs}} = 215$$

$$N_{dowels} := n_{dowels\_per\_row} \cdot \pi \cdot \frac{D}{s_{dowels}} = 257$$

$$V_{conc} := \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}^{\ 2} - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 56 \cdot cy$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 688\,ft^2$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1338\,ft^2$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1004\,ft^2 \qquad R_{exc} := R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 7.333\,ft$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 249 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} = 208 \cdot cy$$

Seismic Evaluation

for

Division 22 Reservoir Option C

for

Lake Whatcom Water & Sewer District
Bellingham, WA

Preliminary Design of Anchored Tank

General layout similar to Sumner Springs Reservoir shown below

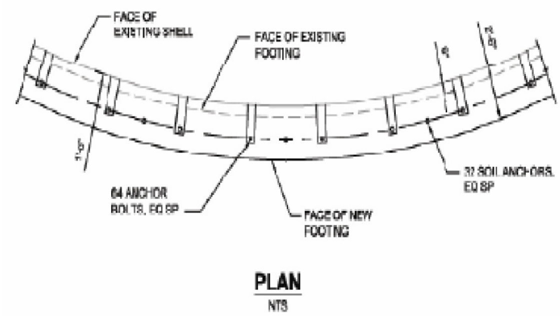


Figure 5. Enlarged Foundation Plan Detail

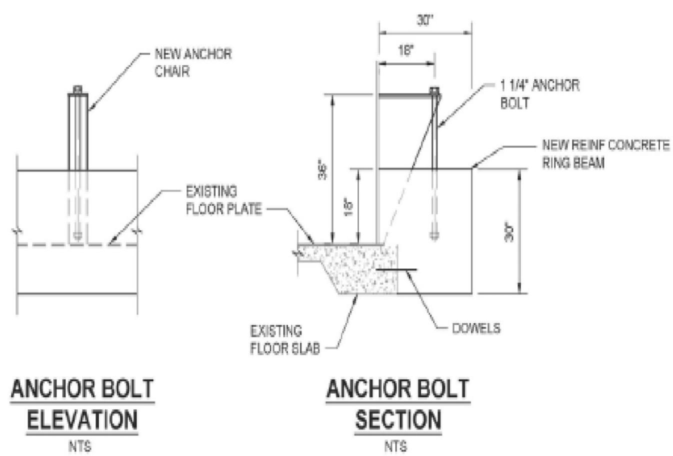
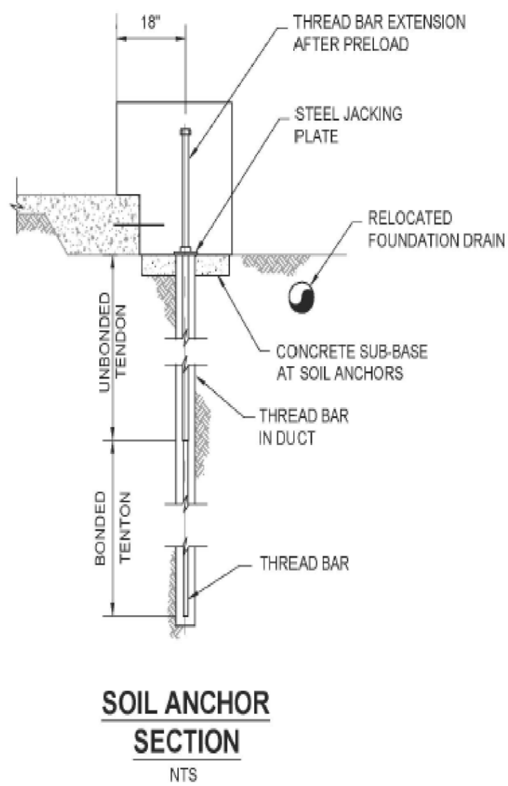
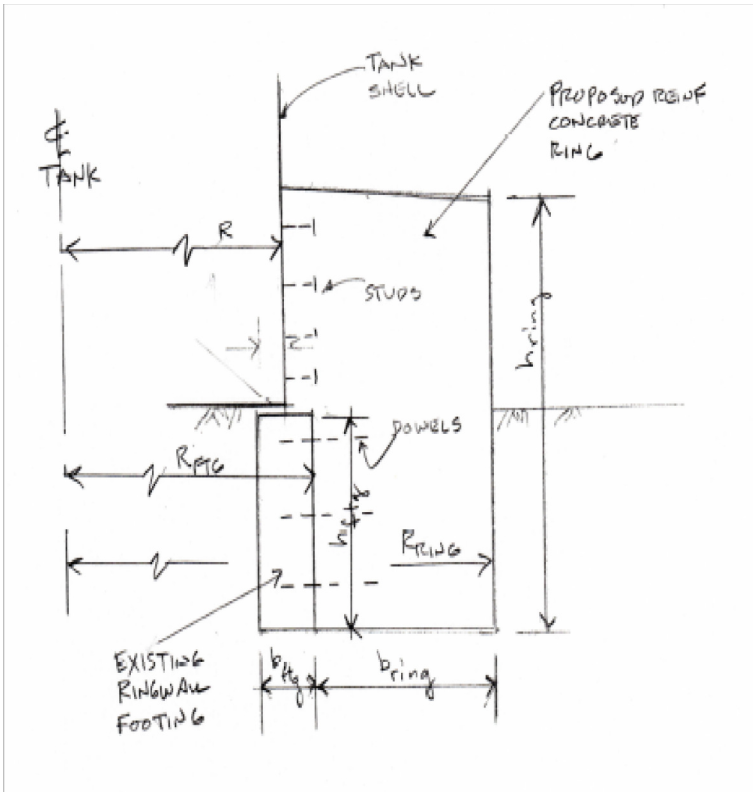


Figure 6. Anchor Bolt and Chair Detail



Supplemental units and unit weights

$$cy := yd^3$$



Existing ringwall and tank dimensions

Existing footing

$R_{ftg} := 25.5 \cdot ft$ outside radius, ex. ftg.

$b_{ftg} := b_{ringwall} = 2.333\,ft$

$h_{ftg} := h_{ringwall} = 3.333\,ft$

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi \cdot \left(R_{ftg}{}^2 - R_{in}{}^2\right)$ footprint

Additional exterior ring

$h_{ring} := h_{ringwall}$ Ring depth

$b_{ring} := 30 \cdot in$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 28\,ft$

$A_{gross} := \pi \cdot R_{ring}{}^2 = 2463\,ft^2$

$A_{ring} := A_{gross} - \pi \cdot R_{ftg}{}^2$

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0) \quad P_{static} = 591 \cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi) \cdot \frac{\left(4 \cdot M_s(0)\right)}{\pi \cdot D^2}$$ Seismic load at base of shell from lateral ground motion

$P_{seismic}(0) = 7121 \cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -7121 \cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 21 \cdot plf$

c. Existing footing Dead Load Component

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 178.4 \cdot kip$ Total weight of existing ringwall

$w_{ftg} := \dfrac{W_{ftg}}{2 \cdot \pi \cdot R} = 1136 \cdot plf$ Ringwall weight per ft of shell

d. Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\, dz\, dr\, d\phi\right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring}-h_{ftg}} r\, dz\, dr\, d\phi\right) = 51.875 \cdot cy$$ Ring volume

$W_{ring} := V_{ring} \cdot \gamma_{conc}$ $W_{ring} = 210 \cdot kip$

$w_{ring} := \dfrac{W_{ring}}{2 \cdot \pi \cdot R} = 1337 \cdot plf$ Anchor ring weight per ft of shell

e. Weight of water over footing

$p_{static} := \gamma_{water} \cdot H = 2090 \cdot psf$

$w_{water} := p_{static} \cdot \dfrac{A_{ftg}}{2 \cdot \pi \cdot R}$

f. Seismic pressure increase/decrease on footing

$w_{water} = 4748 \cdot plf$

(base pressure functions hidden below for brevity)

$\Delta p := p_{base}(R, 0) = 546 \cdot psf$ Plus or minus water pressure at the toe or heel of the tank due to seismic effects

$$w_{seismic} := \int_{\frac{-\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R} p_{base}(r, \phi) \cdot \frac{r}{ft}\, dr\, d\phi$$

Calculate the required anchor transfer capacity between tank and new anchor ring per foot of shell

$SF_{ot} := 1.67$ target safety factor

$Uplift := P_{seismic}(0)$ $Uplift = 7.121 \cdot klf$ Transfer force at face of shell

The resistance of various components is

$D_{tank\_resist} := P_{static} \cdot (1 - .4 \cdot A_v) = 0.57 \cdot klf$

$w_{water\_resist} := (1 - .4 \cdot A_v) \cdot w_{water} - w_{seismic} = 4.431 \cdot klf$

Set number of anchors and compute load.

$n_{anchors} := 36$ $s_{anchor} := \pi \cdot \dfrac{D}{n_{anchors}} = 4.363\,ft$

$$T_{anchor} := \frac{\left[\pi \cdot D \cdot \left(Uplift - D_{tank\_resist} - w_{water\_resist}\right)\right]}{n_{anchors}} = 9.247 \cdot kip$$ measured at the shell

Resistance provided by ring $w_{ring} = 1.337 \cdot klf$

Resistance required by ground anchors

$$Ground\_anchor\_resist := SF_{ot} \cdot (Uplift) - D_{tank\_resist} - w_{water\_resist} - w_{ring} = 5.553 \cdot klf$$

$ground\_anchor\_capacity\_ASD := 75 \cdot kip$

$n_{ground\_anchors} := 18$ provide one ground anchor for every two anchors

$$ground\_anchor\_load := Ground\_anchor\_resist \cdot \pi \cdot \frac{D}{n_{ground\_anchors}} = 48.457 \cdot kip$$

$$s_{ground\_anchor} := \pi \cdot \frac{D}{n_{ground\_anchors}} = 8.727\,ft$$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Ring\_dowels := \left(w_{water} + w_{ftg}\right) = 5883 \cdot plf$$

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$s_{dowels} := s_{anchor} = 4.363\,ft$ $\quad n_{dowels\_per\_row} := 3$

$$Load\_per\_dowel := \frac{s_{dowels}}{s_{anchor}} \cdot \frac{T_{anchor}}{n_{dowels\_per\_row}} = 3082 \cdot lbf$$

Half inch dowels should be more than enough $\quad n_{dowels} := n_{anchors} \cdot n_{dowels\_per\_row} = 108$

Quantities

$n_{dowels} = 108$ $\quad n_{anchors} = 36$ $\quad n_{ground\_anchors} = 18$

$$V_{conc} := \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}^{\ 2} - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 52 \cdot cy$$

By compariison to Sumner Springs reservoir, assume reinforcement at $\quad steel\_unit := 210 \cdot \frac{lbf}{cy}$

$$rebar := V_{conc} \cdot steel\_unit = 10894\,lbf$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 785\,ft^2$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1448\,ft^2$$

$$R_{exc} := R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 7.833\,ft$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1107\,ft^2$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 274 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} = 222.3 \cdot cy$$

Anchor Bolt Sizing

Assume A36 anchor bolts $F_y := 36 \cdot ksi$ $F_u := 58 \cdot ksi$

$F_{anchor} := \min(.80 \cdot 36 \cdot ksi, .50 \cdot 58 \cdot ksi) = 28.8 \cdot ksi$ Allowable seismic load stress on anchors per Ref 5 section 3.3.3.2

$A_{root\_min} := \frac{T_{anchor}}{F_{anchor}} = 0.321 \cdot in^2$ $d_{root\_calc} := \sqrt{\frac{4}{\pi} \cdot A_{root\_min}} = 0.639 \cdot in$

Per Ref 5, 3.8.5.1, add a .25" corrosion allowance to the root diameter for bolts less than 1.25", and use not less than a 1" bolt. This makes an 1.25" bolt the practical minimum

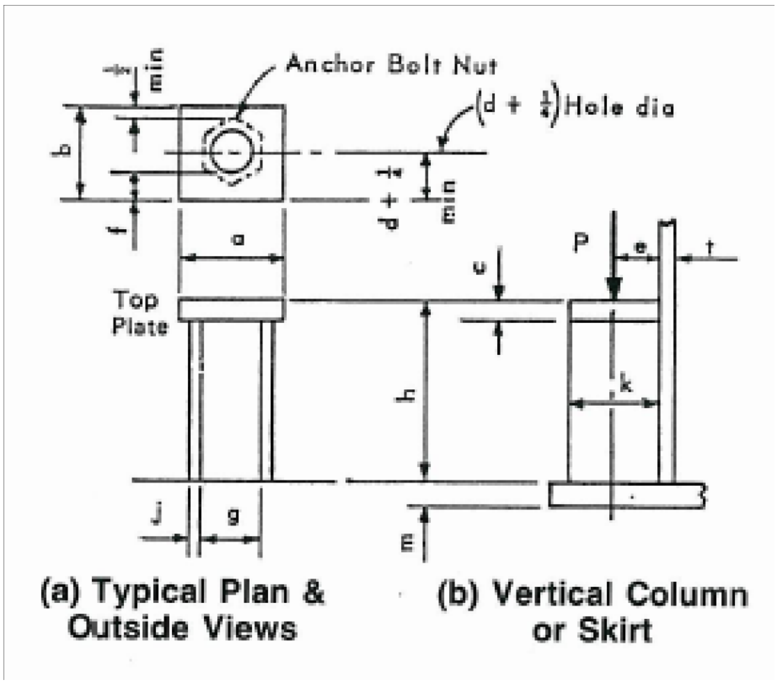
Bolt Dia (in)	Root Dia (in)	Root Area (in^2)	Gross Area (in^2)	Root Dia + .25" (in)	Min Bolt Dia (in)
1.000	0.865	0.587	0.785	1.115	1.375
1.125	0.970	0.74	0.994	1.220	1.500
1.250	1.100	0.942	1.23		1.250
1.375	1.190	1.12	1.49		1.375
1.500	1.320	1.37	1.77		1.500
1.750	1.530	1.85	2.41		1.750
2.000	1.760	2.43	3.14		2.000

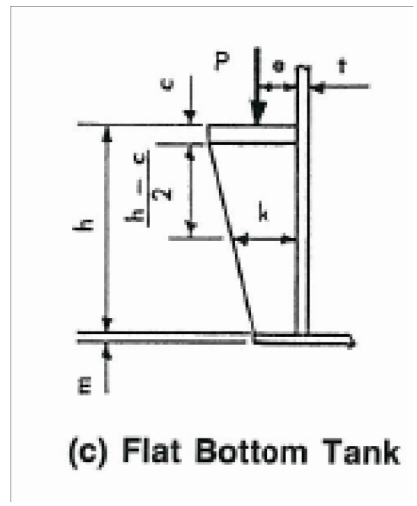
Ref 10, Table 7-18

$d := 1.25 \cdot in$ anchor diameter $A_{bolt} := \pi \cdot \frac{d^2}{4} = 1.227 \cdot in^2$ gross area of bolt

Anchor Chair Design

Methodology is from Ref 11, Part VII - Anchor Bolt Chairs





$e := 18 \cdot in$ bolt centerline distance from shell

Minimum bolt hole size per Ref 11 is

Oversized hole size per Ref 18 Table J.3.3 is $d + \frac{5}{16} \cdot in = 1.563 \cdot in$ for bolts >= 1.25 in. Use

$d_{hole} := d + \frac{5}{16} \cdot in$ $d_{hole} = 1.563 \cdot in$

Edge distance per Ref 10 Tables J.3.4 and J3.5 (from center of hole) is

$c_{edge} := 2.25 \cdot in + \frac{1}{8} \cdot in = 2.375 \cdot in$

$b := e + c_{edge} = 20.375 \cdot in$

$f := c_{edge} - \frac{d_{hole}}{2} = 1.594 \cdot in$

$g := d + 1 \cdot in = 2.25 \cdot in$ minimum side plate separation recommended by Ref 21, however this is very tight for seal welding on interior of plates. Increase this dimension to

$g := 8 \cdot in$

$t := t_s(0)$ $t = 0.406 \cdot in$ Shell bottom course thickness

$P := T_{anchor} = 9.247 \cdot kip$

$S := 1.33 \cdot 15 \cdot ksi = 19.95 \cdot ksi$ Ref 4 allowable stress < 25 ksi recommended by Ref 11 OK

Compute top plate thickness

$$c_{min} := \left[\frac{P}{S \cdot f} \cdot (0.37 \cdot g - 0.22 \cdot d)\right]^{.5} = 0.884 \cdot in$$

use $c := 1.5 \cdot in$

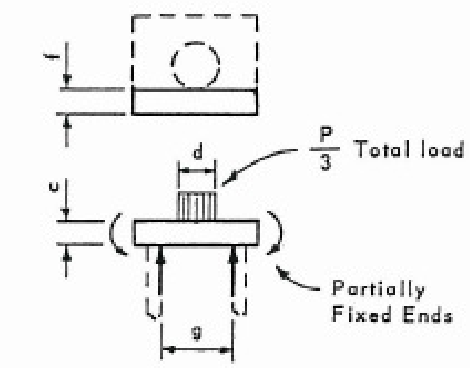


Figure 7-2. Assumed Top-Plate Beam.

top plate thickness

$h := 27 \cdot in$

$j_{min} := \max[.5 \cdot in, 0.04 \cdot (h - c)] = 1.02 \cdot in$ use $j := 1in$

$m := .25 \cdot in$ bottom plate thickness assumption $proj := 2 \cdot in - t$ bottom plate projection from shell face

$a := g + 2 \cdot j + .5 \cdot in = 10.5 \cdot in$ > $2 \cdot c_{edge} = 4.75 \cdot in$ OK   Use $a := 12 \cdot in$

Recess the side plate not more than 1/2" from front edge of top plate per Ref 21. Use .25" to allow seal weld at front edge.

$plate\_top := b - .25 \cdot in$ $k := \frac{(plate\_top + proj)}{2} = 10.859 \cdot in$ mean side plate width

$$\frac{j \cdot k}{\frac{P \cdot in^2}{25 \cdot kip}} = 29.359$$ > 1.0 OK per Ref 21

Compute reduction factor Z for local stress check

$$Z := \frac{1.0}{\frac{(.177 \cdot a \cdot m)}{in\sqrt{R \cdot t}} \cdot \left(\frac{m}{t}\right)^2 + 1.0} = 0.982$$

$$S := \frac{P\cdot e}{in\,t^2}\cdot\left[\frac{1.32\cdot Z}{\left[\frac{1.43\cdot a\cdot h^2}{R\cdot t\cdot in}+\left(4\cdot\frac{a}{in^3}\cdot h^2\right)\right]^{.333}}+\frac{.031\cdot in}{\sqrt{R\cdot t}}\right]$$

$S = 12.5\cdot ksi$

localized vertical shell stress just above the chair. Ref 21 recommends 25ksi max.

Weld Design

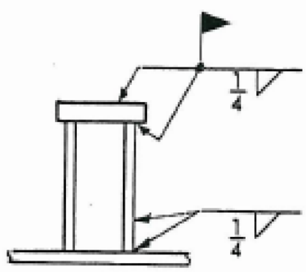


Figure 7-4. Typical Welding, Base Plate Shop Attached.

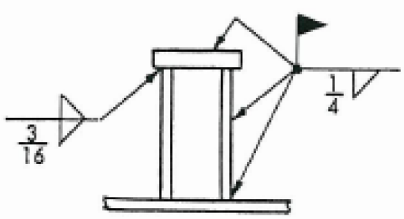


Figure 7-5. Typical Welding, Base or Bottom Field Attached.

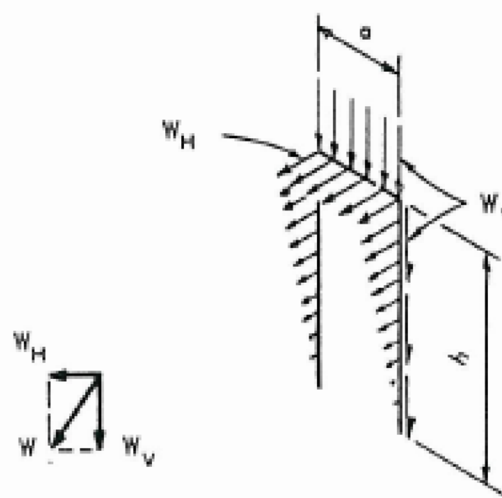


Figure 7-6. Loads on Welds.

$$W_v := \frac{P}{a+2\cdot h} = 140\cdot\frac{lbf}{in}$$

$$W_h := \frac{P\cdot e}{a\cdot h + 0.667\cdot h^2} = 205\cdot\frac{lbf}{in}$$

$$W := \sqrt{W_v^2 + W_h^2} = 249\cdot\frac{lbf}{in}$$

By inspection, a .25" weld will be more than adequate.

Shell shear capacity per inch exceeds weld, OK

Anchor Quantities

$V_{bp} := a \cdot b \cdot c$ $\qquad V_{bp} = 366.75 \cdot in^3$

$V_{sp} := 2 \cdot \dfrac{(b + 2 \cdot in) \cdot (h - c) \cdot j}{2}$ $\qquad V_{sp} = 570.563 \cdot in^3$

$W_{anchor} := \gamma_{steel} \cdot \left(V_{bp} + V_{sp}\right) = 265.789\,lbf$

$W_{anchor\_total} := W_{anchor} \cdot n_{anchors} = 9568\,lbf$

$L_{weld} := 2 \cdot h + a + (a - g - 2 \cdot j) = 68 \cdot in$

$L_{weld\_total} := n_{anchors} \cdot L_{weld} = 204 \cdot ft$

Seismic Evaluation
for
Division 22 Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Compute mat weight and location of center of gravity above the base

$h_{mat} := 2.5\cdot ft$ Mat thickness $BCL_{exist} := 0$ $cy := 27\cdot ft^3$

Existing Bottom Capacity Level (elevation of base of tank)

$BCL := BCL_{exist} + h_{mat}$ $BCL = 2.5\,ft$ Bottom Capacity Level (water elevation at top of mat)

$MOL := H$ Assumed maximum operating level

$TCL := 655.5\cdot ft$ Top Capacity Level (elevation at lip of overflow)

$D = 50\,ft$ Shell diameter

$A_{tank} := \pi\cdot\frac{D^2}{4}$ $A_{tank} = 1963\,ft^2$ Tank footprint

$V_{mat} := A_{tank}\cdot(BCL - BCL_{exist})$ $V_{mat} = 181.8\cdot cy$

$\gamma_{conc} := 150\cdot pcf$ Unit weight of concrete

$W_{mat} := V_{mat}\cdot\gamma_{conc}$ $W_{mat} = 736\cdot kip$ $X_{mat} := \frac{h_{mat}}{2}$ $X_{mat} = 1.25\,ft$

Compute existing floor plate weight

$Floor\_flange := 2\cdot in$ Bottom plate projection beyond shell plate

$D_{plate} := D + 2\cdot Floor\_flange$ $D_{plate} = 50.333\,ft$

$t_{plate} := .25\cdot in$ $W_f := \gamma_{steel}\cdot t_{plate}\cdot\pi\cdot\frac{D_{plate}^2}{4}$ $W_f = 20\cdot kip$

Compute weight of assumed steel plate installed above mat to seal the bottom

$t_{seal} := .25\cdot in$ $W_{seal} := \gamma_{steel}\cdot t_{seal}\cdot\pi\cdot\frac{D^2}{4}$ $W_{seal} = 20\cdot kip$ $X_{seal} := h_{mat}$

Calculate Loads to Foundation

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0)$ $P_{static} = 591\cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

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$$P_{seismic}(\varphi) := \cos(\varphi)\cdot\frac{\left(4\cdot M_s(0)\right)}{\pi\cdot D^2}$$

Seismic load at base of shell from lateral ground motion

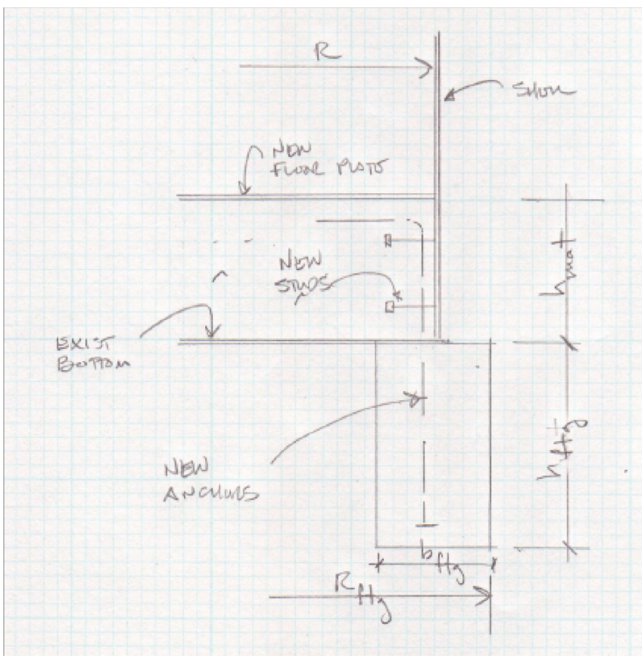
$P_{seismic}(0) = 7121\cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -7121\cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40\cdot A_v\cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 21\cdot plf$

c. Ringwall Dead Load Component



$R_{ftg} := R + 6\cdot in$ from as-built topo

$b_{ftg} := b_{ringwall}$

$b_{ftg} = 2.333\,ft$ from impact-echo measurement

$h_{ftg} := h_{ringwall}$

$h_{ftg} = 3.333\,ft$ field measurement

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$$A_{ftg} := \pi\cdot\left(R_{ftg}^{\;2} - R_{in}^{\;2}\right)$$

$$W_{ftg} := \gamma_{conc}\cdot A_{ftg}\cdot h_{ftg}$$

Total weight of existing ringwall

$$w_{ftg} := \frac{W_{ftg}}{2\cdot\pi\cdot R} = 1136\cdot plf$$

Ringwall weight per ft of shell

<u>Calculate the radial centroid for the ringwall area</u>

$\theta_1 := \frac{1\cdot ft}{R}$ tank angle subtended by one ft of shell length

$$A_{ringwall} := \int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r \, dr \, d\theta \qquad A_{ringwall} = 2.271\,ft^2$$

ringwall footprint per foot of shell

$$r_{ringwall} := \frac{\int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r^2 \, dr \, d\theta}{A_{ringwall}} \qquad r_{ringwall} = 24.352\,ft$$

Radial distance to ringwall center of gravity

d. Mat and New Floor Plate Unit Weight

$$w_{mat} := \frac{(W_{mat} + W_{seal})}{\pi \cdot R^2} \qquad w_{mat} = 385 \cdot psf$$

The required safety factor is not stated directly in the design standards Ref 1 and Ref 3, nor for anchored tanks in Ref 4. It may be inferred from Ref 3 section 12.14.8.4 and the load combinations in Ref 3 section 2.4.

Safety factor >= 0.75 (from 12.14.8.4) * .98 (0.7 earthquake load factor x 1.4 scale up factor to convert Ref 4 earthquake loads to Ref 3 basis) / 0.6 (dead load factor, Ref 3 equation 8, section 3.2.4.1) = 1.23

e. Check Sliding Safety Factor

$V_f = 908 \cdot kip$ Base shear at base of mat

Weight of soil confined by ringwall $A_{soil} := \pi \cdot R_{in}^2$ $\gamma_{soil} := 125 \cdot pcf$ $W_{soil} := \gamma_{soil} \cdot A_{soil} \cdot h_{ftg}$

Ratio of base shear to total dead weight at the plane defined by the base of the footing

$$V_{allow} := \tan(30 \cdot deg) \cdot (W_s + W_r + W_i + W_c + W_f + W_{mat} + W_{seal} + W_{ftg} + W_{soil}) \cdot (1 - .40 \cdot A_v)$$

$V_{allow} = 3267 \cdot kip$ Ref 4 Eq 13-57

$$SF_{sliding} := \frac{V_{allow}}{V_f} \qquad SF_{sliding} = 3.598$$

> 1.0 OK for sliding

f. Check Overturning Safety Factor about the Base of the Mat

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

Calculate overturning moment at the base of the mat

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2}$$

Ref 4 Eq 13-23

$M_s = 10599 \cdot kip \cdot ft$ $M_{ssave} := M_s$ placeholder for later calculation

$$M_{ssum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_c$$

$M_{shell} := M_{ssum}$ placeholder for later calculation

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2}$$

Ref 4 Eq 13-32

$M_{mf} = 16837 \cdot kip \cdot ft$ Result using SRSS method

Results using straight sum method (more conservative)

$$M_{mfsum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_{cmf}$$

$M_{mfsum} = 19559 \cdot kip \cdot ft$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left(W_s + W_r + W_f + W_i + W_{mat} + W_{seal}\right)\right]^2 + \left(A_c \cdot W_c\right)^2}$$

Ref 4 Eq 13-31

$V_f = 997 \cdot kip$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

$M_{mfsum} = 19559 \cdot kip \cdot ft$ Total overturning moment about the base of the mat, including base pressure effects

$W_{resist} := (1 - .40 \cdot A_v) \cdot (W_s + W_r + W_i + W_c + W_{mat} + W_{seal} + W_{ftg})$ $W_{resist} = 4962 \cdot kip$

$M_{res} := W_{resist} \cdot R = 124041 \cdot kip \cdot ft$

$SF_{ot} := \frac{M_{res}}{M_{mfsum}}$ $SF_{ot} = 6.342$ Global safety factor against overturning without regard to uplift, soil pressure, or concrete capacity

g. Check Pressure at Base of Mat Floor Plate - Static - Rigid Mat Assumption

$$q_{static} := \frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2} + (H - h_{mat}) \cdot \gamma_{water}$$ $q_{static} = 4559 \cdot psf$

Weight of structure and water at emergency operating level applied uniformly to the mat.

h. Check Soil Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Down

$$q1_{max} := (1 + .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3}$$ $q1_{max} = 4312 \cdot psf$

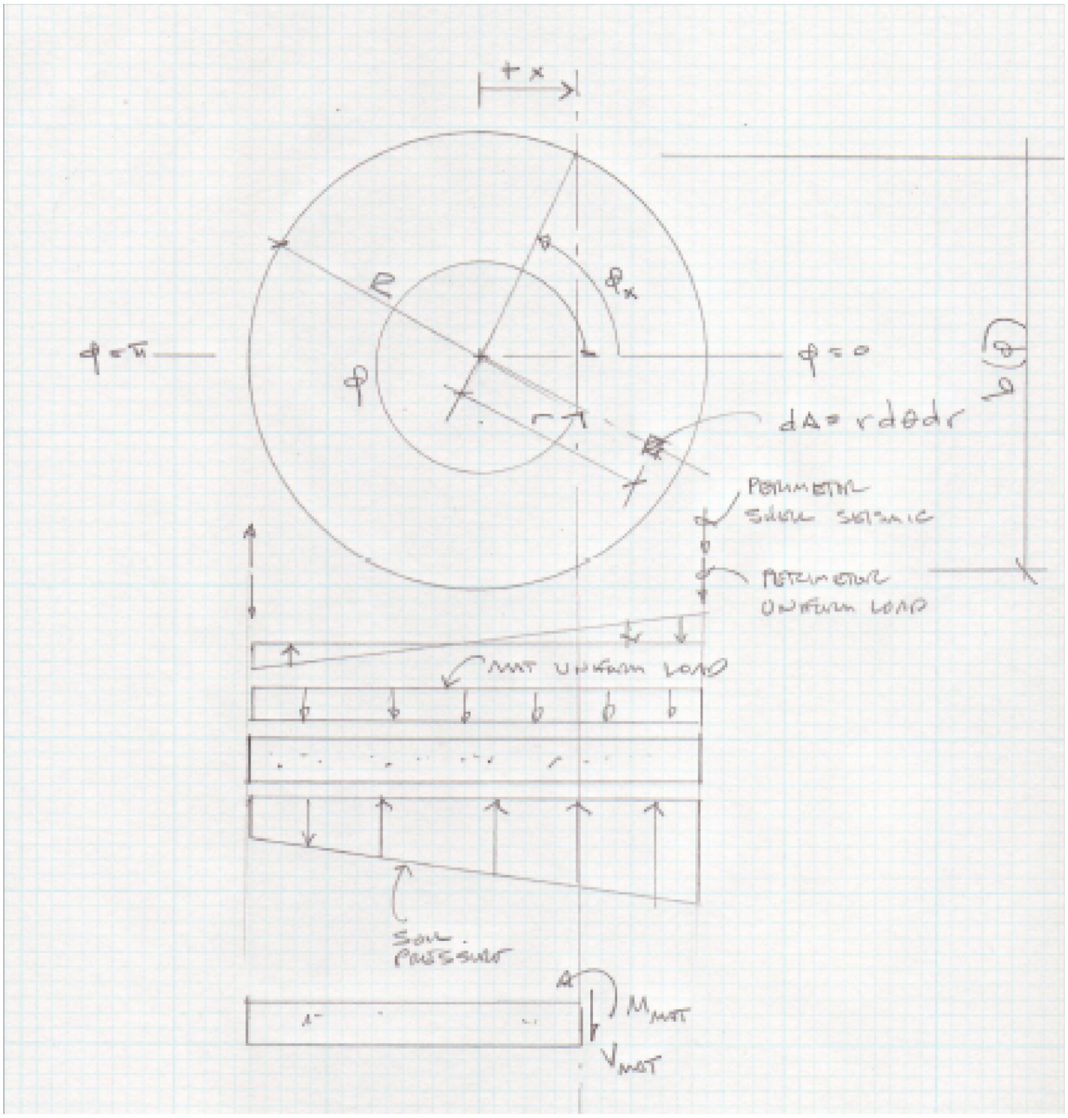
$$q1_{min} := (1 + .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2}\right] - \frac{4M_{mfsum}}{\pi \cdot R^3}$$ $q1_{min} = 1125 \cdot psf$

i. Check Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Up

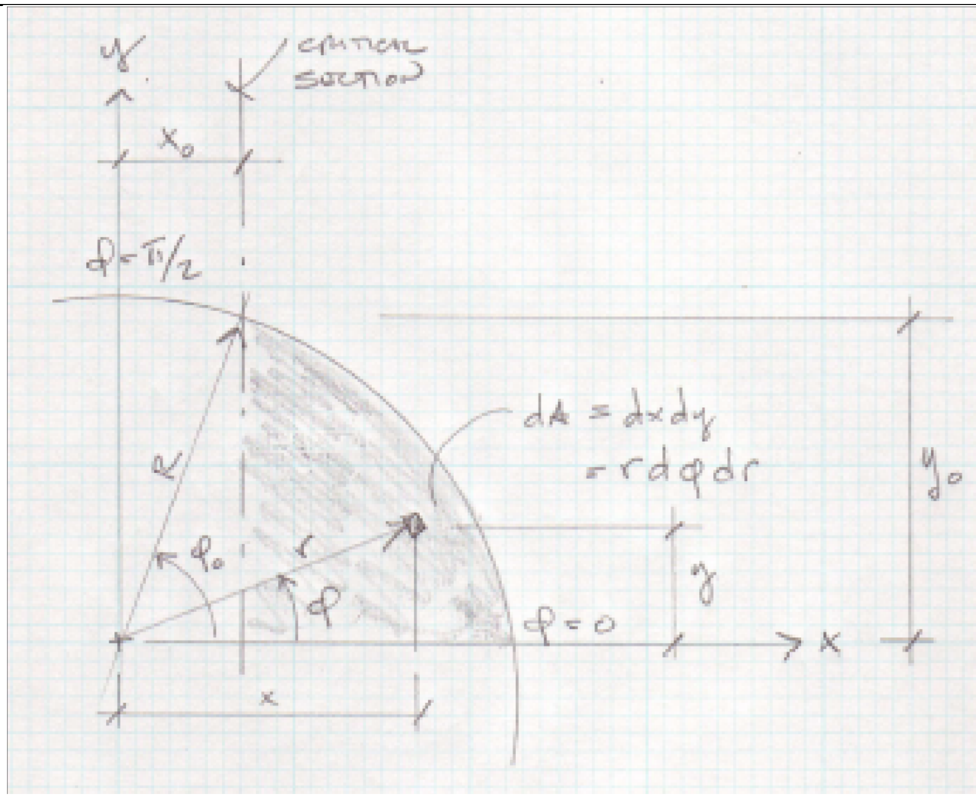
$$q2_{max} := (1 - .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3}$$ $q2_{max} = 4124 \cdot psf$

$$q2_{min} := (1 - .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2}\right] - \frac{4M_{mfsum}}{\pi \cdot R^3}$$ $q2_{min} = 936 \cdot psf$

i. Compute the mat shear and moment under seismic load



(1) First define some basic geometric relationships for the range $0 < \varphi < \pi$



$x(r,\varphi) := r\cdot\cos(\varphi)$ $y(r,\varphi) := r\cdot\sin(\varphi)$ x,y coordinates as functions of polar coordinates r,φ

$r(x,y) := \sqrt{x^2 + y^2}$ $\varphi(x,y) := \mathrm{angle}(x,y)$ polar coordinates as funtions of x,y coordinates

$\varphi_o(x_o) := \mathrm{acos}\left(\frac{x_o}{R}\right)$ $y_o(x_o) := R\cdot\sin(\varphi_o(x_o))$ coordinates of x_o intercept with shell

$x_p(\varphi) := x(R,\varphi)$ $y_p(\varphi) := y(R,\varphi)$ Coordinates of the shell perimeter vs angle from toe

$y_R(x_R) := \sqrt{R^2 - x_R^2}$ $y'_R(x_R) := \frac{d}{dx_R}y_R(x_R)$ Equation for the shell perimeter and its derivative

$L(y) := \sqrt{R^2 - y^2}$

(2) Define functions for soil pressure and for associated mat shear and moment

Write soil pressure functions vs x (soil pressure must be greater than zero at all locations)

$q1_{av} := \frac{(q1_{max} + q1_{min})}{2}$ $q1(x) := q1_{av} + \left(\frac{x}{R}\right)\cdot(q1_{max} - q1_{av})$

$q2_{av} := \frac{(q2_{max} + q2_{min})}{2}$ $q2(x) := q2_{av} + \left(\frac{x}{R}\right)\cdot(q2_{max} - q2_{av})$ Case of vertical seismic loads up

Write functions for shear and moment due to soil pressure at section cut xo due to total soil reaction to the right of the cut

$$Vq1(x_o) := 2\cdot\int_{x_o}^{R} q1(x)\sqrt{R^2 - x^2}\,dx \qquad Mq1(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q1(x)\sqrt{R^2 - x^2}\,dx$$

$$Vq2(x_o) := 2\cdot\int_{x_o}^{R} q2(x)\sqrt{R^2 - x^2}\,dx \qquad Mq2(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q2(x)\sqrt{R^2 - x^2}\,dx$$

(3) Define functions for mat shear and moment due to hydrostatic load and mat, floor, and seal plate loads

$$w_{unif} := \frac{(W_T + W_{mat} + W_{seal} + W_f)}{\pi\cdot R^2} \qquad w_{unif} = 2486\cdot psf$$ uniform load acting down on interior

$$V_{unif}(x_o) := -2\cdot\int_{x_o}^{R} w_{unif}\sqrt{R^2 - x^2}\,dx \qquad M_{unif}(x_o) := -2\cdot\int_{x_o}^{R} (x - x_o)\cdot w_{unif}\sqrt{R^2 - x^2}\,dx$$

(4) Define functions for mat shear and moment due to hydrodynamic base pressure (excluding Av effects)

Total moment due to impulsive and convective effects

$$\Delta M_{imp} := A_i\cdot W_i\cdot(X_{imf} - X_i) = 6324\cdot kip\cdot ft$$

$$\Delta M_{conv} := A_c\cdot W_c\cdot(X_{cmf} - X_c) = 272\cdot kip\cdot ft$$

The impulsive base pressure varies as

$$\frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

From Ref 5, Equation F80

Integration constant for impulsive base pressure is

$$Const_{imp} := \frac{\Delta M_{imp}}{2\int_{-R}^{R}\int_{0}^{y_o(x)} \frac{x\cdot\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}\,dy\,dx}$$

$$Const_{imp} = 615\cdot psf$$

And the pressure function can be written as

$$p_{base\_i}(x,y) := Const_{imp} \cdot \frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

The convective base pressure varies as $\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3$ From Ref 5, Equation F108

Integration constant for convective base pressure is

$$Const_{conv} := \frac{\Delta M_{conv}}{2\int_{-R}^{R}\int_{0}^{y_o(x)} x\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right] dy\,dx}$$

$$Const_{conv} = 27\cdot psf$$

And the pressure function can be written as

$$p_{base\_c}(x,y) := Const_{conv}\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right]$$

The combined base pressure associated with convective and impulsive effects is

$$p_{base}(x,y) := p_{base\_i}(x,y) + p_{base\_c}(x,y)$$

$p_{base}(R,0) = 546\cdot psf$ Maximum pressure at toe

As a check, compare maximum bottom pressure if an approximate linear distribution of base pressure is assumed by dividing the total moment by the section modulus of the foundation footprint

$$p_{toe\_linear} := 4\cdot\frac{\left(\Delta M_{imp} + \Delta M_{conv}\right)}{\pi\cdot R^3}$$

$p_{toe\_linear} = 537\cdot psf$ $\quad \frac{p_{toe\_linear}}{p_{base}(R,0)} = 0.984$ OK

$$V_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} p_{base}(x,y)\,dy\,dx$$

$$M_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} (x - x_o)\,p_{base}(x,y)\,dy\,dx$$

(5) Define functions for mat shear and moment due to Av only (up or down, not including loads at shell)

$$V_{Av1}(x_o) := -2 \cdot \int_{x_o}^{R} .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx \qquad M_{Av1}(x_o) := -2 \cdot \int_{x_o}^{R} (x - x_o) \cdot .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx$$

$$V_{Av2}(x_o) := 2 \cdot \int_{x_o}^{R} .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx \qquad M_{Av2}(x_o) := 2 \cdot \int_{x_o}^{R} (x - x_o) \cdot .4 \cdot A_v w_{unif} \sqrt{R^2 - x^2}\, dx$$

(6) Define functions for mat shear and moment due to roof shell and footing dead load applied at the perimeter

$$V_{shell\_static}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg}) \cdot R\, d\varphi$$

$$M_{shell\_static}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg}) \cdot (R \cdot \cos(\varphi) - x_o) \cdot R\, d\varphi$$

(7) Define functions for mat shear and moment due to lateral seismic loads all applied at the perimeter

Write hydrodynamic force intensity at the shell as a function of φ

$$E_{shell}(\varphi) := \left(\frac{M_{shell}}{\pi \cdot R^2}\right) \cdot \cos(\varphi)$$

$$V_{E\_shell}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi) \cdot R\, d\varphi$$

$$M_{E\_shell}(x_o) := -2 \cdot \int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi) \cdot (R \cdot \cos(\varphi) - x_o) \cdot R\, d\varphi$$

(8) Define functions for mat shear and moment due to Av loads applied at the perimeter

$$V_{shell\_Av1}(x_o) := .40 \cdot A_v \cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av1}(x_o) := .40 \cdot A_v \cdot M_{shell\_static}(x_o)$$

$$V_{shell\_Av2}(x_o) := -.40 \cdot A_v \cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av2}(x_o) := -.40 \cdot A_v \cdot M_{shell\_static}(x_o)$$

(9) Define functions for mat shear and moment due to center column force

$$P_{D\_ctr} := W_{roof\_center} + W_{col\_base} + W_{col} = 10.7\cdot kip$$

$$V_{ctr}(x_o) := \text{if}(x_o > 0, 0, -P_{D\_ctr})$$

$$M_{ctr}(x_o) := \text{if}(x_o > 0, 0, x_o \cdot P_{D\_ctr})$$

(10) Define functions for total mat shear and moment due to combined loadins for the case of Av up or down

