Lake Whatcom Water and Sewer District
Division 22-1 Reservoir Replacement

ADDENDUM NO. 2
March 25, 2025

REVISIONS TO THE REQUEST FOR QUALIFICATIONS:

The purpose of this addendum is to modify the Request for Qualifications documents for the referenced project. This addendum shall become a part of these documents.

Firms are hereby given notice that the RFQ documents are modified/amended as hereinafter set forth:

<u>Attachment A. Project Information:</u>

1. ADD the attached reference documents:
    a. Reservoir Seismic Vulnerability Assessment (BHC 2016)
    b. Division 22-1, Division 30 and Geneva Reservoir Coating Condition Assessment (Evergreen Coating Engineers 2022)
    c. Division 22-1 Condition Assessment (H2O Solutions 2024)

Questions and Answers:

QUESTION: For the Division 22-1 Water Reservoir Replacement Project, will an archaeologist be needed?

ANSWER: It is anticipated that there will be ground disturbing work included in the project and, although the ground disturbing work will likely be largely within the footprint of the existing reservoir, it is the District's expectation that an archaeologist will likely be required to meet regulatory requirements.

LAKE WHATCOM WATER AND SEWER DISTRICT

Division 22-1 Reservoir Inspection Report

April 1, 2024

Standards

The inspection report of this tank was preformed by H2O Solutions, LLC using surface supplied air, totally encapsulated in a sealed dry suit mated to a sealed dry divers hard hat and conducted in accordance with all applicable OSHA, EPA, AWWA,NACE,SSPC and ADC Requirements and recommendations.

The inspection consisted of a visual observation of the tanks exterior and interior components and coating system. The tank was not drained for the inspection and all interior assessment data was recorded using real time video with live voice narration as well as still photographs.

Condition Observations

Conditions noted during the inspection are documented in the following pages and are supplemented with color photographs. Condition ratings used to describe the inspection findings are annotated as follows:

Excellent: No deficiencies noted.

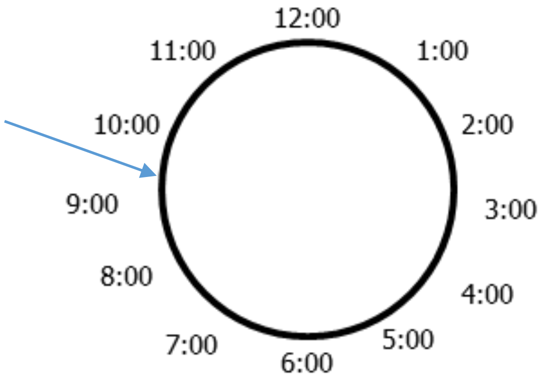
Good: Minor deficiencies noted. Item is functioning as designed.

Fair: Major deficiencies noted. Item is in need of repairs to continue functioning as designed.

Poor: Repair or replacement required immediately. Item may no longer function as designed.

Date of Cleaning & Inspection :	April 1, 2024	Tank Name :	Division 22-1 Reservoir
Water Loss from Cleaning:	18,000 Gallons	Diameter :	50′
Construction Type:	Welded Steel	Height :	35′
Capacity(gal):	520,000	Year Built :	1971

Exterior Wall



Description

Appeared to be in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with signs of staining, delamination and areas of mossy overgrowth.

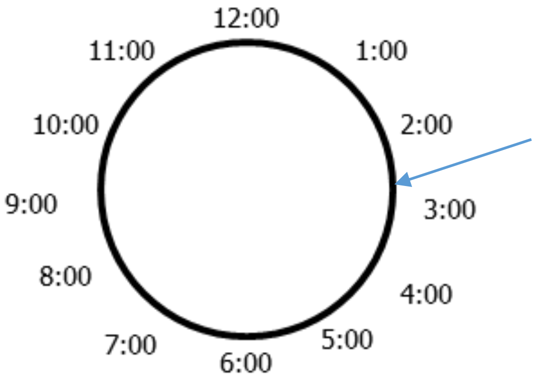
Coating Failure

10%

Recommendations

None at this time.

Exterior Wall



Description

Appeared to be in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with signs of staining, delamination and areas of mossy overgrowth.

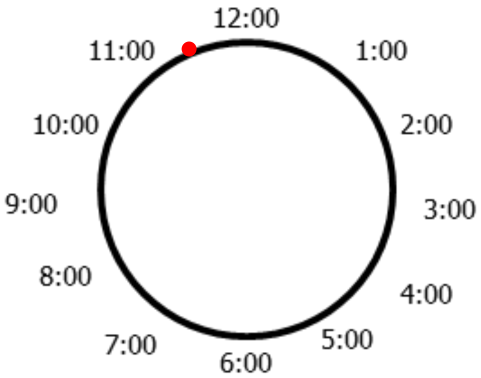
Coating Failure

10%

Recommendations

None at this time.

Exterior Manway



Description

The gasket appeared to be fully intact and the hatch appeared to be in good working condition with corrosion present.

Corrosion Present

5%

Rust Grade

5

Coating System

Appeared to be in fair condition with delamination present.

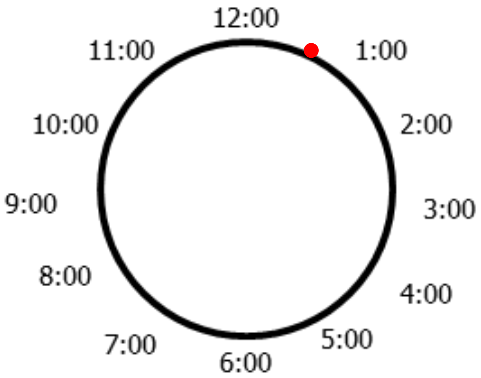
Coating Failure

15%

Recommendations

None at this time.