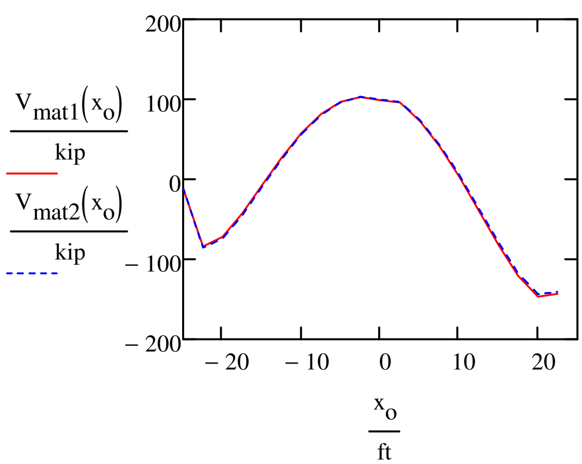
$$V_{mat1}(x_o) := Vq1(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av1}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av1}(x_o) + V_{ctr}(x_o)\cdot(1 + .40\cdot A_v)$$

$$V_{mat2}(x_o) := Vq2(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av2}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av2}(x_o) + V_{ctr}(x_o)\cdot(1 - .40A_v)$$

$$M_{mat1}(x_o) := Mq1(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av1}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av1}(x_o) + M_{ctr}(x_o)\cdot(1 + .40\cdot A_v)$$

$$M_{mat2}(x_o) := Mq2(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av2}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av2}(x_o) + M_{ctr}(x_o)\cdot(1 - .40\cdot A_v)$$

$x_o := -R, -R + \frac{R}{10} .. R$ Set plot parameters



$V_{mat1}(R) = 0\cdot kip$

$V_{mat2}(R) = 0\cdot kip$

$V_{mat1}(-R) = -11.1\cdot kip$

$V_{mat2}(-R) = -10.3\cdot kip$ All values zero, check

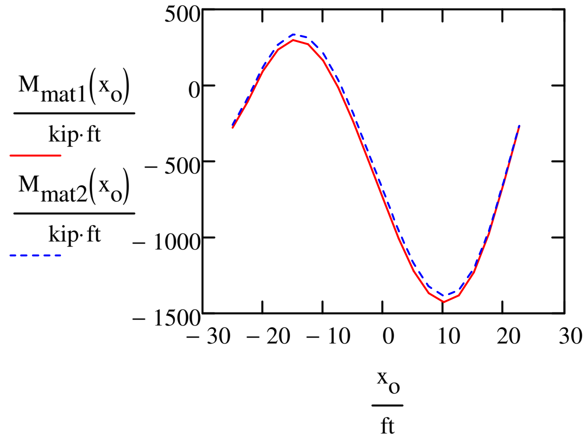
$M_{mat1}(R) = 0 \cdot kip \cdot ft$

$M_{mat2}(R) = 0 \cdot kip \cdot ft$

$M_{mat1}(-R) = -277 \cdot kip \cdot ft$

$M_{mat2}(-R) = -257.742 \cdot kip \cdot ft$

All values zero, check



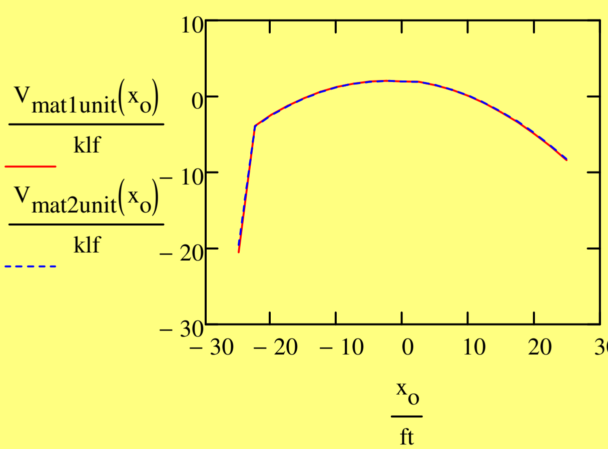
These forces are distributed over a variable mat width. Convert to average unit forces in the mat

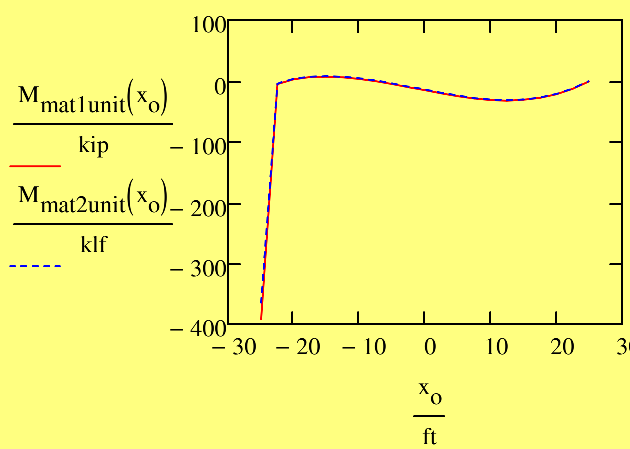
Note: These expressions cannot be evaluated at R or -R because the denominato is zero at the limits. Evaluate at values of x close to +/- R

$$V_{mat1unit}(x_o) := \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat1unit}(x_o) := \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$$V_{mat2unit}(x_o) := \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat2unit}(x_o) := \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$x_o := -.9999R, -R + \frac{R}{10} .. .9999R$ Plot parameters





<u>Average unit shear and moment in the mat, ASD basis</u>

Compute maxima and minimima

$x_o := 0$

Given

$V_{mat1unit}(x_o) = \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)}$

$V_{mat1unitmax} := V_{mat1unit}(Maximize(V_{mat1unit}, x_o)) = 2.208 \cdot klf$

$V_{mat1unitmin} := min(V_{mat1unit}(-.9999R), V_{mat1unit}(.9999R)) = -20.478 \cdot klf$

$Vu_{mat1} := 1.4 max(|V_{mat1unitmax}|, |V_{mat1unitmin}$ $Vu_{mat1} = 28.669 \cdot klf$

Given

$V_{mat2unit}(x_o) = \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)}$

$V_{mat2unitmax} := V_{mat2unit}(Maximize(V_{mat2unit}, x_o)) = 2.208 \cdot klf$

$V_{mat2unitmin} := min(V_{mat2unit}(-.9999R), V_{mat2unit}(.9999R)) = -19.516 \cdot klf$

$Vu_{mat2} := 1.4 max(|V_{mat2unitmax}|, |V_{mat2unitmin}$ $Vu_{mat2} = 27.323 \cdot klf$

$Vu_{mat} := max(Vu_{mat1}, Vu_{mat2})$ $Vu_{mat} = 28.669 \cdot klf$

$x_o := \frac{-R}{2}$

Given

$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$

$M_{mat1unitmax} := M_{mat1unit}(Maximize(M_{mat1unit}, x_o))$ $M_{mat1unitmax} = 7.512 \cdot kip$

$x_o := \frac{R}{2}$

Given

$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$

$M_{mat1unitmin} := M_{mat1unit}(Minimize(M_{mat1unit}, x_o))$ $M_{mat1unitmin} = -31.883 \cdot kip$

$x_o := \frac{-R}{2}$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmax} := M_{mat2unit}(\text{Maximize}(M_{mat2unit}, x_o))$ $M_{mat2unitmax} = 8.456 \cdot kip$

$x_o := \frac{R}{2}$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmin} := M_{mat2unit}(\text{Minimize}(M_{mat2unit}, x_o))$ $M_{mat2unitmin} = -31.105 \cdot kip$

$Mu_{mat\_pos} := 1.4\max(M_{mat1unitmax}, M_{mat2unitmax}$ $Mu_{mat\_pos} = 11.838 \cdot kip$

$Mu_{mat\_neg} := 1.4\min(M_{mat1unitmin}, M_{mat2unitmin})$ $Mu_{mat\_neg} = -44.636 \cdot kip$

Capacity Check and Preliminary Quantities

Material assumptions

$f'c := 4000 \cdot psi$ $f_y := 60 \cdot ksi$ $d := h_{mat} - 4 \cdot in$ $d = 2.167\,ft$ $h_{mat} = 2.5\,ft$

Check shear capacity

$\varphi Vc := .75 \cdot 2 \cdot d \cdot \sqrt{\frac{f'c}{psi}} \cdot psi$ $\varphi Vc = 29.599 \cdot klf$ $\frac{Vu_{mat}}{\varphi Vc} = 0.969$ 1.0 OK

Compute approximate bottom steel requirement

$A_{s\_bot} := \frac{Mu_{mat\_pos}}{.90 \cdot .90 \cdot d \cdot f_y}$ $A_{s\_bot} = 0.112 \cdot \frac{in^2}{ft}$ Computed steel requirement

$A_{s\_bot} := \text{if}\left[\left(\frac{A_{s\_bot}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_bot}, A_{s\_bot}\right]$ $A_{s\_bot} = 0.15 \cdot \frac{in^2}{ft}$ Adjust steel requirement if computed steel ratio less than 200/fy

$A_{s\_top} := \frac{-Mu_{mat\_neg}}{.90 \cdot .90 \cdot d \cdot f_y}$ $A_{s\_top} = 0.424 \cdot \frac{in^2}{ft}$ Computed steel requirement

$A_{s\_top} := \text{if}\left[\left(\frac{A_{s\_top}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_top}, A_{s\_top}\right]$ $A_{s\_top} = 0.565 \cdot \frac{in^2}{ft}$ Adjust steel requirement if computed steel ratio less than 200/fy

Reinforcement requirement per unit area of mat

$w_{reinf} := \gamma_{steel} \cdot 2 \cdot \left(A_{s\_bot} + A_{s\_top}\right)$ $w_{reinf} = 4.865 \cdot psf$

$W_{reinf} := w_{reinf} \cdot \pi \cdot R^2$ $W_{reinf} = 9553\,lbf$ $cy := 27 \cdot ft^3$ $h_{mat} = 2.5\,ft$

Concrete and seal steel quantities

$V_{conc} := h_{mat} \cdot \pi \cdot R^2$ $V_{conc} = 181.805 \cdot cy$ $W_{seal} = 20044\,lbf$ $V_{conc} = 36720\,gal$

Placeholder unit costs for concrete and steel $reinf\_cost := \frac{1}{lbf}$ $conc\_cost := \frac{500}{cy}$ $steel\_cost := \frac{2}{lbf}$

$Cost := W_{reinf} \cdot reinf\_cost + V_{conc} \cdot conc\_cost + W_{seal} \cdot steel\_cost$ $Cost = 140544$

$b_{ftg} = 2.333\,ft$ $b_{ftg} = 2.333\,ft$ b

APPENDIX B.3

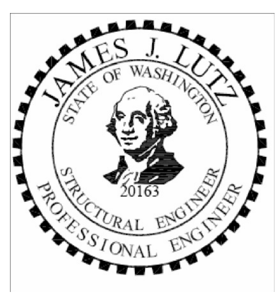
DIVISION 7 RESERVOIR CALCULATIONS

Seismic Evaluation
for
Division 7 Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Methodology Remarks

These calculations are limited to an assessment of the primary elements of the lateral force resisting system for the reservoir under seismic loading. Following is a summary of the methodology used:

1. All dimensions and weights are based on record drawings furnished by the client, supplemented by field measurements.In case of discrepancies, field measurements were used..

2. Water level assumed for seismic calculations is based on maximum current operating level provided by the District..

3. Methodology for determination of seismic loads for tanks with a free water surface is based on the 2012 International Building Code, ASCE 7-10, and AWWA Standard D100-11. These codes and standards post-date and are more stringent than codes and standards used at the time of original tank design.

4. For tanks where the free surface sloshing wave amplitude exceeds the roof elevation, the additional amplification of seismic load is based on an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave. The force is modeled by computing an increase in mass and adjusting the convective period of the water mass. The pressure distribution is assumed the same as for a tank with a free water surface.

5. For tanks where the static water surface level already contacts the roof, the free surface sloshing amplitude is based on a cylinder of the same height and radius with zero freeboard, however the actual water mass is assumed. The ratio of sloshing amplitude to roof height is computed using roof height measured from the free water surface. Adjustments in seismic load are otherwise the same as for the preceding step.

6. Ground motion spectral accelerations S_S and S_1 are those currently available from the USGS on their web site calculator for the latitude and longitude of the tank as taken from Google Earth.

7. Soil site class "D" is assumed as a default in the absence of a soils report for this reservoir..

8. Wind loads, hydrostatic loads at overflow elevation, and roof live loads were not considered in the analysis. However where calculated roof loads exceed 40 psf, a mass equal to .20 times the uniform roof snow load is added to the roof mass for seismic calculations. The gravity effects of snow load were considered whete applicable for determining loads on the shell, however no analysis of roof members was included.

Location and Site Data

Lat 48.7111, Long -122.3189
El 673 ft
(Google Earth)

Superstructure Geometry

From record drawings or field measurement

Tank diameter $D := 70 \cdot ft$

Tank radius $R := \frac{D}{2} = 35\,ft$

Shell height $H_s := 35 \cdot ft$

Floor elevation at shell
(Bottom capacity level)

$BCL := 669 \cdot ft$ (District)

Overflow height above floor

$h_{overflow} := H_s - 6 \cdot in = 34.5\,ft$ (Estimated)

Overflow elevation
(Top capacity level)

$TCL := BCL + h_{overflow}$

$H := 33.5 \cdot ft$ Maximum operating level

$NOL := BCL + H = 702.5\,ft$

$BCL + H_s = 704\,ft$

This level is below the top of the shell ($H < H_S$)

Describe the roof geometry

$roof\_slope := \frac{.75}{12} = 0.063$ (Actual varies between .50" and .81" per 12")

The roof height is $h_r := roof\_slope \cdot R = 2.188\,ft$

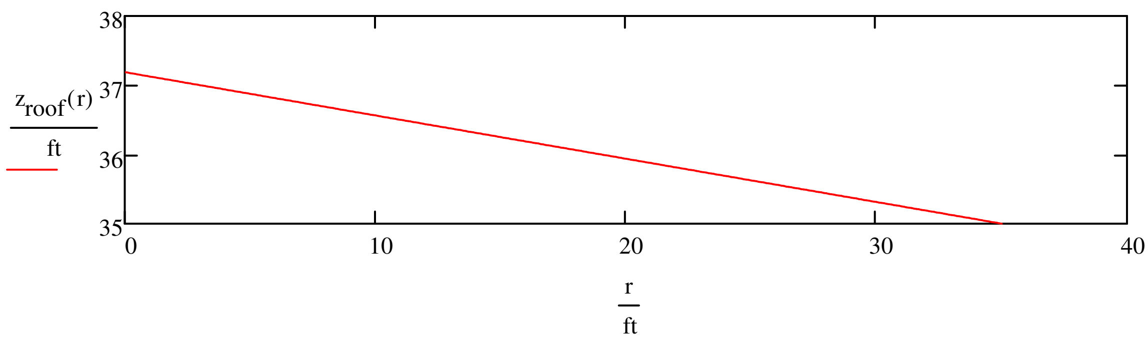
Let "z" be the distance measured vertically from the floor, and "r" the horizontal distance from the center

$z_{apex} := H_s + h_r = 37.188\,ft$

The expression for z for the roof for 0 < r < R is

$z_{roof}(r) := \left(if\left(r > R, 0, z_{apex} - roof\_slope \cdot r\right)\right)$

Plot the roof elevation vs radius $r := 0, .1 \cdot ft .. R$



Enter shell and roof plate thickness.

Mathcad General Input - See Appendix for Mathcad nomenclature and symbols

$ORIGIN := 1$

Special unit definitions $each := 1$ $sf := ft^2$

number of shell plate courses,
numbering starting with the base as
course 1

$n_{course} := 5$ (the vertical leg of the top angle is included with the top shell plate course)

Calculate the elevation of the top of each shell course relative to the floor

$i := 1, 2 .. n_{course}$ i is the number of each shell course, starting from the bottom

$\gamma_{steel} := 490 \cdot pcf$ unit weight of steel zshell is the elevation of the top of each course relative to the top of the bottom plate

$$z_{shell} := \begin{pmatrix} 7.02 \\ 14.02 \\ 21.03 \\ 28.04 \\ 35 \end{pmatrix} \cdot ft \qquad t_{shell} := \begin{pmatrix} \frac{11}{32} \\ \frac{9}{32} \\ .25 \\ .25 \\ .25 \end{pmatrix} \cdot in \qquad w_{shell} := t_{shell} \cdot \gamma_{steel} = \begin{pmatrix} 14.036 \\ 11.484 \\ 10.208 \\ 10.208 \\ 10.208 \end{pmatrix} \cdot psf \qquad class_{shell} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Shell thickness is per field measurements, rounded to the nearest 1/32 inch. Original specifications not known. Assume minimum yield stress to qualify as AWWA D100 Class 1, Fy=27 ksi.

Class 1 material has a yield stress 27 ksi < Fy < 34 ksi. Class 2 material has a yield stress Fy > 34 ksi

$t_{roof\_plate} := \frac{5}{16} \cdot in$ roof plate thickness as measured, rounded to nearest 1/32 inch

Compute weight of roof and shell

Define the roof slope at any point

$$z'_{roof}(r) := \frac{d}{dr} z_{roof}(r)$$

Compute the surface area of the roof plate tributary to the perimeter and the center column. . Ignore laps

For a surface of revolution, the general equation for the surface area is

$$A := 2 \cdot \pi \cdot \int r\,ds \qquad \text{where} \qquad ds := \sqrt{1 + \left(\frac{dz}{dr}\right)^2} \cdot dr$$

$$A_{roof\_plate} := 2 \cdot \pi \cdot \left(\int_0^R r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 3856\, ft^2 \text{(roof surface area)}$$

$$W_{roof\_plate} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate} = 49.204 \cdot kip$$

$$A_{roof\_plate\_center} := 2 \cdot \pi \cdot \left(\int_0^{\frac{R}{2}} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 964\, ft^2$$

$W_{roof\_plate\_center} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_center} = 12.301 \cdot kip$ Portion of roof weight tributary to center column

$$A_{roof\_plate\_edge} := 2 \cdot \pi \cdot \left(\int_{\frac{R}{2}}^{R} r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right) = 2892\, ft^2$$

$W_{roof\_plate\_edge} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate\_edge} = 36.903 \cdot kip$ Portion of roof weight tributary to shell

Calculate the vertical center of gravity from the tank floor for the roof plate

$$x_{cg} := 2 \cdot \pi \cdot \frac{\left(\int_{0}^{R} r^2 \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr \right)}{A_{roof\_plate}} = 23\, ft$$

$$X_{roof\_plate} := z_{roof}\left(x_{cg}\right) = 35.729\, ft$$

Define the number of the shell course for any value of $0 < z < Hs$ using a series of functions

$i_{course}(z) := n_{course}$ Default value

$$i_{course}(z) := \text{if}\left(z < z_{shell_{n_{course}}}, n_{course}, i_{course}(z)\right)$$

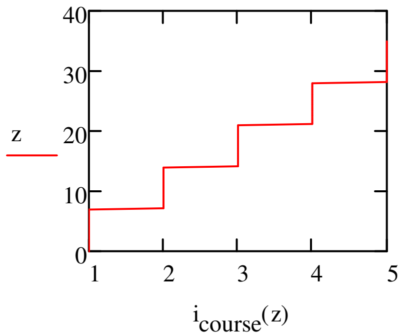
$$i_{course}(z) := \text{if}\left(z < z_{shell_4}, 4, i_{course}(z)\right)$$

$$i_{course}(z) := \text{if}\left(z < z_{shell_3}, 3, i_{course}(z)\right)$$

$$i_{course}(z) := \text{if}\left(z < z_{shell_2}, 2, i_{course}(z)\right)$$

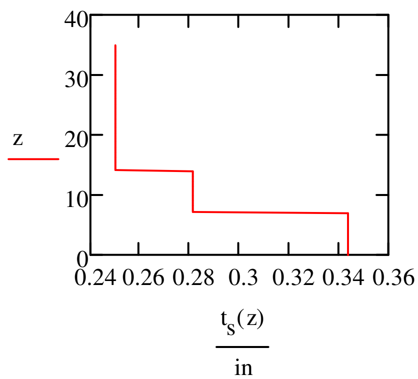
$$i_{course}(z) := \text{if}\left(z < z_{shell_1}, 1, i_{course}(z)\right)$$

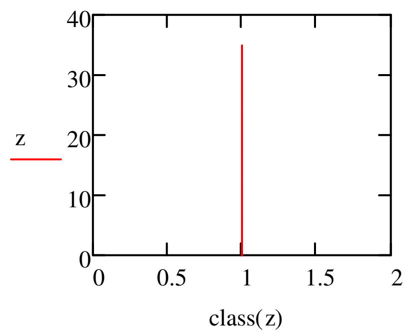
$z := 0 \cdot ft, 0.2 \cdot ft .. H_s$ Set plotting interval for graphs



write functions that return the shell plate thickness and class as a function of height above the base

$t_s(z) := t_{shell_{i_{course}(z)}}$ $class(z) := class_{shell_{i_{course}(z)}}$





<u>Shell thickness vs elevation</u> **<u>Shell class vs elevation</u>**

Floor plate thickness $t_{floor} := \frac{5}{16} \cdot in$ field measurement

$floor\_flange := 2. \cdot in$ Bottom plate projection beyond shell plate $D_{floor} := D + 2 \cdot floor\_flange$

<u>Compute floor weight</u>

$W_f := \gamma_{steel} \cdot t_{floor} \cdot \pi \cdot \frac{D_{floor}^{\;2}}{4}$ $W_f = 49.6 \cdot kip$

<u>Compute the weight of the shell and establish its center of gravity from the base</u>

$W_s := \pi \cdot D \cdot \int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \, dz$ $W_s = 86.43 \cdot kip$

$$X_s := \pi \cdot D \cdot \frac{\int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \cdot z \, dz}{W_s} \qquad X_s = 16.387\,ft$$

Compute the weight of the roof and establish its center of gravity from the base

The total roof mass is a combination of the part tributary to the center column and the part tributary to the edge. The center portion includes part of the roof, partial weight of the rafters, the column cap, and half of the column. (The other half of the column and its base plate are assigned to the floor mass). The edge portion includes part of the roof, partial weight of the rafters, clips and the flange of the top angle. The weight of top angle and clips are ignored.

Based on video, there are 18 wide-flange primary rafters spanning from the shell to the center column top plate. Midway between the primary rafters are 18 channel shaped short rafters spanning from the shell to a channel shaped header supported by the primary rafters. The headers appear to be roughly a quarter of the distance from the shell based on review of inspection videos.There are no records for the member sizes and the rafters were not accessible for field measurements.

Assume all channel rafters/headers are C6X8.2 and long wide-flange rafters are W8X10 based on scratch calculations (not shown)

$L_{rafter\_long} = 34\,ft \qquad L_{rafter\_short} = 8.5\,ft \qquad L_{header} = 9.345\,ft$

$$W_{rafters} := 18 \cdot \left(L_{rafter\_long} \cdot 10 \cdot plf + L_{rafter\_short} \cdot 8.2 \cdot plf + L_{header} \cdot 8.2 \cdot plf\right) = 8.754 \cdot kip$$

$$W_{col\_cap} := \pi (12 \cdot in)^2 \cdot .5 \cdot in \cdot \gamma_{steel} = 0.064 \cdot kip$$

$$W_{col} := 33.6 \cdot ft \cdot 18.7 \cdot \frac{lbf}{ft} = 0.628 \cdot kip$$

$$W_{col\_base} := \gamma_{steel} \cdot \left[.5 \cdot in \cdot \pi \cdot (18 \cdot in)^2 + .375 \cdot in \cdot 2 \cdot 1 \cdot ft^2\right] = 0.175 \cdot kip$$ assumed base plate and gussets

$$W_{roof\_center} := W_{roof\_plate\_center} + \frac{W_{rafters}}{2} + W_{col\_cap} + \frac{W_{col}}{2} = 17.056 \cdot kip$$ Roof weight tributary to center column

$$W_{roof\_edge} := W_{roof\_plate\_edge} + \frac{W_{rafters}}{2} = 41.28 \cdot kip$$ Roof weight tributary to top of shell

$\Delta W_f := W_{col\_base} + \dfrac{W_{col}}{2} = 0.489 \cdot kip$ Column and base plate tributary to floor

Total roof structure mass for seismic calculation $W_r := W_{roof\_center} + W_{roof\_edge} = 58.336 \cdot kip$

Check to see if roof snow load mass must be included per ASCE 7-10

$p_g := 50 \cdot psf$ from "Snow Load Analysis for Washington", 2nd ed, SEAW

$I_s := 1.20$ Snow load importance factor for risk category IV, ASCE 7-10

$C_e := 1.2$ ASCE 7-10, Table 7-2. Exposure Factor, Terrain B, Sheltered

$C_t := 1.2$ ASCE 7-10, Table 7-3, Thermal Factor, Unheated

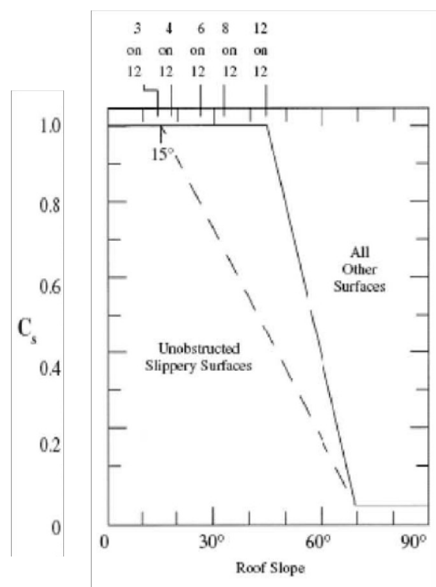
$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 60.48 \cdot psf$ Flat roof snow load, ASCE 7-10 Eq 7.3-1. Since flat roof snow load exceeds 30 psf, add 20% of the design snow load to the roof mass per ASCE 7-10, section 12.7.2.

The roof slope is $\mathrm{atan}(roof\_slope) = 3.576 \cdot deg$

From ASCE 7-10 Fig 7-2c, the roof slope factor is

$C_s := 1.0$

$p_s := C_s \cdot p_f = 60.48 \cdot psf$



7-2c: Cold roofs with C = 1.2 or larger

Snow weight to include with roof weight

$w_{snow} := .20 \cdot p_s = 12.096 \cdot psf$

$W_{snow} := w_{snow} \cdot \pi \cdot R^2 = 46.551 \cdot kip$

Snow weight tributary to edge

$W_{snow\_shell} := W_{snow} \cdot \dfrac{A_{roof\_plate\_edge}}{A_{roof\_plate}} = 34.913 \cdot kip$

$P_{snow} := \dfrac{W_{snow\_shell}}{\pi \cdot D} = 158.76 \cdot \dfrac{lbf}{ft}$ Snow load applied at top of shell concurrent with seismic

Snow weight tributary to floor

$W_{snow\_floor} := W_{snow} - W_{snow\_shell} = 11.638 \cdot kip$

All the lateral resistance for the roof is assumed to be by the shell, except for the lower half of the column

Compute the center of gravity of the roof and column mass for seismic calculation

$$X_r := \frac{\left[W_{roof\_plate} \cdot X_{roof\_plate} \cdots + z_{apex} \cdot W_{col\_cap} + .75 \cdot z_{apex} \cdot \frac{W_{col}}{2} + W_{rafters} \cdot \left(H_s + \frac{h_r}{2}\right)\right]}{W_r} = 35.743\,\text{ft}$$

Compute the center of gravity of the roof snow load for seismic calculations

Snow density per ASCE 7-10 equation 7.7.1 is

$$\gamma_{snow} := \min\left(30 \cdot \text{pcf}, 0.13 \cdot \frac{p_g}{\text{ft}} + 30 \cdot \text{pcf}\right) = 30 \cdot \text{pcf}$$ snow depth $$h_d := \frac{w_{snow}}{\gamma_{snow}} = 0.403\,\text{ft}$$

$$X_{snow} := X_{roof\_plate} + \frac{h_d}{2} = 35.931\,\text{ft}$$ centroid of snow mass

Compute total water weight for seismic calculations

$$\gamma_{water} := 62.4 \cdot \text{pcf}$$

$$W_T := \gamma_{water} \cdot H \cdot \pi \cdot \frac{D^2}{4} = 8044.8 \cdot \text{kip}$$

Calculate the impulsive and convective water weights and vertical centroids

$$\frac{D}{H} = 2.09$$

$$W_i := W_T \cdot \frac{\tanh\left(.866 \cdot \frac{D}{H}\right)}{.866 \cdot \frac{D}{H}}$$ if D/H > 1.333

$$W_i := \text{if}\left[\frac{D}{H} < 1.333, W_T \cdot \left(1.0 - 0.218 \cdot \frac{D}{H}\right), W_i\right]$$ if D/H < 1.33

$$W_i = 4213.613 \cdot \text{kip}$$ Impulsive water weight $$\frac{W_i}{W_T} = 0.524$$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated or the total moment on the foundation, shell plus floor.

$$X_i := H \cdot \text{if}\left[\left(\frac{D}{H}\right) > 1.333, 0.375, 0.50 - 0.094 \cdot \frac{D}{H}\right]$$ $$X_i = 12.563\,\text{ft}$$ centroid for calculation of just the shell moment

$$X_{imf} := 0.375 \cdot \left[1.0 + 1.333 \cdot \left(\frac{0.866 \cdot \frac{D}{H}}{\tanh\left(0.866 \cdot \frac{D}{H}\right)} - 1 \right) \right] \cdot H$$

centroid for calculation of total bottom moment if D/H > 1.33

$$X_{imf} := \text{if}\left[\frac{D}{H} < 1.333, \left(0.50 + 0.06 \cdot \frac{D}{H}\right) \cdot H, X_{imf} \right]$$

centroid for calculation of total bottom moment if D/H < 1.33

$$X_{imf} = 27.788\,\text{ft}$$

Compute convective water weight and effective centroid above the base

$$W_c := W_T \cdot \left(.230 \cdot \frac{D}{H} \cdot \tanh\left(3.67 \cdot \frac{H}{D}\right) \right) \qquad W_c = 3642.43 \cdot \text{kip} \qquad \frac{W_c}{W_T} = 0.453$$

Ref 4, Eq 13-26

$$X_c := H \cdot \left[1 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1}{3.67 \cdot \left(\frac{H}{D}\right) \cdot \sinh\left(3.67 \frac{H}{D}\right)} \right] \qquad X_c = 20.043\,\text{ft}$$

centroid for calculation of just the shell moment

$$X_{cmf} := H \cdot \left(1.0 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1.937}{3.67 \cdot \frac{H}{D} \cdot \sinh\left(3.67 \cdot \frac{H}{D}\right)} \right) \qquad X_{cmf} = 26.405\,\text{ft}$$

centroid for calculation of total bottom moment

Seismic Design Criteria

Importance Factor: $I_E := 1.50$ Risk category IV

Ground Motion Parameters

Site Class D Site Class based on soils report for proposed adjacent reservoir

$S_S := .940$ $S_1 := .367$ Mapped earthquake short period and long period spectral accelerations. For Site Class B, 5% damping, expressed as fraction of g.

$F_a := 1.12$ $F_v := 1.67$ Site coefficients from 2012 IBC Table 1613.3.3(2). Seismic Design Category "D"

Adjusted maximum considered earthquake for site class

$S_{MS} := F_a \cdot S_S$ $S_{MS} = 1.053$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.613$

Design spectral response parameters

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$ $S_{DS} = 0.702$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$ $S_{D1} = 0.409$

Compute points on the design response spectrum

$T_0 := 0.2 \cdot sec \cdot \frac{S_{D1}}{S_{DS}}$ $T_0 = 0.116 \cdot sec$

$T_S := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot sec$ $T_S = 0.582 \cdot sec$

$T_L := 6 \cdot sec$ Mapped value, ASCE 7-10, Figure 22-12

$T_L := \text{if}\left(T_L > 4.sec, 4.sec, T_L\right) = 4 \cdot sec$ Maximum required for tank sloshing wave calculations, ASCE 7-10, Section 15.7.6.1.d

$S_{ac}(T) := \text{if}\left(T > T_L, \frac{1.5 \cdot S_{D1} \cdot T_L \cdot sec}{T^2}, \min\left(\frac{1.5 \cdot S_{D1} \cdot sec}{T}, 1.5 \cdot S_{DS}\right)\right)$ Convective acceleration function

$S_{ac}(T) := \text{if}\left(S_{ac}(T) > 1.5S_{DS}, 1.5S_{DS}, S_{ac}(T)\right)$ Upper bound for S_{ac} for low values of T

$$S_{ai}(T) := \text{if}\left(T > T_L, \frac{S_{D1} \cdot T_L \cdot \text{sec}}{T^2}, \text{if}\left(T > T_S, \frac{S_{D1}}{T} \cdot \text{sec}, S_{DS}\right)\right)$$

Impulsive acceleration function

Calculate Free Surface Wave Height and Compare to Freeboard Requirements

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{D}{3.68\cdot g\cdot \tanh\left(3.68\cdot\frac{H}{D}\right)}} \qquad T_c = 4.976\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \text{if}\left(SUG = 3, \text{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\,2}}\right), \text{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\,2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.099$

$d := 0.5\cdot D\cdot A_f = 3.465\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude.

freeboard $f := H_s - H = 1.5\,ft$

$\frac{d}{f} = 2.31$ > 1.0, therefore **freeboard is insufficient**

Compute Base Shear and Overturning Moments As If Free Surface

$S_{ai} := S_{DS}$ $R_i := 2.5$ $R_c := 1.5$ AWWA D100-11, Table 28 and section 13.2.9.2. Unnchored tank

$$A_i := \max\left(\frac{S_{ai} \cdot I_E}{1.4 \cdot R_i}, \frac{0.36 \cdot S_1 \cdot I_E}{R_i}\right) \quad A_i = 0.301$$ Impulsive design acceleration

$$A_c := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c} \quad A_c = 0.071$$ Convective design acceleration

Calculate overturning moment at the base of the shell

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_i\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2} \qquad M_s = 18225 \cdot kip \cdot ft$$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_{imf}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2} \qquad M_{mf} = 37401 \cdot kip \cdot ft$$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_i\right]\right]^2 + \left(A_c \cdot W_c\right)^2} \qquad V_f = 1364.6 \cdot kip$$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Adjust Effective Masses for Roof Contact

The methodology for roof contact effects is an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave.

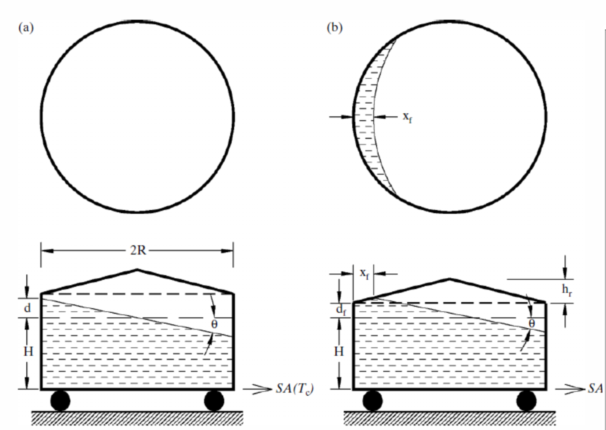
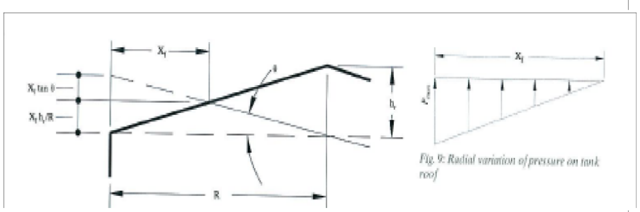


Fig. 5: Liquid-filled tank translating with an acceleration $SA(T_c)$: (a) sufficient freeboa and (b) insufficient freeboard



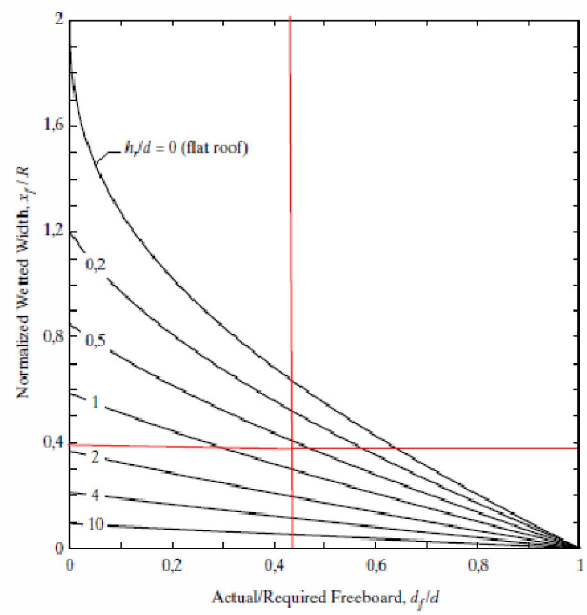


Fig. 6: Cone roof tank. Normalized wetted width of tank roof x_f/R as a function of actual/required freeboard d_f/d and normalized roof height h_r/d

Compute the angle θ

$$\theta := \operatorname{atan}\left(\frac{I_E \cdot S_{ac}(T_c) \cdot \frac{ft}{sec^2}}{g}\right) = 0.264 \cdot deg$$

Where $S_{ac}(T_c) = 0.099$ $I_E = 1.5$ $g = 32.174 \frac{ft}{s^2}$

$d_f := H_s - H = 1.5\,ft$ $d = 3.465\,ft$ $\frac{d_f}{d} = 0.433$ Compute input variables for graph above

$h_r = 2.188\,ft$ $\frac{h_r}{d} = 0.631$

From graph figure 6

$x_f := .39 \cdot R = 13.65\,ft$ horizontal extent of wetted dome surface from the shell $\frac{x_f}{R} = 0.39$ << 1.0 OK

$\rho := \frac{\gamma_{water}}{g} = 62.4 \cdot \frac{lbm}{ft^3}$ unit mass of water

$F_{max} := \frac{\rho}{2} \cdot g \cdot x_f^{\,2} \cdot \frac{(d + h_r)}{R}$ $\quad F_{max} = 939 \cdot \frac{lbf}{ft}$

Maximum uplift on shell due to hydrodynamic pressure caused by sloshing. Impact effects are considered minor and ignored

adjust mass for recalculation of seismic demand

$$\bar{m}_i = \begin{cases} m_i + m_c \cdot \left(1 - \dfrac{d_f + h_r/3}{d}\right) & \text{for } d_f + h_r/3 < d \\ m_i & \text{for } d_f + h_r/3 \geq d \end{cases}$$

$W_i = 4214 \cdot kip$

$W_T = 8045 \cdot kip$

$\frac{\left(d_f + \frac{h_r}{3}\right)}{d} = 0.643 \qquad W_{bar\_i} := W_i + W_c \cdot \left(1 - \frac{d_f + \frac{h_r}{3}}{d}\right) = 5513 \cdot kip$

$W_{bar\_i} := \text{if}\left[\frac{\left(d_f + \frac{h_r}{3}\right)}{d} < 1, W_{bar\_i}, W_i\right] = 5513 \cdot kip$

$$\bar{m}_c = m_l - \bar{m}_i$$

$W_c = 3642.4 \cdot kip \qquad W_{bar\_c} := W_T - W_{bar\_i} = 2531.8 \cdot kip$

$\frac{W_{bar\_i}}{W_i} = 1.308 \qquad \frac{W_{bar\_c}}{W_c} = 0.695$

<u>Factors by which mass must be multiplied due to the slosh contact with the roof</u>

Recaclulate convective period using adjusted mass. Maintain asssumption of T = 0 for impulsive mass

$$\bar{T}_i = T_i \cdot \sqrt{\frac{\bar{m}_i}{m_i}}$$

$$\bar{T}_c = T_c \cdot \sqrt{\frac{\bar{m}_c}{m_c}}$$

$T_c = 4.976\,s$ original convective period

$T_{c\_bar} := T_c \cdot \sqrt{\frac{W_{bar\_c}}{W_c}} = 4.149\,s$ modified convective period

$S_{ac}(T_c) = 0.099 \qquad A_c = 0.071$ original convective seismic factor

$S_{ac}(T_{c\_bar}) = 0.142 \qquad A_{c\_bar} := A_c \cdot \frac{S_{ac}(T_{c\_bar})}{S_{ac}(T_c)} = 0.102$ revised convective seismic factor

<u>Recompute base shear and overturning moment</u>

Change formula weights to adjusted values

$M_s = 18225 \cdot kip \cdot ft$ original overturning moment

$$M_{s\_rev} := \sqrt{\left[A_i \cdot \left[W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + \left(W_{bar\_i}\right) \cdot X_i\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_c\right)^2}$$

$M_{s\_rev} = 22976 \cdot kip \cdot ft$ revised moment

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$M_{mf} = 37401 \cdot kip \cdot ft$ original overturning moment

$$M_{mf\_rev} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{bar\_i} \cdot X_{imf}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_{cmf}\right)^2}$$

$M_{mf\_rev} = 48121 \cdot kip \cdot ft$ revised moment

Calculate base shear at top of foundation

$V_f = 1364.6 \cdot kip$ original base shear

$$V_{f\_rev} := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{col\_base} + \frac{W_{col}}{2}\right) + W_{bar\_i}\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c}\right)^2}$$

$V_{f\_rev} = 1749.97 \cdot kip$ revised base shear

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Compute Shell Hoop Forces and Stresses

Impulsive and convective forces are distributed using Housner's distribution formulas

Define the following variables:

z	Height of a point above the tank floor
Y	Depth of a point below the water surface
n_I	Distributed hoop force, klf, due to impulsive load N_I
n_C	Distributed hoop force, klf, due to convective load N_C
n_V	Distributed hoop force, klf, due to vertical seismic force N_V
n_F	Distributed hoop force, klf, due to hydrostatic force at maximum normal operating level
n_{Fol}	Distributed hoop force, klf, due to hydrostatic force at overflow operating level

Define elevation, distribution, and force component functions

$Y(z) := H - z$ distance from MOL to z

Housner's distribution of impulsive load as a function of elevation above the base and, in the case of impulsive loads, depends on the ratio of D/H

For the case of D/H < 1.33 and Y(z) < 0.75 D (z > .75D, upper section)

$$Dist_{ia}(z) := \frac{\left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2}{\left[\int_{.75 \cdot D}^{H} \left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2 dz + \int_0^{.75 \cdot D} 0.5\, dz\right]}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H < 1.33 at lower elevations, the factor is a constant equal to

$$Dist_{ib}(z) := \frac{0.5}{\left[\int_{.75 \cdot D}^{H} \left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2 dz\right] + \int_0^{.75 \cdot D} 0.5\, dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H > 1.33

$$\mathrm{Dist}_{ic}(z) := \frac{\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)}{\int_{0\cdot ft}^{H}\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

$$\mathrm{Dist}_i(z) := \mathrm{if}\left[\left(\frac{D}{H}\right) \geq 1.333, \mathrm{Dist}_{ic}(z), \mathrm{if}\left(Y(z) < 0.75\cdot D, \mathrm{Dist}_{ia}(z), \mathrm{Dist}_{ib}(z)\right)\right]$$

select appropriate formula based on depth and diameter ratio

Housner's distribution of convective load as a function of elevation above the base

$$\mathrm{Dist}_c(z) := \frac{\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}}{\displaystyle\int_{0\cdot ft}^{H}\frac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}dz}$$

The above formula is the convective force per unit depth at elevation "z" expressed as a fraction of the total convective force.

$V_i := A_i\cdot W_{bar\_i}$ $\quad V_i = 1658.301\cdot kip$ — Total base shear component due to impulsive fluid load

$N_i(z) := \left(\frac{V_i}{2}\right)\cdot\mathrm{Dist}_i(z)$ — Shell hoop force due to impulsive fluid load

$V_c := A_{c\_bar}\cdot W_{bar\_c}$ $\quad V_c = 257.598\cdot kip$ — Total base shear component due to convective fluid load

$N_c(z) := \frac{V_c}{2}\cdot\mathrm{Dist}_c(z)$ — Shell hoop force due to convective fluid load

$N_h(z) := \gamma_{water}\cdot\left(\frac{D}{2}\right)\cdot Y(z)$ — Shell hoop force due to hydrostatic load with water at MOL

$A_v := 0.14\cdot S_{DS}$ $\quad A_v = 0.098$ — Vertical seismic factor

$$\sigma_{static}(z) := \frac{N_h(z)}{t_s(z)}$$

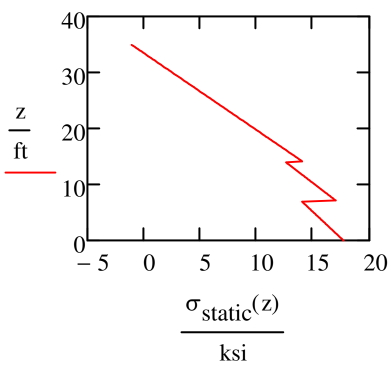
Hoop stress due to static fluid pressure at MOL

$$\sigma_s(z) := \frac{\sqrt{N_i(z)^2 + N_c(z)^2 + \left(N_h(z) \cdot A_v\right)^2}}{t_s(z)}$$

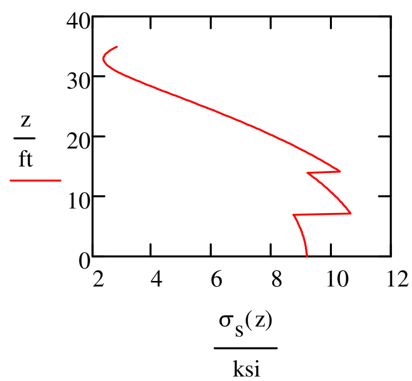
Hoop stress due to hydrodynamic pressure, Ref 4 Eq 13-42

$$\sigma_{total}(z) := \sigma_{static}(z) + \sigma_s(z)$$

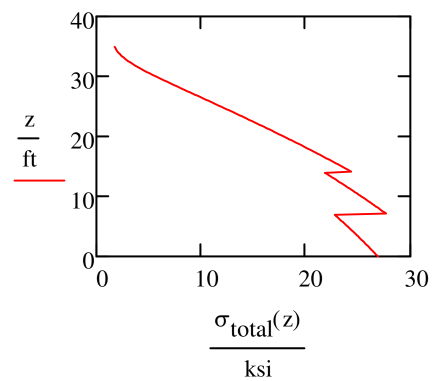
Combined static and seismic hoop stress at MOL



Hydrostatic Stress



Seismic Stress



Static + Seismic Stress

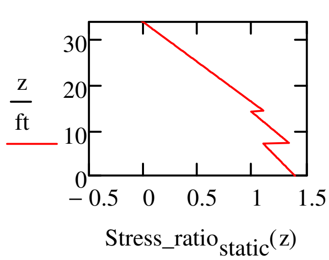
Note: the above plots are nominal based on treating each hoop course as acting independently. Actual stresses each side of girth joints are the same since strains are identical if the courses are attached, so the real stress near transition zones falls somewhere between the apparent discontinuous stress levels shown on the graphs. The actual maximum stress levels tend to occur about a foot above the joint and are not as high as predicted by the more simplified model. The simplified model is conservative and is the method reflected in the AWWA D-100 standard.

Check actual versus allowable stress based on the class of steel used.

Assumed joint efficiency and allowable stress $E_{joint} := 85\%$ $F_t(z) := E_{joint} \cdot 15 \cdot ksi$

Chapter 14 of AWWA D100-11 does not apply

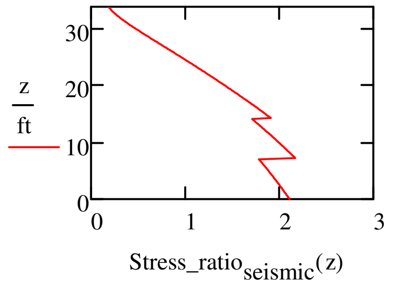
$$Stress\_ratio_{static}(z) := \frac{\sigma_{static}(z)}{F_t(z)}$$



Maximum static stress ratio is $Stress\_ratio_{static}(0) = 1.391$ > 1.0 NG

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{total}(z)}{F_t(z)}$$

The worst case stress ratio is at the bottom of the second shell course



> 1,33 NG

$$Stress\_ratio\_max_{seismic} := Stress\_ratio_{seismic}\left(z_{shell_1}\right) = 2.181$$

The worst case of overstress is at the bottom of the second shell course, but overstress occurs in all three of the lowest courses

Compute Shell Longitudinal Forces and Stresses

Define axial compressive force in the shell due to dead load for $0 < z < H_s$, in klf.

$$P_D(z) := \frac{W_r}{\pi \cdot D} + \int_z^{H_s} \gamma_{steel} \cdot t_s(z)\,dz$$

Define overturning moment functions at elevation z, in kip-ft

$$M_{rs}(z) := A_i \cdot \left[W_r \cdot (X_r - z) + W_{snow} \cdot X_{snow} + \pi \cdot \gamma_{steel} \cdot D \cdot \int_z^{H} y \cdot t_s(y)\,dy \right]$$

Moment associated with roof, snow and shell mass

$$M_i(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_i(y)\,dy$$

Moment associated with impulsive fluid mass, z < H

$$M_c(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_c(y)\,dy$$

Moment associated with convective fluid mass, z < H

$$M_s(z) := M_{rs}(z) + M_i(z) + M_c(z)$$

Total moment at elevation z on the shell for z < H

Define functions for compressive stress under static or seismic load conditions

$$\sigma_{static}(z) := \frac{P_D(z) + P_{snow}}{t_s(z)}$$

$$\sigma_{comp}(z) := \frac{(1 + 0.4 \cdot A_v)(P_D(z) + P_{snow}) - F_{max} + \frac{4\,M_s(z)}{\pi \cdot D^2}}{t_s(z)}$$

Includes deduction for roof uplift, $F_{max.}$

Check allowable stress for compression with local buckling and slenderness considered

Use Method 1. Yield stress of shell plate does not permit use of Method 2.

Local buckling stress formulas for Class 1 Materials

$$F_{L1a}(z) := \left[17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R} \right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R} \right)^2 \right] \cdot psi \right]$$

For Class 1 materials with 0 < t/R < t/Rc = .0031088, elastic buckling

$$F_{L1b}(z) := 5775 \cdot psi + 738 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$$

For Class 1 materials with t/Rc = .0031088 < t/R < 0.0125, inelastic buckling

$$F_{L1c}(z) := 15 \cdot ksi$$

For Class 1 materials with t/R > 0.0125, plastic buckling

Local buckling stress formulas for Class 2 Materials

$$F_{L2a}(z) := \min\left[15 \cdot ksi, 17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R}\right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R}\right)^2\right] \cdot psi\right]$$

For Class 2 materials with 0 < t/R < t/Rc = .0035372, elastic buckling

$$F_{L2b}(z) := 6925 \cdot psi + 886 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$$

For Class 2 materials with t/Rc = .0035372 < t/R < 0.0125, inelastic buckling

$$F_{L2c}(z) := 18 \cdot ksi$$

For Class 1 materials with t/R > 0.0125, plastic buckling

Write equation selection functions for F_L depending on t/R ratio and class

$$ratio1 := .0031088 \quad ratio2 := .0035372$$

$$F_{L1}(z) := \min\left(\mathrm{if}\left(\frac{t_s(z)}{R} < ratio1, F_{L1a}(z), \mathrm{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L1b}(z), F_{L1c}(z)\right)\right), 15 \cdot ksi\right)$$

$$F_{L2}(z) := \min\left(\mathrm{if}\left(\frac{t_s(z)}{R} < ratio2, F_{L2a}(z), \mathrm{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L2b}(z), F_{L2c}(z)\right)\right), 18 \cdot ksi\right)$$

$$F_L(z) := \mathrm{if}\left(class(z) = 1, F_{L1}(z), F_{L2}(z)\right)$$

Slenderness reduction factor equations

$r := \frac{D \cdot \sqrt{2}}{4}$ radius of gyration of tank shell

$K := 1.0$ effective column length factor, pinned ends assumed

$E := 29 \cdot 10^6 \cdot psi$ modulus of elasticity for steel

Slenderness ratio at which overall elastic column buckling can occur (not local buckling)

$$C'_c(z) := \sqrt{\pi^2 \cdot \frac{E}{F_L(z)}} \qquad L := H_s$$

$K_{\varphi 1}(z) := 1 - \frac{1}{2}\cdot\left(\frac{\frac{K\cdot L}{r}}{C'_c(z)}\right)^2$ For 25 < KL/r < C'c

$K_{\varphi 2}(z) := \frac{1}{2}\cdot\left(\frac{C'_c(z)}{\frac{K\cdot L}{r}}\right)^2$ For KL/r > C'c

$K_{\varphi 3}(z) := 1.0$ For KL/r < 25

$ratio := K\cdot\frac{L}{r}$ $ratio = 1.414$

$K_{\varphi}(z) := if\left(ratio < 25, K_{\varphi 3}(z), if\left(ratio > C'_c(z), K_{\varphi 2}(z), K_{\varphi 1}(z)\right)\right)$

$F_a(z) := F_L(z)\cdot K_{\varphi}(z)$ allowable compressive stress due to axial load

However, for unanchored tanks the allowable stress is permitted to be increased by accounting for the stability provided by hydrostatic pressure

Write a function for hydrostatic pressure for 0 < z < H $P(z) := \gamma_{water}\cdot Y(z)$ $E = 2.9\times 10^4\cdot ksi$

$$\Delta C_c(z) := if\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2 \le .064, .072\cdot\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2\right]^{0.84}, .045\cdot\ln\left[\frac{P(z)}{E}\cdot\left(\frac{R}{t_s(z)}\right)^2 + .0018\right] + .194\right]$$

$\Delta C_c(z) := \min\left(\Delta C_c(z), 0.22\right)$ See AWWA D100 Eq 13-50 and 13-51

$\Delta\sigma_{cr}(z) := \frac{\left(\Delta C_c(z)\cdot E\cdot t_s(z)\right)}{R}$ $\Delta\sigma_{cr}(0) = 4.296\cdot ksi$ Eq 13-49

$\sigma_a(z) := F_a(z)$

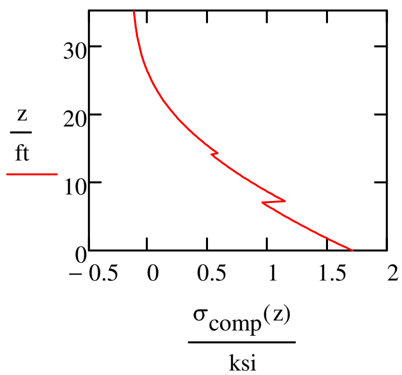
$\sigma_e(z) := 1.33\cdot\left(\sigma_a(z) + \frac{\Delta\sigma_{cr}(z)}{2}\right)$ Eq 13-47

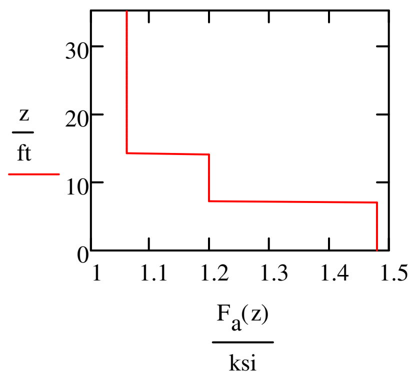
$Stress\_ratio_{seismic}(z) := \frac{\sigma_{comp}(z)}{\sigma_e(z)}$

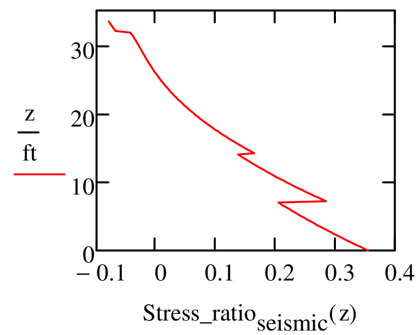
Plot static plus seismic compressive stress and compare to allowables

$$\mathrm{Stress\_ratio}_{\mathrm{seismic}}(z) := \frac{\sigma_{\mathrm{comp}}(z)}{\sigma_e(z)}$$

Plot static plus seismic compressive stress and compare to allowables





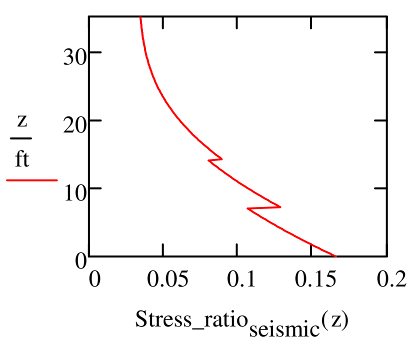


$\mathrm{Stress\_ratio}_{\mathrm{seismic}}(0) = 0.355$ < 1.0 OK. This number is not 1.33 because the 1.33 multiplier is already included in the Fa calculation

Check seismic longitudinal tensile stress

$$\sigma_{\mathrm{tens}}(z) := \frac{(1 - .40\cdot A_v)P_D(z) + F_{\max} + \frac{4\,M_s(z)}{\pi\cdot D^2}}{t_s(z)} \qquad \mathrm{Stress\_ratio}_{\mathrm{seismic}}(z) := \frac{\sigma_{\mathrm{tens}}(z)}{F_t(z)}$$

$\mathrm{Stress\_ratio}_{\mathrm{seismic}}(0) = 0.166$



All stress ratios << 1.333 are **OK for static plus seismic stress in longitudinal tension**

Horizontal Shear Transfer Capacity

The previously calculated base shear is $V_f = 1365 \cdot kip$

From AWWA D100-11 Eq 13-57, the allowable resistance attributable to friction is (for the full tank, seismic condition)

$$V_{ALLOW} := \tan(30 \cdot deg) \cdot (W_s + W_r + W_T + W_f) \cdot (1 - A_v) = 4289 \cdot kip$$

>> V_f OK. No shear connection between the superstructure and base is required for shear. Shear resistance is provided by the bottom plate acting as a diaphragm kept in place by bottom friction. Check shell to bottom transfer capacity

The maximum shell to bottom plate shear load is $v := 2 \cdot \frac{V_f}{\pi \cdot D} = 12.41 \cdot klf$ $\quad \frac{V_f}{V_{ALLOW}} = 0.318$

There is no annular plate, just the 5/16" floor plate

$$t_f := \frac{5}{16} \cdot in$$

And the maximum shear stress on the plate is $\tau := \frac{v}{t_f} = 3 \cdot ksi$ $\quad \frac{\tau}{12 \cdot ksi} = 0.276$

AWWA D100 permits 12 ksi in shear, and this can be increased by 1.33 for seismic, so **floor plate should not tear in shear parallel to the floor plate**

Check Foundation

No record drawings exist giving the dimensions of the foundation. The foundation provides no resistance to uplift since it is unanchored.

Calculate Foundation Dead Weight

$\gamma_{conc} := 150 \cdot pcf$

$h_{ftg} := 40 \cdot in$ height and depth measurements assumed from field dimensions at discrete locations

$b_{ftg} := 30 \cdot in$

$R_{ftg} := R + 6 \cdot in = 35.5\,ft$ $\quad R_{in} := R_{ftg} - b_{ftg}$ footing outside and inside radii

$A_{ftg} := \pi \cdot \left(R_{ftg}^{\,2} - R_{in}^{\,2}\right) = 537.998\,ft^2$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 269.0 \cdot kip$ $\quad w_{ftg} := \dfrac{W_{ftg}}{\pi \cdot D} = 1.223 \cdot klf$ total and unit footing weight

$W_{water} := H \cdot \gamma_{water} \cdot \pi \cdot \left(R^2 - R_{in}^{\,2}\right) = 893.1 \cdot kip$ $\quad w_{water} := \dfrac{W_{water}}{\pi \cdot D} = 4.061 \cdot klf$ total and unit weight of water over footing

$\gamma_{soil} := 125 \cdot pcf$ typical weight of compacted soil

$A_{soil} := 0$ area of soil over footing

$A_{wedge} := \dfrac{\left(h_{ftg} - 8 \cdot in\right)^2}{2 \cdot 2} = 1.778\,ft^2$ area of soil resisting uplift in friction at 1H:2V, backfill to within 8" of top of footing. Skin friction assumed 0.4 between footing and soil

$w_{soil} := \gamma_{soil} \cdot \left(A_{soil} + 0.4A_{wedge}\right)$ $\quad w_{soil} = 0.1 \cdot klf$ unit soil resistance

$W_s = 86.43 \cdot kip$ $\quad w_{shell} := \dfrac{W_s}{\pi \cdot D} = 0.393 \cdot klf$ shell weight

$W_{roof\_edge} = 41.28 \cdot kip$ $\quad w_{roof\_edge} := \dfrac{W_{roof\_edge}}{\pi \cdot D} = 0.188 \cdot klf$ roof edge weight

Compute overturning safety factor for pivoting about the toe of the shell

$M_{s\_rev} = 22976 \cdot kip \cdot ft$

$$SF_{overturning} := (1 - A_v) \cdot (W_{roof\_edge} + W_s + W_{ftg} + W_{water}) \cdot \frac{R}{M_{s\_rev}} = 1.772$$ OK

Required safety factor based on ASCE 7 load combos is .7E/.6D where .7E is the earthquake load in allowable stress terms, an effective ratio of 1.67

Check ratio of resistance to uplift at the foundation

$$SF_{uplift} := \frac{\left[(1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water}) + w_{soil} - F_{max}\right]}{4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2}} = 0.744$$

< 1.0 so there will be some foundation uplift

Check bearing pressure

$$\sigma_{comp}(0) = 1.712 \times 10^3\ psi$$

$$w_{static} := w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water} = 5.865 \cdot klf$$

$$q_{bearing\_static} := \frac{w_{static}}{b_{ftg}} = 2.346 \cdot ksf$$

$$w_{seismic} := (1 + A_v) \cdot (w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water}) + F_{max} + 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 13.351 \cdot klf$$

$$q_{bearing\_seismic} := \frac{w_{seismic}}{b_{ftg}} = 5.34 \cdot ksf$$

$q_{allow} := 2.5 \cdot ksf$ Static allowable bearing pressure

$$\frac{q_{bearing\_static}}{q_{allow}} = 0.938$$ OK

$$\frac{q_{bearing\_seismic}}{q_{allow}} = 2.136$$ > 1.33 NG

Check As Self-Anchored Tank

Per AWWA D100 section 13.5.4.1

$w_t := P_D(0) = 658 \cdot \frac{lbf}{ft}$ Weight of shell and roof supported by shell

$t_b := t_{floor} = 0.313 \cdot in$ $F_y := 27 \cdot ksi$ $G := 1.0$ A283 Grade B steel assumed

$w_L := \min\left(1.28 \cdot \frac{H}{ft} \cdot \frac{D}{ft} \cdot G, 7.29 \cdot \frac{t_b}{in} \sqrt{\frac{F_y}{ksi} \cdot \frac{H}{ft} \cdot G}\right) \cdot plf = 69 \cdot \frac{lbf}{ft}$ Eq 13-37, normalized for units

Overturning ratio

$J := \frac{M_s(0)}{D^2 \cdot \left[w_t \cdot \left(1 - 0.4 \cdot A_v\right) + w_L\right]} = 8.013$ **>> 1.54 therefore the tank is not stable without anchorage**

References

1. 2012 *International Building Code*
2. Washington State Adoption of and Amendments to 2012 International Building Code (State Building Code)
3. ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
4. AWWA Standard D100-11 *Welded Carbon Steel Tanks for Water Storage*
5. Nuclear Reactors and Earthquakes, Chap. 6 and Appendix F. U.S. Nuclear Regulatory Commission publication, Division of Technical Information, TID-7024, National Technical Information Service (1963).
6. Not used
7. Not used
8. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" Praveen K. Malhotra, Structural Engineering International, March 2006
9. Not used
10. "Dynamic Pressures on Accelerated Fluid Containers," G.W. Housner, 1955, Bulletin of the Seismological Society of America.
11. "Snow Load Analysis for Washington, 2nd Ed." Structural Engineers Association of Washington,1995
12. Not used
13. Not used
14. ACI 318-11 Building Code Requirements for Structural Concrete
15. ANSI/AISC 360-10 Specification for Structural Steel Buildings
16. AWS D1.1 Structural Welding Code - Steel

Units and Mathcad Notation

All calculations are shown in U.S. customary units. Calculations have been performed using MathSoft's Mathcad Version 14.0 software, which automatically checks for unit consistency and applies any necessary unit conversion factors internally to the program. Where computations are imported from Excel, SAP2000, or other software, the source is identified. Input values are shaded. Others are computed.

Where equations are shown with a ":=" sign, the left hand side of the equation is being defined by the right hand side. Where equations are shown with a "=" sign, the current value of the expression on the left hand side is being displayed.

=	An ordinary "equals" sign indicates the value being shown is for the most current evaluation of the variable on the left hand side of the equation
:=	An "equals" sign with a colon indicates the value on the left hand side is being defined by the expression on the right. Variables may be redefined, the last definition taking precedence
=	A bold "equals" sign indicates the symbol is being used in a logical expression
if(a,b,c)	An "if" statement is evaluated as "b" if "a" is true, and as "c' is "a' is false. These expressions may be nested
(matrix$_{i,j}$)	In matrix expressions, the first subscript is the row, and the second is the column. Numbering starts with the value indicated as "ORIGIN" for the first row and column unless otherwise noted
submatrix (A,i1,i2,j1,j2)	Defines a vector or submatrix of matrix "A" from row i1 thru i2, and column j1 thru j2
-----------> ( )	An expression with a vector arrow over it indicates that the expression involves subscripted variables, and that the expression is being evaluated for each subscript in the range
| | | |	A bold vertical line to the left of a series of expressions indicates that they are acting as a programming loop in the calculations
ORIGIN := 1	Sets initial subscript value for subscripted variables
M<j>	The vector in column "j" of matrix "M"
$sf := ft^2$	
$\Phi(x)$	Step function. Returns -1 for x < 0, +1 for x > 0 and .5 if x = 0

BHC CONSULTANTS

Seismic Evaluation
for

Division 7 Reservoir - Retrofit Option A

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

These calculations are preliminary in nature for design approach analysis and are not to be used for construction

Incorporate calculations from existing tank analysis by reference.

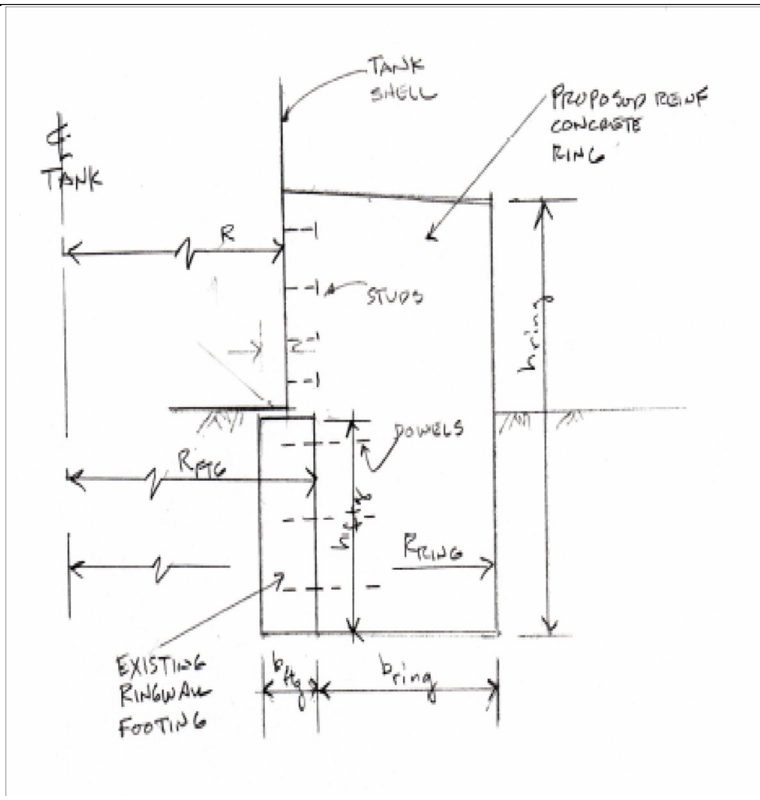
Reference:S:\Projects\Lake Whatcom W&S District\Reservoir Seismic VA 2015\Structural Calculations\Division 7\Division 7 Reservoi

$cy := yd^3$

Existing footing

Existing ringwall and tank dimensions

$b_{ftg} = 2.5\,ft$ $h_{ftg} = 3.333\,ft$ $R_{ftg} = 35.5\,ft$ outside radius, ex. ftg.



$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi \cdot \left(R_{ftg}^{\,2} - R_{in}^{\,2}\right)$ footprint

Additional exterior ring

$h_{ring} := 7 \cdot ft$ Ring depth

$b_{ring} := 3 \cdot ft$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 38.5\,ft$

$A_{gross} := \pi \cdot R_{ring}^{\,2} = 4657\,ft^2$

$A_{ring} := A_{gross} - \pi \cdot R_{ftg}^{\,2}$

Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\,dz\,dr\,d\phi\right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring}-h_{ftg}} r\,dz\,dr\,d\phi\right) = 101.142 \cdot cy$$

$W_{ring} := V_{ring} \cdot \gamma_{conc}$ $W_{ring} = 410 \cdot kip$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 1863 \cdot plf$$

Anchor ring weight per ft of shell

Check overturning stability safety factor

$$SF_{overturning} := \left(1 - A_v\right) \cdot \left(W_{roof\_edge} + W_s + W_{ftg} + W_{water} + W_{ring}\right) \cdot \frac{R}{M_{s\_rev}} = 2.815$$ > 1.67 OK

Calculate the required shear transfer capacity between footing and new anchor ring per foot of shell

$$Uplift := 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 5.054 \cdot klf$$ Transfer force at face of shell

The resistance available along the perimeter is

$$Resistance := (1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water} + w_{ring}) + w_{soil} - F_{max} = 6.264 \cdot klf$$

Check resistance/uplift safety factor with added block

$$Resistance\_ratio := \frac{Resistance}{Uplift} = 1.239$$ > 1.0 OK

The load to be transfered by the shell to the new ring is $Stud\_load := Uplift = 5.054 \cdot klf$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Dowel\_load := (w_{water} + w_{ftg} + F_{max}) = 6.223 \cdot klf$$

$\Omega_o := 2.0$ From Ref 3, Table 15.4-2, for tanks the overstrength factor

Stud design

$s_{studs} := 32 \cdot in$ horizontal stud spacing

Try $s_{studs\_vert} := 20 \cdot in$

$$n_{studs\_per\_row} := \frac{(h_{ring} - h_{ftg})}{s_{studs\_vert}} = 2.2$$

Use at least 3 studs per row $n_{studs\_per\_row} := 3$ $s_{studs\_vert} := \frac{(h_{ring} - h_{ftg})}{n_{studs\_per\_row}} = 14.667 \cdot in$

$$Load\_per\_stud := s_{studs} \cdot \frac{Stud\_load}{n_{studs\_per\_row}} = 4492 \cdot lbf$$

$$V_u := \Omega_o \cdot 1.4 \cdot Load\_per\_stud = 12578\,lbf$$

Shear strength for a 5/8" Nelson stud is $Q_N := 15113 \cdot lbf$ per AISC for f'c=4.5 ksi, Fu=65 ksi

$\phi_{shear} := .90$ $\frac{V_u}{\phi_{shear} \cdot Q_N} = 0.925$ < 1.0 OK

$l_{stud} := 8 \cdot in$ $\quad d_{stud} := .625 \cdot in$

$f'_c := 4.5 \cdot ksi$ $\quad \dfrac{V_u}{l_{stud} \cdot d_{stud}} = 2.516 \cdot ksi$

$DCR := \dfrac{V_u}{.85 \cdot f'_c \, l_{stud} \cdot d_{stud}} = 0.658$ OK for crushing

Dowel Design

$s_{dowels} := 19 \cdot in$ horizontal stud spacing

$Dowel\_load = 6.223 \cdot klf$

$n_{dowels\_per\_row} := 3$

$s_{dowels\_vert} := \dfrac{h_{ftg}}{n_{dowels\_per\_row} + 1} = 0.833\,ft$

$Load\_per\_dowel := s_{dowels} \cdot \dfrac{Dowel\_load}{n_{dowels\_per\_row}} = 3285 \cdot lbf$

$V_u := \Omega_o \cdot 1.4 \cdot Load\_per\_dowel = 9197\,lbf$

for a #6 Grade 60 dowel, Hilti HIT-RE 500 adhesive in shear $\quad V_{sa} := 15840 \cdot lbf$

$DCR := \dfrac{V_u}{.60 \cdot V_{sa}} = 0.968$ < 1 OK

Quantities

$$N_{studs} := n_{studs\_per\_row} \cdot \pi \cdot \frac{D}{s_{studs}} = 247$$

$$N_{dowels} := n_{dowels\_per\_row} \cdot \pi \cdot \frac{D}{s_{dowels}} = 417$$

$$V_{conc} := \pi \cdot \left(R_{ring}{}^{2} - R_{ftg}{}^{2}\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}{}^{2} - R^{2}\right) \cdot \left(h_{ring} - h_{ftg}\right) = 101 \cdot cy$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^{2} - \pi \cdot R_{ftg}{}^{2} = 1194\,ft^{2}$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^{2} - \pi \cdot R_{ftg}{}^{2} = 2077\,ft^{2}$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^{2} - \pi \cdot R_{ftg}{}^{2} = 1627\,ft^{2}$$

$$R_{exc} := R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 8.333\,ft$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 402 \cdot cy$$

Backfill quantity

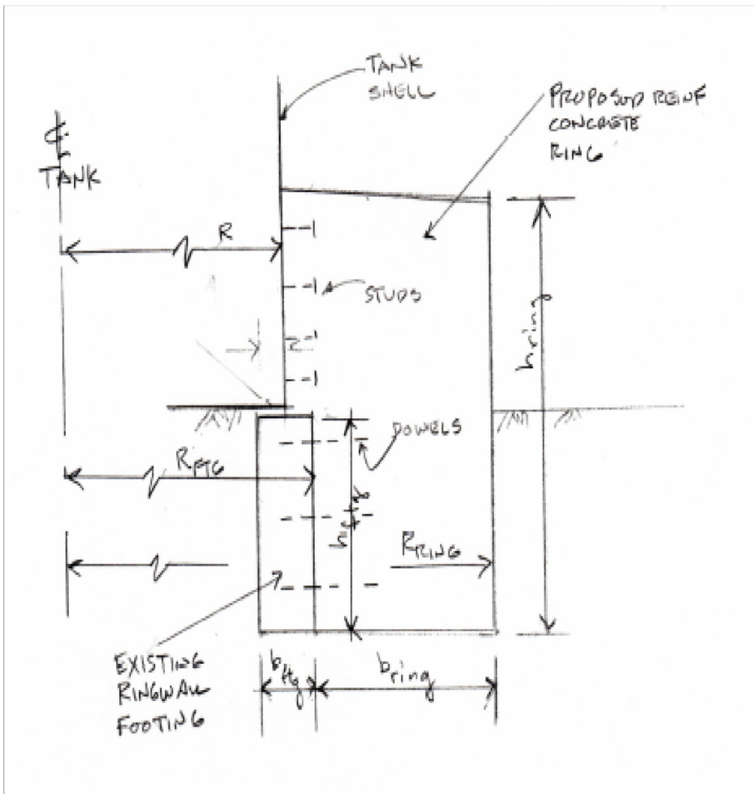
$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}{}^{2} - R_{ftg}{}^{2}\right) \cdot h_{ftg} = 316 \cdot cy$$

Shell wrap weight $20 \cdot ft \cdot \pi \cdot D \cdot 7.66 \cdot psf = 33690 \cdot lbf$

Seismic Evaluation
for
Division 7 Reservoir Option C

for

Lake Whatcom Water & Sewer District
Bellingham, Washington



Existing ringwall and tank dimensions

Existing footing

$R_{ftg} := 35.5\cdot ft$ outside radius, ex. ftg.

$b_{ftg} := b_{ringwall} = 2.5\,ft$

$h_{ftg} := h_{ringwall} = 3.333\,ft$

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$A_{ftg} := \pi\cdot\left(R_{ftg}^{\ 2} - R_{in}^{\ 2}\right)$ footprint

Additional exterior ring

$h_{ring} := h_{ftg}$ Ring depth

$b_{ring} := 30\cdot in$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 38\,ft$

$A_{gross} := \pi\cdot R_{ring}^{\ 2} = 4536\,ft^2$

$A_{ring} := A_{gross} - \pi\cdot R_{ftg}^{\ 2}$

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0)$ $P_{static} = 658\cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi)\cdot\frac{\left(4\cdot M_s(0)\right)}{\pi\cdot D^2}$$ Seismic load at base of shell from lateral ground motion

$P_{seismic}(0) = 7151\cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -7151\cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 26 \cdot plf$

c. Existing footing Dead Load Component

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 269 \cdot kip$ Total weight of existing ringwall

$w_{ftg} := \dfrac{W_{ftg}}{2 \cdot \pi \cdot R} = 1223 \cdot plf$ Ringwall weight per ft of shell

d. Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r \, dz \, dr \, d\phi \right) + \left(2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring} - h_{ftg}} r \, dz \, dr \, d\phi \right) = 71.268 \cdot cy$$ Ring volume

$W_{ring} := V_{ring} \cdot \gamma_{conc}$ $W_{ring} = 289 \cdot kip$

$w_{ring} := \dfrac{W_{ring}}{2 \cdot \pi \cdot R} = 1313 \cdot plf$ Anchor ring weight per ft of shell

e. Weight of water over footing

$p_{static} := \gamma_{water} \cdot H = 2090 \cdot psf$

$$w_{water} := p_{static} \cdot \frac{\pi \cdot \left(R^2 - R_{in}^{\ 2}\right)}{2 \cdot \pi \cdot R}$$

f. Seismic pressure increase/decrease on footing

$w_{water} = 4061 \cdot plf$

(base pressure functions hidden below for brevity)

$\Delta p := p_{base}(R, 0) = 653 \cdot psf$ Plus or minus water pressure at the toe or heel of the tank due to seismic effects

$$w_{seismic} := \int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R} p_{base}(r,\phi)\cdot\frac{r}{ft}\,dr\,d\phi$$

$$w_{seismic} = 191.826\cdot plf$$

Calculate the required anchor transfer capacity between tank and new anchor ring per foot of shell

$SF_{ot} := 1.67$ target safety factor

$Uplift := P_{seismic}(0)$ $Uplift = 7.151\cdot klf$ Transfer force at face of shell

The resistance of various components is

$$D_{tank\_resist} := P_{static}\cdot(1 - .4\cdot A_v) = 0.632\cdot klf$$

$$w_{water\_resist} := (1 - .4\cdot A_v)\cdot w_{water} - w_{seismic} = 3.71\cdot klf$$

Set number of anchors and compute load. Assume three new anchors between each of the 12 existing

$n_{anchors} := 40$ $s_{anchor} := \pi\cdot\frac{D}{n_{anchors}} = 5.498\,ft$

$$T_{anchor} := \frac{\left[\pi\cdot D\cdot\left(Uplift - D_{tank\_resist} - w_{water\_resist}\right)\right]}{n_{anchors}} = 15.442\cdot kip$$ measured at the shell

Resistance provided by ring $w_{ring} = 1.313\cdot klf$

Resistance required by ground anchors

$$Ground\_anchor\_resist := SF_{ot}\cdot(Uplift) - D_{tank\_resist} - w_{water\_resist} - w_{ring} = 6.288\cdot klf$$

$ground\_anchor\_capacity\_ASD := 75\cdot kip$

$n_{ground\_anchors} := \frac{n_{anchors}}{2} = 20$ provide one ground anchor for every two anchors

$$ground\_anchor\_load := Ground\_anchor\_resist \cdot \pi \cdot \frac{D}{n_{ground\_anchors}} = 69.136 \cdot kip$$

$$s_{ground\_anchor} := \pi \cdot \frac{D}{n_{ground\_anchors}} = 10.996\,ft$$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Ring\_dowels := \left(w_{water} + w_{ftg}\right) = 5285 \cdot plf$$

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$s_{dowels} := s_{anchor} = 5.498\,ft$ $n_{dowels\_per\_row} := 3$

$$Load\_per\_dowel := \frac{s_{dowels}}{s_{anchor}} \cdot \frac{T_{anchor}}{n_{dowels\_per\_row}} = 5147 \cdot lbf$$

Half inch dowels should be more than enough $n_{dowels} := n_{anchors} \cdot n_{dowels\_per\_row} = 120$

Quantities

$n_{dowels} = 120$ $n_{anchors} = 40$ $n_{ground\_anchors} = 20$

$$V_{conc} := \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}^{\ 2} - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 71 \cdot cy$$

By compariison to Sumner Springs reservoir, assume reinforcement at $steel\_unit := 210 \cdot \frac{lbf}{cy}$

$$rebar := V_{conc} \cdot steel\_unit = 14966\,lbf$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1067\,ft^2$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1940\,ft^2$$

$$R_{exc} := R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 7.833\,ft$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 1495\,ft^2$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 370 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} = 298.577 \cdot cy$$

Anchor Bolt Sizing

Assume A36 anchor bolts $F_y := 36 \cdot ksi$ $F_u := 58 \cdot ksi$

$F_{anchor} := \min(.80 \cdot 36 \cdot ksi, .50 \cdot 58 \cdot ksi) = 28.8 \cdot ksi$ Allowable seismic load stress on anchors per Ref 5 section 3.3.3.2

$A_{root\_min} := \frac{T_{anchor}}{F_{anchor}} = 0.536 \cdot in^2$ $d_{root\_calc} := \sqrt{\frac{4}{\pi} \cdot A_{root\_min}} = 0.826 \cdot in$

Per Ref 5, 3.8.5.1, add a .25" corrosion allowance to the root diameter for bolts less than 1.25", and use not less than a 1" bolt. This makes an 1.25" bolt the practical minimum

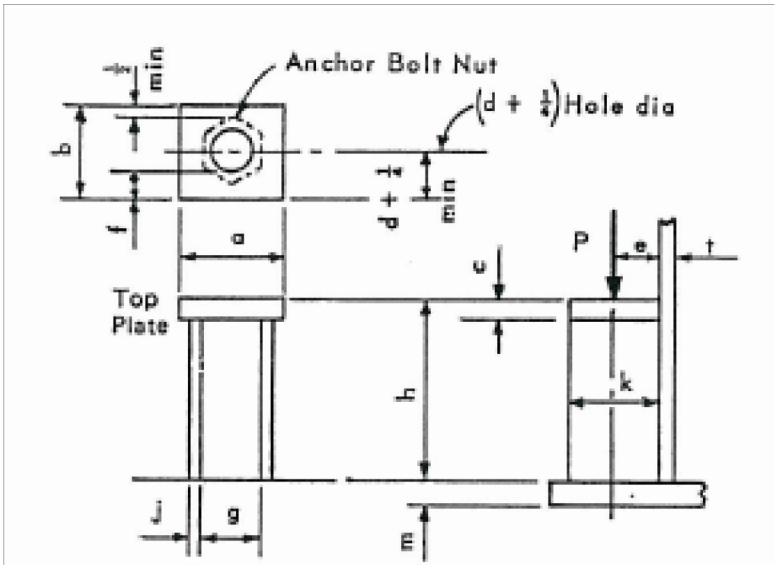
Bolt Dia (in)	Root Dia (in)	Root Area (in^2)	Gross Area (in^2)	Root Dia + .25" (in)	Min Bolt Dia (in)
1.000	0.865	0.587	0.785	1.115	1.375
1.125	0.970	0.74	0.994	1.220	1.500
1.250	1.100	0.942	1.23		1.250
1.375	1.190	1.12	1.49		1.375
1.500	1.320	1.37	1.77		1.500
1.750	1.530	1.85	2.41		1.750
2.000	1.760	2.43	3.14		2.000

Ref 10, Table 7-18

$d := 1.25 \cdot in$ anchor diameter $A_{bolt} := \pi \cdot \frac{d^2}{4} = 1.227 \cdot in^2$ gross area of bolt

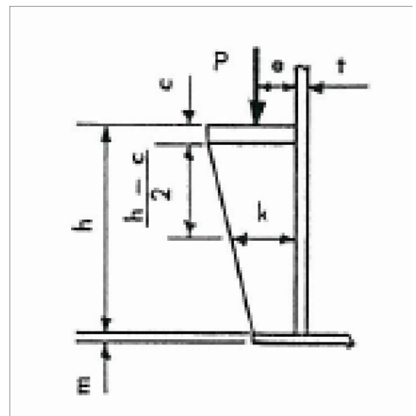
Anchor Chair Design

Methodology is from Ref 11, Part VII - Anchor Bolt Chairs



(a) Typical Plan & Outside Views

(b) Vertical Column or Skirt



(c) Flat Bottom Tank

$e := 18 \cdot in$ bolt centerline distance from shell

Minimum bolt hole size per Ref 11 is

Oversized hole size per Ref 18 Table J.3.3 is $d + \frac{5}{16} \cdot in = 1.563 \cdot in$ for bolts >= 1.25 in. Use

$d_{hole} := d + \frac{5}{16} \cdot in$ $d_{hole} = 1.563 \cdot in$

Edge distance per Ref 10 Tables J.3.4 and J3.5 (from center of hole) is

$c_{edge} := 2.25 \cdot in + \frac{1}{8} \cdot in = 2.375 \cdot in$

$b := e + c_{edge} = 20.375 \cdot in$

$f := c_{edge} - \frac{d_{hole}}{2} = 1.594 \cdot in$

$g := d + 1 \cdot in = 2.25 \cdot in$ minimum side plate separation recommended by Ref 21, however this is very tight for seal welding on interior of plates. Increase this dimension to

$g := 8 \cdot in$

$t := t_s(0)$ $t = 0.344 \cdot in$ Shell bottom course thickness

$P := T_{anchor} = 15.442 \cdot kip$

$S := 1.33 \cdot 15 \cdot ksi = 19.95 \cdot ksi$ Ref 4 allowable stress < 25 ksi recommended by Ref 11 OK

Compute top plate thickness

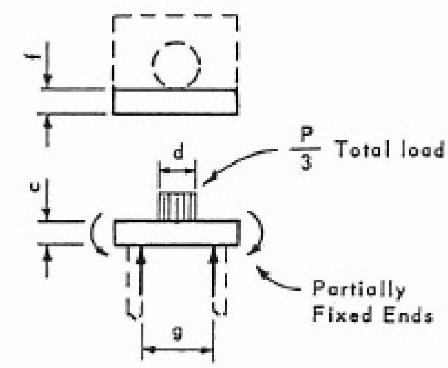


Figure 7-2. Assumed Top-Plate Beam.

$$c_{min} := \left[\frac{P}{S \cdot f} \cdot (0.37 \cdot g - 0.22 \cdot d)\right]^{.5} = 1.142 \cdot in$$

use $c := 1.5 \cdot in$

top plate thickness

$h := 30 \cdot in$

$j_{min} := \max[.5 \cdot in, 0.04 \cdot (h - c)] = 1.14 \cdot in$ use $j := 1in$

$m := .25 \cdot in$ bottom plate thickness assumption $proj := 2 \cdot in - t$ bottom plate projection from shell face

$a := g + 2 \cdot j + .5 \cdot in = 10.5 \cdot in$ $>$ $2 \cdot c_{edge} = 4.75 \cdot in$ OK   Use $a := 14 \cdot in$

Recess the side plate not more than 1/2" from front edge of top plate per Ref 21. Use .25" to allow seal weld at front edge.