Exterior Manual Level Indicator



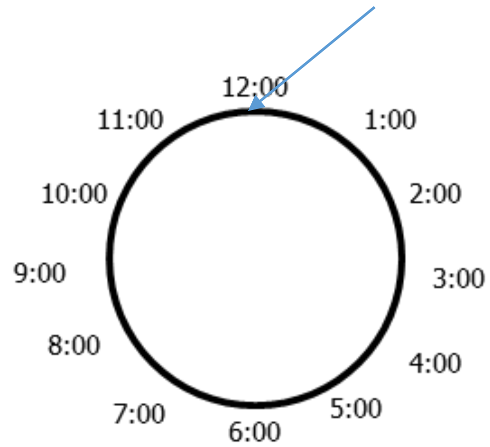
Description

Appeared to be in good working condition with no visible discrepancies.

Recommendations

None at this time.

Exterior Ladder



Description

Appeared to be structurally sound and in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with signs of staining, delamination and areas of mossy overgrowth.

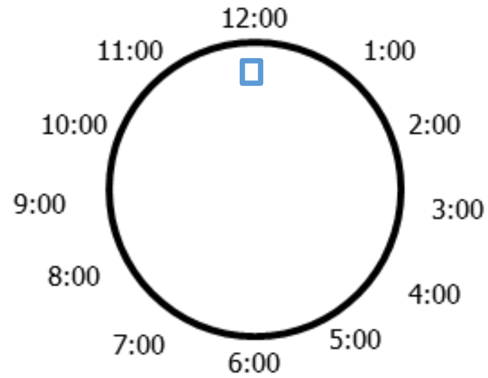
Coating Failure

10%

Recommendations

None at this time.

Exterior Hatch



Description

Appeared to be in fair condition with corrosion present.

Corrosion Present

15%

Rust Grade

4

Coating System

Appeared to be in fair condition with rust staining and delamination present.

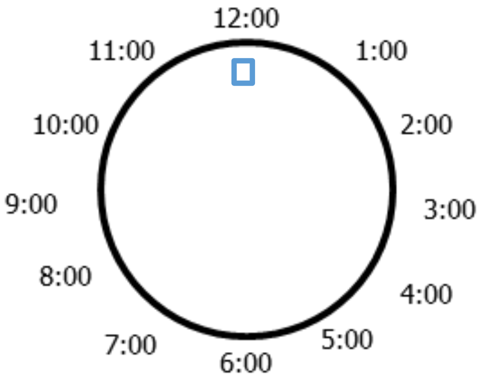
Coating Failure

15%

Recommendations

None at this time.

Exterior Hatch Lid



Description

Appeared to be in fair working condition with corrosion present.

Corrosion Present

15%

Rust Grade

4

Coating System

Appeared to be in fair condition with rust staining and delamination present.

Coating Failure

15%

Recommendations

None at this time.

Exterior Roof

Description

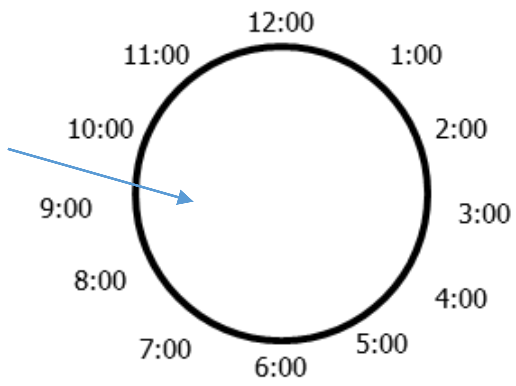
Appeared to be in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10



Coating System

Appeared to be in good condition with signs of staining and areas of mossy overgrowth.

Coating Failure

5%

Recommendations

None at this time.

Exterior Roof

Description

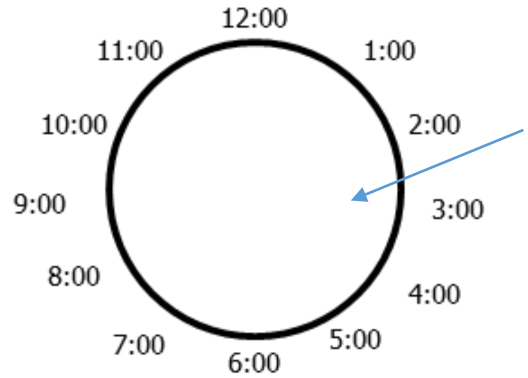
Appeared to be in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10



Coating System

Appeared to be in good condition with signs of staining and areas of mossy overgrowth.

Coating Failure

5%

Recommendations

None at this time.

Exterior Vent

Description

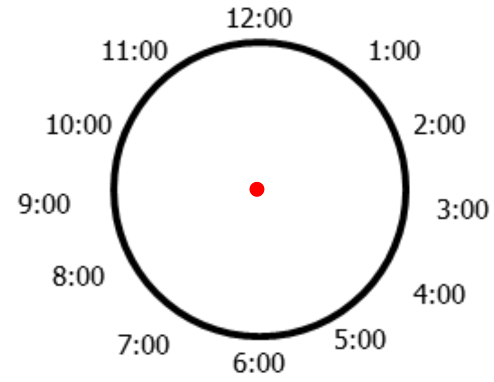
Appeared to be in good working condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10



Coating System

Appeared to be in good condition with light staining present.

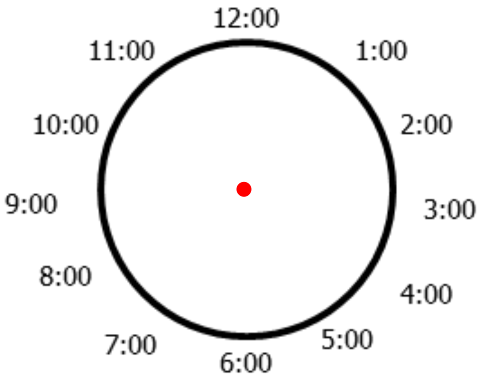
Coating Failure

< 5%

Recommendations

None at this time.