$plate\_top := b - .25 \cdot in$ $k := \frac{(plate\_top + proj)}{2} = 10.891 \cdot in$ mean side plate width

$$\frac{j \cdot k}{\frac{P \cdot in^2}{25 \cdot kip}} = 17.631$$ > 1.0 OK per Ref 21

Compute reduction factor Z for local stress check

$$Z := \frac{1.0}{\frac{(.177 \cdot a \cdot m)}{in\sqrt{R \cdot t}} \cdot \left(\frac{m}{t}\right)^2 + 1.0} = 0.973$$

$$S := \frac{P \cdot e}{in\, t^2} \cdot \left[\frac{1.32 \cdot Z}{\frac{1.43 \cdot a \cdot h^2}{R \cdot t \cdot in} + \left(4 \cdot \frac{a}{in^3} \cdot h^2\right)^{.333}} + \frac{.031 \cdot in}{\sqrt{R \cdot t}} \right]$$

$S = 24.773 \cdot ksi$

localized vertical shell stress just above the chair. Ref 21 recommends 25ksi max.

Weld Design

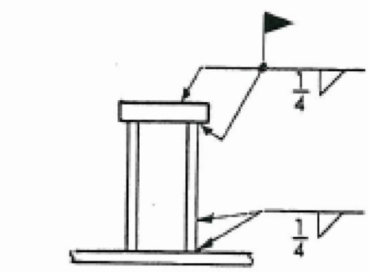


Figure 7-4. Typical Welding, Base Plate Shop Attached.

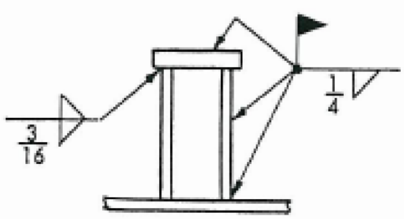


Figure 7-5. Typical Welding, Base or Bottom Field Attached.

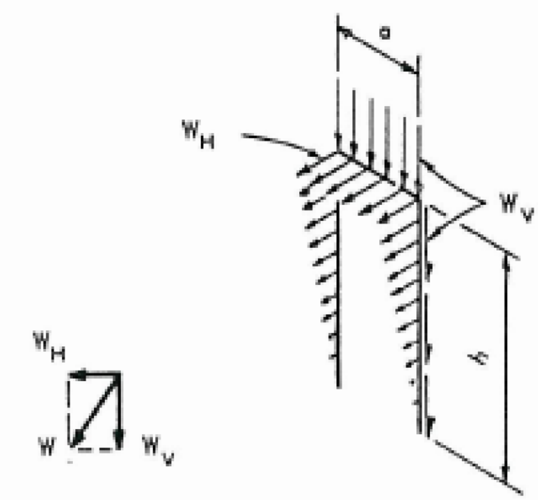


Figure 7-6. Loads on Welds.

$$W_v := \frac{P}{a + 2 \cdot h} = 209 \cdot \frac{lbf}{in}$$

$$W_h := \frac{P \cdot e}{a \cdot h + 0.667 \cdot h^2} = 272 \cdot \frac{lbf}{in}$$

$$W := \sqrt{W_v^{\,2} + W_h^{\,2}} = 343 \cdot \frac{lbf}{in}$$

By inspection, a .25" weld will be more than adequate.

Shell shear capacity per inch exceeds weld, OK

Anchor Quantities

$V_{bp} := a \cdot b \cdot c$

$V_{bp} = 427.875 \cdot in^3$

$$V_{sp} := 2 \cdot \frac{(b + 2 \cdot in) \cdot (h - c) \cdot j}{2} \qquad V_{sp} = 637.688 \cdot in^3$$

$$W_{anchor} := \gamma_{steel} \cdot \left(V_{bp} + V_{sp}\right) = 302.156\,lbf$$

$$W_{anchor\_total} := W_{anchor} \cdot n_{anchors} = 12086\,lbf$$

$$L_{weld} := 2 \cdot h + a + (a - g - 2 \cdot j) = 78 \cdot in$$

$$L_{weld\_total} := n_{anchors} \cdot L_{weld} = 3120 \cdot in$$

CONSULTANTS

Seismic Evaluation

for

Division 7 Reservoir Option D

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Compute mat weight and location of center of gravity above the base

$h_{mat} := 2.67 \cdot ft$ Mat thickness $BCL_{exist} := 0$ $cy := 27 \cdot ft^3$

Existing Bottom Capacity Level (elevation of base of tank)

$BCL := BCL_{exist} + h_{mat}$ $BCL = 2.67\,ft$ Bottom Capacity Level (water elevation at top of mat)

$MOL := H$ Assumed maximum operating level

$TCL := 655.5 \cdot ft$ Top Capacity Level (elevation at lip of overflow)

$D = 70\,ft$ Shell diameter

$A_{tank} := \pi \cdot \frac{D^2}{4}$ $A_{tank} = 3848\,ft^2$ Tank footprint

$V_{mat} := A_{tank} \cdot (BCL - BCL_{exist})$ $V_{mat} = 380.6 \cdot cy$

$\gamma_{conc} := 150 \cdot pcf$ Unit weight of concrete

$W_{mat} := V_{mat} \cdot \gamma_{conc}$ $W_{mat} = 1541 \cdot kip$ $X_{mat} := \frac{h_{mat}}{2}$ $X_{mat} = 1.335\,ft$

Compute existing floor plate weight

$Floor\_flange := 2 \cdot in$ Bottom plate projection beyond shell plate

$D_{plate} := D + 2 \cdot Floor\_flange$ $D_{plate} = 70.333\,ft$

$t_{plate} := .25 \cdot in$ $W_f := \gamma_{steel} \cdot t_{plate} \cdot \pi \cdot \frac{D_{plate}^2}{4}$ $W_f = 40 \cdot kip$

Compute weight of assumed steel plate installed above mat to seal the bottom

$t_{seal} := .25 \cdot in$ $W_{seal} := \gamma_{steel} \cdot t_{seal} \cdot \pi \cdot \frac{D^2}{4}$ $W_{seal} = 39 \cdot kip$ $X_{seal} := h_{mat}$

Hydrodynamic Wall Pressure Functions

$MOL := H \qquad BCL_{exist} := 0$

$z_f := z$

$z(z_f) := z_f - h_{mat}$

$z(z_f) := \text{if}\left[\left[z_f > (MOL - BCL_{exist})\right], H, z(z_f)\right]$ z cannot be greater than H when calculating water effects

Define fluid pressure functions

Hydrodynamic pressures due to impulsive and convective lateral loads vary around the shell as a function of the angle from the toe of the tank, ϕ. (See Ref 5)

The pressure distribution for impulsive forces is proportional to the function

$$\Psi_i(\phi) := \cos(\phi)$$

The pressure distribution for convective forces is proportional to the function

$$\Psi_c(\phi) := \cos(\phi)\cdot\left(1 - \frac{1}{3}\cdot\cos(\phi)^2\right)$$

Half of the impulsive and convective base shear, taken at the top of the mat, is represented by the region where $-\pi/2 < \phi < \pi/2$

$$\frac{V_i}{2} = 829.151\cdot kip \qquad \frac{V_c}{2} = 128.799\cdot kip$$

The maximum impulsive pressure distribution is

$$p_i(z_f) := \left(\frac{V_i}{2\cdot R}\right)\cdot\left(\frac{1}{\displaystyle\int_{\frac{-\pi}{2}}^{\frac{\pi}{2}} \Psi_i(\phi)\cdot\cos(\phi)\, d\phi}\right)\cdot Dist_i(z(z_f))$$

The maximum convective pressure distribution is

$$p_c(z_f) := \left(\frac{V_c}{2\cdot R}\right)\cdot\left(\frac{1}{\displaystyle\int_{\frac{-\pi}{2}}^{\frac{\pi}{2}} \Psi_c(\phi)\cdot\cos(\phi)\, d\phi}\right)\cdot Dist_c(z(z_f))$$

The static and vertical hydrodynamic wall pressures are

$$p_{static}(z_f) := \gamma_{water}\cdot Y(z(z_f))$$

$$p_z(z_f) := A_v \cdot p_{static}(z_f)$$

Set pressures equal to zero unless $h_{mat} < z_f < H + h_{mat}$

$$p_i(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_i(z_f)\right]\right]$$

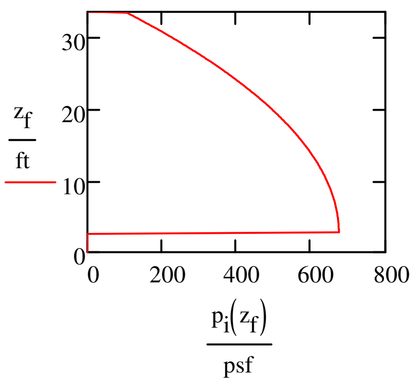
$$p_c(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_c(z_f)\right]\right]$$

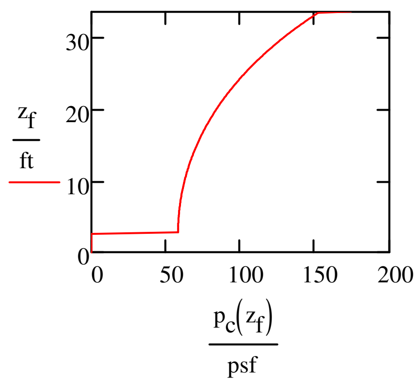
$$p_{static}(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_{static}(z_f)\right]\right]$$

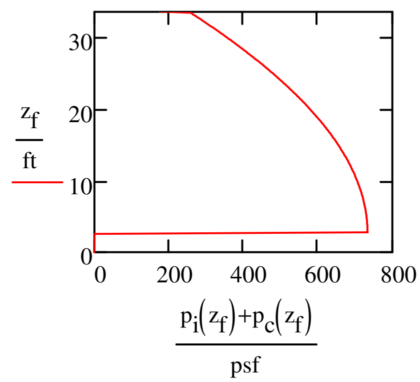
$$p_z(z_f) := \text{if}\left[z_f < h_{mat}, 0, \text{if}\left[z_f > (H + h_{mat}), 0, p_z(z_f)\right]\right]$$

$$p_i(5 \cdot ft) = 672.026 \cdot psf$$

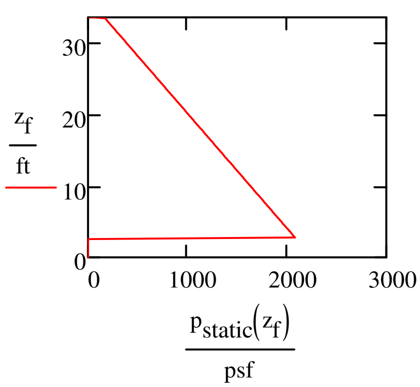
The maximum hydrodynamic impulsive, convective, and combined wall pressures are graphed below vs z_f at $\phi = 0$

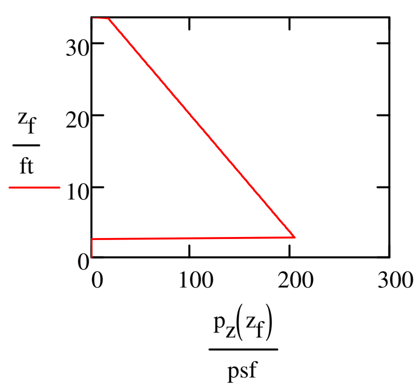






The static and vertical seismic wall pressures are graphed below for all φ_f





Hydrodynamic pressures are added (or subtracted) from hydrostatic pressure to obtain net water fluid pressures, along with the vertical seismic pressure (+ or -). Use the slightly higher straight addition values for the impulsive and convective components so the sign of the pressure will be correct when integrating over the mat surface. When using direct sum instead of SRSS (square root of the sum of the squares) Ref 4 allows the vertical acceleration component to be taken as .40Av. (See Ref 4 section 13.5.4.3)

The base pressure varies in a more complicated way and is computed in the following section

Calculate Loads to Foundation

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0) \quad P_{static} = 658 \cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi) \cdot \frac{(4 \cdot M_s(0))}{\pi \cdot D^2}$$

Seismic load at base of shell from lateral ground motion

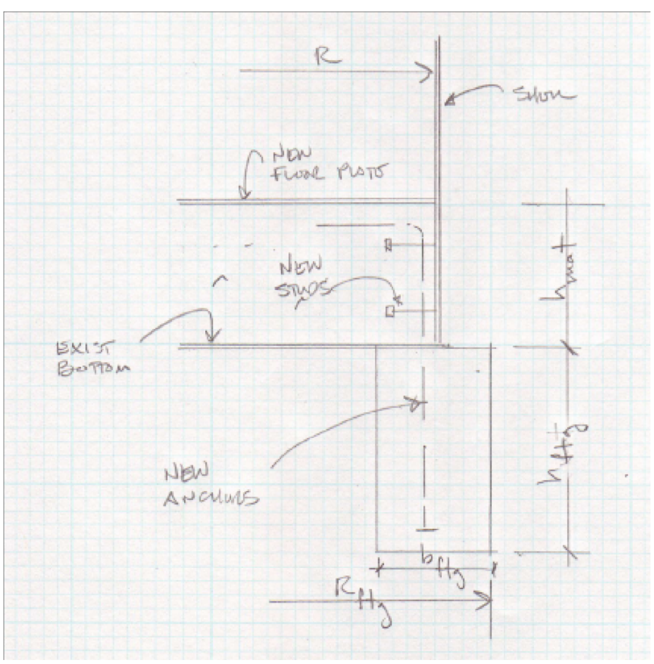
$P_{seismic}(0) = 7151 \cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -7151 \cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40 \cdot A_v \cdot P_{static}$ Seismic load at base of shell from vertical ground motion

$P_{seismic\_v} = 26 \cdot plf$

c. Ringwall Dead Load Component



$R_{ftg} := 51.5 \cdot ft$ from as-built topo

$b_{ftg} := 2 \cdot ft$ from impact-echo measurement

$h_{ftg} := 4 \cdot ft$ field measurement

$R_{in} := R_{ftg} - b_{ftg}$ footing inside radius

$$A_{ftg} := \pi \cdot \left(R_{ftg}^2 - R_{in}^2\right)$$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} \quad W_{ftg} = 380.761 \cdot kip$

Total weight of existing ringwall

$$w_{ftg} := \frac{W_{ftg}}{2 \cdot \pi \cdot R} = 1731 \cdot plf$$

Ringwall weight per ft of shell

Calculate the radial centroid for the ringwall area

$\theta_1 := \frac{1 \cdot ft}{R}$ tank angle subtended by one ft of shell length

$A_{ringwall} := \int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r \, dr \, d\theta$ $A_{ringwall} = 2.886\,ft^2$ ringwall footprint per foot of shell

$r_{ringwall} := \frac{\int_{-\frac{\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R_{ftg}} r^2 \, dr \, d\theta}{A_{ringwall}}$ $r_{ringwall} = 50.507\,ft$ Radial distance to ringwall center of gravity

d. Mat and New Floor Plate Unit Weight

$w_{mat} := \frac{(W_{mat} + W_{seal})}{\pi \cdot R^2}$ $w_{mat} = 411 \cdot psf$

The required safety factor is not stated directly in the design standards Ref 1 and Ref 3, nor for anchored tanks in Ref 4. It may be inferred from Ref 3 section 12.14.8.4 and the load combinations in Ref 3 section 2.4.

Safety factor >= 0.75 (from 12.14.8.4) * .98 (0.7 earthquake load factor x 1.4 scale up factor to convert Ref 4 earthquake loads to Ref 3 basis) / 0.6 (dead load factor, Ref 3 equation 8, section 3.2.4.1) = 1.23

e. Check Sliding Safety Factor

$V_f = 1750 \cdot kip$ Base shear at base of mat

Weight of soil confined by ringwall $A_{soil} := \pi \cdot R_{in}^2$ $\gamma_{soil} := 125 \cdot pcf$ $W_{soil} := \gamma_{soil} \cdot A_{soil} \cdot h_{ftg}$

Ratio of base shear to total dead weight at the plane defined by the base of the footing

$V_{allow} := \tan(30 \cdot deg) \cdot (W_s + W_r + W_i + W_c + W_f + W_{mat} + W_{seal} + W_{ftg} + W_{soil}) \cdot (1 - .40 \cdot A_v)$

$V_{allow} = 7682 \cdot kip$ Ref 4 Eq 13-57

$SF_{sliding} := \frac{V_{allow}}{V_f}$ $SF_{sliding} = 4.39$ > 1.0 OK for sliding

f. Check Overturning Safety Factor about the Base of the Mat

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

Calculate overturning moment at the base of the mat

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2}$$ Ref 4 Eq 13-23

$M_s = 18367 \cdot kip \cdot ft$ $M_{ssave} := M_s$ placeholder for later calculation

$$M_{ssum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_i + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_c$$

$M_{shell} := M_{ssum}$ placeholder for later calculation

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2}$$ Ref 4 Eq 13-32

$M_{mf} = 37546 \cdot kip \cdot ft$ Result using SRSS method

Results using straight sum method (more conservative)

$$M_{mfsum} := A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_i \cdot X_{imf} + W_{mat} \cdot X_{mat} + W_{seal} \cdot X_{seal}\right) + A_c \cdot W_c \cdot X_{cmf}$$

$M_{mfsum} = 43726 \cdot kip \cdot ft$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left(W_s + W_r + W_f + W_i + W_{mat} + W_{seal}\right)\right]^2 + \left(A_c \cdot W_c\right)^2}$$

Ref 4 Eq 13-31

$V_f = 1817 \cdot kip$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

$M_{mfsum} = 43726 \cdot kip \cdot ft$ Total overturning moment about the base of the mat, including base pressure effects

$$W_{resist} := \left(1 - .40 \cdot A_v\right) \cdot \left(W_s + W_r + W_i + W_c + W_{mat} + W_{seal} + W_{ftg}\right) \qquad W_{resist} = 9571 \cdot kip$$

$$M_{res} := W_{resist} \cdot R = 334971 \cdot kip \cdot ft$$

$$SF_{ot} := \frac{M_{res}}{M_{mfsum}} \qquad SF_{ot} = 7.661$$

Global safety factor against overturning without regard to uplift, soil pressure, or concrete capacity

g. Check Pressure at Base of Mat Floor Plate - Static - Rigid Mat Assumption

$$q_{static} := \frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2} + \left(H - h_{mat}\right) \cdot \gamma_{water} \qquad q_{static} = 4572 \cdot psf$$

Weight of structure and water at emergency operating level applied uniformly to the mat.

h. Check Soil Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Down

$$q1_{max} := \left(1 + .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q1_{max} = 4051 \cdot psf$$

$$q1_{min} := \left(1 + .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] - \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q1_{min} = 1454 \cdot psf$$

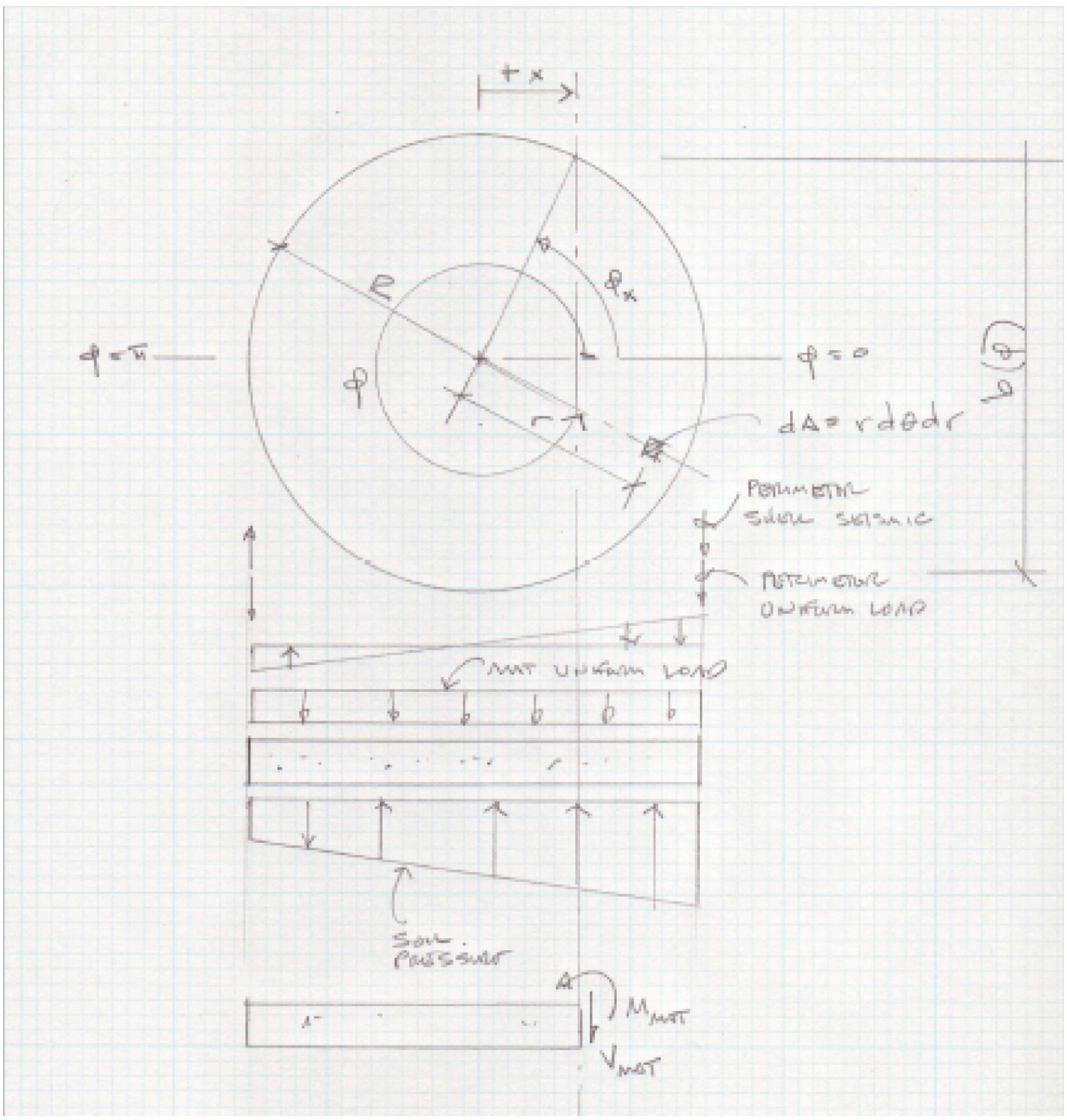
i. Check Pressure at Base of Mat - Dynamic - Rigid Mat - Vertical Seismic Acting Up

$$q2_{max} := \left(1 - .40 \cdot A_v\right) \cdot \left[\frac{\left(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg}\right)}{\pi \cdot R^2}\right] + \frac{4M_{mfsum}}{\pi \cdot R^3} \qquad q2_{max} = 3842 \cdot psf$$

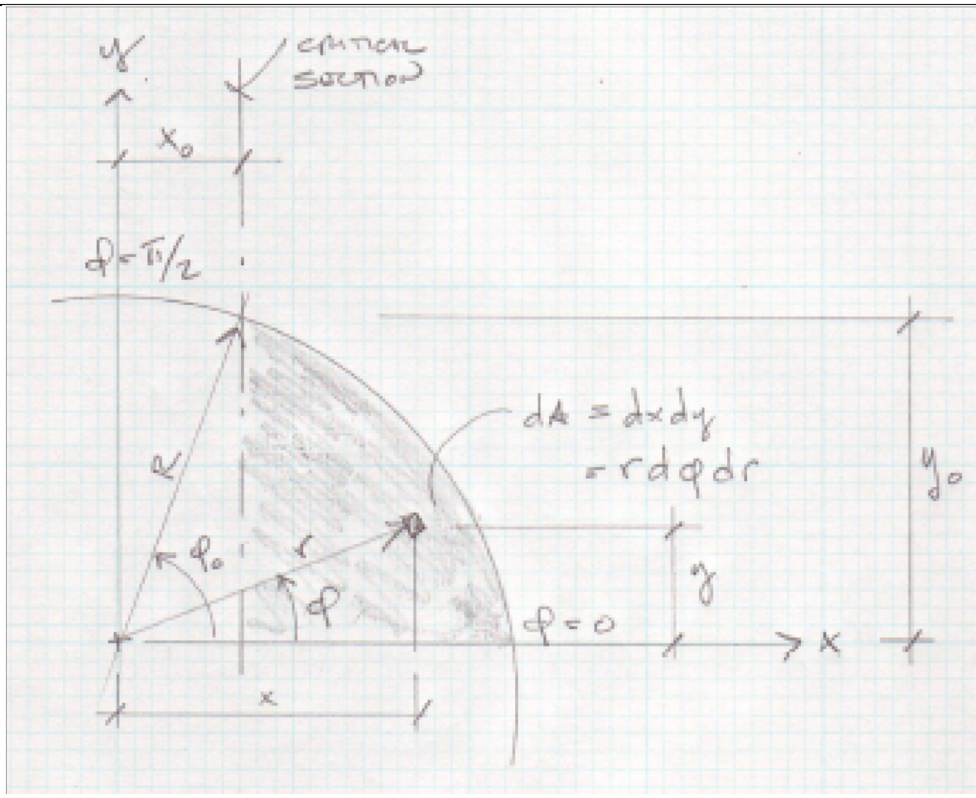
$$q2_{min} := (1 - .40 \cdot A_v) \cdot \left[\frac{(W_s + W_r + W_T + W_{mat} + W_{seal} + W_f + W_{ftg})}{\pi \cdot R^2} \right] - \frac{4M_{mfsum}}{\pi \cdot R^3}$$

$q2_{min} = 1245 \cdot psf$

i. Compute the mat shear and moment under seismic load



(1) First define some basic geometric relationships for the range   0 < φ < π



$x(r,\varphi) := r\cdot\cos(\varphi)$ $y(r,\varphi) := r\cdot\sin(\varphi)$ x,y coordinates as functions of polar coordinates r,φ

$r(x,y) := \sqrt{x^2 + y^2}$ $\varphi(x,y) := \text{angle}(x,y)$ polar coordinates as funtions of x,y coordinates

$\varphi_o(x_o) := \text{acos}\left(\frac{x_o}{R}\right)$ $y_o(x_o) := R\cdot\sin(\varphi_o(x_o))$ coordinates of x_o intercept with shell

$x_p(\varphi) := x(R,\varphi)$ $y_p(\varphi) := y(R,\varphi)$ Coordinates of the shell perimeter vs angle from toe

$y_R(x_R) := \sqrt{R^2 - x_R^{\,2}}$ $y'_R(x_R) := \frac{d}{dx_R}y_R(x_R)$ Equation for the shell perimeter and its derivative

$L(y) := \sqrt{R^2 - y^2}$

<u>(2) Define functions for soil pressure and for associated mat shear and moment</u>

Write soil pressure functions vs x (soil pressure must be greater than zero at all locations)

$q1_{av} := \frac{(q1_{max} + q1_{min})}{2}$ $q1(x) := q1_{av} + \left(\frac{x}{R}\right)\cdot(q1_{max} - q1_{av})$

$q2_{av} := \frac{(q2_{max} + q2_{min})}{2}$ $q2(x) := q2_{av} + \left(\frac{x}{R}\right)\cdot(q2_{max} - q2_{av})$ Case of vertical seismic loads up

Write functions for shear and moment due to soil pressure at section cut xo due to total soil reaction to the right of the cut

$$Vq1(x_o) := 2\cdot\int_{x_o}^{R} q1(x)\sqrt{R^2 - x^2}\,dx \qquad Mq1(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q1(x)\sqrt{R^2 - x^2}\,dx$$

$$Vq2(x_o) := 2\cdot\int_{x_o}^{R} q2(x)\sqrt{R^2 - x^2}\,dx \qquad Mq2(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot q2(x)\sqrt{R^2 - x^2}\,dx$$

(3) Define functions for mat shear and moment due to hydrostatic load and mat, floor, and seal plate loads

$$w_{unif} := \frac{(W_T + W_{mat} + W_{seal} + W_f)}{\pi\cdot R^2} \qquad w_{unif} = 2511\cdot psf$$

uniform load acting down on interior

$$V_{unif}(x_o) := -2\cdot\int_{x_o}^{R} w_{unif}\sqrt{R^2 - x^2}\,dx \qquad M_{unif}(x_o) := -2\cdot\int_{x_o}^{R} (x - x_o)\cdot w_{unif}\sqrt{R^2 - x^2}\,dx$$

(4) Define functions for mat shear and moment due to hydrodynamic base pressure (excluding Av effects)

Total moment due to impulsive and convective effects

$$\Delta M_{imp} := A_i\cdot W_i\cdot(X_{imf} - X_i) = 19298\cdot kip\cdot ft$$

$$\Delta M_{conv} := A_c\cdot W_c\cdot(X_{cmf} - X_c) = 1639\cdot kip\cdot ft$$

The impulsive base pressure varies as

$$\frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

From Ref 5, Equation F80

Integration constant for impulsive base pressure is

$$Const_{imp} := \frac{\Delta M_{imp}}{2\displaystyle\int_{-R}^{R}\int_{0}^{y_o(x)} \frac{x\cdot\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}\,dy\,dx}$$

$$Const_{imp} = 647\cdot psf$$

And the pressure function can be written as

$$p_{base\_i}(x,y) := Const_{imp} \cdot \frac{\sinh\left(\sqrt{3}\cdot\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\cdot\frac{L(y)}{H}\right)}$$

The convective base pressure varies as $\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3$ From Ref 5, Equation F108

Integration constant for convective base pressure is

$$Const_{conv} := \frac{\Delta M_{conv}}{2\int_{-R}^{R}\int_{0}^{y_o(x)} x\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right] dy\, dx}$$

$$Const_{conv} = 58\cdot psf$$

And the pressure function can be written as

$$p_{base\_c}(x,y) := Const_{conv}\cdot\left[\left(\frac{x}{R}\right) - \frac{1}{3}\cdot\left(\frac{x}{R}\right)^3\right]$$

The combined base pressure associated with convective and impulsive effects is

$$p_{base}(x,y) := p_{base\_i}(x,y) + p_{base\_c}(x,y)$$

$p_{base}(R,0) = 653\cdot psf$ Maximum pressure at toe

As a check, compare maximum bottom pressure if an approximate linear distribution of base pressure is assumed by dividing the total moment by the section modulus of the foundation footprint

$$p_{toe\_linear} := 4\cdot\frac{\left(\Delta M_{imp} + \Delta M_{conv}\right)}{\pi\cdot R^3} \qquad p_{toe\_linear} = 622\cdot psf \qquad \frac{p_{toe\_linear}}{p_{base}(R,0)} = 0.953 \quad \text{OK}$$

$$V_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} p_{base}(x,y)\, dy\, dx$$

$$M_{BP}(x_o) := -2\cdot\int_{x_o}^{R}\int_{0}^{y_o(x)} (x - x_o)p_{base}(x,y)\, dy\, dx$$

(5) Define functions for mat shear and moment due to Av only (up or down, not including loads at shell)

$$V_{Av1}(x_o) := -2\cdot\int_{x_o}^{R} .4\cdot A_v w_{unif}\sqrt{R^2 - x^2}\,dx \qquad M_{Av1}(x_o) := -2\cdot\int_{x_o}^{R} (x - x_o)\cdot .4\cdot A_v w_{unif}\sqrt{R^2 - x^2}\,dx$$

$$V_{Av2}(x_o) := 2\cdot\int_{x_o}^{R} .4\cdot A_v w_{unif}\sqrt{R^2 - x^2}\,dx \qquad M_{Av2}(x_o) := 2\cdot\int_{x_o}^{R} (x - x_o)\cdot .4\cdot A_v w_{unif}\sqrt{R^2 - x^2}\,dx$$

(6) Define functions for mat shear and moment due to roof shell and footing dead load applied at the perimeter

$$V_{shell\_static}(x_o) := -2\cdot\int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg})\cdot R\,d\varphi$$

$$M_{shell\_static}(x_o) := -2\cdot\int_{0}^{\varphi_o(x_o)} (P_{static} + w_{ftg})\cdot(R\cdot\cos(\varphi) - x_o)\cdot R\,d\varphi$$

(7) Define functions for mat shear and moment due to lateral seismic loads all applied at the perimeter

Write hydrodynamic force intensity at the shell as a function of φ

$$E_{shell}(\varphi) := \left(\frac{M_{shell}}{\pi\cdot R^2}\right)\cdot\cos(\varphi)$$

$$V_{E\_shell}(x_o) := -2\cdot\int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi)\cdot R\,d\varphi$$

$$M_{E\_shell}(x_o) := -2\cdot\int_{0}^{\varphi_o(x_o)} E_{shell}(\varphi)\cdot(R\cdot\cos(\varphi) - x_o)\cdot R\,d\varphi$$

(8) Define functions for mat shear and moment due to Av loads applied at the perimeter

$$V_{shell\_Av1}(x_o) := .40\cdot A_v\cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av1}(x_o) := .40\cdot A_v\cdot M_{shell\_static}(x_o)$$

$$V_{shell\_Av2}(x_o) := -.40\cdot A_v\cdot V_{shell\_static}(x_o) \qquad M_{shell\_Av2}(x_o) := -.40\cdot A_v\cdot M_{shell\_static}(x_o)$$

(9) Define functions for mat shear and moment due to center column force

$$P_{D\_ctr} := W_{roof\_center} + W_{col\_base} + W_{col} = 17.9 \cdot kip$$

$$V_{ctr}(x_o) := if(x_o > 0, 0, -P_{D\_ctr})$$

$$M_{ctr}(x_o) := if(x_o > 0, 0, x_o \cdot P_{D\_ctr})$$

(10) Define functions for total mat shear and moment due to combined loadins for the case of Av up or down

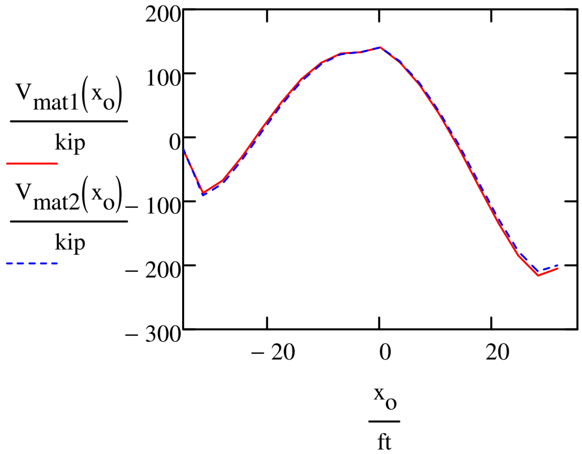
$$V_{mat1}(x_o) := Vq1(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av1}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av1}(x_o) + V_{ctr}(x_o) \cdot (1 + .40 \cdot A_v)$$

$$V_{mat2}(x_o) := Vq2(x_o) + V_{unif}(x_o) + V_{BP}(x_o) \ldots + V_{Av2}(x_o) + V_{shell\_static}(x_o) + V_{E\_shell}(x_o) + V_{shell\_Av2}(x_o) + V_{ctr}(x_o) \cdot (1 - .40A_v)$$

$$M_{mat1}(x_o) := Mq1(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av1}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av1}(x_o) + M_{ctr}(x_o) \cdot (1 + .40 \cdot A_v)$$

$$M_{mat2}(x_o) := Mq2(x_o) + M_{unif}(x_o) + M_{BP}(x_o) \ldots + M_{Av2}(x_o) + M_{shell\_static}(x_o) + M_{E\_shell}(x_o) + M_{shell\_Av2}(x_o) + M_{ctr}(x_o) \cdot (1 - .40 \cdot A_v)$$

$x_o := -R, -R + \frac{R}{10} .. R$ Set plot parameters



$V_{mat1}(R) = 0 \cdot kip$

$V_{mat2}(R) = 0 \cdot kip$

$V_{mat1}(-R) = -18.6 \cdot kip$

$V_{mat2}(-R) = -17.2 \cdot kip$ All values zero, check

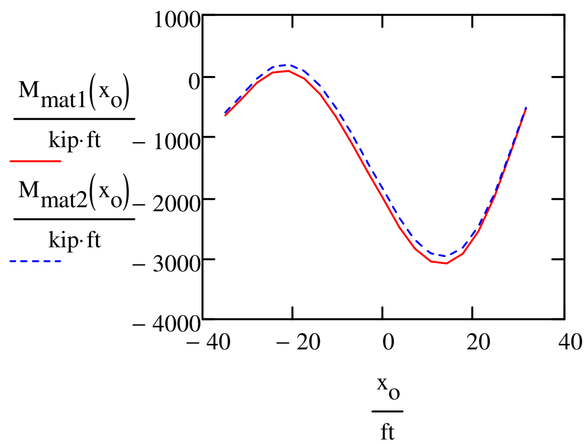
$M_{mat1}(R) = 0 \cdot kip \cdot ft$

$M_{mat2}(R) = 0 \cdot kip \cdot ft$

$M_{mat1}(-R) = -650 \cdot kip \cdot ft$

$M_{mat2}(-R) = -600.438 \cdot kip \cdot ft$

All values zero, check



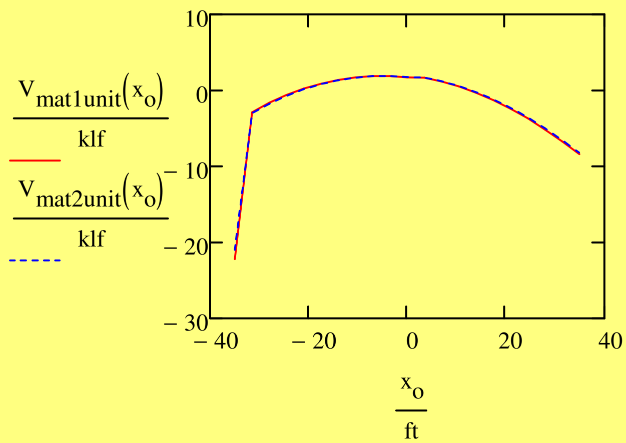
These forces are distributed over a variable mat width. Convert to average unit forces in the mat

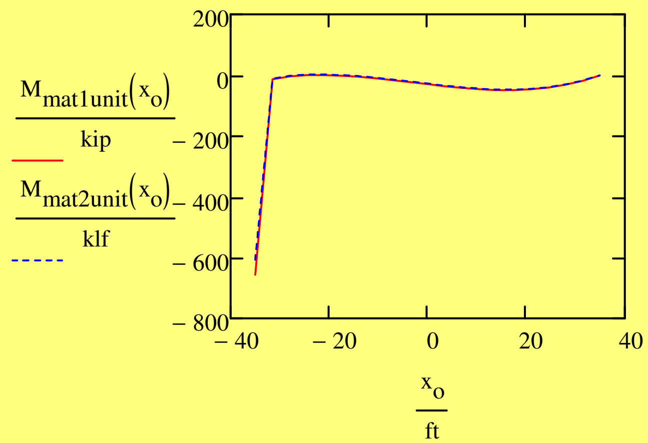
Note: These expressions cannot be evaluated at R or -R because the denominato is zero at the limits. Evaluate at values of x close to +/- R

$$V_{mat1unit}(x_o) := \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat1unit}(x_o) := \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$$V_{mat2unit}(x_o) := \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)} \qquad M_{mat2unit}(x_o) := \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$x_o := -.9999R, -R + \frac{R}{10} .. .9999R$ Plot parameters





Average unit shear and moment in the mat, ASD basis

Compute maxima and minimima

$x_o := 0$

Given

$$V_{mat1unit}(x_o) = \frac{V_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$V_{mat1unitmax} := V_{mat1unit}(\text{Maximize}(V_{mat1unit}, x_o)) = 2.016 \cdot klf$

$V_{mat1unitmin} := \min(V_{mat1unit}(-.9999R), V_{mat1unit}(.9999R)) = -22.183 \cdot klf$

$Vu_{mat1} := 1.4 \max(|V_{mat1unitmax}|, |V_{mat1unitmin}$ $Vu_{mat1} = 31.056 \cdot klf$

Given

$$V_{mat2unit}(x_o) = \frac{V_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$V_{mat2unitmax} := V_{mat2unit}(\text{Maximize}(V_{mat2unit}, x_o)) = 2.016 \cdot klf$

$V_{mat2unitmin} := \min(V_{mat2unit}(-.9999R), V_{mat2unit}(.9999R)) = -20.953 \cdot klf$

$Vu_{mat2} := 1.4 \max(|V_{mat2unitmax}|, |V_{mat2unitmin}$ $Vu_{mat2} = 29.335 \cdot klf$

$Vu_{mat} := \max(Vu_{mat1}, Vu_{mat2})$ $Vu_{mat} = 31.056 \cdot klf$

$x_o := \frac{-R}{2}$

Given

$$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat1unitmax} := M_{mat1unit}(\text{Maximize}(M_{mat1unit}, x_o))$ $\quad M_{mat1unitmax} = 1.71 \cdot kip$

$$x_o := \frac{R}{2}$$

Given

$$M_{mat1unit}(x_o) = \frac{M_{mat1}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat1unitmin} := M_{mat1unit}(\text{Minimize}(M_{mat1unit}, x_o))$ $\quad M_{mat1unitmin} = -48.406 \cdot kip$

$$x_o := \frac{-R}{2}$$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmax} := M_{mat2unit}(\text{Maximize}(M_{mat2unit}, x_o))$ $\quad M_{mat2unitmax} = 3.592 \cdot kip$

$$x_o := \frac{R}{2}$$

Given

$$M_{mat2unit}(x_o) = \frac{M_{mat2}(x_o)}{2 \cdot y_o(x_o)}$$

$M_{mat2unitmin} := M_{mat2unit}(\text{Minimize}(M_{mat2unit}, x_o))$ $\quad M_{mat2unitmin} = -46.676 \cdot kip$

$Mu_{mat\_pos} := 1.4 \max(M_{mat1unitmax}, M_{mat2unitmax}$ $\quad Mu_{mat\_pos} = 5.028 \cdot kip$

$Mu_{mat\_neg} := 1.4 \min(M_{mat1unitmin}, M_{mat2unitmin})$ $\quad Mu_{mat\_neg} = -67.768 \cdot kip$

Capacity Check and Preliminary Quantities

Material assumptions

$f'c := 4000 \cdot psi$ $\quad f_y := 60 \cdot ksi$ $\quad d := h_{mat} - 4 \cdot in$

Check shear capacity

$\varphi Vc := .75 \cdot 2 \cdot d \cdot \sqrt{\frac{f'c}{psi}} \cdot psi$ $\quad \varphi Vc = 31.921 \cdot klf$ $\quad \frac{Vu_{mat}}{\varphi Vc} = 0.973$ $\quad$ 1.0 OK

Compute approximate bottom steel requirement

$A_{s\_bot} := \frac{Mu_{mat\_pos}}{.90 \cdot .90 \cdot d \cdot f_y}$ $\quad A_{s\_bot} = 0.044 \cdot \frac{in^2}{ft}$ $\quad$ Computed steel requirement

$A_{s\_bot} := \text{if}\left[\left(\frac{A_{s\_bot}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_bot}, A_{s\_bot}\right]$ $\quad A_{s\_bot} = 0.059 \cdot \frac{in^2}{ft}$ $\quad$ Adjust steel requirement if computed steel ratio less than 200/fy

$A_{s\_top} := \frac{-Mu_{mat\_neg}}{.90 \cdot .90 \cdot d \cdot f_y}$ $\quad A_{s\_top} = 0.597 \cdot \frac{in^2}{ft}$ $\quad$ Computed steel requirement

$A_{s\_top} := \text{if}\left[\left(\frac{A_{s\_top}}{d}\right) < \left(200 \cdot \frac{psi}{f_y}\right), 1.333 \cdot A_{s\_top}, A_{s\_top}\right]$ $\quad A_{s\_top} = 0.795 \cdot \frac{in^2}{ft}$ $\quad$ Adjust steel requirement if computed steel ratio less than 200/fy

Reinforcement requirement per unit area of mat

$w_{reinf} := \gamma_{steel} \cdot 2 \cdot \left(A_{s\_bot} + A_{s\_top}\right)$ $\quad w_{reinf} = 5.815 \cdot psf$ $\quad \pi \cdot R^2 \cdot h_{mat} = 76865\,gal$

$W_{reinf} := w_{reinf} \cdot \pi \cdot R^2$ $\quad W_{reinf} = 22380\,lbf$ $\quad cy := 27 \cdot ft^3$

Concrete and seal steel quantities

$V_{conc} := h_{mat} \cdot \pi \cdot R^2$ $\quad V_{conc} = 380.569 \cdot cy$ $\quad W_{seal} = 39286\,lbf$

Placeholder unit costs for concrete and steel $\quad reinf\_cost := \frac{1}{lbf}$ $\quad conc\_cost := \frac{500}{cy}$ $\quad steel\_cost := \frac{2}{lbf}$

$Cost := W_{reinf} \cdot reinf\_cost + V_{conc} \cdot conc\_cost + W_{seal} \cdot steel\_cost$ $\quad Cost = 291237$

APPENDIX B.4

DIVISION 30 RESERVOIR CALCULATIONS

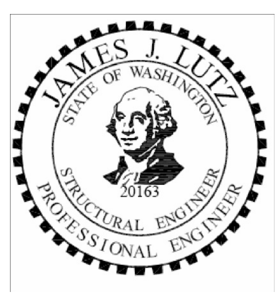
CONSULTANTS

Seismic Evaluation
for
Division 30 Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Methodology Remarks

These calculations are limited to an assessment of the primary elements of the lateral force resisting system for the reservoir under seismic loading. Following is a summary of the methodology used:

1. All dimensions and weights are based on record drawings furnished by the client, supplemented by field measurements.In case of discrepancies, field measurements were used..