Exterior Vent Screen



Description

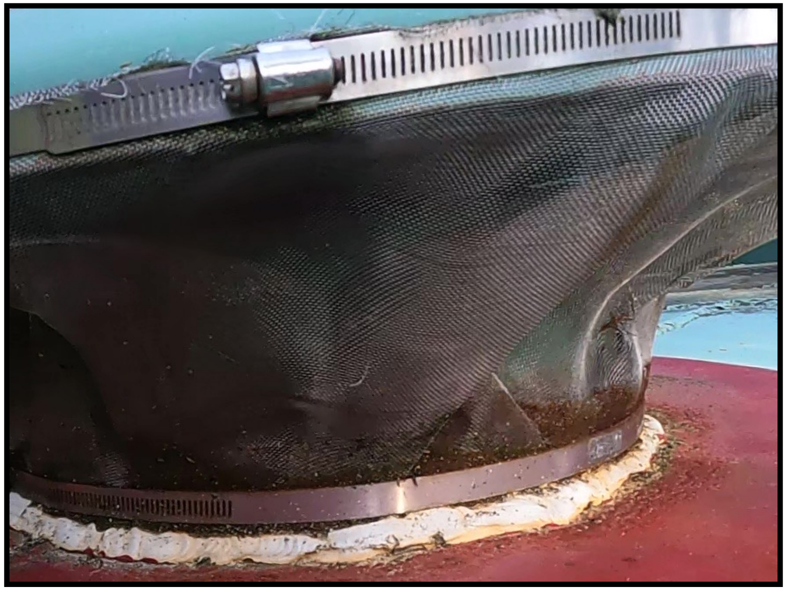
Appeared to be fully intact and in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10



Coating System

N/A

Coating Failure

N/A

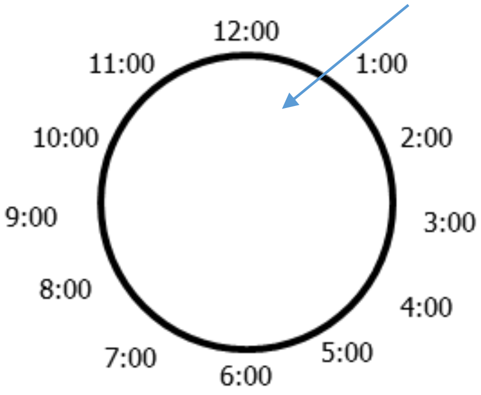
Recommendations

None at this time.

Interior Sediment

Description

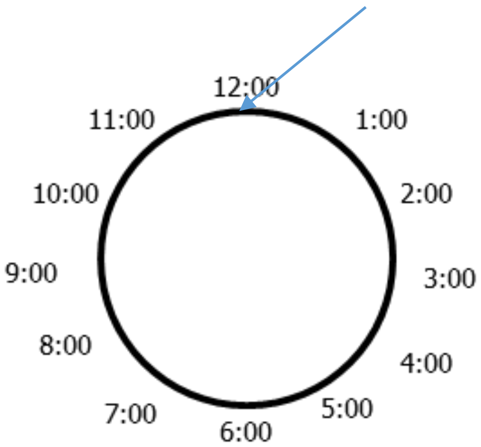
¼” of sediment.



Recommendations

None at this time.

Interior Ladder



Description

Appeared to be structurally sound and in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with light staining present.

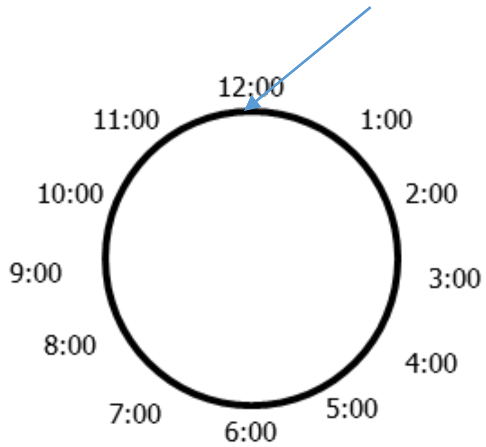
Coating Failure

< 5%

Recommendations

None at this time.

Interior Ladder



Description

Appeared to be structurally sound and in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with light staining present.

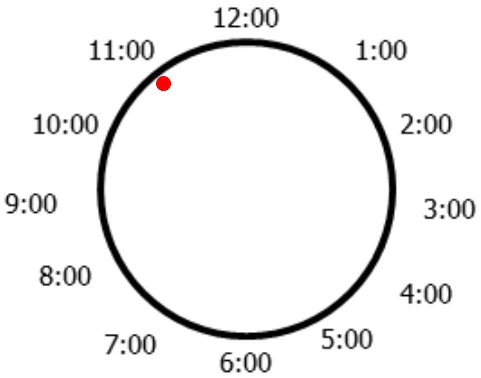
Coating Failure

< 5%

Recommendations

None at this time.

Interior High-Fill Inlet



Description

Appeared to be in good working condition with corrosion present.

Corrosion Present

5%

Rust Grade

5

Coating System

Appeared to be in good condition with rust staining and delamination present.

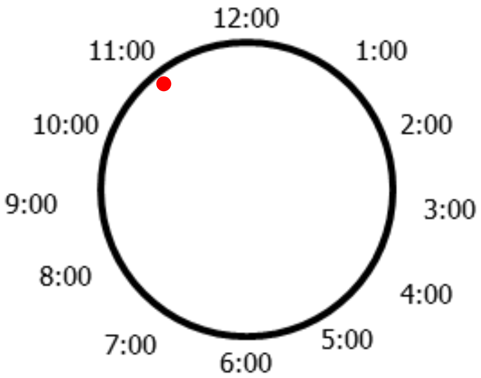
Coating Failure

5%

Recommendations

None at this time.

Interior High-Fill Inlet



Description

Appeared to be in good working condition with corrosion present.

Corrosion Present

5%

Rust Grade

5

Coating System

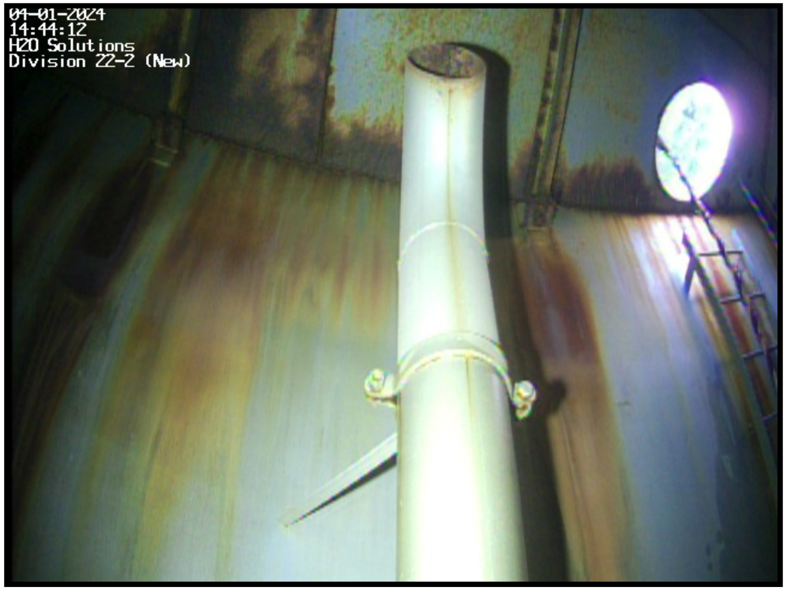
Appeared to be in good condition with rust staining and delamination present.

Coating Failure

5%

Recommendations

None at this time.



Interior Outlet & Drain

Description

Appeared to be in good working condition with spots of minor corrosion present.

Corrosion Present

< 5%

Rust Grade

5

Coating System

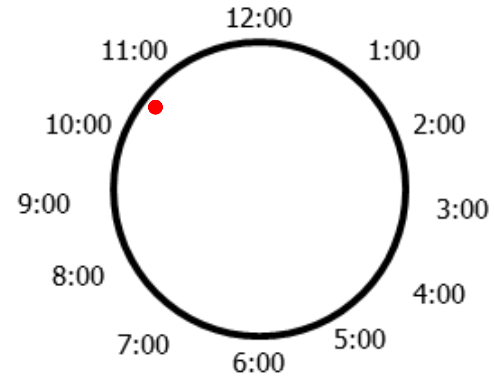
Appeared to be in good condition with light staining present.

Coating Failure

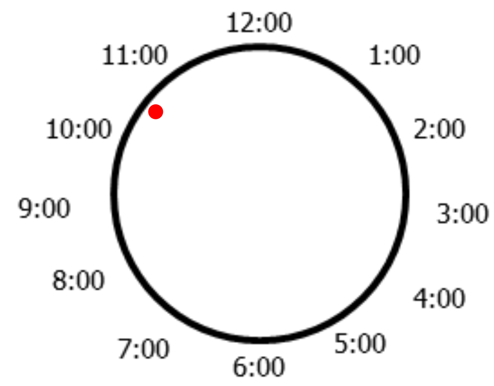
< 5%

Recommendations

None at this time.



Interior Outlet



Description

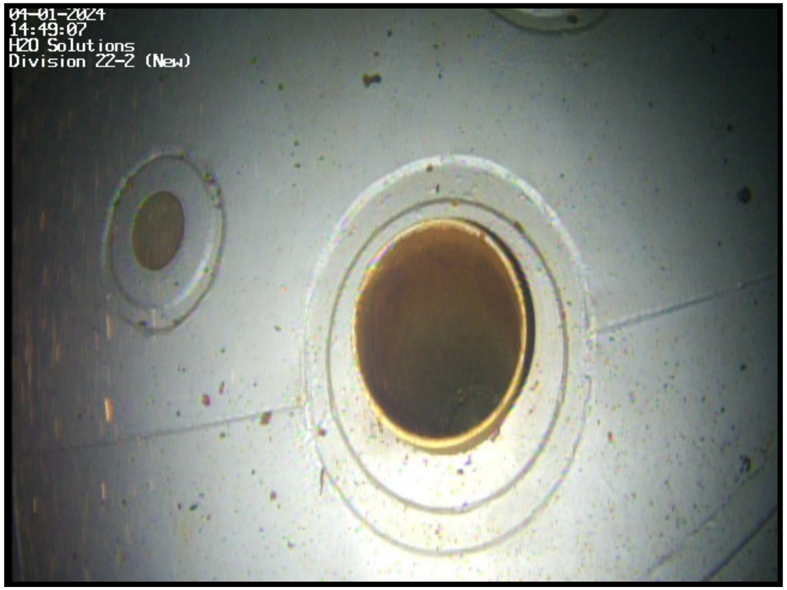
Appeared to be in good working condition with spots of minor corrosion present.

Corrosion Present

< 5%

Rust Grade

5



Coating System

Appeared to be in good condition with light staining present.

Coating Failure

< 5%

Recommendations

None at this time.

Interior Drain

Description

Appeared to be in good working condition with spots of minor corrosion present.

Corrosion Present

< 5%

Rust Grade

5

Coating System

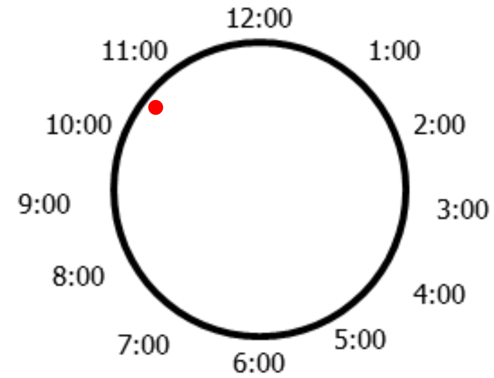
Appeared to be in good condition with light staining present.