2. Water level assumed for seismic calculations is based on maximum current operating level provided by the District..

3. Methodology for determination of seismic loads for tanks with a free water surface is based on the 2012 International Building Code, ASCE 7-10, and AWWA Standard D100-11. These codes and standards post-date and are more stringent than codes and standards used at the time of original tank design.

4. For tanks where the free surface sloshing wave amplitude exceeds the roof elevation, the additional amplification of seismic load is based on an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave. The force is modeled by computing an increase in mass and adjusting the convective period of the water mass. The pressure distribution is assumed the same as for a tank with a free water surface.

5. For tanks where the static water surface level already contacts the roof, the free surface sloshing amplitude is based on a cylinder of the same height and radius with zero freeboard, however the actual water mass is assumed. The ratio of sloshing amplitude to roof height is computed using roof height measured from the free water surface. Adjustments in seismic load are otherwise the same as for the preceding step.

6. Ground motion spectral accelerations S_S and S_1 are those currently available from the USGS on their web site calculator for the latitude and longitude of the tank as taken from Google Earth.

7. Soil site class "B" is assumed for this reservoir based on rock found at the base of test pits near the ringwall..

8. Wind loads, hydrostatic loads at overflow elevation, and roof live loads were not considered in the analysis. However where calculated roof loads exceed 40 psf, a mass equal to .20 times the uniform roof snow load is added to the roof mass for seismic calculations. The gravity effects of snow load were considered whete applicable for determining loads on the shell, however no analysis of roof members was included.

Location and Site Data

Lat 48.7028, Long -122.3333
El 1030
(Google Earth)

Superstructure Geometry

From record drawings

Tank diameter $D := 25.42 \cdot ft$

Tank radius $R := \frac{D}{2} = 12.71\,ft$

Shell height $H_s := 40.38 \cdot ft$

Floor elevation at shell
(Bottom capacity level)

$BCL := 1025.5 \cdot$ (District)

Overflow height above floor

$h_{overflow} := H_s - 6 \cdot in$

Overflow elevation
(Top capacity level)

$TCL := BCL + h_{overflow}$

$H := 39.3 \cdot ft$ Maximum operating level

$NOL := BCL + H = 1.065 \times 10^3\,ft$

$BCL + H_s = 1.066 \times 10^3\,ft$

This level is below the top of the shell.

Describe the roof geometry

This tank has a dome roof of constant radius. The measured slope distance from top of shell to the vertex is

$arc_{roof} := 13 \cdot ft$ Solve for the roof radius. Start with "guess" values $r_{roof} := H_s$

$\theta_{roof} := atan\left(\frac{R}{H_s}\right) = 17.472 \cdot deg$

Given

$r_{roof} \cdot \sin(\theta_{roof}) = R$

$r_{roof} \cdot \theta_{roof} = arc_{roof}$

$Solution := Find\left(\frac{r_{roof}}{ft}, \frac{\theta_{roof}}{deg}\right) = \begin{pmatrix} 35.414 \\ 21.032 \end{pmatrix}$ $r_{roof} := Solution_0 \cdot ft = 35.414\,ft$ $\theta_{roof} := Solution_1 \cdot deg = 21.032 \cdot deg$

Find the vertical distance from the top of the shell to the roof radius point

$rp_{delta\_Hs} := \sqrt{r_{roof}^2 - R^2} = 33.055\,ft$ $\quad z_{rp} := H_s - rp_{delta\_Hs} = 7.325\,ft$ height of radius point above floor

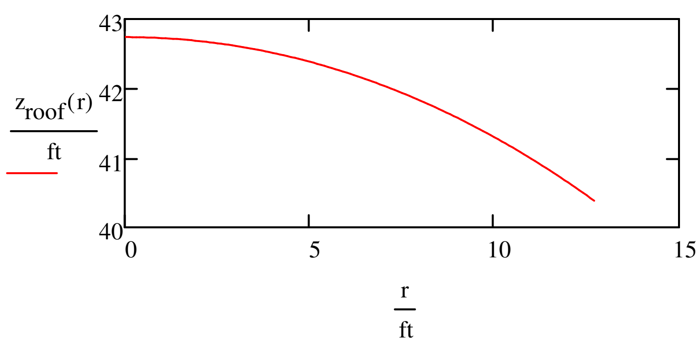
The height of the apex above the floor is $z_{apex} := z_{rp} + r_{roof} = 42.739\,ft$

The roof height is $h_r := z_{apex} - H_s = 2.359\,ft$

The expression for z for the roof for $0 < r < R$ is

$$z_{roof}(r) := \mathrm{if}\left(r > R, 0, z_{rp} + \sqrt{r_{roof}^2 - r^2}\right)$$

Plot the roof elevation vs radius $r := 0, .1 \cdot ft .. R$



The slope at distance "r" is

$$z'_{roof}(r) := \frac{d}{dr} z_{roof}(r)$$

For a surface of revolution, the general equation for the surface area is

$$A := 2 \cdot \pi \cdot \int x\, ds \quad \text{where} \quad ds := \sqrt{1 + \left(\frac{dz}{dx}\right)^2} \cdot dx$$

$$A_r := 2 \cdot \pi \cdot \left(\int_0^R r \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr\right) = 525\,ft^2 \qquad A_{tank} := \pi \cdot R^2 = 507.506\,ft^2$$

$$X_r := \frac{\left[2 \cdot \pi \cdot \left(\int_0^R r \cdot z_{roof}(r) \cdot \sqrt{1 + z'_{roof}(r)^2}\, dr\right)\right]}{A_r} = 41.56\,ft$$ height to centroid of roof area

Compute the horizontal distance to the center of area from the center for a wedge of roof

$$R_r := \frac{\left[2\cdot\pi\cdot\left(\int_0^R r^2\cdot\sqrt{1+z'_{roof}(r)^2}\,dr\right)\right]}{A_r} = 8.531\,ft$$

Enter shell and roof plate thickness.

Mathcad General Input - See Appendix for Mathcad nomenclature and symbols

$ORIGIN := 1$

Special unit definitions $each := 1$ $sf := ft^2$

number of shell plate courses, numbering starting with the base as course 1

$n_{course} := 5$ (the vertical leg of the top angle is included with the top shell plate course)

Calculate the elevation of the top of each shell course relative to the floor

$i := 1, 2 .. n_{course}$ i is the number of each shell course, starting from the bottom $\gamma_{steel} := 490\cdot pcf$ unit weight of steel

zshell is the elevation of the top of each course relative to the top of the bottom plate

$$z_{shell} := \begin{pmatrix} 8.02 \\ 16.02 \\ 24.01 \\ 32.04 \\ 40.36 \end{pmatrix}\cdot ft \qquad t_{shell} := \begin{pmatrix} .25 \\ .25 \\ .25 \\ .25 \\ .25 \end{pmatrix}\cdot in \qquad w_{shell} := t_{shell}\cdot\gamma_{steel} = \begin{pmatrix} 10.208 \\ 10.208 \\ 10.208 \\ 10.208 \\ 10.208 \end{pmatrix}\cdot psf \qquad class_{shell} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Shell thickness is per field measurement, rounded to the nearest 1/32". Records do not indicate steel which was used. Assume at least ASTM 283 Grade B, the minimum to qualify for AWWA Class 1 material..

Class 1 material has a yield stress 27 ksi < Fy < 34 ksi. Class 2 material has a yield stress Fy > 34 ksi

Roof thickness is 3/16" per nameplate, but thickness gauge measurements were .120". Use 3/16" to be conservative for roof weight calcualtions.

$t_{roof\_plate} := .15 \cdot in$ roof plate thickness as measured in the field

Compute weight of roof and shell

$W_r := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_r = 3.216 \cdot kip$

Define the number of the shell course for any value of 0 < z < Hs using a series of functions

$i_{course}(z) := n_{course}$ Default value

$i_{course}(z) := if\left(z < z_{shell_{n_{course}}}, n_{course}, i_{course}(z)\right)$

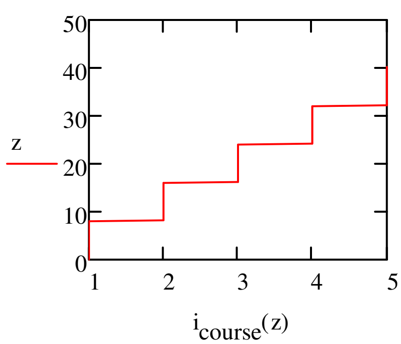
$i_{course}(z) := if\left(z < z_{shell_4}, 4, i_{course}(z)\right)$

$i_{course}(z) := if\left(z < z_{shell_3}, 3, i_{course}(z)\right)$

$i_{course}(z) := if\left(z < z_{shell_2}, 2, i_{course}(z)\right)$

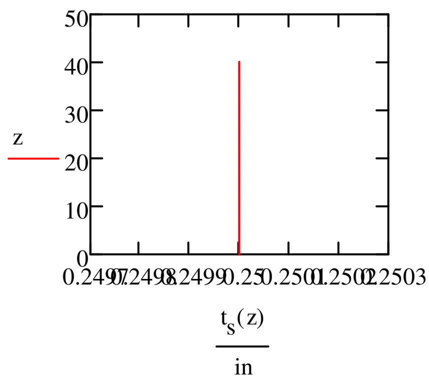
$i_{course}(z) := if\left(z < z_{shell_1}, 1, i_{course}(z)\right)$

$z := 0 \cdot ft, 0.2 \cdot ft .. H_s$ Set plotting interval for graphs

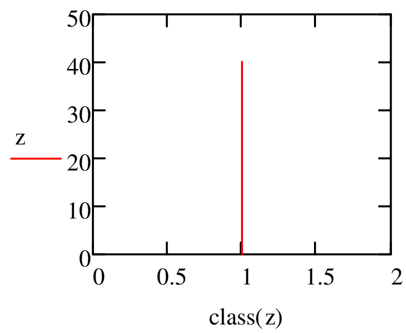


write functions that return the shell plate thickness and class as a function of height above the base

$t_s(z) := t_{shell_{i_{course}(z)}}$ $class(z) := class_{shell_{i_{course}(z)}}$



Shell thickness vs elevation



Shell class vs elevation

Floor plate thickness $t_{floor} := \frac{.9}{32} \cdot in$ measured

$floor\_flange := 1.5 \cdot in$ Bottom plate projection beyond shell plate $D_{floor} := D + 2 \cdot floor\_flange$

Compute floor weight

$$W_f := \gamma_{steel} \cdot t_{floor} \cdot \pi \cdot \frac{D_{floor}^{\ 2}}{4} \qquad W_f = 0.6 \cdot kip$$

Compute the weight of the shell and establish its center of gravity from the base

$$W_s := \pi \cdot D \cdot \int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z)\, dz \qquad W_s = 32.919 \cdot kip$$

$$X_s := \pi \cdot D \cdot \frac{\int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \cdot z\, dz}{W_s} \qquad X_s = 20.19\, ft$$

Check to see if roof snow load mass must be included per ASCE 7-10

$p_g := 77 \cdot psf$ from "Snow Load Analysis for Washington", 2nd ed, SEAW

$I_s := 1.20$ Snow load importance factor for risk category IV, ASCE 7-10

$C_e := 1.2$ ASCE 7-10, Table 7-2. Exposure Factor, Terrain B, Sheltered

$C_t := 1.2$ ASCE 7-10, Table 7-3, Thermal Factor, Unheated

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 93.139 \cdot psf$ Flat roof snow load, ASCE 7-10 Eq 7.3-1. Since flat roof snow load exceeds 30 psf, add 20% of the design snow load to the roof mass per ASCE 7-10, section 12.7.2.

The roof slope at the shell is less than or equal to $\theta_{roof} = 21.032 \cdot deg$

From ASCE 7-10 Fig 7-2c and 7-3, the roof slope factor is

$C_s := 1.0$

$p_s := C_s \cdot p_f = 93 \cdot psf$

Snow weight to include with roof weight

$w_{snow} := .20 \cdot p_s = 19 \cdot psf$

$W_{snow} := w_{snow} \cdot \pi \cdot R^2 = 9.454 \cdot kip$

$P_{snow} := \dfrac{W_{snow}}{\pi \cdot D} = 118.38 \cdot \dfrac{lbf}{ft}$ Snow load applied at top of shell concurrent with seismic

Compute the center of gravity of the roof snow load for seismic calculations

Snow density per ASCE 7-10 equation 7.7.1 is

$\gamma_{snow} := \min\left(30 \cdot pcf, 0.13 \cdot \dfrac{p_g}{ft} + 30 \cdot pcf\right) = 30 \cdot pcf$ snow depth $h_d := \dfrac{w_{snow}}{\gamma_{snow}} = 0.621\,ft$

$X_{snow} := X_r + \dfrac{h_d}{2} = 41.87\,ft$ centroid of snow mass

Compute total water weight for seismic calculations

$\gamma_{water} := 62.4 \cdot pcf$

$W_T := \gamma_{water} \cdot H \cdot \pi \cdot \dfrac{D^2}{4} = 1244.57 \cdot kip$

Calculate the impulsive and convective water weights and vertical centroids

$\dfrac{D}{H} = 0.647$

$W_i := W_T \cdot \dfrac{\tanh\left(.866 \cdot \dfrac{D}{H}\right)}{.866 \cdot \dfrac{D}{H}}$ if D/H > 1.333

$W_i := \text{if}\left[\frac{D}{H} < 1.333, W_T \cdot \left(1.0 - 0.218 \cdot \frac{D}{H}\right), W_i\right]$ if D/H < 1.33

$W_i = 1069.074 \cdot kip$ Impulsive water weight $\frac{W_i}{W_T} = 0.859$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated or the total moment on the foundation, shell plus floor.

$X_i := H \cdot \text{if}\left[\left(\frac{D}{H}\right) > 1.333, 0.375, 0.50 - 0.094 \cdot \frac{D}{H}\right]$ $X_i = 17.261\,ft$ centroid for calculation of just the shell moment

$X_{imf} := 0.375 \cdot \left[1.0 + 1.333 \cdot \left(\frac{0.866 \cdot \frac{D}{H}}{\tanh\left(0.866 \cdot \frac{D}{H}\right)} - 1\right)\right] \cdot H$ centroid for calculation of total bottom moment if D/H > 1.33

$X_{imf} := \text{if}\left[\frac{D}{H} < 1.333, \left(0.50 + 0.06 \cdot \frac{D}{H}\right) \cdot H, X_{imf}\right]$ centroid for calculation of total bottom moment if D/H < 1.33

$X_{imf} = 21.175\,ft$

<u>Compute convective water weight and effective centroid above the base</u>

$W_c := W_T \cdot \left(.230 \cdot \frac{D}{H} \cdot \tanh\left(3.67 \cdot \frac{H}{D}\right)\right)$ $W_c = 185.15 \cdot kip$ $\frac{W_c}{W_T} = 0.149$ Ref 4, Eq 13-26

$X_c := H \cdot \left[1 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1}{3.67 \cdot \left(\frac{H}{D}\right) \cdot \sinh\left(3.67 \frac{H}{D}\right)}\right]$ $X_c = 32.421\,ft$ centroid for calculation of just the shell moment

$X_{cmf} := H \cdot \left(1.0 - \frac{\cosh\left(3.67 \cdot \frac{H}{D}\right) - 1.937}{3.67 \cdot \frac{H}{D} \cdot \sinh\left(3.67 \cdot \frac{H}{D}\right)}\right)$ $X_{cmf} = 32.466\,ft$ centroid for calculation of total bottom moment

Seismic Design Criteria

Importance Factor: $I_E := 1.50$ Risk category IV

Ground Motion Parameters

Site Class B — Based on rock uncovered at base of ringwall during site investigations

$S_S := .944$ $S_1 := .369$ Mapped earthquake short period and long period spectral accelerations. For Site Class B, 5% damping, expressed as fraction of g.

$F_a := 1.00$ $F_v := 1.00$ Site coefficients from 2012 IBC Table 1613.3.3(2). Seismic Design Category "D"

Adjusted maximum considered earthquake for site class

$S_{MS} := F_a \cdot S_S$ $S_{MS} = 0.944$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.369$

Design spectral response parameters

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$ $S_{DS} = 0.629$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$ $S_{D1} = 0.246$

Compute points on the design response spectrum

$T_0 := 0.2 \cdot sec \cdot \frac{S_{D1}}{S_{DS}}$ $T_0 = 0.078 \cdot sec$

$T_S := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot sec$ $T_S = 0.391 \cdot sec$

$T_L := 6 \cdot sec$ Mapped value, ASCE 7-10, Figure 22-12

$T_L := if\left(T_L > 4.sec, 4.sec, T_L\right) = 4 \cdot sec$ Maximum required for tank sloshing wave calculations, ASCE 7-10, Section 15.7.6.1.d

$$S_{ac}(T) := if\left(T > T_L, \frac{1.5 \cdot S_{D1} \cdot T_L \cdot sec}{T^2}, min\left(\frac{1.5 \cdot S_{D1} \cdot sec}{T}, 1.5 \cdot S_{DS}\right)\right)$$

Convective acceleration function

$S_{ac}(T) := \text{if}\left(S_{ac}(T) > 1.5S_{DS}, 1.5S_{DS}, S_{ac}(T)\right)$ Upper bound for S_{ac} for low values of T

$$S_{ai}(T) := \text{if}\left(T > T_L, \frac{S_{D1} \cdot T_L \cdot \text{sec}}{T^2}, \text{if}\left(T > T_S, \frac{S_{D1}}{T} \cdot \text{sec}, S_{DS}\right)\right)$$

Impulsive acceleration function

Calculate Free Surface Wave Height and Compare to Freeboard Requirements

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{D}{3.68\cdot g\cdot\tanh\left(3.68\cdot\frac{H}{D}\right)}} \qquad T_c = 2.911\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \text{if}\left(SUG = 3, \text{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\;2}}\right), \text{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\;2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.127$

$d := 0.5\cdot D\cdot A_f = 1.611\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude.

freeboard $f := H_s - H = 1.08\,ft$

$\frac{d}{f} = 1.492$ > 1.0, therefore **freeboard is insufficient**

Compute Base Shear and Overturning Moments As If Free Surface

$S_{ai} := S_{DS}$ $R_i := 3.0$ $R_c := 1.5$ AWWA D100-11, Table 28 and section 13.2.9.2. Anchored tank

$$A_i := \max\left(\frac{S_{ai} \cdot I_E}{1.4 \cdot R_i}, \frac{0.36 \cdot S_1 \cdot I_E}{R_i}\right) \quad A_i = 0.225$$

Impulsive design acceleration

$$A_c := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c} \quad A_c = 0.091$$

Convective design acceleration

Calculate overturning moment at the base of the shell

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_i\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2} \quad M_s = 4449 \cdot kip \cdot ft$$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_{imf}\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2} \quad M_{mf} = 5384 \cdot kip \cdot ft$$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left(W_s + W_r + W_{snow} + W_f + W_i\right)\right]^2 + \left(A_c \cdot W_c\right)^2} \quad V_f = 251.23 \cdot kip$$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Adjust Effective Masses for Roof Contact

The methodology for roof contact effects is an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave.

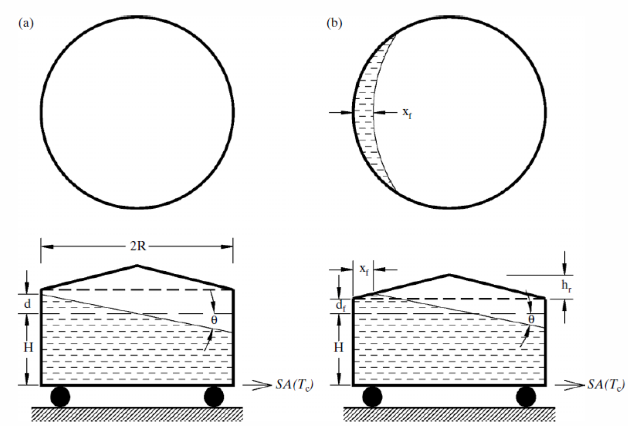


Fig. 5: Liquid-filled tank translating with an acceleration SA(T_c): (a) sufficient freeboard; and (b) insufficient freeboard

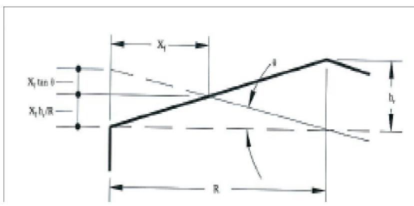


Fig. 9: Radial variation of pressure on tank roof

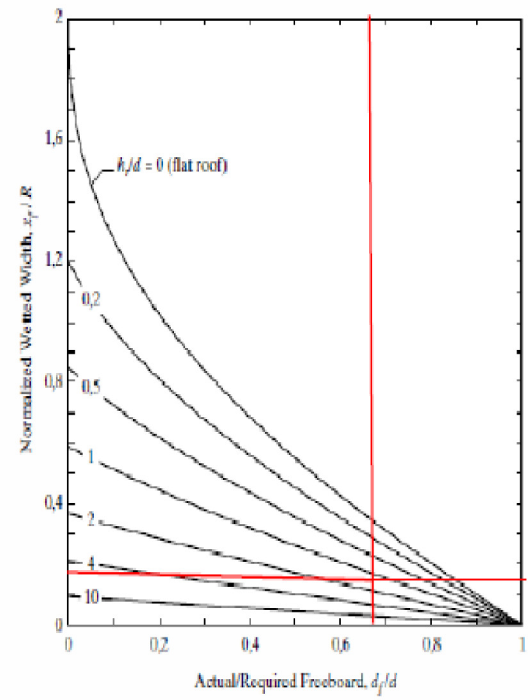


Fig. 6: Cone roof tank. Normalized wetted width of tank roof x_f/R as a function of actual/required freeboard d_f/d and normalized roof height h_r/d

Compute the angle θ

$$\theta := \operatorname{atan}\left(\frac{I_E \cdot S_{ac}(T_c) \cdot \frac{ft}{sec^2}}{g}\right) = 0.339 \cdot deg$$

Where

$S_{ac}(T_c) = 0.127$ $\quad I_E = 1.5$ $\quad g = 32.174 \frac{ft}{s^2}$

$d_f := H_s - H = 1.08\,ft$ $\quad d = 1.611\,ft$ $\quad \frac{d_f}{d} = 0.67$ Compute input variables for graph above

$h_r = 2.359\,ft$ $\quad \frac{h_r}{d} = 1.465$

From graph figure 6

$x_f := .17 \cdot R = 2.161\,ft$ horizontal extent of wetted dome surface from the shell $\frac{x_f}{R} = 0.17$ << 1.0 OK

$\rho := \frac{\gamma_{water}}{g} = 62.4 \cdot \frac{lbm}{ft^3}$ unit mass of water

Note: per above reference, use of this method for curved roofs is slightly conservative

$F_{max} := \frac{\rho}{2} \cdot g \cdot x_f^{\,2} \cdot \frac{(d + h_r)}{R}$ $F_{max} = 46 \cdot \frac{lbf}{ft}$ Maximum uplift on shell due to hydrodynamic pressure caused by sloshing. Impact effects are considered minor and ignored

adjust mass for recalculation of seismic demand

$$\bar{m}_i = \begin{cases} m_i + m_c \cdot \left(1 - \dfrac{d_f + h_r/3}{d}\right) & \text{for } d_f + h_r/3 < d \\ m_i & \text{for } d_f + h_r/3 \geq d \end{cases}$$

$W_i = 1069 \cdot kip$

$W_T = 1245 \cdot kip$

$\frac{\left(d_f + \frac{h_r}{3}\right)}{d} = 1.159$ $W_{bar\_i} := W_i + W_c \cdot \left(1 - \frac{d_f + \frac{h_r}{3}}{d}\right) = 1039.7 \cdot kip$

$W_{bar\_i} := \mathrm{if}\left[\frac{\left(d_f + \frac{h_r}{3}\right)}{d} < 1, W_{bar\_i}, W_i\right] = 1069 \cdot kip$

$$\bar{m}_c = m_l - \bar{m}_i$$

$W_c = 185.1 \cdot kip$ $W_{bar\_c} := W_T - W_{bar\_i} = 175.5 \cdot kip$

$\frac{W_{bar\_i}}{W_i} = 1$ $\frac{W_{bar\_c}}{W_c} = 0.948$ Factors by which mass must be multiplied due to the slosh contact with the roof

Recaclulate convective period using adjusted mass. Maintain asssumption of T = 0 for impulsive mass

$$\bar{T}_i = T_i \cdot \sqrt{\frac{\bar{m}_i}{m_i}}$$

$$\bar{T}_c = T_c \cdot \sqrt{\frac{\bar{m}_c}{m_c}}$$

$T_c = 2.911\,s$ original convective period

$T_{c\_bar} := T_c \cdot \sqrt{\frac{W_{bar\_c}}{W_c}} = 2.834\,s$ modified convective period

$S_{ac}(T_c) = 0.127$ $A_c = 0.091$ original convective seismic factor

$S_{ac}(T_{c\_bar}) = 0.13$ $A_{c\_bar} := A_c \cdot \frac{S_{ac}(T_{c\_bar})}{S_{ac}(T_c)} = 0.093$ revised convective seismic factor

Recompute base shear and overturning moment

Change formula weights to adjusted values

$M_s = 4449 \cdot kip \cdot ft$ original overturning moment

$$M_{s\_rev} := \sqrt{\left[A_i \cdot \left[W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + \left(W_{bar\_i}\right) \cdot X_i\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_c\right)^2}$$

$M_{s\_rev} = 4447 \cdot kip \cdot ft$ revised moment

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$M_{mf} = 5384 \cdot kip \cdot ft$ original overturning moment

$$M_{mf\_rev} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{bar\_i} \cdot X_{imf}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_{cmf}\right)^2}$$

$M_{mf\_rev} = 5383 \cdot kip \cdot ft$ revised moment

Calculate base shear at top of foundation

$V_f = 251.23 \cdot kip$ original base shear

$$V_{f\_rev} := \sqrt{\left[A_i \cdot \left(W_s + W_r + W_{snow} + W_f + W_{bar\_i}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c}\right)^2}$$

$V_{f\_rev} = 251.2 \cdot kip$ revised base shear

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Compute Shell Hoop Forces and Stresses

Impulsive and convective forces are distributed using Housner's distribution formulas

Define the following variables:

z	Height of a point above the tank floor
Y	Depth of a point below the water surface
n_I	Distributed hoop force, klf, due to impulsive load N_I
n_C	Distributed hoop force, klf, due to convective load N_C
n_V	Distributed hoop force, klf, due to vertical seismic force N_V
n_F	Distributed hoop force, klf, due to hydrostatic force at maximum normal operating level
n_{Fol}	Distributed hoop force, klf, due to hydrostatic force at overflow operating level

Define elevation, distribution, and force component functions

$Y(z) := H - z$ distance from MOL to z

Housner's distribution of impulsive load as a function of elevation above the base
and, in the case of impulsive loads, depends on the ratio of D/H

For the case of D/H < 1.33 and Y(z) < 0.75 D (z > .75D, upper section)

$$Dist_{ia}(z) := \frac{\left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2}{\left[\int_{.75\cdot D}^{H} \left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz + \int_0^{.75\cdot D} 0.5\,dz\right]}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H < 1.33 at lower elevations, the factor is a constant equal to

$$Dist_{ib}(z) := \frac{0.5}{\left[\int_{.75\cdot D}^{H} \left(\frac{Y(z)}{0.75\cdot D}\right) - 0.5\cdot\left(\frac{Y(z)}{0.75\cdot D}\right)^2 dz\right] + \int_0^{.75\cdot D} 0.5\,dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H > 1.33

$$\mathrm{Dist_{ic}}(z) := \frac{\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)}{\int_{0\cdot ft}^{H}\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

$$\mathrm{Dist_i}(z) := \mathrm{if}\left[\left(\frac{D}{H}\right) \geq 1.333, \mathrm{Dist_{ic}}(z), \mathrm{if}\left(Y(z) < 0.75\cdot D, \mathrm{Dist_{ia}}(z), \mathrm{Dist_{ib}}(z)\right)\right]$$

select appropriate formula based on depth and diameter ratio

Housner's distribution of convective load as a function of elevation above the base

$$\mathrm{Dist_c}(z) := \frac{\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}}{\displaystyle\int_{0\cdot ft}^{H}\frac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}dz}$$

The above formula is the convective force per unit depth at elevation "z" expressed as a fraction of the total convective force.

$V_i := A_i\cdot W_{bar\_i}$ $\quad V_i = 240.287\cdot kip$ — Total base shear component due to impulsive fluid load

$N_i(z) := \left(\frac{V_i}{2}\right)\cdot \mathrm{Dist_i}(z)$ — Shell hoop force due to impulsive fluid load

$V_c := A_{c\_bar}\cdot W_{bar\_c}$ $\quad V_c = 16.319\cdot kip$ — Total base shear component due to convective fluid load

$N_c(z) := \frac{V_c}{2}\cdot \mathrm{Dist_c}(z)$ — Shell hoop force due to convective fluid load

$N_h(z) := \gamma_{water}\cdot\left(\frac{D}{2}\right)\cdot Y(z)$ — Shell hoop force due to hydrostatic load with water at MOL

$A_v := 0.14\cdot S_{DS}$ $\quad A_v = 0.088$ — Vertical seismic factor

$$\sigma_{static}(z) := \frac{N_h(z)}{t_s(z)}$$

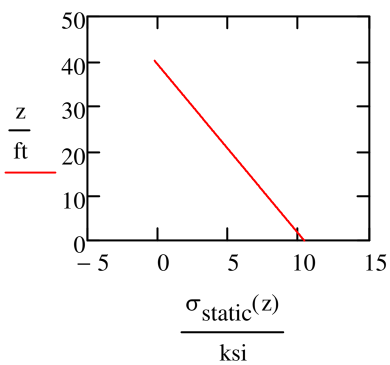
Hoop stress due to static fluid pressure at MOL

$$\sigma_s(z) := \frac{\sqrt{N_i(z)^2 + N_c(z)^2 + \left(N_h(z) \cdot A_v\right)^2}}{t_s(z)}$$

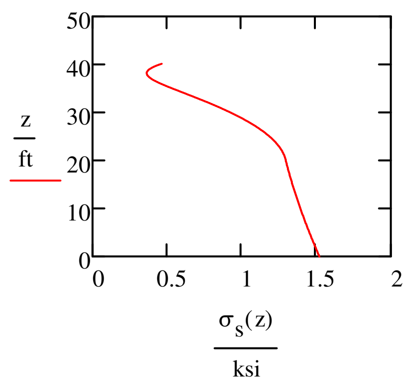
Hoop stress due to hydrodynamic pressure, Ref 4 Eq 13-42

$$\sigma_{total}(z) := \sigma_{static}(z) + \sigma_s(z)$$

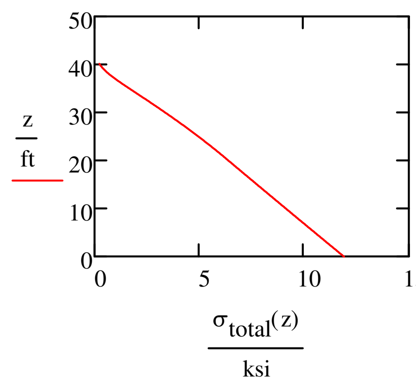
Combined static and seismic hoop stress at MOL



Hydrostatic Stress



Seismic Stress



Static + Seismic Stress

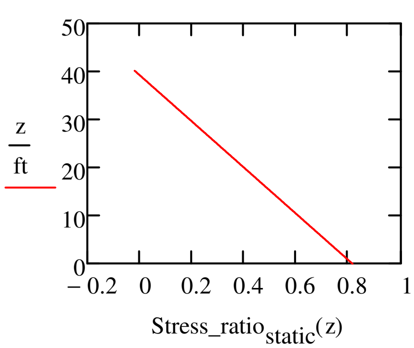
Note: the above plots are nominal based on treating each hoop course as acting independently. Actual stresses each side of girth joints are the same since strains are identical if the courses are attached, so the real stress near transition zones falls somewhere between the apparent discontinuous stress levels shown on the graphs. The actual maximum stress levels tend to occur about a foot above the joint and are not as high as predicted by the more simplified model. The simplified model is conservative and is the method reflected in the AWWA D-100 standard.

Check actual versus allowable stress based on the class of steel used.

Assumed joint efficiency and allowable stress $E_{joint} := 85\%$ $F_t(z) := E_{joint} \cdot 15 \cdot ksi$

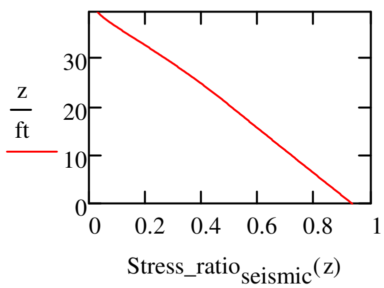
Chapter 14 of AWWA D100-11 does not apply

$$Stress\_ratio_{static}(z) := \left(\frac{\sigma_{static}(z)}{F_t(z)}\right)$$



Maximum static stress ratio is $Stress\_ratio_{static}(0) = 0.815$ < 1.0 OK

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{total}(z)}{F_t(z)}$$



The worst case stress ratio is at
the bottom of the first shell course

$Stress\_ratio_{seismic}(0) = 0.934$ < 1.33 OK

Compute Shell Longitudinal Forces and Stresses

Define axial compressive force in the shell due to dead load for $0 < z < H_s$, in klf.

$$P_D(z) := \frac{W_r}{\pi \cdot D} + \int_z^{H_s} \gamma_{steel} \cdot t_s(z)\, dz$$

Define overturning moment functions at elevation z, in kip-ft

$$M_{rs}(z) := A_i \cdot \left[W_r \cdot (X_r - z) + W_{snow} \cdot X_{snow} + \pi \cdot \gamma_{steel} \cdot D \cdot \int_z^{H} y \cdot t_s(y)\, dy \right]$$

Moment associated with roof, snow and shell mass

$$M_i(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_i(y)\, dy$$

Moment associated with impulsive fluid mass, z < H

$$M_c(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_c(y)\, dy$$

Moment associated with convective fluid mass, z < H

$$M_s(z) := M_{rs}(z) + M_i(z) + M_c(z)$$

Total moment at elevation z on the shell for z < H

Define functions for compressive stress under static or seismic load conditions

$$\sigma_{static}(z) := \frac{P_D(z) + P_{snow}}{t_s(z)}$$

$$\sigma_{comp}(z) := \frac{(1 + 0.4 \cdot A_v)(P_D(z) + P_{snow}) - F_{max} + \frac{4\, M_s(z)}{\pi \cdot D^2}}{t_s(z)}$$

Includes deduction for roof uplift, $F_{max.}$

Check allowable stress for compression with local buckling and slenderness considered

Use Method 1. Yield stress of shell plate does not permit use of Method 2.

Local buckling stress formulas for Class 1 Materials

$$F_{L1a}(z) := \left[17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R} \right) \right] \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R} \right)^2 \right] \cdot psi$$

For Class 1 materials with 0 < t/R < t/Rc = .0031088, elastic buckling

$F_{L1b}(z) := 5775 \cdot psi + 738 \cdot 10^{3} \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 1 materials with t/Rc = .0031088 < t/R < 0.0125, inelastic buckling

$F_{L1c}(z) := 15 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Local buckling stress formulas for Class 2 Materials

$$F_{L2a}(z) := \min\left[15 \cdot ksi, 17.5 \cdot 10^{5} \cdot \left(\frac{t_s(z)}{R}\right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R}\right)^{2}\right] \cdot psi\right]$$

For Class 2 materials with 0 < t/R < t/Rc = .0035372, elastic buckling

$F_{L2b}(z) := 6925 \cdot psi + 886 \cdot 10^{3} \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 2 materials with t/Rc = .0035372 < t/R < 0.0125, inelastic buckling

$F_{L2c}(z) := 18 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Write equation selection functions for F_L depending on t/R ratio and class

$ratio1 := .0031088 \quad ratio2 := .0035372$

$$F_{L1}(z) := \min\left(\text{if}\left(\frac{t_s(z)}{R} < ratio1, F_{L1a}(z), \text{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L1b}(z), F_{L1c}(z)\right)\right), 15 \cdot ksi\right)$$

$$F_{L2}(z) := \min\left(\text{if}\left(\frac{t_s(z)}{R} < ratio2, F_{L2a}(z), \text{if}\left(\frac{t_s(z)}{R} < 0.0125, F_{L2b}(z), F_{L2c}(z)\right)\right), 18 \cdot ksi\right)$$

$F_L(z) := \text{if}\left(class(z) = 1, F_{L1}(z), F_{L2}(z)\right)$

Slenderness reduction factor equations

$r := \frac{D \cdot \sqrt{2}}{4}$ radius of gyration of tank shell

$K := 1.0$ effective column length factor, pinned ends assumed

$E := 29 \cdot 10^{6} \cdot psi$ modulus of elasticity for steel

Slenderness ratio at which overall elastic column buckling can occur (not local buckling)

$C'_c(z) := \sqrt{\pi^{2} \cdot \frac{E}{F_L(z)}} \qquad L := H_s$

$$K_{\varphi 1}(z) := 1 - \frac{1}{2} \cdot \left(\frac{\frac{K \cdot L}{r}}{C'_c(z)} \right)^2$$ For 25 < KL/r < C'c

$$K_{\varphi 2}(z) := \frac{1}{2} \cdot \left(\frac{C'_c(z)}{\frac{K \cdot L}{r}} \right)^2$$ For KL/r > C'c

$$K_{\varphi 3}(z) := 1.0$$ For KL/r < 25

$$ratio := K \cdot \frac{L}{r} \qquad ratio = 4.493$$

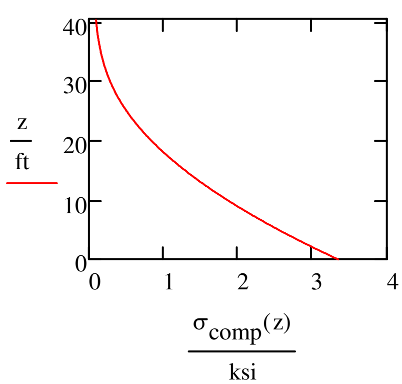
$$K_{\varphi}(z) := \text{if}\left(ratio < 25, K_{\varphi 3}(z), \text{if}\left(ratio > C'_c(z), K_{\varphi 2}(z), K_{\varphi 1}(z)\right)\right)$$

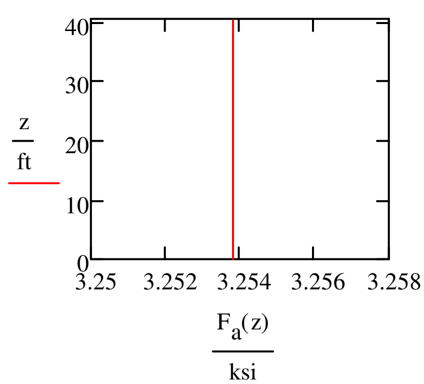
$$F_a(z) := F_L(z) \cdot K_{\varphi}(z)$$ allowable compressive stress due to axial load

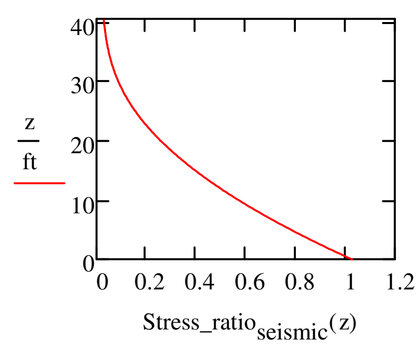
For shell longitudinal stress, treat all stress as axial

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{comp}(z)}{F_a(z)}$$

Plot static plus seismic compressive stress and compare to allowables





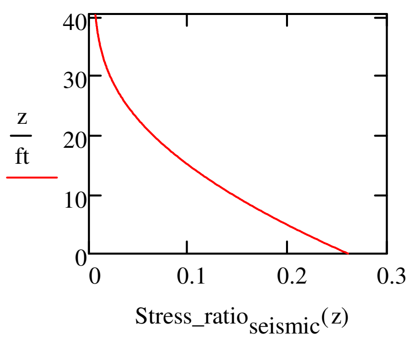


$Stress\_ratio_{seismic}(0) = 1.029$ << 1.33, **OK for static plus seismic longitudinal compression**

Check seismic longitudinal tensile stress

$$\sigma_{tens}(z) := \frac{\left(1 - .40 \cdot A_v\right) P_D(z) + F_{max} + \dfrac{4\,M_s(z)}{\pi \cdot D^2}}{t_s(z)} \qquad Stress\_ratio_{seismic}(z) := \frac{\sigma_{tens}(z)}{F_t(z)}$$

$$Stress\_ratio_{seismic}(0) = 0.261$$



All stress ratios << 1.333 are **OK for static plus seismic stress in longitudinal tension**

Horizontal Shear Transfer Capacity

The previously calculated base shear is $V_f = 251 \cdot kip$

From AWWA D100-11 Eq 13-57, the allowable resistance attributable to friction is (for the full tank, seismic condition)

$$V_{ALLOW} := \tan(30 \cdot deg) \cdot \left(W_s + W_r + W_T + W_f\right) \cdot \left(1 - A_v\right) = 675 \cdot kip$$

$$\frac{V_f}{V_{ALLOW}} = 0.372$$

>> V_f OK. No shear connection between the superstructure and base is required for shear. Shear resistance is provided by the bottom plate acting as a diaphragm kept in place by bottom friction. Check shell to bottom transfer capacity

The maximum shell to bottom plate shear load is $v := 2 \cdot \frac{V_f}{\pi \cdot D} = 6.292 \cdot klf$

There is no annular plate, just the 5/32" floor plate

$$t_f := \frac{5}{32} \cdot in$$

And the maximum shear stress on the plate is $\tau := \frac{v}{t_f} = 3 \cdot ksi$ $\frac{\tau}{12 \cdot ksi} = 0.28$

AWWA D100 permits 12 ksi in shear, and this can be increased by 1.33 for seismic, so **floor plate should not tear in shear parallel to the floor plate**

Check Foundation

Check nominal anchor capacity $\sigma_{tens}(0)\cdot t_s(0) = 9.983\cdot klf$

Compute existing anchor load

$n_{anchors} := 12$ $T_{anchor} := \left(\frac{\pi\cdot D}{n_{anchors}}\right)\cdot\left(\sigma_{tens}(0)\cdot t_s(0)\right)$ $T_{anchor} = 66.4\cdot\frac{kip}{each}$

Allowable stress $F_t := 15\cdot ksi = 15\cdot ksi$

Check stress in embedded plate $A_{anchor} := 3\cdot in\cdot\frac{3}{8}\cdot in = 1.125\cdot in^2$

$\sigma_{anchor} := \frac{\pi\cdot D\cdot\sigma_{tens}(0)\cdot t_s(0)}{n_{anchors}\cdot A_{anchor}} = 59.055\cdot ksi$ $\frac{\sigma_{anchor}}{F_t} = 3.937$ >> 1.33 No Good for backing plate

Anchors are overstressed

Compute anchor weld load vs allowable

$l_{weld\_longitudinal} := 8\cdot in$ $l_{weld\_transverse} := 3\cdot in$ Strap to shell

$t_{weld} := \frac{3}{8}\cdot in$ $F_t = 15000\,psi$ Note: record drawing says fillet weld of strap to shell is 1/4", but plate is only called out as 3/16"

$T_{allowable} := .7071\cdot t_{weld}\cdot F_t\cdot\left(.65\cdot l_{weld\_transverse} + .50\cdot l_{weld\_longitudinal}\right) = 23.666\cdot kip$

$\frac{T_{anchor}}{T_{allowable}} = 2.807$ >1.33 No good for strap to shell weld, even with offset ignored

Welds are overstressed

Compute embedded plate bond capacity

approximate method, use ACI 318-63 which allows the following allowable bond stress for plain bars

The perimeter of the embedded anchor is $P_{anchor} := (2\cdot.375 + 2\cdot 3)\cdot in = 6.75\cdot in$

(this is for typical anchors only. anchors are shorter over pipe entrance, so capacity is less)

An equivalent round bar diameter would be $D_{equiv} := \frac{P_{anchor}}{\pi} = 2.149\cdot in$

For deformed bars, the ACI 318-63 allowable bond stress is $F_{bond} := 4.8 \cdot \sqrt{3000} \cdot \frac{in \cdot psi}{D_{equiv}} = 122.362\, psi$

For plain bars $F_{bond} := \min(.5 \cdot F_{bond}, 160 \cdot psi) = 61.181\, psi$

The embedded length of the anchor, including the hook, is unknown. Assume at best the anchor extends to withing 6 inches of the bottom of the footing, with a 6 inch 90 degree "hook" based on typical details for other tanks. The measured height of the foundation ringwall is 58.5".

$l_{embed} := 58.5 \cdot in$

Allowable load based on bond $T_{allowable} := P_{anchor} \cdot l_{embed} \cdot F_{bond} = 24.159 \cdot kip$

$\frac{T_{anchor}}{T_{allowable}} = 2.75$ **Pullout capacity is inadequate**

Check Foundation For Uplift and Overturning

$\gamma_{conc} := 150 \cdot pcf$

$b_{ftg} := 18 \cdot in$ $b_{ftg\_lower} := b_{ftg} + 2.5 \cdot in = 20.5 \cdot in$ $h_{ftg} := 32 \cdot in$ $h_{ftg\_lower} := 40 \cdot in - h_{ftg}$

footing width and depth

$R_{ftg} := R + 6 \cdot in = 13.21\, ft$ $R_{in} := R_{ftg} - b_{ftg}$ $R_{ftg\_lower} := R_{in} + b_{ftg\_lower} = 13.418\, ft$

footing outside and inside radii

$A_{ftg} := \pi \cdot \left(R_{ftg}^2 - R_{in}^2\right) = 117.433\, ft^2$ $A_{ftg\_lower} := \pi \cdot \left(R_{ftg\_lower}^2 - R_{in}^2\right) = 134.861\, ft^2$

$W_{ftg} := \gamma_{conc} \cdot \left(A_{ftg} \cdot h_{ftg} + A_{ftg\_lower} \cdot h_{ftg\_lower}\right) = 60.5 \cdot kip$ $w_{ftg} := \frac{W_{ftg}}{\pi \cdot D} = 0.757 \cdot klf$ total and unit footing weight

$W_{water} := H \cdot \gamma_{water} \cdot \pi \cdot \left(R^2 - R_{in}^2\right) = 188.1 \cdot kip$ $w_{water} := \frac{W_{water}}{\pi \cdot D} = 2.356 \cdot klf$ total and unit weight of water over footing

$\gamma_{soil} := 125 \cdot pcf$ typical weight of compacted soil

$A_{soil} := h_{ftg} \cdot \left(R_{ftg\_lower} - R_{ftg}\right) = 0.6\, ft^2$ vertical area of soil over footing

$A_{wedge} := \frac{\left(h_{ftg} + .h_{ftg\_lower} - 8 \cdot in\right)^2}{2 \cdot 2} = 1\, ft^2$ area of soil resisting uplift in friction at 1H:2V, backfill to within 7" of top of footing. Skin friction assumed 0.4 between footing and soil

$w_{soil} := \gamma_{soil} \cdot (A_{soil} + 0.4A_{wedge})$ $\quad w_{soil} = 0.1 \cdot klf$ unit soil resistance

$W_s = 32.919 \cdot kip$ $\quad w_{shell} := \dfrac{W_s}{\pi \cdot D} = 0.412 \cdot klf$ shell weight

$W_r = 3.216 \cdot kip$ $\quad w_{roof\_edge} := \dfrac{W_r}{\pi \cdot D} = 0.04 \cdot klf$ roof edge weight

Compute overturning safety factor for pivoting about the toe of the shell

$M_{s\_rev} = 4447 \cdot kip \cdot ft$

$$SF_{overturning} := (1 - A_v) \cdot (W_r + W_s + W_{ftg} + W_{water}) \cdot \frac{R}{M_{s\_rev}} = 0.742$$ < 1.67 NG

Required safety factor based on ASCE 7 load combos is .7E/.6D where .7E is the earthquake load in allowable stress terms, an effective ratio of 1.67

Check ratio of resistance to uplift at the foundation

$$SF_{uplift} := \frac{\left[(1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water}) + w_{soil} - F_{max}\right]}{4 \cdot \dfrac{M_{s\_rev}}{\pi \cdot D^2}} = 0.379$$ NG

Tank is not stable under assumed seismic load

Check bearing pressure

The total load on the perimeter under static conditions is

$$w_{static} := w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water} = 3.565 \cdot klf$$

$$w_{seismic} := (1 + A_v) \cdot (w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water}) + F_{max} + 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 12.688 \cdot klf$$

Static allowable bearing pressure if no uplift

$q_{allow} := 12 \cdot ksf$

$q_{bearing\_static} := \dfrac{w_{static}}{b_{ftg}} = 2.377 \cdot ksf$ $\quad \dfrac{q_{bearing\_static}}{q_{allow}} = 0.198$ OK

$q_{bearing\_seismic} := \frac{w_{seismic}}{b_{ftg}} = 8.459 \cdot ksf$ $\frac{q_{bearing\_seismic}}{q_{allow}} = 0.705$ < 1.33 OK

Check As Self-Anchored Tank

Per AWWA D100 section 13.5.4.1

$w_t := P_D(0) = 452\cdot\frac{lbf}{ft}$ Weight of shell and roof supported by shell

$t_b := t_f = 0.156\cdot in$ $F_y := 27\cdot ksi$ $G := 1.0$ A283 Grade B steel assumed

$$w_L := \min\left(1.28\cdot\frac{H}{ft}\cdot\frac{D}{ft}\cdot G, 7.29\cdot\frac{t_b}{in}\sqrt{\frac{F_y}{ksi}\cdot\frac{H}{ft}\cdot G}\right)\cdot plf = 37\cdot\frac{lbf}{ft}$$

Eq 13-37, normalized for units

Overturning ratio

$$J := \frac{M_s(0)}{D^2\cdot\left[w_t\cdot\left(1 - 0.4\cdot A_v\right) + w_L\right]} = 15.755$$

Calculated for Ri = 3.0. For Ri = 2.5, result is 18.56

>> 1.54 therefore the tank is not stable without anchorage

References

1. 2012 *International Building Code*
2. Washington State Adoption of and Amendments to 2012 International Building Code (State Building Code)
3. ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
4. AWWA Standard D100-11 *Welded Carbon Steel Tanks for Water Storage*
5. Nuclear Reactors and Earthquakes, Chap. 6 and Appendix F. U.S. Nuclear Regulatory Commission publication, Division of Technical Information, TID-7024, National Technical Information Service (1963).
6. Not used
7. Not used
8. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" Praveen K. Malhotra, Structural Engineering International, March 2006
9. Not used
10. "Dynamic Pressures on Accelerated Fluid Containers," G.W. Housner, 1955, Bulletin of the Seismological Society of America.
11. "Snow Load Analysis for Washington, 2nd Ed." Structural Engineers Association of Washington,1995
12. Not used
13. Not used
14. ACI 318-11 Building Code Requirements for Structural Concrete
15. ANSI/AISC 360-10 Specification for Structural Steel Buildings
16. AWS D1.1 Structural Welding Code - Steel

Units and Mathcad Notation

All calculations are shown in U.S. customary units. Calculations have been performed using MathSoft's Mathcad Version 14.0 software, which automatically checks for unit consistency and applies any necessary unit conversion factors internally to the program. Where computations are imported from Excel, SAP2000, or other software, the source is identified. Input values are shaded. Others are computed.

Where equations are shown with a ":=" sign, the left hand side of the equation is being defined by the right hand side. Where equations are shown with a "=" sign, the current value of the expression on the left hand side is being displayed.

=	An ordinary "equals" sign indicates the value being shown is for the most current evaluation of the variable on the left hand side of the equation
:=	An "equals" sign with a colon indicates the value on the left hand side is being defined by the expression on the right. Variables may be redefined, the last definition taking precedence
=	A bold "equals" sign indicates the symbol is being used in a logical expression
if(a,b,c)	An "if" statement is evaluated as "b" if "a" is true, and as "c' is "a' is false. These expressions may be nested
(matrix$_{i,j}$)	In matrix expressions, the first subscript is the row, and the second is the column. Numbering starts with the value indicated as "ORIGIN" for the first row and column unless otherwise noted
submatrix (A,i1,i2,j1,j2)	Defines a vector or submatrix of matrix "A" from row i1 thru i2, and column j1 thru j2
-----------> ( )	An expression with a vector arrow over it indicates that the expression involves subscripted variables, and that the expression is being evaluated for each subscript in the range
| | | |	A bold vertical line to the left of a series of expressions indicates that they are acting as a programming loop in the calculations
$ORIGIN := 1$	Sets initial subscript value for subscripted variables
M<j>	The vector in column "j" of matrix "M"
$sf := ft^2$	
$\Phi(x)$	Step function. Returns -1 for x < 0, +1 for x > 0 and .5 if x = 0

Seismic Evaluation
for
Division 30 Reservoir - Retrofit Option A

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

These calculations are preliminary in nature for design approach analysis and are not to be used for construction

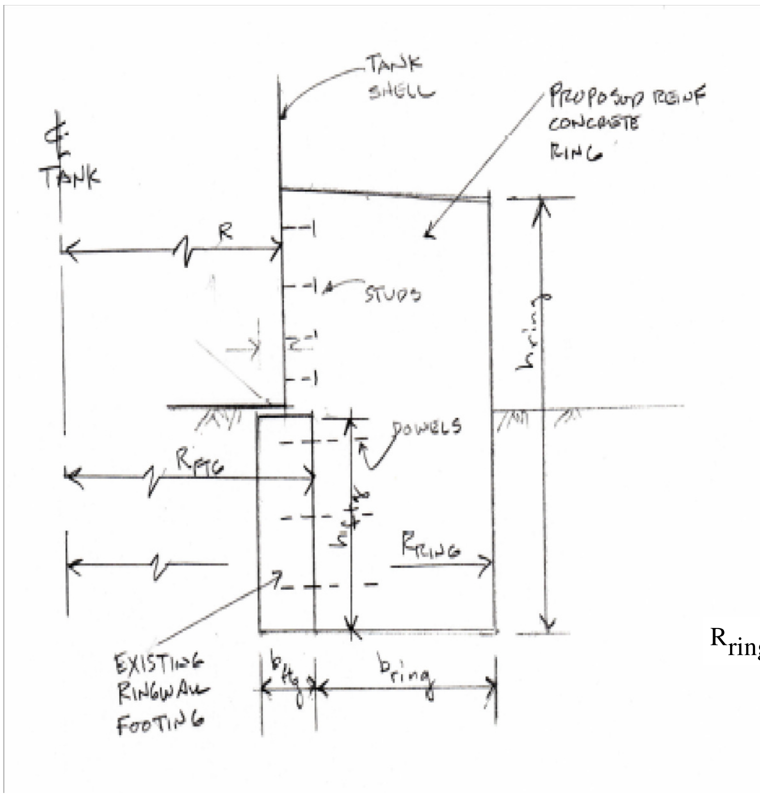
Incorporate calculations from existing tank analysis by reference.

Reference:S:\Projects\Lake Whatcom W&S District\Reservoir Seismic VA 2015\Structural Calculations\Division 30\Site Class B\Divi

$offset\_upper := 6 \cdot in$ $offset\_lower := 2.5 \cdot in$ $b_{ftg} = 1.5\,ft$ $b_{ftg\_lower} = 20.5 \cdot in$

$R_{ftg} = 13.21\,ft$ $R_{ftg\_lower} = 13.418\,ft$ $R_{in} = 11.71\,ft$ footing inside radius (presumed)

$h_{ftg} = 2.667\,ft$ $h_{ftg\_lower} = 0.667\,ft$ $R_{ftg} = 13.21\,ft$



Existing ringwall and tank dimensions

Existing footing

$A_{ftg} = 117.433\ ft^2$

$A_{ftg\_lower} = 134.861\ ft^2$

Additional exterior ring

$h_{ring} := 10 \cdot ft$ Ring depth

$b_{ring} := 8 \cdot ft$ Ring width at bottom

$R_{ring} := R_{ftg\_lower} + b_{ring} = 21.418\ ft$

Added ring dead load

$cy := yd^3$

$$V_{ring} := 2 \cdot \int_0^{\pi} \int_{R_{ftg\_lower}}^{R_{ring}} \int_0^{h_{ftg\_lower}} r\, dz\, dr\, d\phi \ldots = 119.866 \cdot cy$$
$$+ \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\, dz\, dr\, d\phi \right) + 2 \cdot \int_0^{\pi} \int_{R}^{R_{ftg}} \int_0^{h_{ring} - h_{ftg} - h_{ftg\_lower}} r\, dz\, dr\, d\phi$$

$$V_{ring\_lower} := 2 \cdot \int_0^{\pi} \int_{R_{ftg\_lower}}^{R_{ring}} \int_0^{h_{ftg\_lower}} r\, dz\, dr\, d\phi \ldots = 109.813 \cdot cy$$
$$+ \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r\, dz\, dr\, d\phi \right)$$

$$W_{ring} := \left(V_{ring}\right) \cdot \gamma_{conc} = 4.855 \times 10^5\ lbf$$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 6079 \cdot plf$$

Anchor ring weight per ft of shell

Check overturning stability safety factor

$$SF_{overturning} := (1 - A_v) \cdot (W_r + W_s + W_{ftg} + W_{water} + W_{ring}) \cdot \frac{R}{M_{s\_rev}} = 2.007$$

OK

Calculate the required shear transfer capacity between footing and new anchor ring per foot of shell

$$Uplift := 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 8.763 \cdot klf$$

Transfer force at face of shell

The resistance available along the perimeter is

$$Resistance := (1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water} + w_{ring}) + w_{soil} - F_{max} = 8.869 \cdot klf$$

Check resistance/uplift safety factor with added block

$$Resistance\_ratio := \frac{Resistance}{Uplift} = 1.012$$

> 1.0 OK

The load to be transfered by the shell to the new ringwall is $Stud\_load := Uplift = 8.763 \cdot klf$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$$Dowel\_load := (w_{water} + w_{ftg} + F_{max}) = 3.158 \cdot klf$$

$\Omega_o := 2.0$ From Ref 3, Table 15.4-2, for tanks the overstrength factor

Stud design

$s_{studs} := 30 \cdot in$ horizontal stud spacing

Try $s_{studs\_vert} := 20 \cdot in$

$$n_{studs\_per\_row} := \frac{(h_{ring} - h_{ftg} - h_{ftg\_lower})}{s_{studs\_vert}} = 4$$

try

$$n_{studs\_per\_row} := 6 \qquad s_{studs\_vert} := \frac{(h_{ring} - h_{ftg})}{n_{studs\_per\_row}} = 14.667 \cdot in$$

$$\text{Load\_per\_stud} := s_{studs} \cdot \frac{\text{Stud\_load}}{n_{studs\_per\_row}} = 3651 \cdot \text{lbf}$$

$$V_u := \Omega_o \cdot 1.4 \cdot \text{Load\_per\_stud} = 10224\,\text{lbf}$$

Shear strength for a 5/8" Nelson stud is $Q_N := 15113 \cdot \text{lbf}$ per AISC for f'c=4.5 ksi, Fu=65 ksi

$$\phi_{shear} := .90 \qquad \frac{V_u}{\phi_{shear} \cdot Q_N} = 0.752$$

$$l_{stud} := 8 \cdot \text{in} \qquad d_{stud} := .625 \cdot \text{in}$$

$$f'_c := 4.5 \cdot \text{ksi} \qquad \frac{V_u}{l_{stud} \cdot d_{stud}} = 2.045 \cdot \text{ksi}$$

$$DCR := \frac{V_u}{.85 \cdot f'_c \, l_{stud} \cdot d_{stud}} = 0.535$$ OK for crushing

Dowel Design

$s_{dowels} := 19 \cdot \text{in}$ horizontal stud spacing

$n_{dowels\_per\_row} := 3$

$\text{Dowel\_load} = 3.158 \cdot \text{klf}$

$$s_{dowels\_vert} := \frac{h_{ftg}}{n_{dowels\_per\_row} + 1} = 0.667\,\text{ft}$$

$$\text{Load\_per\_dowel} := s_{dowels} \cdot \frac{\text{Dowel\_load}}{n_{dowels\_per\_row}} = 1667 \cdot \text{lbf}$$

$$V_u := \Omega_o \cdot 1.4 \cdot \text{Load\_per\_dowel} = 4667\,\text{lbf}$$

for a #6 Grade 60 dowel, Hilti HIT-RE 500 adhesive in shear $V_{sa} := 15840 \cdot \text{lbf}$

$$DCR := \frac{V_u}{.60 \cdot V_{sa}} = 0.491$$ < 1 OK

Quantities

$$N_{studs} := n_{studs\_per\_row} \cdot \pi \cdot \frac{D}{s_{studs}} = 192$$

$$N_{dowels} := n_{dowels\_per\_row} \cdot \pi \cdot \frac{D}{s_{dowels}} = 151$$

$$W_{ring} = 485.457 \cdot kip \qquad V_{ring} := \frac{W_{ring}}{\gamma_{conc}} = 119.866 \cdot cy$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg\_lower}{}^2 = 1157\,ft^2 \qquad R_{exc} := R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 12.875\,ft$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg\_lower} + h_{ftg}\right)^2 - \pi \cdot R_{ftg}{}^2 = 1700\,ft^2$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg} + h_{ftg\_lower}}{2}\right)^2 - \pi \cdot R_{ftg}{}^2 = 1429\,ft^2$$

$$V_{exc} := \frac{h_{ftg} + h_{ftg\_lower}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 353 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - V_{ring\_lower} = 243 \cdot cy$$

Seismic Evaluation

for

Division 30 Reservoir Option C

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Job No.:15-10420.00 LWWSD
Division 30 Reservoir
Sheet No.: 3 of 15
Calculated by: JJL Date: 2/4/2016
Checked by: Date:_______

Job No.:15-10420.00 LWWSD
Division 30 Reservoir
Sheet No.: 4 of 15
Calculated by: JJL   Date: 2/4/2016
Checked by: Date:_______

Preliminary Design of Anchored Tank

General layout similar to Sumner Springs Reservoir shown below

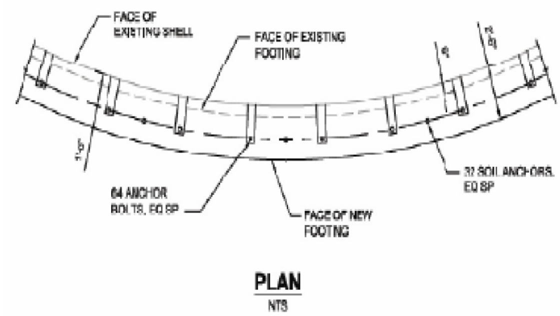


Figure 5. Enlarged Foundation Plan Detail

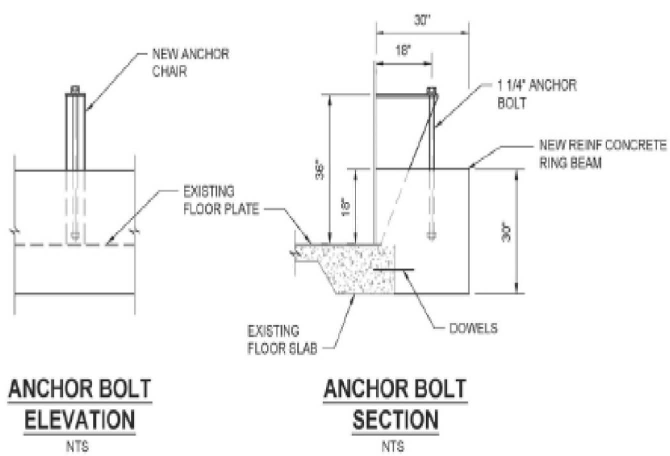
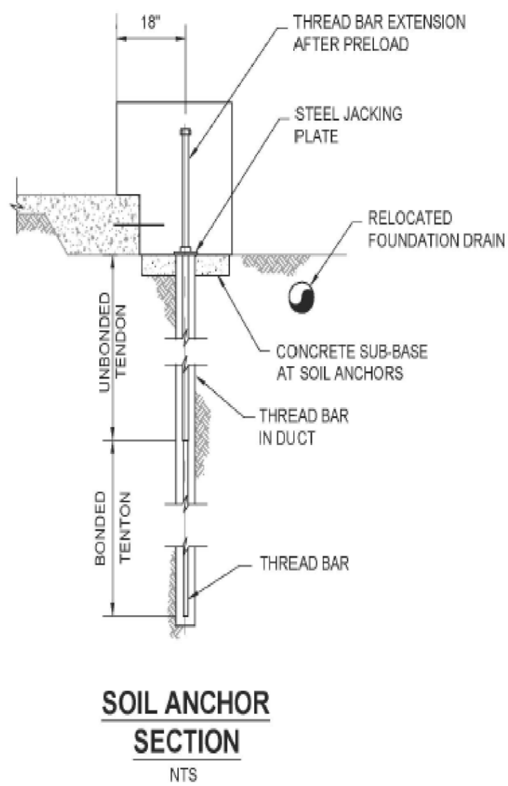


Figure 6. Anchor Bolt and Chair Detail



$R = 12.71\,\text{ft}$

Supplemental units and unit weights

$cy := yd^3$

$offset\_upper := 6 \cdot in \qquad offset\_lower := 2.5 \cdot in$

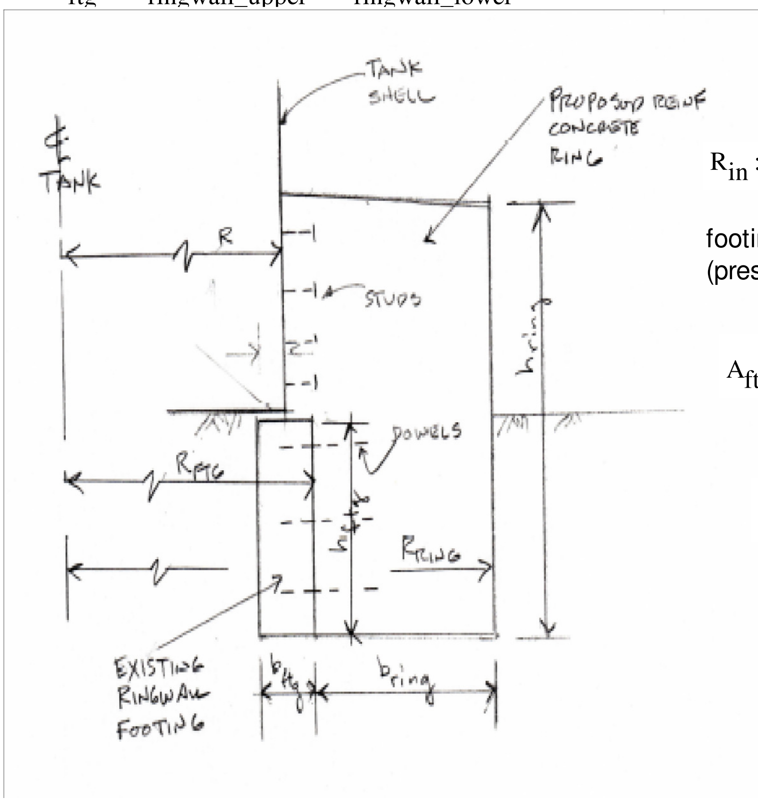
$R_{ftg\_upper} := R + offset\_upper = 13.21\,ft$

$b_{ftg\_upper} := b_{ringwall} = 2\,ft$

$b_{ftg\_lower} := b_{ftg\_upper} + offset\_lower$

$h_{ringwall\_upper} := 32\cdot in$ $h_{ringwall\_lower} := 26.5\cdot in$ $R_{ftg\_lower} := R_{ftg\_upper} + offset\_lower$

$h_{ftg} := h_{ringwall\_upper} + h_{ringwall\_lower} = 4.875\,ft$



Existing ringwall and tank dimensions

Existing footing

$R_{in} := R_{ftg\_upper} - b_{ftg\_upper} = 11.21\,ft$

footing inside radius (presumed)

$A_{ftg\_upper} := \pi\cdot\left(R_{ftg\_upper}{}^2 - R_{in}{}^2\right)$

footprint

$A_{ftg\_lower} := \pi\cdot\left(R_{ftg\_lower}{}^2 - R_{in}{}^2\right)$

Additional exterior ring

$h_{ring} := h_{ftg}$ Ring depth

$b_{ring} := 30\cdot in$ Ring width at bottom

$R_{ring\_outer} := R_{ftg\_lower} + b_{ring} = 15.918\,ft$

$A_{if\_ftg} := \pi\cdot R_{in}{}^2$ $A_{if\_ftg} = 394.785\,ft^2$

$A_{of\_ftg\_upper} := \pi\cdot R_{ftg\_upper}{}^2 - A_{if\_ftg}$ $A_{of\_ftg\_upper} = 153.435\,ft^2$

$A_{of\_ftg\_lower} := \pi\cdot R_{ftg\_lower}{}^2 - A_{if\_ftg}$ $A_{of\_ftg\_lower} = 170.864\,ft^2$

$R_{ring\_lower} := R_{ftg\_lower} + b_{ring}$ $R_{ring\_lower} = 15.918\,ft$

$R_{ring\_upper} := R_{ring\_lower} - offset\_lower \qquad R_{ring\_upper} = 15.71\,ft$

$R_{ring\_tank} := R_{ring\_upper} - offset\_upper$

a. Dead Load Component from shell, roof supported on shell

$P_{static} := P_D(0) \qquad P_{static} = 452\cdot plf$ Dead load, constant for all values of φ

b. Seismic Component from shell and roof supported on shell

$$P_{seismic}(\varphi) := \cos(\varphi)\cdot\frac{(4\cdot M_s(0))}{\pi\cdot D^2}$$ Seismic load at base of shell from lateral ground motior

$P_{seismic}(0) = 9501\cdot plf$ Maximum value at toe of shell

$P_{seismic}(\pi) = -9501\cdot plf$ Minimum value (uplift) at heel of shell

$P_{seismic\_v} := .40\cdot A_v\cdot P_{static}$ Seismic load at base of shell from vertical ground motic

$P_{seismic\_v} = 16\cdot plf$

c. Existing footing Dead Load Component

Compute the concrete volume of the existing footing

$V_{ftg} := A_{ftg\_upper}\cdot h_{ringwall\_upper} + A_{ftg\_lower}\cdot h_{ringwall\_lower} = 29.129\cdot cy$

$W_{ftg} := \gamma_{conc}\cdot V_{ftg} = 117.973\cdot kip$

$$w_{ftg} := \frac{W_{ftg}}{2\cdot\pi\cdot R} = 1477\cdot plf$$ Ringwall weight per ft of shell

d. Added ring dead load

$$V_{ring\_lower} := \pi\cdot\left(R_{ring\_lower}^{\;2} - R_{ftg\_lower}^{\;2}\right)\cdot h_{ringwall\_lower}$$

$$V_{ring\_upper} := \pi \cdot \left(R_{ring\_upper}^{\ 2} - R_{ftg\_upper}^{\ 2}\right) \cdot h_{ringwall\_upper}$$

$$V_{ring} := V_{ring\_lower} + V_{ring\_upper} = 41 \cdot cy$$

$$W_{ring} := V_{ring} \cdot \gamma_{conc} \quad W_{ring} = 167 \cdot kip$$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 2093 \cdot plf$$ Anchor ring weight per ft of shell

e. Weight of water over footing

$$p_{static} := \gamma_{water} \cdot H = 2452 \cdot psf$$

$$w_{water} := p_{static} \cdot \frac{\pi \cdot \left(R^2 - R_{in}^{\ 2}\right)}{2 \cdot \pi \cdot R} \qquad w_{water} = 3461 \cdot plf$$

f. Seismic pressure increase/decrease on footing

$$w_{water} = 3461 \cdot plf$$

(base pressure functions hidden below for brevity)

$$\Delta p := p_{base}(R, 0) = 584 \cdot psf$$ Plus or minus water pressure at the toe or heel of the tank due to seismic effects

$$w_{seismic} := \int_{\frac{-\theta_1}{2}}^{\frac{\theta_1}{2}} \int_{R_{in}}^{R} p_{base}(r, \phi) \cdot \frac{r}{ft} \, dr \, d\phi \qquad w_{seismic} = 123.028 \cdot plf$$

Calculate the required anchor transfer capacity between tank and new anchor ring per foot of shell

$SF_{ot} := 1.67$ target safety factor

$Uplift := P_{seismic}(0)$ $\quad Uplift = 9.501 \cdot klf$ Transfer force at face of shell

The resistance of various components is

$$D_{tank\_resist} := P_{static} \cdot \left(1 - .4 \cdot A_v\right) = 0.437 \cdot klf$$

$w_{water\_resist} := (1 - .4 \cdot A_v) \cdot w_{water} - w_{seismic} = 3.216 \cdot klf$

Set number of anchors and compute load. Assume three new anchors between each of the 12 existing

$n_{anchors} := 36 \qquad s_{anchor} := \pi \cdot \frac{D}{n_{anchors}} = 2.218\,ft$

$T_{anchor} := \frac{\left[\pi \cdot D \cdot \left(Uplift - D_{tank\_resist} - w_{water\_resist}\right)\right]}{n_{anchors}} = 12.973 \cdot kip$ measured at the shell

Resistance provided by ring $w_{ring} = 2.093 \cdot klf$

Resistance required by ground anchors

$Ground\_anchor\_resist := SF_{ot} \cdot (Uplift) - D_{tank\_resist} - w_{water\_resist} - w_{ring} = 10.12 \cdot klf$

$ground\_anchor\_capacity\_ASD := 75 \cdot kip$

$n_{ground\_anchors} := 18$ provide one ground anchor for every two anchors

$ground\_anchor\_load := Ground\_anchor\_resist \cdot \pi \cdot \frac{D}{n_{ground\_anchors}} = 44.9 \cdot kip$

$s_{ground\_anchor} := \pi \cdot \frac{D}{n_{ground\_anchors}} = 4.437\,ft$

If the new ring picks up the weight of the existing ringwall and water resistance via dowel transfer, then

$Ring\_dowels := \left(w_{water} + w_{ftg}\right) = 4939 \cdot plf$

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$s_{dowels} := s_{anchor} = 2.218\,ft \qquad n_{dowels\_per\_row} := 3$

$Load\_per\_dowel := \frac{s_{dowels}}{s_{anchor}} \cdot \frac{T_{anchor}}{n_{dowels\_per\_row}} = 4324 \cdot lbf$

Half inch dowels should be more than enough $n_{dowels} := n_{anchors} \cdot n_{dowels\_per\_row} = 108$

Quantities

$n_{dowels} = 108$ $n_{anchors} = 36$ $n_{ground\_anchors} = 18$

$W_{ring} = 167.178 \cdot kip$ $V_{ring} := \dfrac{W_{ring}}{\gamma_{conc}} = 41.279 \cdot cy$

By compariison to Sumner Springs reservoir, assume reinforcement at $steel\_unit := 210 \cdot \dfrac{lbf}{cy}$

$rebar := V_{ring} \cdot steel\_unit = 8668\,lbf$

$R_{ftg} :=$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring\_outer} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg\_lower}^{\ 2} = 443\,ft^2$$

$$R_{exc} := R_{ring\_outer} + 2 \cdot ft + h_{ftg} - R_{ftg\_upper} = 9.583\,ft$$

$$A_{top} := \pi \cdot \left(R_{ring\_outer} + 2 \cdot ft + h_{ringwall\_lower} + h_{ringwall\_upper}\right)^2 - \pi \cdot R_{ftg\_upper}^{\ 2} = 1084\,ft^2$$

$$A_{mid} := \pi \cdot \left(R_{ring\_outer} + 2 \cdot ft + \frac{h_{ringwall\_upper} + h_{ringwall\_lower}}{2}\right)^2 - \pi \cdot R_{ftg\_upper}^{\ 2} = 754\,ft^2$$

$$V_{exc} := \frac{h_{ringwall\_upper} + h_{ringwall\_lower}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 273 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - V_{ring\_lower} - V_{ring\_upper} = 232.027 \cdot cy$$

Anchor Bolt Sizing

Assume A36 anchor bolts $F_y := 36 \cdot ksi$ $F_u := 58 \cdot ksi$

$F_{anchor} := \min(.80 \cdot 36 \cdot ksi, .50 \cdot 58 \cdot ksi) = 28.8 \cdot ksi$ Allowable seismic load stress on anchors per Ref 5 section 3.3.3.2

$A_{root\_min} := \dfrac{T_{anchor}}{F_{anchor}} = 0.45 \cdot in^2$ $d_{root\_calc} := \sqrt{\dfrac{4}{\pi} \cdot A_{root\_min}} = 0.757 \cdot in$

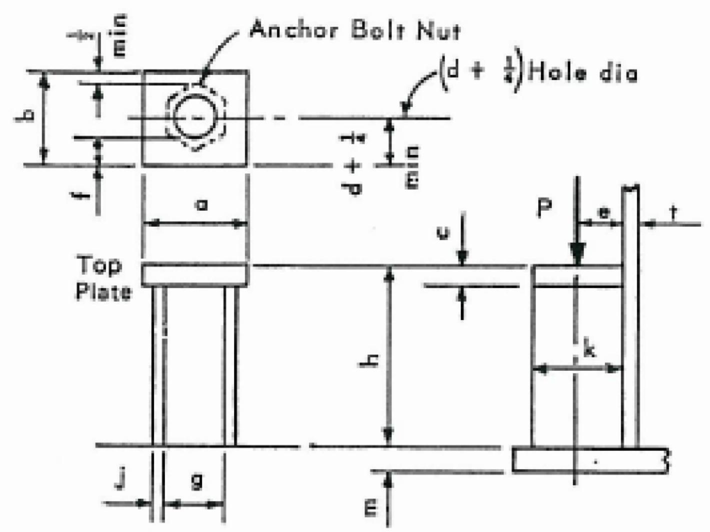
Per Ref 5, 3.8.5.1, add a .25" corrosion allowance to the root diameter for bolts less than 1.25", and use not less than a 1" bolt. This makes an 1.25" bolt the practical minimum

Bolt Dia (in)	Root Dia (in)	Root Area (in^2)	Gross Area (in^2)	Root Dia + .25" (in)	Min Bolt Dia (in)
1.000	0.865	0.587	0.785	1.115	1.375
1.125	0.970	0.74	0.994	1.220	1.500
1.250	1.100	0.942	1.23		1.250
1.375	1.190	1.12	1.49		1.375
1.500	1.320	1.37	1.77		1.500
1.750	1.530	1.85	2.41		1.750
2.000	1.760	2.43	3.14		2.000

$d := 1.25 \cdot in$ anchor diameter $A_{bolt} := \pi \cdot \dfrac{d^2}{4} = 1.227 \cdot in^2$ gross area of bolt

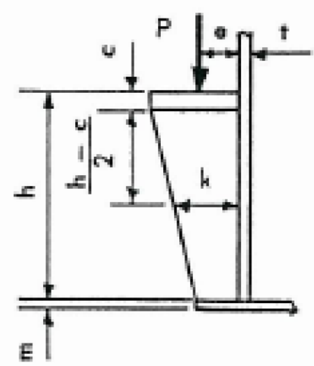
Anchor Chair Design

Methodology is from Ref 11, Part VII - Anchor Bolt Chairs



(a) Typical Plan & Outside Views

(b) Vertical Column or Skirt



(c) Flat Bottom Tank

$e := 16 \cdot in$ bolt centerline distance from shell

Minimum bolt hole size per Ref 11 is

Oversized hole size per Ref 18 Table J.3.3 is $d + \frac{5}{16} \cdot in = 1.563 \cdot in$ for bolts >= 1.25 in. Use

$d_{hole} := d + \frac{5}{16} \cdot in$ $d_{hole} = 1.563 \cdot in$

Edge distance per Ref 10 Tables J.3.4 and J3.5 (from center of hole) is

$c_{edge} := 2.25 \cdot in + \frac{1}{8} \cdot in = 2.375 \cdot in$

$b := e + c_{edge} = 18.375 \cdot in$

$f := c_{edge} - \frac{d_{hole}}{2} = 1.594 \cdot in$

$g := d + 1 \cdot in = 2.25 \cdot in$ minimum side plate separation recommended by Ref 21, however this is very tight for seal welding on interior of plates. Increase this dimension to

$g := 8 \cdot in$

$t := t_s(0)$ $t = 0.25 \cdot in$ Shell bottom course thickness

$P := T_{anchor} = 12.973 \cdot kip$

$S := 1.33 \cdot 15 \cdot ksi = 19.95 \cdot ksi$ Ref 4 allowable stress < 25 ksi recommended by Ref 11 OK

Compute top plate thickness

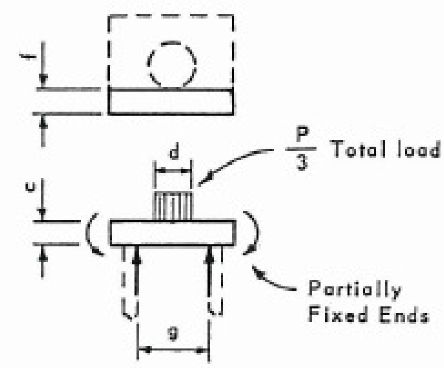


Figure 7-2. Assumed Top-Plate Beam.

$$c_{min} := \left[\frac{P}{S \cdot f} \cdot (0.37 \cdot g - 0.22 \cdot d)\right]^{.5} = 1.047 \cdot in$$

use $c := 1.5 \cdot in$

top plate thickness

$h := 40 \cdot in$

$j_{min} := \max[.5 \cdot in, 0.04 \cdot (h - c)] = 1.54 \cdot in$ use $j := 1.5in$

$m := .25 \cdot in$ bottom plate thickness assumption $proj := 2 \cdot in - t$ bottom plate projection from shell face

$a := g + 2 \cdot j + .5 \cdot in = 11.5 \cdot in$ $> \; 2 \cdot c_{edge} = 4.75 \cdot in$ OK   Use $a := s_{anchor} = 2.218\,ft$

Assumes a continuous top plate,full circumference

Recess the side plate not more than 1/2" from front edge of top plate per Ref 21. Use .25" to allow seal weld at front edge.

$plate\_top := b - .25 \cdot in$ $k := \dfrac{(plate\_top + proj)}{2} = 9.938 \cdot in$ mean side plate width

$$\frac{j \cdot k}{\dfrac{P \cdot in^2}{25 \cdot kip}} = 28.726$$ > 1.0 OK per Ref 21

Compute reduction factor Z for local stress check

$$Z := \frac{1.0}{\frac{(.177 \cdot a \cdot m)}{in\sqrt{R \cdot t}} \cdot \left(\frac{m}{t}\right)^2 + 1.0} = 0.84$$

$$S := \frac{P \cdot e}{in\, t^2} \cdot \left[\frac{1.32 \cdot Z}{\frac{1.43 \cdot a \cdot h^2}{R \cdot t \cdot in} + \left(4 \cdot \frac{a}{in^3} \cdot h^2\right)^{.333}} + \frac{.031 \cdot in}{\sqrt{R \cdot t}} \right]$$

$S = 18.9 \cdot ksi$ localized vertical shell stress just above the chair. Ref 21 recommends 25ksi max.

close enuf for preliminary estimate

Weld Design

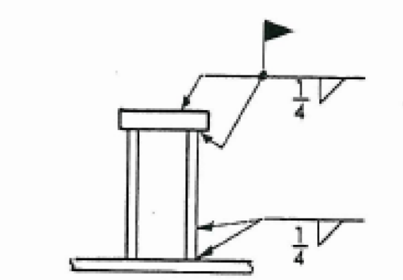


Figure 7-4. Typical Welding, Base Plate Shop Attached.

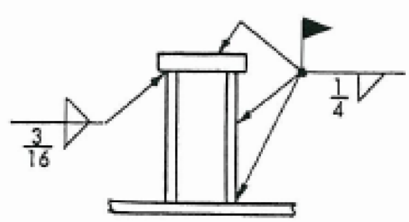


Figure 7-5. Typical Welding, Base or Bottom Field Attached.

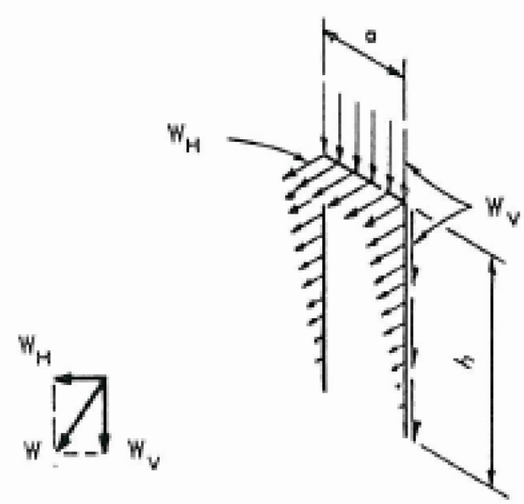


Figure 7-6. Loads on Welds.

$$W_v := \frac{P}{a + 2 \cdot h} = 122 \cdot \frac{lbf}{in} \qquad W_h := \frac{P \cdot e}{a \cdot h + 0.667 \cdot h^2} = 97 \cdot \frac{lbf}{in}$$

$$W := \sqrt{W_v^2 + W_h^2} = 156 \cdot \frac{lbf}{in}$$

By inspection, a .25" weld will be more than adequate.