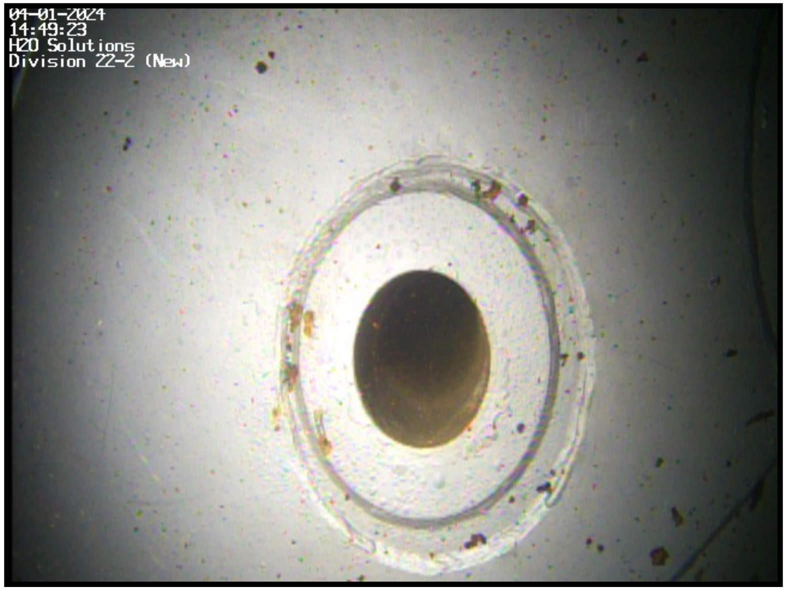
Coating Failure

< 5%

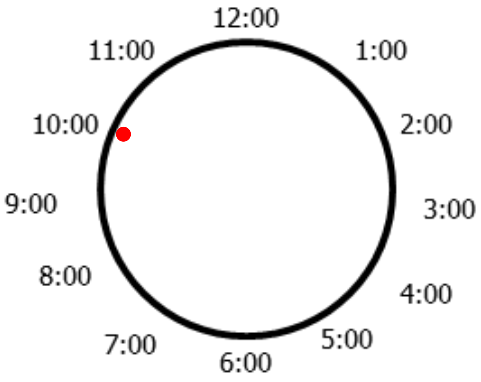
Recommendations

None at this time.





Interior Overflow



Description

Appeared to be in good working condition with corrosion present.

Corrosion Present

5%

Rust Grade

5

Coating System

Appeared to be in good condition with rust staining and delamination present.

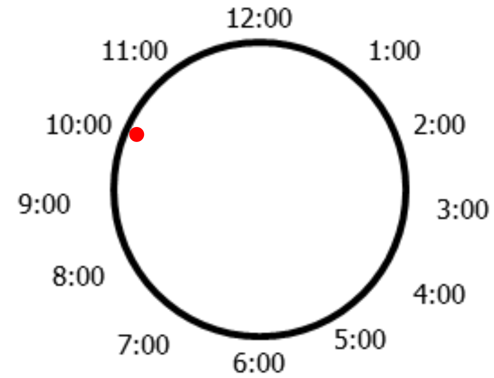
Coating Failure

5%

Recommendations

None at this time.

Interior Overflow



Description

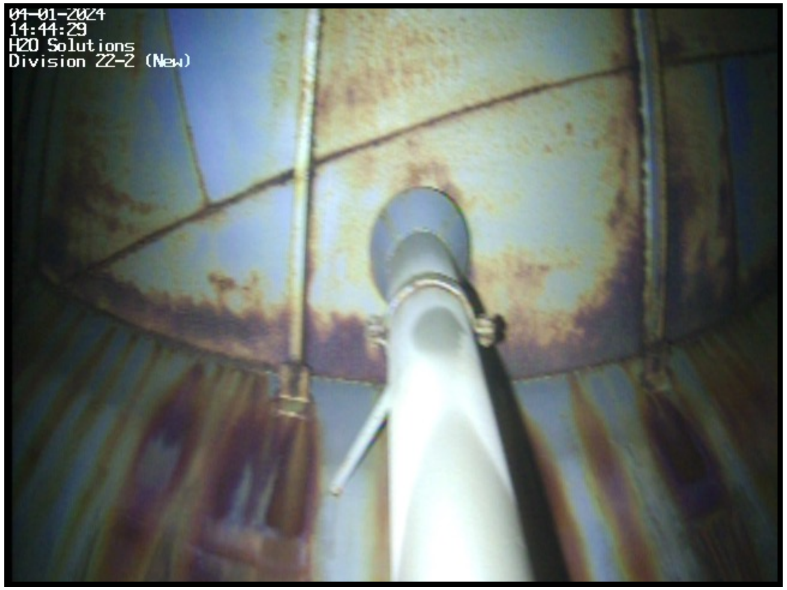
Appeared to be in good working condition with corrosion present.

Corrosion Present

5%

Rust Grade

5



Coating System

Appeared to be in good condition with rust staining and delamination present.

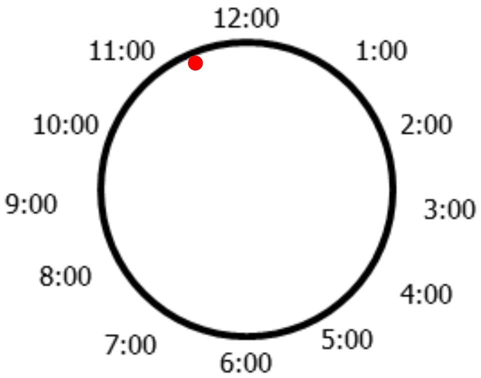
Coating Failure

5%

Recommendations

None at this time.

Interior Manway



Description

The gasket appeared to be fully intact and the hatch appeared to be in good working condition with corrosion present.

Corrosion Present

10%

Rust Grade

4

Coating System

Appeared to be in good condition with rust staining present.

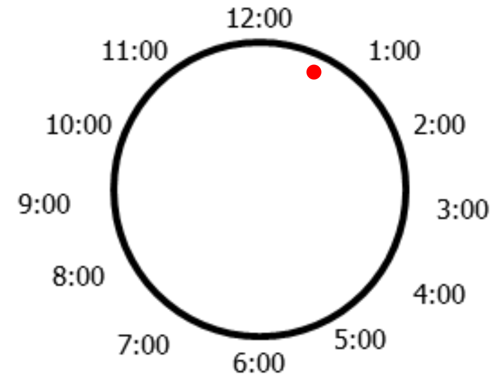
Coating Failure

10%

Recommendations

None at this time.

Interior Manual Level Indicator

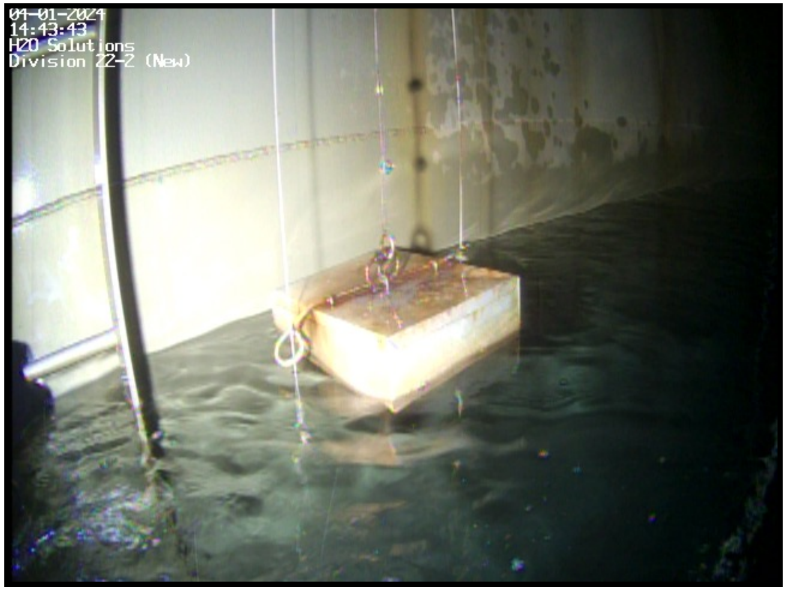


Description

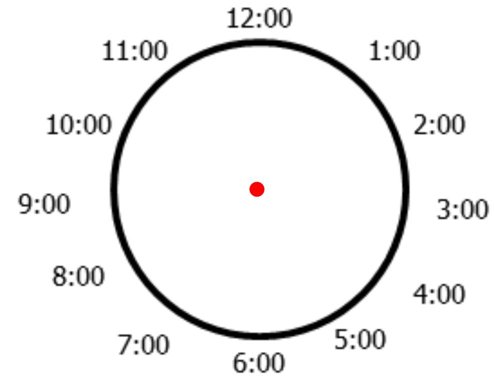
Appeared to be in good working condition with no visible discrepancies.

Recommendations

None at this time.



Interior Column Base



Description

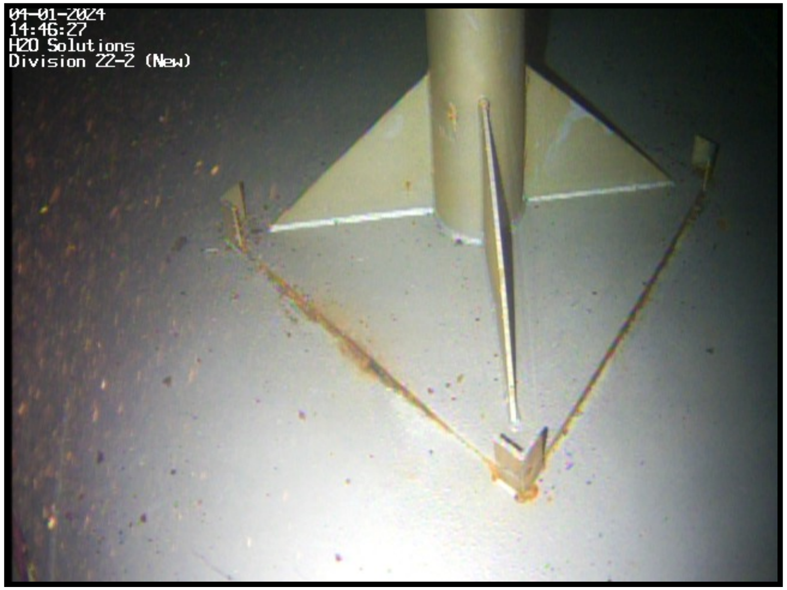
Appeared to be structurally sound and in good condition with corrosion present.

Corrosion Present

5%

Rust Grade

5



Coating System

Appeared to be in good condition with rust staining present.

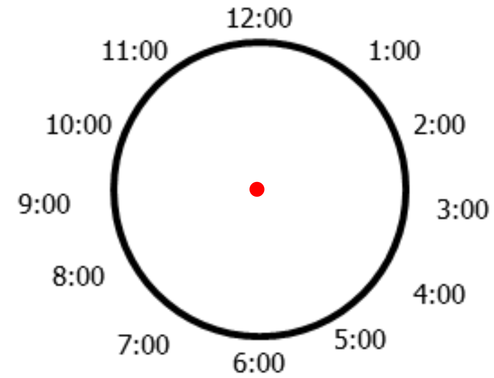
Coating Failure

< 5%

Recommendations

None at this time.

Interior Column



Description

Appeared to be structurally sound and in good condition with no visible signs of corrosion.

Corrosion Present

0%

Rust Grade

10

Coating System

Appeared to be in good condition with no visible discrepancies.

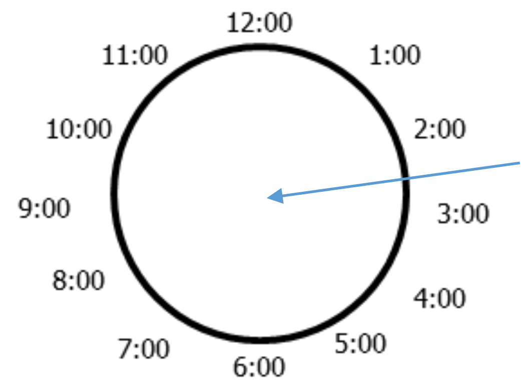
Coating Failure

0%

Recommendations

None at this time.

Interior Ceiling



Description

Appeared to be in poor condition with heavy corrosion present.