Shell shear capacity per inch exceeds weld, OK

Anchor Quantities

$V_{bp} := a \cdot b \cdot c$ $\qquad V_{bp} = 733.707 \cdot in^3$

$V_{sp} := 2 \cdot \frac{(b + 2 \cdot in) \cdot (h - c) \cdot j}{2}$ $\qquad V_{sp} = 1.177 \times 10^3 \cdot in^3$

$W_{anchor} := \gamma_{steel} \cdot (V_{bp} + V_{sp}) = 541.712\ lbf$

$W_{anchor\_total} := W_{anchor} \cdot n_{anchors} = 19502\ lbf$

$L_{weld} := 2 \cdot h + a + (a - g - 2 \cdot j) = 122.24 \cdot in$

$L_{weld\_total} := n_{anchors} \cdot L_{weld} = 367\ ft$

APPENDIX B.5

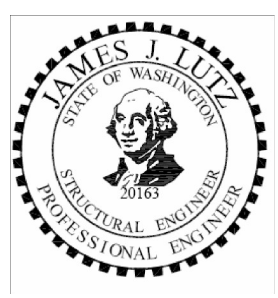
SVWTP RESERVOIR CALCULATIONS

Seismic Evaluation
for
SVWTP Reservoir

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

Calculation Index



Methodology Remarks

These calculations are limited to an assessment of the primary elements of the lateral force resisting system for the reservoir under seismic loading. Following is a summary of the methodology used:

1. All dimensions and weights are based on record drawings furnished by the client, supplemented by field measurements.In case of discrepancies, field measurements were used..

2. Water level assumed for seismic calculations is based on maximum current operating level provided by the District..

3. Methodology for determination of seismic loads for tanks with a free water surface is based on the 2012 International Building Code, ASCE 7-10, and AWWA Standard D100-11. These codes and standards post-date and are more stringent than codes and standards used at the time of original tank design.

4. For tanks where the free surface sloshing wave amplitude exceeds the roof elevation, the additional amplification of seismic load is based on an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave. The force is modeled by computing an increase in mass and adjusting the convective period of the water mass. The pressure distribution is assumed the same as for a tank with a free water surface.

5. For tanks where the static water surface level already contacts the roof, the free surface sloshing amplitude is based on a cylinder of the same height and radius with zero freeboard, however the actual water mass is assumed. The ratio of sloshing amplitude to roof height is computed using roof height measured from the free water surface. Adjustments in seismic load are otherwise the same as for the preceding step.

6. Ground motion spectral accelerations S_S and S_1 are those currently available from the USGS on their web site calculator for the latitude and longitude of the tank as taken from Google Earth.

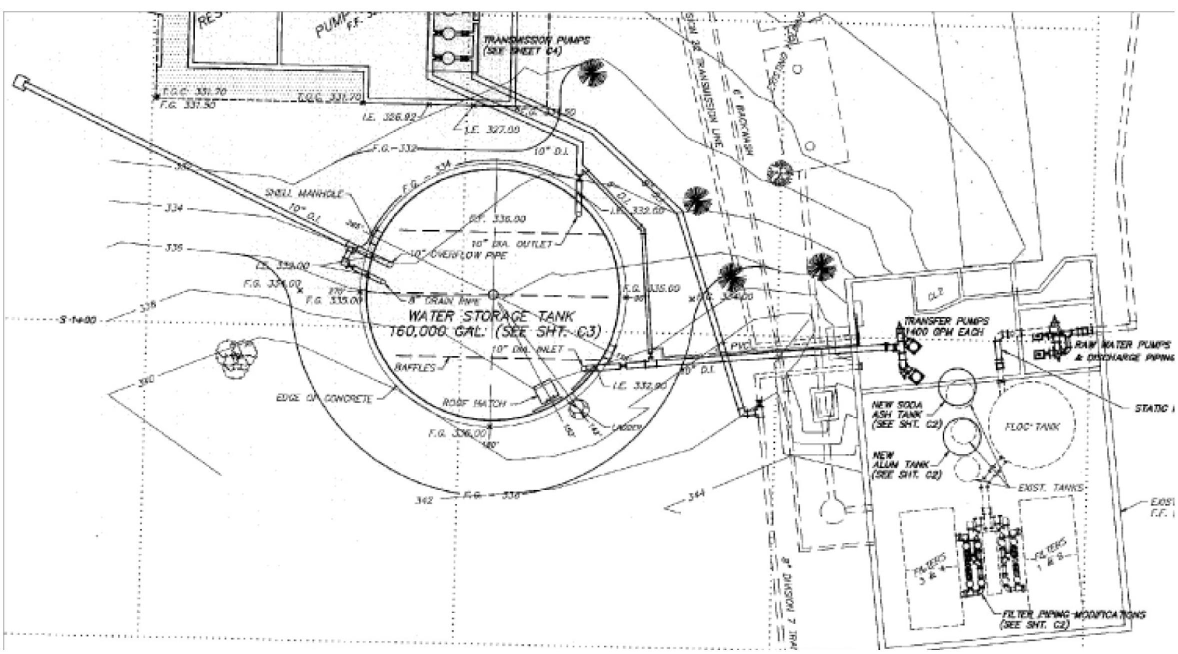
7. Soil site class "D" is assumed as a default in the absence of a soils report for this reservoir..

8. Wind loads, hydrostatic loads at overflow elevation, and roof live loads were not considered in the analysis. However where calculated roof loads exceed 40 psf, a mass equal to .20 times the uniform roof snow load is added to the roof mass for seismic calculations. The gravity effects of snow load were considered whete applicable for determining loads on the shell, however no analysis of roof members was included.

Location and Site Data

Lat 48.7169, Long -122.3172
El 335
(Google Earth)

No soils report available. Record drawings by Reliable Steel dated 1993.



Superstructure Geometry

From record drawings

Tank diameter $D := 40 \cdot ft$

Tank radius $R := \frac{D}{2} = 20\,ft$

Shell height $H_s := 25 \cdot ft$

Floor elevation at shell
(Bottom capacity level)

$BCL := 344.5 \cdot ft$ (District)

Overflow height above floor

$h_{overflow} := 24ft$

Overflow elevation
(Top capacity level)

$h_{baffles} := 24 \cdot ft$

$TCL := BCL + h_{overflow}$

$H := 22 \cdot ft$ Maximum operating level

$NOL := BCL + H = 366.5\,ft$

$BCL + H_s = 369.5\,ft$

This level is below the top of the shell.

Describe the roof geometry

$$roof\_slope := \frac{0.75}{12} = 0.063$$

The roof height is $h_r := roof\_slope \cdot R = 1.25\,ft$

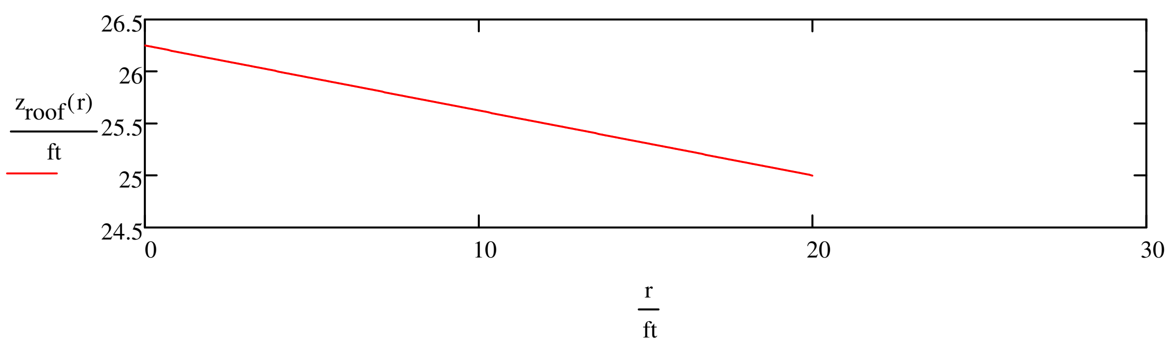
Let "z" be the distance measured vertically from the floor, and "r" the horizontal distance from the center

$$z_{apex} := H_s + h_r = 26.25\,ft$$

The expression for z for the roof for $0 < r < R$ is

$$z_{roof}(r) := \left(if\left(r > R, 0, z_{apex} - roof\_slope \cdot r\right)\right)$$

Plot the roof elevation vs radius $r := 0, .1 \cdot ft .. R$



Enter shell and roof plate thickness.

Mathcad General Input - See Appendix for Mathcad nomenclature and symbols

$ORIGIN := 1$

Special unit definitions $each := 1$ $sf := ft^2$

number of shell plate courses,
numbering starting with the base as
course 1

$n_{course} := 3$ (the vertical leg of the top angle is included with the top shell plate course)

Calculate the elevation of the top of each shell course relative to the floor

$i := 1, 2 .. n_{course}$ i is the number of each shell course, starting from the bottom

$\gamma_{steel} := 490 \cdot pcf$ unit weight of steel

zshell is the elevation of the top of each course relative to the top of the bottom plate

$$z_{shell} := \begin{pmatrix} 8.33 \\ 16.67 \\ 25 \end{pmatrix} \cdot ft \qquad t_{shell} := \begin{pmatrix} \frac{3}{16} \\ \frac{3}{16} \\ \frac{3}{16} \end{pmatrix} \cdot in \qquad w_{shell} := t_{shell} \cdot \gamma_{steel} = \begin{pmatrix} 7.656 \\ 7.656 \\ 7.656 \end{pmatrix} \cdot psf \qquad class_{shell} := \begin{pmatrix} 2 \\ 2 \\ 2 \end{pmatrix}$$

Shell course heights and thicknesses per record drawings, spot checked with field measurements. Shop drawings call out all plate as ASTM A36.

Note 3/16" minimum shell plate permitted by AWWA for tanks under 50 ft in diameter

Class 1 material has a yield stress 27 ksi < Fy < 34 ksi. Class 2 material has a yield stress Fy > 34 ksi

Roof thickness is 3/16" per nameplate, but thickness gauge measurements were .120". Use 3/16" to be conservative for roof weight calcualtions.

$t_{roof\_plate} := \frac{3}{16} \cdot in$ roof plate thickness

Compute weight of roof and shell

Define the roof slope at any point

$$z'_{roof}(r) := \frac{d}{dr} z_{roof}(r)$$

Compute the surface area of the roof plate tributary to the perimeter and the center column. . Ignore laps

For a surface of revolution, the general equation for the surface area is

$$A := 2 \cdot \pi \cdot \int r \, ds \qquad \text{where} \qquad ds := \sqrt{1 + \left(\frac{dz}{dr}\right)^2} \cdot dr$$

$$A_{roof\_plate} := 2 \cdot \pi \cdot \left(\int_0^R r \cdot \sqrt{1 + z'_{roof}(r)^2} \, dr \right) = 1259 \, ft^2 \text{(roof surface area)}$$

$$W_{roof\_plate} := \gamma_{steel} \cdot t_{roof\_plate} \cdot A_{roof\_plate} = 9.64 \cdot kip$$

$$A_{roof\_plate\_center} := 2\cdot\pi\cdot\left(\int_0^{\frac{R}{2}} r\cdot\sqrt{1 + z'_{roof}(r)^2}\,dr\right) = 315\,ft^2$$

$$W_{roof\_plate\_center} := \gamma_{steel}\cdot t_{roof\_plate}\cdot A_{roof\_plate\_center} = 2.41\cdot kip$$

Portion of roof weight tributary to center column

$$A_{roof\_plate\_edge} := 2\cdot\pi\cdot\left(\int_{\frac{R}{2}}^{R} r\cdot\sqrt{1 + z'_{roof}(r)^2}\,dr\right) = 944\,ft^2$$

$$W_{roof\_plate\_edge} := \gamma_{steel}\cdot t_{roof\_plate}\cdot A_{roof\_plate\_edge} = 7.23\cdot kip$$

Portion of roof weight tributary to shell

Calculate the vertical center of gravity from the tank floor for the roof plate

$$x_{cg} := 2\cdot\pi\cdot\frac{\left(\int_0^R r^2\cdot\sqrt{1 + z'_{roof}(r)^2}\,dr\right)}{A_{roof\_plate}} = 13\,ft$$

$$X_{roof\_plate} := z_{roof}\left(x_{cg}\right) = 25.417\,ft$$

Define the number of the shell course for any value of 0 < z < Hs using a series of functions

$i_{course}(z) := n_{course}$ Default value

$$i_{course}(z) := if\left(z < z_{shell_{n_{course}}}, n_{course}, i_{course}(z)\right)$$

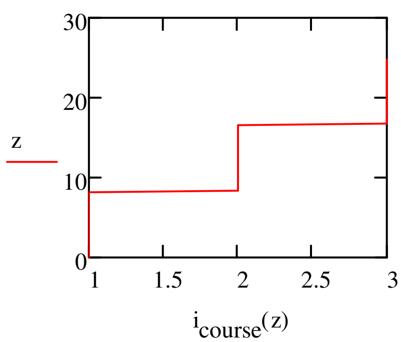
$$i_{course}(z) := if\left(z < z_{shell_4}, 4, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_3}, 3, i_{course}(z)\right)$$

$$i_{course}(z) := if\left(z < z_{shell_2}, 2, i_{course}(z)\right)$$

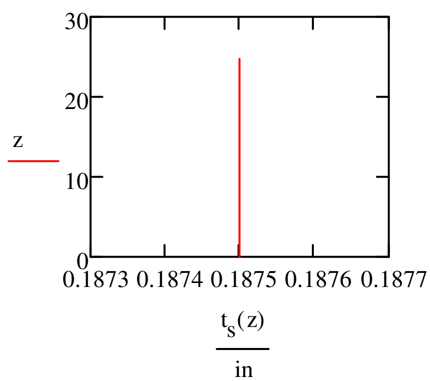
$i_{course}(z) := \text{if}\left(z < z_{shell_1}, 1, i_{course}(z)\right)$

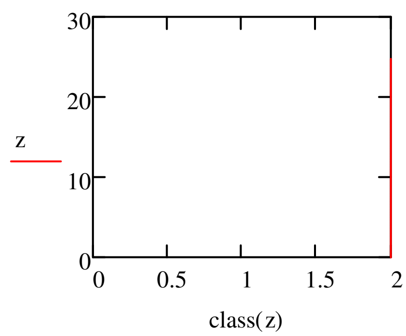
$z := 0 \cdot ft, 0.2 \cdot ft .. H_s$ Set plotting interval for graphs



write functions that return the shell plate thickness and class as a function of height above the base

$t_s(z) := t_{shell_{i_{course}(z)}}$ $class(z) := class_{shell_{i_{course}(z)}}$





<u>Shell thickness vs elevation</u> **<u>Shell class vs elevation</u>**

Floor plate thickness $t_{floor} := .25 \cdot in$

$floor\_flange := 1.5 \cdot in$ Bottom plate projection beyond shell plate $D_{floor} := D + 2 \cdot floor\_flange$

<u>Compute floor weight</u>

$W_f := \gamma_{steel} \cdot t_{floor} \cdot \pi \cdot \frac{D_{floor}^2}{4}$ $W_f = 13 \cdot kip$

<u>Compute the weight of the shell and establish its center of gravity from the base</u>

$$W_s := \pi \cdot D \cdot \int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z)\, dz \qquad W_s = 24.053 \cdot kip$$

$$X_s := \pi \cdot D \cdot \frac{\int_{0 \cdot ft}^{H_s} \gamma_{steel} \cdot t_s(z) \cdot z\, dz}{W_s} \qquad X_s = 12.5\,ft$$

Compute the weight of the roof and establish its center of gravity from the base

The total roof mass is a combination of the part tributary to the center column and the part tributary to the edge. The center portion includes part of the roof, half the weight of the rafters, the column cap, and half of the column. (The other half of the column and its base plate are assigned to the floor mass). The edge portion includes part of the roof, half the weight of the rafters, clips and the flange of the top angle. The weight of top angle and clips and top angle flange are ignored.

Based on record drawings, there are 20 each rafters C6X8.2 shapes, about 20 ft long. Column cap is 1" x 30 inch dia. Center pipe column is 6" diameter, Std (Sched 40), 25'-6-1/2" long. Ignore weight of clips, bolts, laps, and appurtenances..

$$W_{rafters} := 20 \cdot 8.2 \cdot \frac{lbf}{ft} \cdot (20 \cdot ft) = 3.28 \cdot kip$$

$$W_{col\_cap} := \pi\,(15 \cdot in)^2 \cdot 1.0 \cdot in \cdot \gamma_{steel} = 0.200 \cdot kip$$

$$W_{col} := 25.54 \cdot ft \cdot 19.6 \cdot \frac{lbf}{ft} = 0.501 \cdot kip$$

$$W_{col\_base} := \gamma_{steel} \cdot .75 \cdot in \cdot 42 \cdot in \cdot 42 \cdot in = 0.375 \cdot kip$$ square base plate

$$W_{roof\_center} := W_{roof\_plate\_center} + \frac{W_{rafters}}{2} + W_{col\_cap} + \frac{W_{col}}{2} = 4.501 \cdot kip$$ Roof weight tributary to center column

$$W_{roof\_edge} := W_{roof\_plate\_edge} + \frac{W_{rafters}}{2} = 8.87 \cdot kip$$ Roof weight tributary to top of shell

$$\Delta W_f := W_{col\_base} + \frac{W_{col}}{2} = 0.625 \cdot kip$$ Column and base plate tributary to floor

Total roof structure mass for seismic calculation $W_r := W_{roof\_center} + W_{roof\_edge} = 13.371 \cdot kip$

Check to see if roof snow load mass must be included per ASCE 7-10

$p_g := 26 \cdot psf$ from "Snow Load Analysis for Washington", 2nd ed, SEAW

$I_s := 1.20$ Snow load importance factor for risk category IV, ASCE 7-10

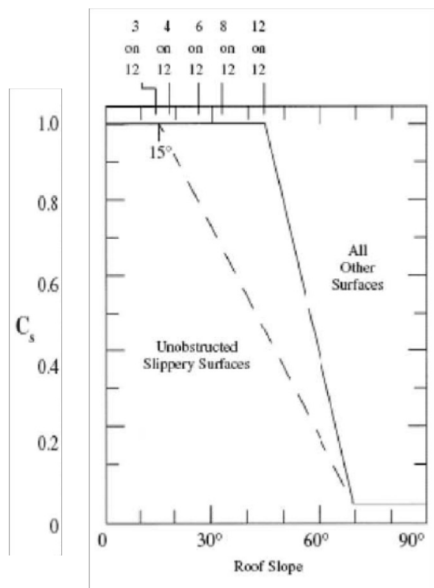
$C_e := 1.2$ ASCE 7-10, Table 7-2. Exposure Factor, Terrain B, Sheltered

$C_t := 1.2$ ASCE 7-10, Table 7-3, Thermal Factor, Unheated

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 31.45 \cdot psf$ Flat roof snow load, ASCE 7-10 Eq 7.3-1. Since flat roof snow load exceeds 30 psf, add 20% of the design snow load to the roof mass per ASCE 7-10, section 12.7.2.

The roof slope is $\text{atan}(roof\_slope) = 3.576 \cdot deg$

From ASCE 7-10 Fig 7-2c, the roof slope factor is



7-2c: Cold roofs with C = 1.2 or larger

$C_s := 1.0$

$p_s := C_s \cdot p_f = 31.45 \cdot psf$

Snow weight to include with roof weight

$w_{snow} := .20 \cdot p_s = 6.29 \cdot psf$

$W_{snow} := w_{snow} \cdot \pi \cdot R^2 = 7.904 \cdot kip$

Snow weight tributary to edge

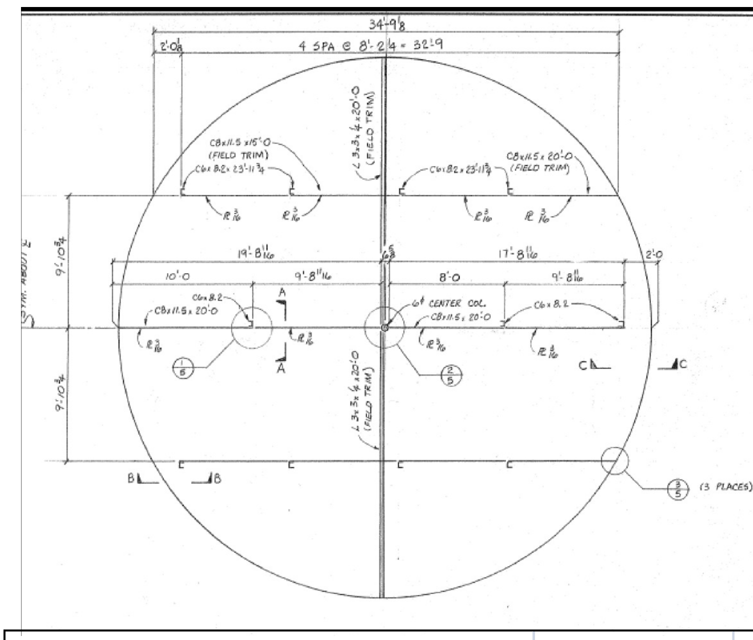
$$W_{snow\_shell} := W_{snow} \cdot \frac{A_{roof\_plate\_edge}}{A_{roof\_plate}} = 5.928 \cdot kip$$

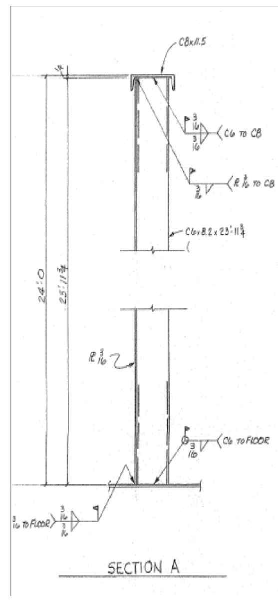
$$P_{snow} := \frac{W_{snow\_shell}}{\pi \cdot D} = 47.174 \cdot \frac{lbf}{ft}$$ Snow load applied at top of shell concurrent with seismic

Snow weight tributary to floor

$$W_{snow\_floor} := W_{snow} - W_{snow\_shell} = 1.976 \cdot kip$$

The tank is used for chlorine contact and has some interior baffles to create plug flow. The baffles consist of stiffened steel plate as hown in the attached figures, and contribute to the mass of the structure..





Item	Thickness (in)	Length (ft)	Height (ft)	Qty (ea)	Weight (lbs)
Steel plate	0.188	32.76	24	3	18107
Vertical stiffeners, C6X8.2		24		11	2165
Horiz stiffeners, top of baffle, C8X11.5		34.6		3	1194
Lateral top brace, L3X3X1/4 @ 4.9		20		2	3430
					24896

$W_{baffles} := 24.896 \cdot kip$ Assign entire mass to the floor. $\gamma_{water} := 62.4 \cdot pcf$ $X_b := 12 \cdot ft$

$$Displacement_{baffles} := \frac{W_{baffles}}{\gamma_{steel}} = 50.808 \cdot ft^3 \qquad \Delta W_T := \gamma_{water} \cdot Displacement_{baffles} = 3.17 \cdot kip$$

All the lateral resistance for the roof is assumed to be by the shell, except for the lower half of the column. The internal tank baffles provide very little stiffness against horizontal loads in the plane of the baffle and are ignored for purposes of evaluating tank lateral resistance.

Compute the center of gravity of the roof and column mass for seismic calculation

$$X_r := \frac{\left[W_{roof\_plate} \cdot X_{roof\_plate} \cdots + z_{apex} \cdot W_{col\_cap} + .75 \cdot z_{apex} \cdot \frac{W_{col}}{2} + W_{rafters} \cdot \left(H_s + \frac{h_r}{2}\right)\right]}{W_r} = 25.373\,ft$$

Compute the center of gravity of the roof snow load for seismic calculations

Snow density per ASCE 7-10 equation 7.7.1 is

$$\gamma_{snow} := \min\left(30 \cdot pcf, 0.13 \cdot \frac{p_g}{ft} + 30 \cdot pcf\right) = 30 \cdot pcf$$ snow depth $$h_d := \frac{w_{snow}}{\gamma_{snow}} = 0.21\,ft$$

$$X_{snow} := X_{roof\_plate} + \frac{h_d}{2} = 25.521\,ft$$ centroid of snow mass

Compute total water weight for seismic calculations

$$\gamma_{water} := 62.4 \cdot pcf$$

$$W_T := \gamma_{water} \cdot H \cdot \pi \cdot \frac{D^2}{4} = 1725.11 \cdot kip \qquad \frac{\Delta W_T}{W_T} = 0.002$$ Ignore deduction for baffle displacement

Calculate the impulsive and convective water weights and vertical centroids

$$\frac{D}{H} = 1.818$$ Assumed ground motion parallel to baffles, no impact on sloshing behaviour

$$W_i := W_T \cdot \frac{\tanh\left(.866 \cdot \frac{D}{H}\right)}{.866 \cdot \frac{D}{H}}$$ if D/H > 1.333

$$W_i := \text{if}\left[\frac{D}{H} < 1.333, W_T \cdot \left(1.0 - 0.218 \cdot \frac{D}{H}\right), W_i\right]$$ if D/H < 1.33

$$W_i = 1005.505 \cdot kip$$ Impulsive water weight $$\frac{W_i}{W_T} = 0.583$$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated or the total moment on the foundation, shell plus floor.

$$X_i := H \cdot \text{if}\left[\left(\frac{D}{H}\right) > 1.333, 0.375, 0.50 - 0.094 \cdot \frac{D}{H}\right] \qquad X_i = 8.25\,ft$$ centroid for calculation of just the shell moment

$$X_{imf} := 0.375\cdot\left[1.0 + 1.333\cdot\left(\frac{0.866\cdot\frac{D}{H}}{\tanh\left(0.866\cdot\frac{D}{H}\right)} - 1\right)\right]\cdot H$$

centroid for calculation of total bottom moment if D/H > 1.33

$$X_{imf} := \text{if}\left[\frac{D}{H} < 1.333, \left(0.50 + 0.06\cdot\frac{D}{H}\right)\cdot H, X_{imf}\right]$$

centroid for calculation of total bottom moment if D/H < 1.33

$$X_{imf} = 16.12\,\text{ft}$$

Compute convective water weight and effective centroid above the base

$$W_c := W_T\cdot\left(.230\cdot\frac{D}{H}\cdot\tanh\left(3.67\cdot\frac{H}{D}\right)\right) \qquad W_c = 696.39\cdot\text{kip} \qquad \frac{W_c}{W_T} = 0.404$$

Ref 4, Eq 13-26

$$X_c := H\cdot\left[1 - \frac{\cosh\left(3.67\cdot\frac{H}{D}\right) - 1}{3.67\cdot\left(\frac{H}{D}\right)\cdot\sinh\left(3.67\frac{H}{D}\right)}\right] \qquad X_c = 13.657\,\text{ft}$$

centroid for calculation of just the shell moment

$$X_{cmf} := H\cdot\left(1.0 - \frac{\cosh\left(3.67\cdot\frac{H}{D}\right) - 1.937}{3.67\cdot\frac{H}{D}\cdot\sinh\left(3.67\cdot\frac{H}{D}\right)}\right) \qquad X_{cmf} = 16.42\,\text{ft}$$

centroid for calculation of total bottom moment

Seismic Design Criteria

Importance Factor: $I_E := 1.50$ Risk category IV

Ground Motion Parameters

Site Class D   Default Site Class in absence of a geotechnical report

$S_S := .939$ $S_1 := .366$ Mapped earthquake short period and long period spectral accelerations. For Site Class B, 5% damping, expressed as fraction of g.

$F_a := 1.12$ $F_v := 1.66$ Site coefficients from 2012 IBC Table 1613.3.3(2). Seismic Design Category "D"

Adjusted maximum considered earthquake for site class

$S_{MS} := F_a \cdot S_S$ $S_{MS} = 1.052$

$S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.608$

Design spectral response parameters

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$ $S_{DS} = 0.701$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$ $S_{D1} = 0.405$

Compute points on the design response spectrum

$T_0 := 0.2 \cdot sec \cdot \frac{S_{D1}}{S_{DS}}$ $T_0 = 0.116 \cdot sec$

$T_S := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot sec$ $T_S = 0.578 \cdot sec$

$T_L := 6 \cdot sec$ Mapped value, ASCE 7-10, Figure 22-12

$T_L := if\left(T_L > 4.sec, 4.sec, T_L\right) = 4 \cdot sec$ Maximum required for tank sloshing wave calculations, ASCE 7-10, Section 15.7.6.1.d

$S_{ac}(T) := if\left(T > T_L, \frac{1.5 \cdot S_{D1} \cdot T_L \cdot sec}{T^2}, min\left(\frac{1.5 \cdot S_{D1} \cdot sec}{T}, 1.5 \cdot S_{DS}\right)\right)$ Convective acceleration function

$S_{ac}(T) := \text{if}\left(S_{ac}(T) > 1.5S_{DS}, 1.5S_{DS}, S_{ac}(T)\right)$ Upper bound for S_{ac} for low values of T

$$S_{ai}(T) := \text{if}\left(T > T_L, \frac{S_{D1} \cdot T_L \cdot \sec}{T^2}, \text{if}\left(T > T_S, \frac{S_{D1}}{T} \cdot \sec, S_{DS}\right)\right)$$

Impulsive acceleration function

Calculate Free Surface Wave Height and Compare to Freeboard Requirements

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{D}{3.68\cdot g\cdot \tanh\left(3.68\cdot\frac{H}{D}\right)}} \qquad T_c = 3.716\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \text{if}\left(SUG = 3, \text{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\,2}}\right), \text{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\,2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.163$

$d := 0.5\cdot D\cdot A_f = 3.27\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude.

freeboard $f := H_s - H = 3\,ft$

$\frac{d}{f} = 1.09$ > 1.0, therefore **freeboard is insufficient, but not by much**

Compute Base Shear and Overturning Moments As If Free Surface

$S_{ai} := S_{DS}$ $R_i := 3.0$ $R_c := 1.5$ AWWA D100-11, Table 28 and section 13.2.9.2. Anchored tank

$A_i := \max\left(\frac{S_{ai} \cdot I_E}{1.4 \cdot R_i}, \frac{0.36 \cdot S_1 \cdot I_E}{R_i}\right)$ $A_i = 0.25$ Impulsive design acceleration

$A_c := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c}$ $A_c = 0.117$ Convective design acceleration

Calculate overturning moment at the base of the shell

$$M_s := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{baffles} \cdot X_b + W_i \cdot X_i\right)\right]^2 + \left(A_c \cdot W_c \cdot X_c\right)^2} \qquad M_s = 2611 \cdot kip \cdot ft$$

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$$M_{mf} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_i \cdot X_{imf} + W_{baffles} \cdot X_b\right)\right]^2 + \left(A_c \cdot W_c \cdot X_{cmf}\right)^2} \qquad M_{mf} = 4545 \cdot kip \cdot ft$$

Calculate base shear at top of foundation

$$V_f := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{baffles} + W_{col\_base} + \frac{W_{col}}{2}\right) + W_i\right]\right]^2 + \left(A_c \cdot W_c\right)^2} \qquad V_f = 284.64 \cdot kip$$

The above base shears and moments are expressed in allowable stress design (ASD) basis.

Adjust Effective Masses for Roof Contact

The methodology for roof contact effects is an approximate method published in Structural Engineering International, March 2006. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" by Dr. Praveen K. Malhotra. This simplified method assumes a linear shape for the sloshing wave.

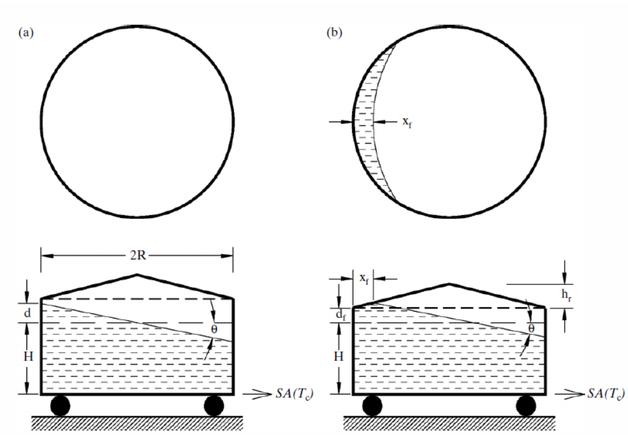
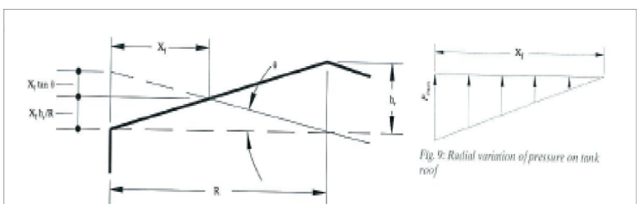


Fig. 5: Liquid-filled tank translating with an acceleration $SA(T_c)$: (a) sufficient freeboard; and (b) insufficient freeboard



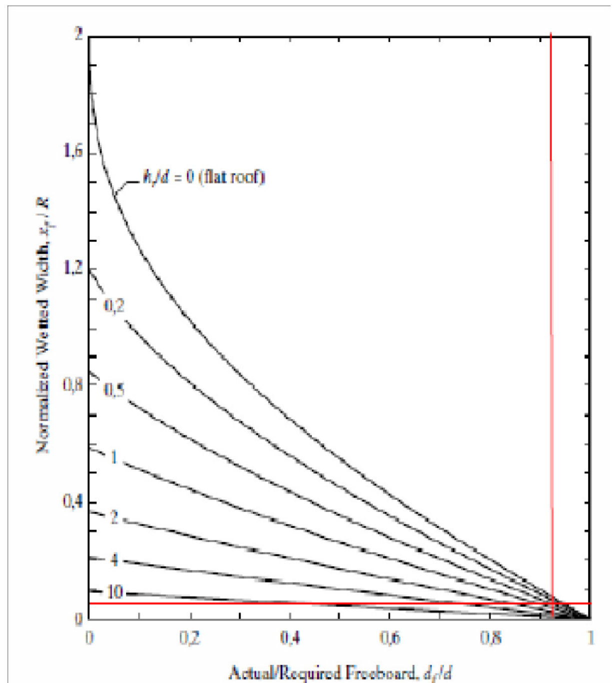


Fig. 6: Cone roof tank. Normalized wetted width of tank roof x_f/R as a function of actual/required freeboard d_f/d and normalized roof height h_r/d

Compute the angle θ

$$\theta := \operatorname{atan}\left(\frac{I_E \cdot S_{ac}(T_c) \cdot \frac{ft}{sec^2}}{g}\right) = 0.437 \cdot deg$$

Where $S_{ac}(T_c) = 0.163$ $I_E = 1.5$ $g = 32.174\frac{ft}{s^2}$

$d_f := H_s - H = 3\,ft$ $d = 3.27\,ft$ $\frac{d_f}{d} = 0.918$ Compute input variables for graph above

$h_r = 1.25\,ft$ $\frac{h_r}{d} = 0.382$

From graph figure 6

$x_f := .05 \cdot R = 1\,ft$ horizontal extent of wetted dome surface from the shell $\frac{x_f}{R} = 0.05$ << 1.0 OK

$\rho := \frac{\gamma_{water}}{g} = 62.4 \cdot \frac{lbm}{ft^3}$ unit mass of water

$$F_{max} := \frac{\rho}{2} \cdot g \cdot x_f^{\,2} \cdot \frac{(d + h_r)}{R} \qquad F_{max} = 7 \cdot \frac{lbf}{ft}$$

Maximum uplift on shell due to hydrodynamic pressure caused by sloshing. Impact effects are considered minor and ignored

adjust mass for recalculation of seismic demand

$$\boxed{\bar{m}_i = \begin{cases} m_i + m_c \cdot \left(1 - \dfrac{d_f + h_r/3}{d}\right) & \text{for } d_f + h_r/3 < d \\ m_i & \text{for } d_f + h_r/3 \ge d \end{cases}}$$

$$W_i = 1006 \cdot kip \qquad W_T = 1725 \cdot kip$$

$$\frac{\left(d_f + \frac{h_r}{3}\right)}{d} = 1.045 \qquad W_{bar\_i} := W_i + W_c \cdot \left(1 - \frac{d_f + \frac{h_r}{3}}{d}\right) = 974.2 \cdot kip$$

$$W_{bar\_i} := \text{if}\left[\frac{\left(d_f + \frac{h_r}{3}\right)}{d} < 1, W_{bar\_i}, W_i\right] = 1006 \cdot kip$$

$$\boxed{\bar{m}_c = m_l - \bar{m}_i}$$

$$W_c = 696.4 \cdot kip \qquad W_{bar\_c} := W_T - W_{bar\_i} = 719.6 \cdot kip$$

$$\frac{W_{bar\_i}}{W_i} = 1 \qquad \frac{W_{bar\_c}}{W_c} = 1.033$$

Factors by which mass must be multiplied due to the slosh contact with the roof

Recaclulate convective period using adjusted mass. Maintain asssumption of T = 0 for impulsive mass

$$\boxed{\begin{aligned} \bar{T}_i &= T_i \cdot \sqrt{\frac{\bar{m}_i}{m_i}} \\ \bar{T}_c &= T_c \cdot \sqrt{\frac{\bar{m}_c}{m_c}} \end{aligned}}$$

$T_c = 3.716\,s$ original convective period

$T_{c\_bar} := T_c \cdot \sqrt{\frac{W_{bar\_c}}{W_c}} = 3.778\,s$ modified convective period

$S_{ac}(T_c) = 0.163 \qquad A_c = 0.117$ original convective seismic factor

$S_{ac}(T_{c\_bar}) = 0.161 \qquad A_{c\_bar} := A_c \cdot \frac{S_{ac}(T_{c\_bar})}{S_{ac}(T_c)} = 0.115$ revised convective seismic factor

Recompute base shear and overturning moment

Change formula weights to adjusted values

$M_s = 2611 \cdot kip \cdot ft$ original overturning moment

$$M_{s\_rev} := \sqrt{\left[A_i \cdot \left[W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + \left(W_{bar\_i}\right) \cdot X_i\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_c\right)^2}$$

$M_{s\_rev} = 2551 \cdot kip \cdot ft$ revised moment

Calculate overturning moment at the top of foundation, including floor plate differential pressure effects

$M_{mf} = 4545 \cdot kip \cdot ft$ original overturning moment

$$M_{mf\_rev} := \sqrt{\left[A_i \cdot \left(W_s \cdot X_s + W_r \cdot X_r + W_{snow} \cdot X_{snow} + W_{baffles} \cdot X_b + W_{bar\_i} \cdot X_{imf}\right)\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c} \cdot X_{cmf}\right)^2}$$

$M_{mf\_rev} = 4551 \cdot kip \cdot ft$ revised moment

Calculate base shear at top of foundation

$V_f = 284.64 \cdot kip$ original base shear

$$V_{f\_rev} := \sqrt{\left[A_i \cdot \left[W_s + W_r + W_{snow} + \left(W_f + W_{baffles} + W_{col\_base} + \frac{W_{col}}{2}\right) + W_{bar\_i}\right]\right]^2 + \left(A_{c\_bar} \cdot W_{bar\_c}\right)^2}$$

$V_{f\_rev} = 285.02 \cdot kip$ revised base shear

The above base shears and moments are expressed in allowable stress design (ASD) basis.
The slight amount of wave-roof contact has a minimal effect on the seismic loads..

Compute Shell Hoop Forces and Stresses

Impulsive and convective forces are distributed using Housner's distribution formulas for horizontal motion parallel t the baffles.

Define the following variables:

z	Height of a point above the tank floor
Y	Depth of a point below the water surface
n_I	Distributed hoop force, klf, due to impulsive load N_I
n_C	Distributed hoop force, klf, due to convective load N_C
n_V	Distributed hoop force, klf, due to vertical seismic force N_V
n_F	Distributed hoop force, klf, due to hydrostatic force at maximum normal operating level
n_{Fol}	Distributed hoop force, klf, due to hydrostatic force at overflow operating level

Define elevation, distribution, and force component functions

$Y(z) := H - z$ distance from MOL to z

Housner's distribution of impulsive load as a function of elevation above the base and, in the case of impulsive loads, depends on the ratio of D/H

For the case of D/H < 1.33 and Y(z) < 0.75 D (z > .75D, upper section)

$$Dist_{ia}(z) := \frac{\left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2}{\left[\int_{.75 \cdot D}^{H} \left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2 dz + \int_0^{.75 \cdot D} 0.5\, dz\right]}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H < 1.33 at lower elevations, the factor is a constant equal to

$$Dist_{ib}(z) := \frac{0.5}{\left[\int_{.75 \cdot D}^{H} \left(\frac{Y(z)}{0.75 \cdot D}\right) - 0.5 \cdot \left(\frac{Y(z)}{0.75 \cdot D}\right)^2 dz\right] + \int_0^{.75 \cdot D} 0.5\, dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

For the case of D/H > 1.33

$$Dist_{ic}(z) := \frac{\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)}{\int_{0\cdot ft}^{H}\left[\left(\frac{Y(z)}{H}\right) - .5\cdot\left(\frac{Y(z)}{H}\right)^2\right]\cdot\tanh\left(0.866\cdot\frac{D}{H}\right)dz}$$

The above formula is the impulsive force per unit depth at elevation "z" expressed as a fraction of the total impulsive force.

$$Dist_i(z) := \text{if}\left[\left(\frac{D}{H}\right) \geq 1.333, Dist_{ic}(z), \text{if}\left(Y(z) < 0.75\cdot D, Dist_{ia}(z), Dist_{ib}(z)\right)\right]$$

select appropriate formula based on depth and diameter ratio

Housner's distribution of convective load as a function of elevation above the base

$$Dist_c(z) := \frac{\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}}{\int_{0\cdot ft}^{H}\dfrac{\cosh\left(3.68\cdot\frac{H-Y(z)}{D}\right)}{\cosh\left(3.68\cdot\frac{H}{D}\right)}dz}$$

The above formula is the convective force per unit depth at elevation "z" expressed as a fraction of the total convective force.