Corrosion Present

> 50%

Rust Grade

1

Coating System

Appeared to be in poor condition with heavy rust staining and delamination present.

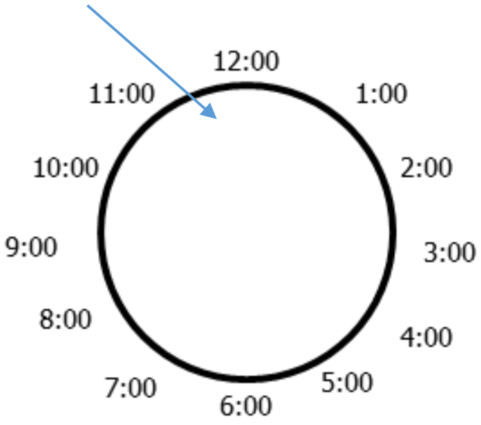
Coating Failure

> 50%

Recommendations

Blast and re-coat the ceiling.

Interior Ceiling



Description

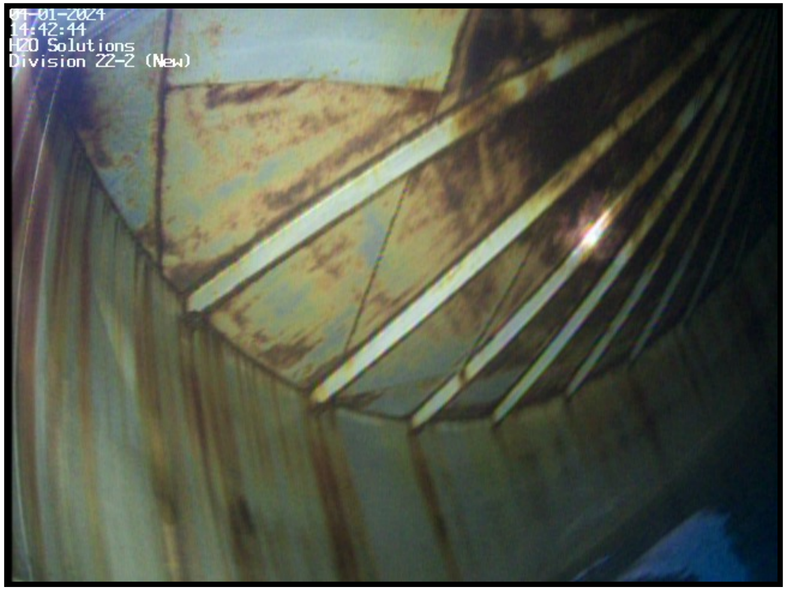
Appeared to be in poor condition with heavy corrosion present.

Corrosion Present

> 50%

Rust Grade

1



Coating System

Appeared to be in poor condition with heavy rust staining and delamination present.

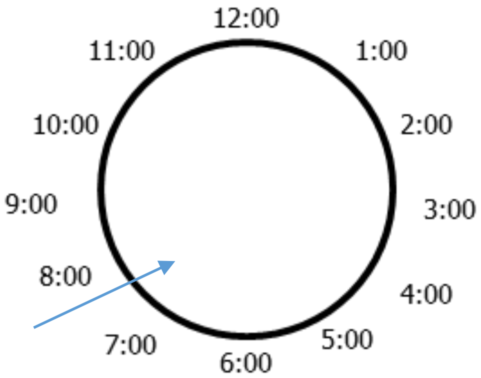
Coating Failure

> 50%

Recommendations

Blast and re-coat the ceiling.

Interior Ceiling



Description

Appeared to be in poor condition with heavy corrosion present.

Corrosion Present

> 50%

Rust Grade

1

Coating System

Appeared to be in poor condition with heavy rust staining and delamination present.

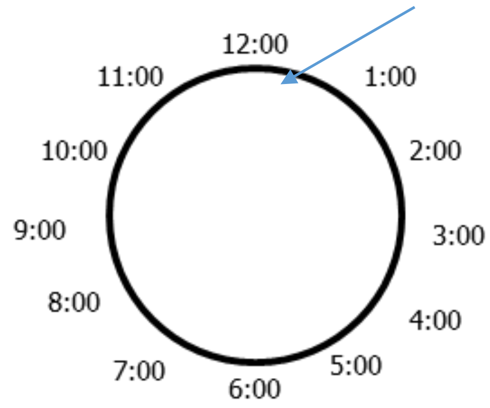
Coating Failure

> 50%

Recommendations

Blast and re-coat the ceiling.

Interior Wall



Description

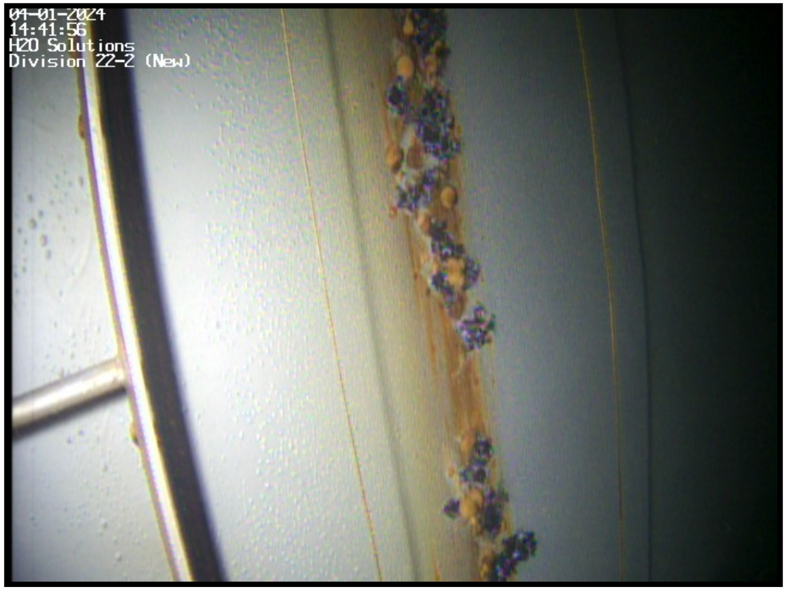
Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4



Coating System

Appeared to be in good condition with areas of rust staining and delamination present.