$V_i := A_i\cdot W_{bar\_i}$ $\quad V_i = 251.779\cdot kip$ — Total base shear component due to impulsive fluid load

$N_i(z) := \left(\frac{V_i}{2}\right)\cdot Dist_i(z)$ — Shell hoop force due to impulsive fluid load

$V_c := A_{c\_bar}\cdot W_{bar\_c}$ $\quad V_c = 82.664\cdot kip$ — Total base shear component due to convective fluid load

$N_c(z) := \frac{V_c}{2}\cdot Dist_c(z)$ — Shell hoop force due to convective fluid load

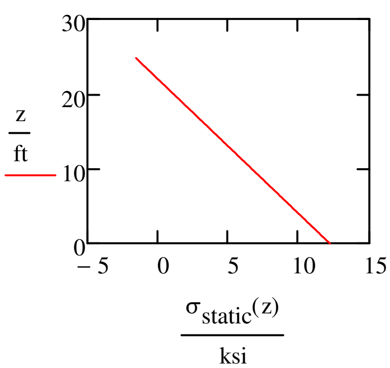
$N_h(z) := \gamma_{water}\cdot\left(\frac{D}{2}\right)\cdot Y(z)$ — Shell hoop force due to hydrostatic load with water at MOL

$A_v := 0.14\cdot S_{DS}$ $\quad A_v = 0.098$ — Vertical seismic factor

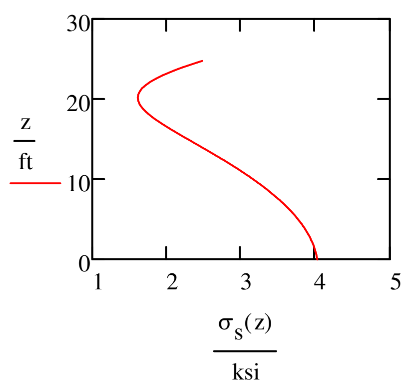
$\sigma_{static}(z) := \dfrac{N_h(z)}{t_s(z)}$ Hoop stress due to static fluid pressure at MOL

$\sigma_s(z) := \dfrac{\sqrt{N_i(z)^2 + N_c(z)^2 + \left(N_h(z) \cdot A_v\right)^2}}{t_s(z)}$ Hoop stress due to hydrodynamic pressure, Ref 4 Eq 13-42

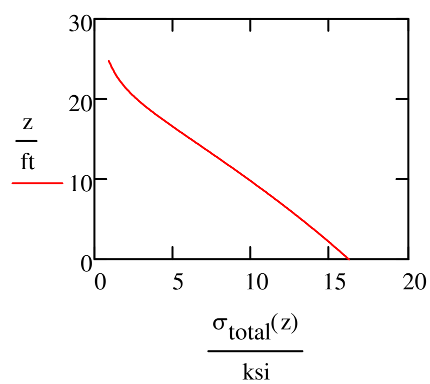
$\sigma_{total}(z) := \sigma_{static}(z) + \sigma_s(z)$ Combined static and seismic hoop stress at MOL



Hydrostatic Stress



Seismic Stress



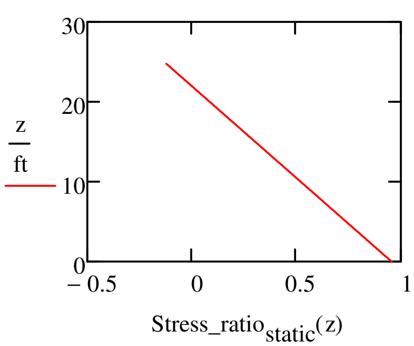
Static + Seismic Stress

Note: the above plots are nominal based on treating each hoop course as acting independently. Actual stresses each side of girth joints are the same since strains are identical if the courses are attached, so the real stress near transition zones falls somewhere between the apparent discontinuous stress levels shown on the graphs. The actual maximum stress levels tend to occur about a foot above the joint and are not as high as predicted by the more simplified model. The simplified model is conservative and is the method reflected in the AWWA D-100 standard.

Check actual versus allowable stress based on the class of steel used.

Assumed joint efficiency and allowable stress $E_{joint} := 85\%$ $F_t(z) := E_{joint} \cdot 15 \cdot ksi$ Chapter 14 of AWWA D100-11 does not apply

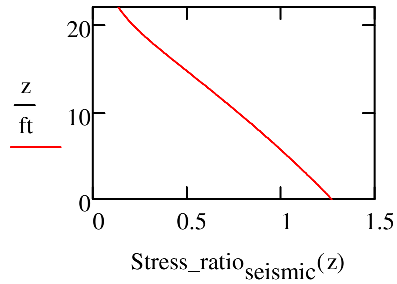
$Stress\_ratio_{static}(z) := \left(\dfrac{\sigma_{static}(z)}{F_t(z)}\right)$



Maximum static stress ratio is $Stress\_ratio_{static}(0) = 0.957$ < 1.0 OK

$$Stress\_ratio_{seismic}(z) := \frac{\sigma_{total}(z)}{F_t(z)}$$

The worst case stress ratio is at the bottom of the first shell course



$$Stress\_ratio\_max_{seismic} := Stress\_ratio_{seismic}(0) = 1.273$$

< 1.33 OK

Compute Shell Longitudinal Forces and Stresses

Define axial compressive force in the shell due to dead load for $0 < z < H_s$, in klf.. Ground motion parallel to baffles.

$$P_D(z) := \frac{W_r}{\pi \cdot D} + \int_z^{H_s} \gamma_{steel} \cdot t_s(z)\, dz$$

Define overturning moment functions at elevation z, in kip-ft

$$M_{rs}(z) := A_i \cdot \left[W_r \cdot (X_r - z) + W_{snow} \cdot X_{snow} + \pi \cdot \gamma_{steel} \cdot D \cdot \int_z^{H} y \cdot t_s(y)\, dy \right]$$

Moment associated with roof, snow and shell mass

$$M_i(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_i(y)\, dy$$

Moment associated with impulsive fluid mass, z < H

$$M_c(z) := 2 \cdot \int_z^{H} (y - z) \cdot N_c(y)\, dy$$

Moment associated with convective fluid mass, z < H

$$M_s(z) := M_{rs}(z) + M_i(z) + M_c(z)$$

Total moment at elevation z on the shell for z < H

Define functions for compressive stress under static or seismic load conditions

$$\sigma_{static}(z) := \frac{P_D(z) + P_{snow}}{t_s(z)}$$

$$\sigma_{comp}(z) := \frac{(1 + 0.4 \cdot A_v)(P_D(z) + P_{snow}) - F_{max} + \frac{4\, M_s(z)}{\pi \cdot D^2}}{t_s(z)}$$

Includes deduction for roof uplift, $F_{max.}$

Check allowable stress for compression with local buckling and slenderness considered

Use AWWA Method 1. Method 2 may be applicable for Fy=36 ksi steel, and allows consideration of water pressure in providing compression stability. Method 2 is more complicated than Method 1, which is more conservative. If Method 1 works, there is no need to use Method 2.

Local buckling stress formulas for Class 1 Materials

$$F_{L1a}(z) := \left[17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R} \right) \right] \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R} \right)^2 \right] \cdot psi$$

For Class 1 materials with 0 < t/R < t/Rc = .0031088, elastic buckling

$F_{L1b}(z) := 5775 \cdot psi + 738 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 1 materials with t/Rc = .0031088 < t/R < 0.0125, inelastic buckling

$F_{L1c}(z) := 15 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Local buckling stress formulas for Class 2 Materials

$F_{L2a}(z) := \min\left[15 \cdot ksi, 17.5 \cdot 10^5 \cdot \left(\frac{t_s(z)}{R}\right) \cdot \left[1 + 50000 \cdot \left(\frac{t_s(z)}{R}\right)^2\right] \cdot psi\right]$

For Class 2 materials with 0 < t/R < t/Rc = .0035372, elastic buckling

$F_{L2b}(z) := 6925 \cdot psi + 886 \cdot 10^3 \cdot psi \cdot \frac{t_s(z)}{R}$

For Class 2 materials with t/Rc = .0035372 < t/R < 0.0125, inelastic buckling

$F_{L2c}(z) := 18 \cdot ksi$

For Class 1 materials with t/R > 0.0125, plastic buckling

Write equation selection functions for F_L depending on t/R ratio and class

$ratio1 := .0031088 \quad ratio2 := .0035372$

$$F_{L1}(z) := \min\left(if\left(\frac{t_s(z)}{R} < ratio1, F_{L1a}(z), if\left(\frac{t_s(z)}{R} < 0.0125, F_{L1b}(z), F_{L1c}(z)\right)\right), 15 \cdot ksi\right)$$

$$F_{L2}(z) := \min\left(if\left(\frac{t_s(z)}{R} < ratio2, F_{L2a}(z), if\left(\frac{t_s(z)}{R} < 0.0125, F_{L2b}(z), F_{L2c}(z)\right)\right), 18 \cdot ksi\right)$$

$F_L(z) := if(class(z) = 1, F_{L1}(z), F_{L2}(z))$

Slenderness reduction factor equations

$r := \frac{D \cdot \sqrt{2}}{4}$ radius of gyration of tank shell

$K := 1.0$ effective column length factor, pinned ends assumed

$E := 29 \cdot 10^6 \cdot psi$ modulus of elasticity for steel

Slenderness ratio at which overall elastic column buckling can occur (not local buckling)

$C'_c(z) := \sqrt{\pi^2 \cdot \frac{E}{F_L(z)}}$ $L := H_s$

$K_{\varphi 1}(z) := 1 - \frac{1}{2} \cdot \left(\frac{\frac{K \cdot L}{r}}{C'_c(z)} \right)^2$ For 25 < KL/r < C'c

$K_{\varphi 2}(z) := \frac{1}{2} \cdot \left(\frac{C'_c(z)}{\frac{K \cdot L}{r}} \right)^2$ For KL/r > C'c

$K_{\varphi 3}(z) := 1.0$ For KL/r < 25

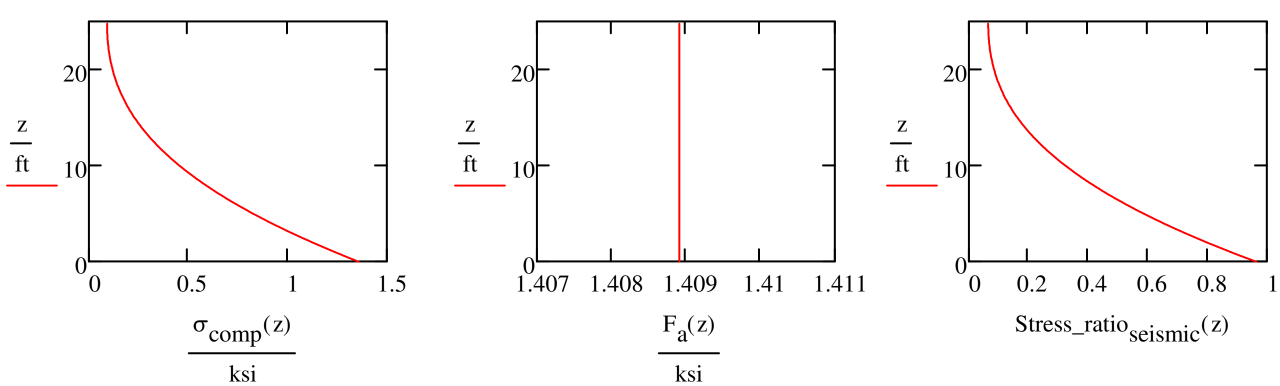
$ratio := K \cdot \frac{L}{r}$ $ratio = 1.768$

$K_\varphi(z) := if\left(ratio < 25, K_{\varphi 3}(z), if\left(ratio > C'_c(z), K_{\varphi 2}(z), K_{\varphi 1}(z)\right)\right)$

$F_a(z) := F_L(z) \cdot K_\varphi(z)$ allowable compressive stress due to axial load

For shell longitudinal stress, treat all stress as axial

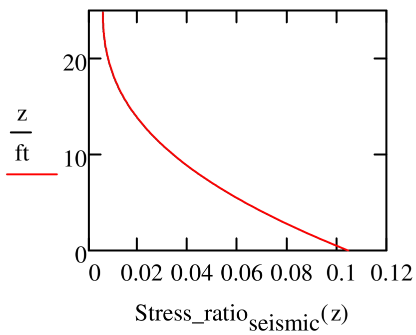
$Stress\_ratio_{seismic}(z) := \frac{\sigma_{comp}(z)}{F_a(z)}$



$Stress\_ratio_{seismic}(0) = 0.965$ < 1.00, **OK for static plus seismic longitudinal compression**

Check seismic longitudinal tensile stress for ground motion parallel to the baffles

$$\sigma_{tens}(z) := \frac{(1 - .40\cdot A_v)P_D(z) + F_{max} + \frac{4\,M_s(z)}{\pi\cdot D^2}}{t_s(z)} \qquad Stress\_ratio_{seismic}(z) := \frac{\sigma_{tens}(z)}{F_t(z)}$$



All stress ratios << 1.333 are **OK for static plus seismic stress in longitudinal tension**

$Stress\_ratio_{seismic}(0) = 0.105$

Horizontal Shear Transfer Capacity

The previously calculated base shear is $V_f = 285 \cdot kip$

From AWWA D100-11 Eq 13-57, the allowable resistance attributable to friction is (for the full tank, seismic condition)

$$V_{ALLOW} := \tan(30 \cdot deg) \cdot \left(W_s + W_r + W_T + W_f + W_{baffles}\right) \cdot \left(1 - A_v\right) = 937 \cdot kip$$

$$\frac{V_f}{V_{ALLOW}} = 0.304$$

>> V_f OK. No shear connection between the superstructure and ringwall is required for shear. Shear resistance is provided by the bottom plate acting as a diaphragm kept in place by bottom friction. Check shell to bottom transfer capacity

The maximum shell to bottom plate shear load is $v := 2 \cdot \frac{V_f}{\pi \cdot D} = 4.53 \cdot klf$

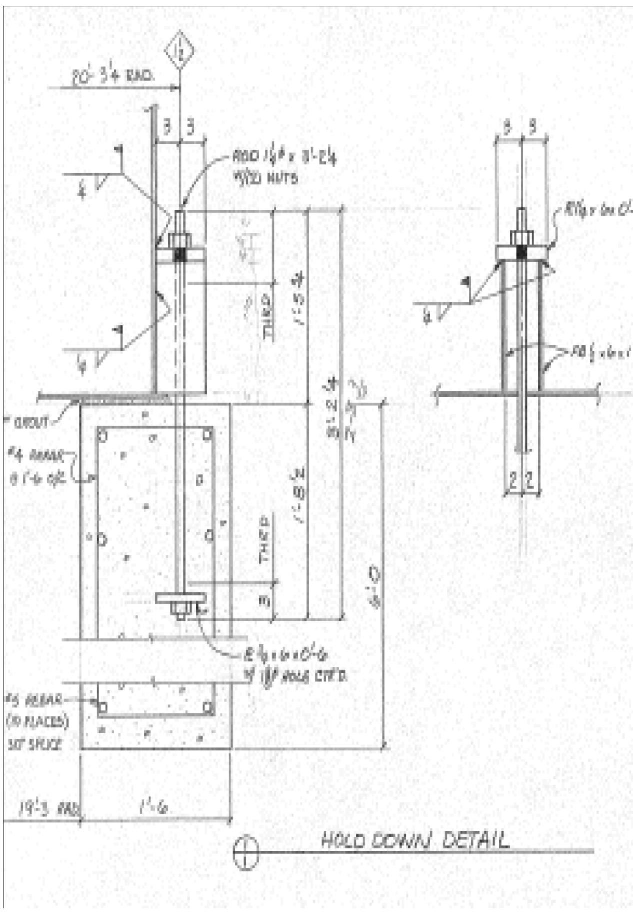
There is no annular plate, just the .25" floor plate

$$t_f := .25 \cdot in$$

And the maximum shear stress on the plate is $\tau := \frac{v}{t_f} = 2 \cdot ksi$ $\frac{\tau}{12 \cdot ksi} = 0.126$

AWWA D100 permits 12 ksi in shear, and this can be increased by 1.33 for seismic, so **floor plate should not tear in shear parallel to the floor plate**

Check Foundation



The foundation detail at the left is from shop drawings

Compute existing anchor load

$$\sigma_{tens}(0)\cdot t_s(0) = 2.999\cdot \text{klf}$$

$n_{anchors} := 13$ $\quad T_{anchor} := \left(\frac{\pi\cdot D}{n_{anchors}}\right)\cdot\left(\sigma_{tens}(0)\cdot t_s(0)\right)$ $\quad T_{anchor} = 29\cdot\frac{\text{kip}}{\text{each}}$ $\quad \frac{\pi\cdot D}{n_{anchors}} = 9.666\,\text{ft}$

$d_{bolt} := 1.25\cdot \text{in}$ $\quad A_{anchor} := \frac{\pi}{4}\cdot d_{bolt}^{\,2} = 1.227\cdot \text{in}^2$ gross area. Use root area per AWWA D100

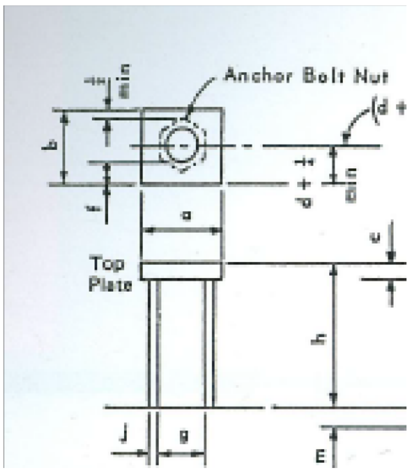
$A_{anchor} := .890\cdot \text{in}^2$ $\quad A_{bolt\_tensile} := .969\cdot \text{in}^2$ $\quad F_y := 36\cdot \text{ksi}$ $\quad F_u := 57\cdot \text{ksi}$

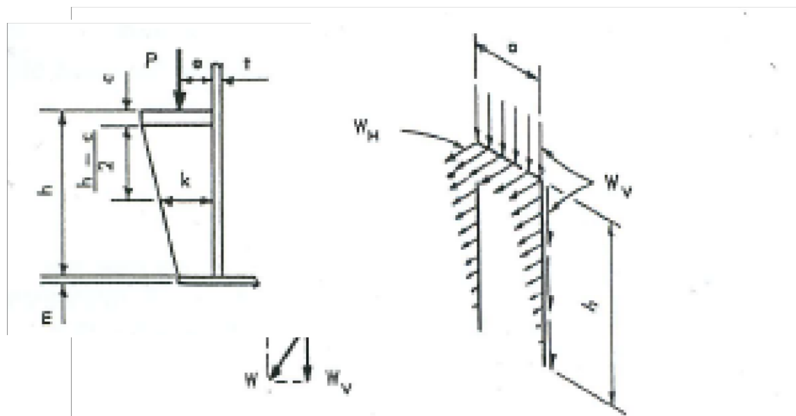
Allowable stress $F_t := \min\left(.80\cdot F_y, .50\cdot F_u\right) = 28.5\cdot \text{ksi}$ A36, per AWWA D100 3.3.3.2

$\sigma_{anchor} := \frac{\pi\cdot D\cdot \sigma_{tens}(0)\cdot t_s(0)}{n_{anchors}\cdot A_{bolt\_tensile}} = 29.921\cdot \text{ksi}$ $\quad \frac{\sigma_{anchor}}{F_t} = 1.05$ >> 1.33 No Good

Check anchor chair welds

From reference 9 guide to anchor chair design





$a := 6 \cdot in \qquad h := 13.5 \cdot in \qquad P := T_{anchor} = 28.993 \cdot kip$

Figure 7-6. Loads on Welds.

$e := 3 \cdot in$ (from record drawing)

Formulas may also be used for cones, although this underrates the vertical welds some.

$$W_v = \frac{P}{a + 2h} \qquad (7\text{-}5)$$

$$W_H = \frac{Pe}{ah + 0.667h^2} \qquad (7\text{-}6)$$

$$W = \sqrt{W_V^2 + W_H^2} \qquad (7\text{-}7)$$

$$W_V := \frac{P}{a + 2 \cdot h} = 879 \cdot \frac{lbf}{in}$$

$$W_H := \frac{P \cdot e}{a \cdot h + 0.667 \cdot h^2} = 429 \cdot \frac{lbf}{in}$$

$$W := \sqrt{W_V^2 + W_H^2} = 978 \cdot \frac{lbf}{in}$$ predicted weld stress

$F_t = 28500\, psi \qquad t_{weld} := .25 \cdot in$

$$W_{V\_allowable\_transverse} := .7071 \cdot t_{weld} \cdot F_t \cdot .65 = 3275 \cdot \frac{lbf}{in}$$

$$W_{V\_allowable\_longitudinal} := .7071 \cdot t_{weld} \cdot F_t \cdot .50 = 2519 \cdot \frac{lbf}{in}$$

$$\frac{W}{W_{V\_allowable\_longitudinal}} = 0.388$$ **< 1.33 OK** Plate dimensions meet Ref 9 minimums

Check anchor using strength basis and ACI 318 Appendix D

$N_{ua} := 1.4 \cdot T_{anchor} = 40.6 \cdot kip$ Express anchor tension ultimate basis $f_y := 36 \cdot ksi$ $f_u := 57 \cdot ksi$ A36

$A_{se\_N} := A_{bolt\_tensile} = 0.969 \cdot in^2$ $f_{uta} := \min(1.9 \cdot f_y, 125 \cdot ksi, f_u) = 57 \cdot ksi$

$N_{sa} := A_{se\_N} \cdot f_{uta} = 55.233 \cdot kip$ Eq D-2

Consider anchors as Condition B (no supplementary reinforcement). $\varphi_{tension} := .70$ D.4.3 or 4 for earthquake

$$\frac{N_{ua}}{\varphi_{tension} \cdot N_{sa}} = 1.05$$ **> 1.0 NG**

Check concrete breakout strength

Estimated embedment depth, from record drawing is $h_{ef} := 18 \cdot in$

The distance to the next anchor is $s_{anchor} := \pi \cdot \frac{(D + 6 \cdot in)}{n_{anchors}} = 9.787\,ft$ **< 10 ft max, OK**

The calculated edge distance is $c_{a1} := 6.0 \cdot in$

Check using ACI 318 Appendix D $1.5 \cdot h_{ef} = 27 \cdot in$

$$\frac{s_{anchor}}{h_{ef}} = 6.525$$ >3 so group action need not be considered for concrete breakout in tension, D.3.1.1

Record drawings do not indicate compressive strength of the concrete. Given construction in the 1990's, use

$f'_c := 4000 \cdot psi$ $k_c := 24$ $\lambda_a := 1.0$

$$N_b := lbf \cdot k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} = 116 \cdot kip$$ Basic concrete breakout strength, Eq D-6

$A_{Nc} := \left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot s_{anchor} = 3876 \cdot in^2$ $\quad A_{Nco} := 9 \cdot h_{ef}^{\ 2} = 2916 \cdot in^2$

stength modification factors

$\psi_{ed\_N} := 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}} = 0.767$ modification factor for edge effects. Eq D-10

$\psi_{ec\_N} := 1.0$ eccentric load modifier, Eq D-8

$\psi_{c\_N} := 1.0$ cracked concrete assumed. D.5.2.6

$\psi_{cp\_N} := 1.0$ D.5.2.7

$N_{cb} := \frac{A_{Nc} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b}{A_{Nco}} = 118.1 \cdot kip$ breakout capacity in tension, D.5.2.1

$\frac{N_{ua}}{\varphi_{tension} \cdot N_{cb}} = 0.491$ <u>< 1.0 OK for concrete breakout</u>

Check pullout strength

Bearing area is $A_{brg} := 36 \cdot in^2 - \frac{\pi}{4} \cdot (1.375 \cdot in)^2 = 34.515 \cdot in^2$ Gross surface less bolt hole

$N_p := 8 \cdot A_{brg} \cdot f'_c = 1104.5 \cdot kip$ $\quad \frac{N_{ua}}{\varphi_{tension} \cdot N_p} = 0.053$ <u>< 1.0 OK for pullout</u>

Check side-face blowout

$N_{sb} := 160 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi = 356.7 \cdot kip$ Eq D-16 applies

$\frac{h_{ef}}{c_{a1}} = 3$ > 2.5 $\quad \frac{s_{anchor}}{c_{a1}} = 19.575$ > 6

$\frac{N_{ua}}{\varphi_{tension} \cdot N_{sb}} = 0.163$ <u>< 1.0 OK for side face blowout</u>

Check Foundation For Uplift and Overturning

$\gamma_{conc} := 150 \cdot pcf$

$b_{ftg} := 1.5 \cdot ft$ $\quad h_{ftg} := 3 \cdot ft$ footing width and depth

$R_{ftg} := R + 9 \cdot in = 20.75\,ft$ $\quad R_{in} := R_{ftg} - b_{ftg}$ footing outside and inside radii

$$A_{ftg} := \pi \cdot \left(R_{ftg}^{\;2} - R_{in}^{\;2}\right) = 188.496\,ft^2$$

$W_{ftg} := \gamma_{conc} \cdot A_{ftg} \cdot h_{ftg} = 84.8 \cdot kip$ $\quad w_{ftg} := \dfrac{W_{ftg}}{\pi \cdot D} = 0.675 \cdot klf$ total and unit footing weight

$W_{water} := H \cdot \gamma_{water} \cdot \pi \cdot \left(R^2 - R_{in}^{\;2}\right) = 127.0 \cdot kip$ $\quad w_{water} := \dfrac{W_{water}}{\pi \cdot D} = 1.01 \cdot klf$ total and unit weight of water over footing

$\gamma_{soil} := 125 \cdot pcf$ typical weight of compacted soil

$A_{soil} := 0$ area of soil over footing

$A_{wedge} := \dfrac{(29 \cdot in)^2}{2 \cdot 2} = 1.46\,ft^2$ area of soil resisting uplift in friction at 1H:2V, backfill to within 7" of top of footing. Skin friction assumed 0.4 between footing and soil

$w_{soil} := \gamma_{soil} \cdot \left(A_{soil} + 0.4A_{wedge}\right)$ $\quad w_{soil} = 0.1 \cdot klf$ unit soil resistance

$W_s = 24.053 \cdot kip$ $\quad w_{shell} := \dfrac{W_s}{\pi \cdot D} = 0.191 \cdot klf$ shell weight

$W_{roof\_edge} = 8.87 \cdot kip$ $\quad w_{roof\_edge} := \dfrac{W_{roof\_edge}}{\pi \cdot D} = 0.071 \cdot klf$ roof edge weight

Compute overturning safety factor for pivoting about the toe of the shell

$M_{s\_rev} = 2551 \cdot kip \cdot ft$

$$SF_{overturning} := \left(1 - A_v\right) \cdot \left(W_{roof\_edge} + W_s + W_{ftg} + W_{water}\right) \cdot \frac{R}{M_{s\_rev}} = 1.73$$ NG

Required safety factor based on ASCE 7 load combos is .7E/.6D where .7E is the earthquake load in allowable stress terms, an effective ratio of 1.67

Check ratio of resistance to uplift at the foundation

Check ratio of resistance to uplift at the foundation

$$SF_{uplift} := \frac{\left[\left(1 - A_v\right)\cdot\left(w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water}\right) + w_{soil} - F_{max}\right]}{4\cdot\frac{M_{s\_rev}}{\pi\cdot D^2}} = 0.897$$

< 1.0 so there will be some foundation uplift

Check bearing pressure

$= \blacksquare\cdot klf$

The total load on the perimeter under static conditions is

$$w_{static} := w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water} = 1.947\cdot klf \qquad q_{bearing\_static} := \frac{w_{static}}{b_{ftg}} = 1.298\cdot ksf$$

$$w_{seismic} := \left(1 + A_v\right)\cdot\left(w_{ftg} + w_{shell} + w_{roof\_edge} + w_{water}\right) + F_{max} + 4\cdot\frac{M_{s\_rev}}{\pi\cdot D^2} = 4.176\cdot klf$$

$$q_{bearing\_seismic} := \frac{w_{seismic}}{b_{ftg}} = 2.784\cdot ksf$$

$q_{allow} := 2.5\cdot ksf$ Static allowable bearing pressure $\frac{q_{bearing\_static}}{q_{allow}} = 0.519$ OK

$\frac{q_{bearing\_seismic}}{q_{allow}} = 1.114$ <1.33 OK

Check As Self-Anchored Tank

Per AWWA D100 section 13.5.4.1

$w_t := P_D(0) = 298 \cdot \frac{lbf}{ft}$ Weight of shell and roof supported by shell

$t_b := t_{floor} = 0.25 \cdot in$ $F_y := 27 \cdot ksi$ $G := 1.0$ A283 Grade B steel assumed

$w_L := \min\left(1.28 \cdot \frac{H}{ft} \cdot \frac{D}{ft} \cdot G, 7.29 \cdot \frac{t_b}{in} \sqrt{\frac{F_y}{ksi} \cdot \frac{H}{ft} \cdot G}\right) \cdot plf = 44 \cdot \frac{lbf}{ft}$ Eq 13-37, normalized for units

Overturning ratio

$J := \frac{M_s(0)}{D^2 \cdot \left[w_t \cdot \left(1 - 0.4 \cdot A_v\right) + w_L\right]} = 6.43$ Value shown is for Ri=3.0. From side calculation at Ri=2.5, J=7.29

>> 1.54 therefore the tank is not stable without anchorage

Accounting for Baffle Damping

Accounting for the effects of baffle damping is a complicated analysis problem generally requiring the use of comptational fluid dynamics (CFD) methods that exceed the scope of this analysis. A gross estimate of the effect can be calculated assuming the relative height to diameter ratio for the four compartments into which the baffles divide the tank. Assume the baffles are rigid and ignore fluid exchange between compartments

First calculate the revised sloshing wave characteristics by substituting .25D where D was used in the previous formulas

Compute the first mode sloshing period

$$T_c := 2\cdot\pi\sqrt{\frac{.25D}{3.68\cdot g\cdot\tanh\left(3.68\cdot\frac{H}{.25D}\right)}} \qquad T_c = 1.826\,s$$

From AWWA D100-11 Eq 13-53 through 13-56

$K := 1.5$ damping scaling factor

$SUG := 3$ Seismic use group

$$A_f := \text{if}\left(SUG = 3, \text{if}\left(T_c \le T_L, \frac{K\cdot S_{D1}\cdot sec}{T_c}, K\cdot S_{D1}\cdot\frac{T_L\cdot sec}{T_c^{\,2}}\right), \text{if}\left(T_c \le 4sec, \frac{K}{T_c}\cdot S_{D1}\cdot I_E\cdot sec, 4\cdot\frac{K}{T_c^{\,2}}\cdot S_{D1}\cdot I_E\cdot T_L\cdot sec\right)\right)$$

$A_f = 0.333$

$d := 0.5\cdot .25D\cdot A_f = 1.664\,ft$ Sloshing wave height, Eq 13-52 - AWWA D100 basis for cylinder at least as high as Hs + d

For Occupancy Category IV and S_{DS} > .50g, the required minimum freeboard is equal to the sloshing amplitude

freeboard $f := H_s - H = 3\,ft$

$\frac{d}{f} = 0.555$ < 1.0, therefore **freeboard is adequate for ground motion perpendicular to the baffles**

Calculate the impulsive and convective water weights and vertical centroids

$\frac{.25D}{H} = 0.455$ Assumed ground motion parallel to baffles, no impact on sloshing behaviour

$$W_{i\_baffled} := W_T\cdot\frac{\tanh\left(.866\cdot\frac{.25D}{H}\right)}{.866\cdot\frac{.25D}{H}}$$ if .25D/H > 1.333

$$W_{i\_baffled} := \text{if}\left[\frac{.25D}{H} < 1.333, W_T\cdot\left(1.0 - 0.218\cdot\frac{.25D}{H}\right), W_{i\_baffled}\right]$$ if D/H < 1.33

$W_i = 1005.505\cdot kip$ Impulsive water weight $\frac{W_{i\_baffled}}{W_T} = 0.901$ vs $\frac{W_i}{W_T} = 0.583$

The effective center of gravity depends on whether just the moment at the base of the shell is being calculated c
the total moment on the foundation, shell plus floor.

$$X_{i\_baffled} := H \cdot \text{if}\left[\left(\frac{.25D}{H}\right) > 1.333, 0.375, 0.50 - 0.094 \cdot \frac{.25D}{H}\right] \quad X_{i\_baffled} = 10.06\,\text{ft}$$

centroid for calculation of just the moment from impulsive water mass

$$\frac{X_{i\_baffled}}{H} = 0.457 \qquad \frac{X_i}{H} = 0.375$$

center of gravity of impulsive mass moves up

$$W_{c\_baffled} := W_T \cdot \left(.230 \cdot \frac{.25D}{H} \cdot \tanh\left(3.67 \cdot \frac{H}{.25D}\right)\right) \quad W_{c\_baffled} = 180.35 \cdot \text{kip}$$

Ref 4, Eq 13-26

$$\frac{W_{c\_baffled}}{W_T} = 0.105 \quad \text{vs} \quad \frac{W_c}{W_T} = 0.404$$

Fraction of impulsive water weight goes up, convective weight goes down due to shorter period

$$X_{c\_baffled} := H \cdot \left[1 - \frac{\cosh\left(3.67 \cdot \frac{H}{.25D}\right) - 1}{3.67 \cdot \left(\frac{H}{.25D}\right) \cdot \sinh\left(3.67 \frac{H}{.25D}\right)}\right] \quad X_{c\_baffled} = 19.277\,\text{ft}$$

centroid for calculation of just the moment due to convective mass

$$\frac{X_c}{H} = 0.621 \qquad \frac{X_{c\_baffled}}{H} = 0.876$$

centroid of convective mass is higher for baffled direction

$$A_{c\_baffled} := \frac{S_{ac}(T_c) I_E}{1.4 \cdot R_c} \qquad A_{c\_baffled} = 0.238$$

Convective design acceleration

Compute the ratio of base shear due to water mass in the baffled and unbaffled directions

$$\Delta V_{water} := \frac{\left[\sqrt{(A_i \cdot W_{i\_baffled})^2 + (A_{c\_baffled} \cdot W_{c\_baffled})^2} - \sqrt{(A_i \cdot W_i)^2 + (A_c \cdot W_c)^2}\right]}{\sqrt{(A_i \cdot W_i)^2 + (A_c \cdot W_c)^2}} = 0.48$$

$$\Delta M_{water} := \frac{\left[\sqrt{(A_i \cdot W_{i\_baffled} \cdot X_{i\_baffled})^2 + (A_{c\_baffled} \cdot W_{c\_baffled} \cdot X_{c\_baffled})^2} - \sqrt{(A_i \cdot W_i \cdot X_i)^2 + (A_c \cdot W_c \cdot X_c)^2}\right]}{\sqrt{(A_i \cdot W_i \cdot X_i)^2 + (A_c \cdot W_c \cdot X_c)^2}}$$

$\Delta M_{water} = 0.70$

The conclusion from this exercise is that the sloshing wave would be reduced, but that the base shear and oerturning moment would be increased considerably by rigid baffles if fluid motion between baffled compartments is ignored. If the baffles were completely flexible, one would expect a much reduced effect.

References

1. 2012 *International Building Code*
2. Washington State Adoption of and Amendments to 2012 International Building Code (State Building Code)
3. ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
4. AWWA Standard D100-11 *Welded Carbon Steel Tanks for Water Storage*
5. Nuclear Reactors and Earthquakes, Chap. 6 and Appendix F. U.S. Nuclear Regulatory Commission publication, Division of Technical Information, TID-7024, National Technical Information Service (1963).
6. Not used
7. Not used
8. "Earthquake Induced Sloshing in Tanks with Insufficient Freeboard" Praveen K. Malhotra, Structural Engineering International, March 2006
9. Not used
10. "Dynamic Pressures on Accelerated Fluid Containers," G.W. Housner, 1955, Bulletin of the Seismological Society of America.
11. "Snow Load Analysis for Washington, 2nd Ed." Structural Engineers Association of Washington,1995
12. Not used
13. Not used
14. ACI 318-11 Building Code Requirements for Structural Concrete
15. ANSI/AISC 360-10 Specification for Structural Steel Buildings
16. AWS D1.1 Structural Welding Code - Steel

Units and Mathcad Notation

All calculations are shown in U.S. customary units. Calculations have been performed using MathSoft's Mathcad Version 14.0 software, which automatically checks for unit consistency and applies any necessary unit conversion factors internally to the program. Where computations are imported from Excel, SAP2000, or other software, the source is identified. Input values are shaded. Others are computed.

Where equations are shown with a ":=" sign, the left hand side of the equation is being defined by the right hand side. Where equations are shown with a "=" sign, the current value of the expression on the left hand side is being displayed.

= — An ordinary "equals" sign indicates the value being shown is for the most current evaluation of the variable on the left hand side of the equation

:= — An "equals" sign with a colon indicates the value on the left hand side is being defined by the expression on the right. Variables may be redefined, the last definition taking precedence

= — A bold "equals" sign indicates the symbol is being used in a logical expression

if(a,b,c) — An "if" statement is evaluated as "b" if "a" is true, and as "c' is "a' is false. These expressions may be nested

($matrix_{i,j}$) — In matrix expressions, the first subscript is the row, and the second is the column. Numbering starts with the value indicated as "ORIGIN" for the first row and column unless otherwise noted

submatrix (A,i1,i2,j1,j2) — Defines a vector or submatrix of matrix "A" from row i1 thru i2, and column j1 thru j2

----------->
( ) — An expression with a vector arrow over it indicates that the expression involves subscripted variables, and that the expression is being evaluated for each subscript in the range

|
|
|
| — A bold vertical line to the left of a series of expressions indicates that they are acting as a programming loop in the calculations

$ORIGIN := 1$ — Sets initial subscript value for subscripted variables

$M^{\langle j \rangle}$ — The vector in column "j" of matrix "M"

$sf := ft^2$

$\Phi(x)$ — Step function. Returns -1 for x < 0, +1 for x > 0 and .5 if x = 0

Seismic Evaluation
for
SVWTP Reservoir - Option A

for

Lake Whatcom Water & Sewer District
Bellingham, Washington

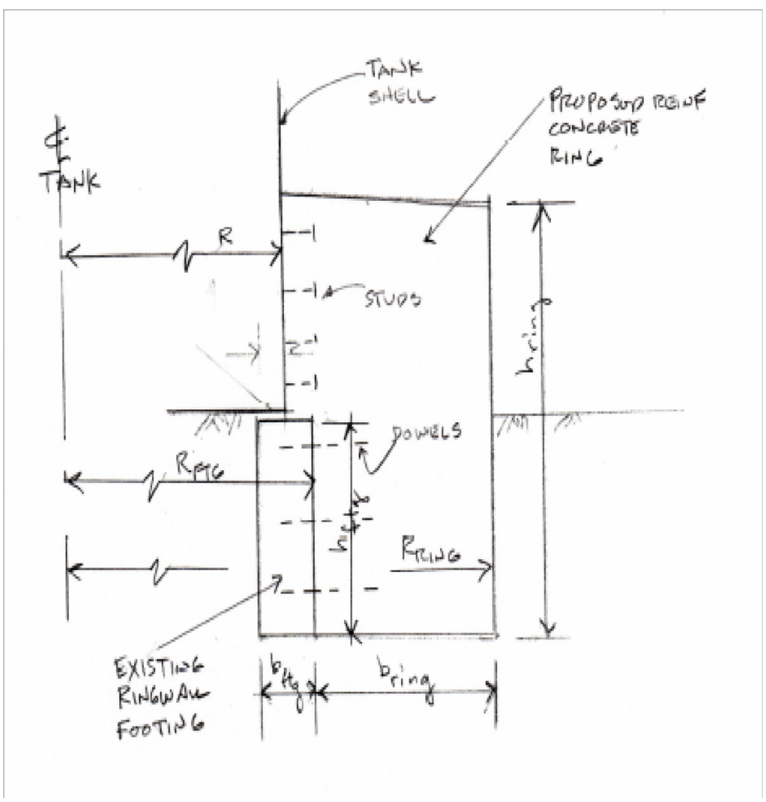
These calculations are preliminary in nature for design approach analysis and are not to be used for construction

Incorporate calculations from existing tank analysis by reference.

Reference:S:\Projects\Lake Whatcom W&S District\Reservoir Seismic VA 2015\Structural Calculations\SVWTP Reservoir\SVWTP Res

$cy := yd^3$

Existing ringwall and tank dimensions



Existing footing

$R_{ftg} = 20.75\,ft$ outside radius, ex. ftg.

$b_{ftg} = 1.5\,ft$

$h_{ftg} = 3\,ft$

$R_{in} = 19.25\,ft$ footing inside radius

$A_{ftg} := \pi\cdot\left(R_{ftg}^{\ 2} - R_{in}^{\ 2}\right)$ footprint

Additional exterior ring

$h_{ring} := 6\cdot ft$ Ring depth

$b_{ring} := 1.5\cdot ft$ Ring width

$R_{ring} := R_{ftg} + b_{ring} = 22.25\,ft$

$A_{gross} := \pi\cdot R_{ring}^{\ 2} = 1555\,ft^2$

$A_{ring} := A_{gross} - \pi\cdot R_{ftg}^{\ 2}$

Added ring dead load

$$V_{ring} := \left(2 \cdot \int_0^{\pi} \int_{R_{ftg}}^{R_{ring}} \int_0^{h_{ftg}} r \, dz \, dr \, d\phi \right) = 22.515 \cdot cy$$ Ring volume

$$W_{ring} := V_{ring} \cdot \gamma_{conc} \quad W_{ring} = 91 \cdot kip$$

$$w_{ring} := \frac{W_{ring}}{2 \cdot \pi \cdot R} = 726 \cdot plf$$ Anchor ring weight per ft of shell

Check overturning stability safety factor

$$SF_{overturning} := (1 - A_v) \cdot (W_{roof\_edge} + W_s + W_{ftg} + W_{water} + W_{ring}) \cdot \frac{R}{M_{s\_rev}} = 2.375$$ > 1.67 OK

$$Uplift := 4 \cdot \frac{M_{s\_rev}}{\pi \cdot D^2} = 2.03 \cdot klf$$ Transfer force at face of shell

The resistance available along the perimeter is

$$Resistance := (1 - A_v) \cdot (w_{roof\_edge} + w_{shell} + w_{ftg} + w_{water} + w_{ring}) + w_{soil} - F_{max} = 2.476 \cdot klf$$

Check resistance/uplift safety factor with added block

$$Resistance\_ratio := \frac{Resistance}{Uplift} = 1.22$$ > 1.0 OK

<u>The required shear transfer force between the ring and foundation is equal to the rig weight</u>

From Ref 3, Table 15.4-2, for tanks the overstrength factor $\Omega_o := 2.0$

$$s_{dowels} := \frac{s_{anchor}}{2} = 4.894 \, ft \qquad n_{dowels\_per\_row} := 4 \qquad s_{anchor} = 9.787 \, ft$$

$$n_{anchors} = 13$$

$$Load\_per\_dowel := s_{dowels} \cdot \frac{w_{ring}}{n_{dowels\_per\_row}} = 888 \cdot lbf$$

$$V_u := \Omega_o \cdot 1.4 \cdot Load\_per\_dowel = 2486 \, lbf$$

Shear strength for a 1/2" dowel (from catalog) is 7320 lbf.

$\phi_{shear} := .90$ $\dfrac{V_u}{\phi_{shear} \cdot 7.32 \cdot kip} = 0.377$ < 1.0 OK

$f'_{c} := 4 \cdot ksi$

Quantities

$$N_{dowelss} := n_{dowels\_per\_row} \cdot \pi \cdot \frac{D}{s_{dowels}} = 103$$

$$V_{conc} := \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} + \pi \cdot \left(R_{ftg}^{\ 2} - R^2\right) \cdot \left(h_{ring} - h_{ftg}\right) = 33 \cdot cy$$

Excavation quantity based on bottom of exc 2 ft beyond the new ring, sloping up to top of ringwall at 1:1

$$A_{bot} := \pi \cdot \left(R_{ring} + 2 \cdot ft\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 495\,ft^2$$

$$A_{top} := \pi \cdot \left(R_{ring} + 2 \cdot ft + h_{ftg}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 980\,ft^2 \qquad R_{ring} + 2 \cdot ft + h_{ftg} - R_{ftg} = 6.5\,ft$$

$$A_{mid} := \pi \cdot \left(R_{ring} + 2 \cdot ft + \frac{h_{ftg}}{2}\right)^2 - \pi \cdot R_{ftg}^{\ 2} = 730\,ft^2$$

$$V_{exc} := \frac{h_{ftg}}{3} \cdot \left(A_{bot} + 4 \cdot A_{mid} + A_{top}\right) = 163 \cdot cy$$

Backfill quantity

$$V_{backfill} := V_{exc} - \pi \cdot \left(R_{ring}^{\ 2} - R_{ftg}^{\ 2}\right) \cdot h_{ftg} = 140.324 \cdot cy$$