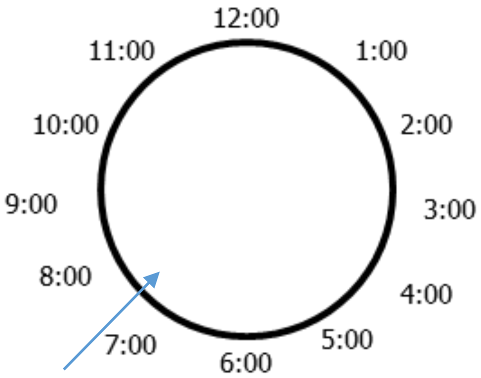
Coating Failure

10%

Recommendations

None at this time.

Interior Wall



Description

Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4

Coating System

Appeared to be in good condition with areas of rust staining and delamination present.

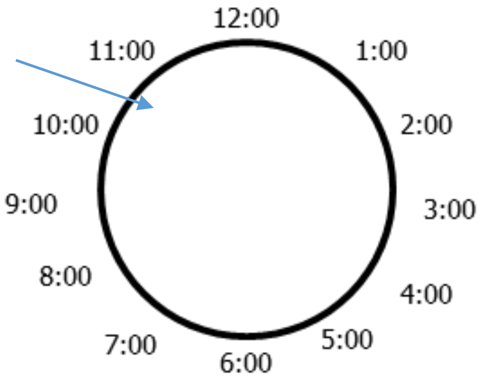
Coating Failure

10%

Recommendations

None at this time.

Interior Wall



Description

Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4

Coating System

Appeared to be in good condition with areas of rust staining and delamination present.

Coating Failure

10%

Recommendations

None at this time.

Interior Floor

Description

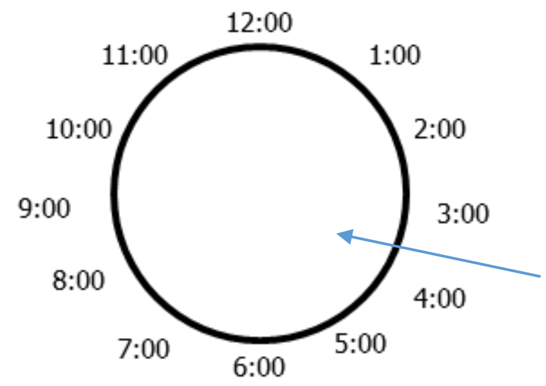
Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4



Coating System

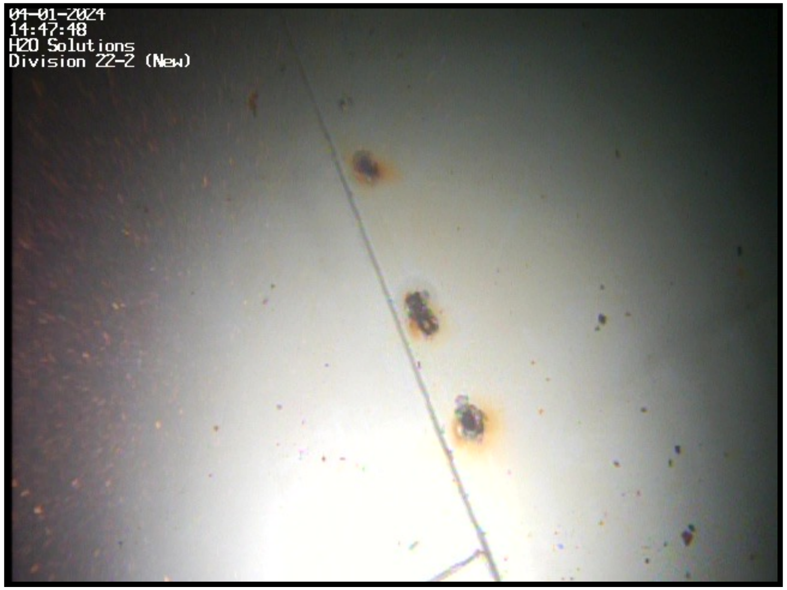
Appeared to be in good condition with areas of rust staining and delamination present.

Coating Failure

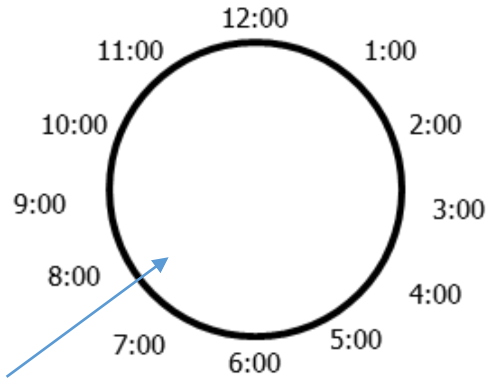
10%

Recommendations

None at this time.



Interior Floor



Description

Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4

Coating System

Appeared to be in good condition with areas of rust staining and delamination present.

Coating Failure

10%

Recommendations

None at this time.

Interior Floor

Description

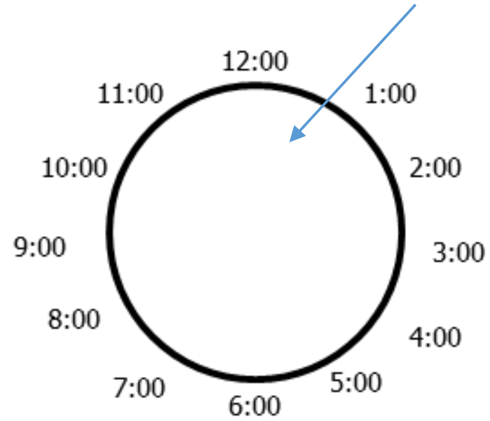
Appeared to be in good condition with areas of corrosion present.

Corrosion Present

10%

Rust Grade

4



Coating System

Appeared to be in good condition with areas of rust staining and delamination present.

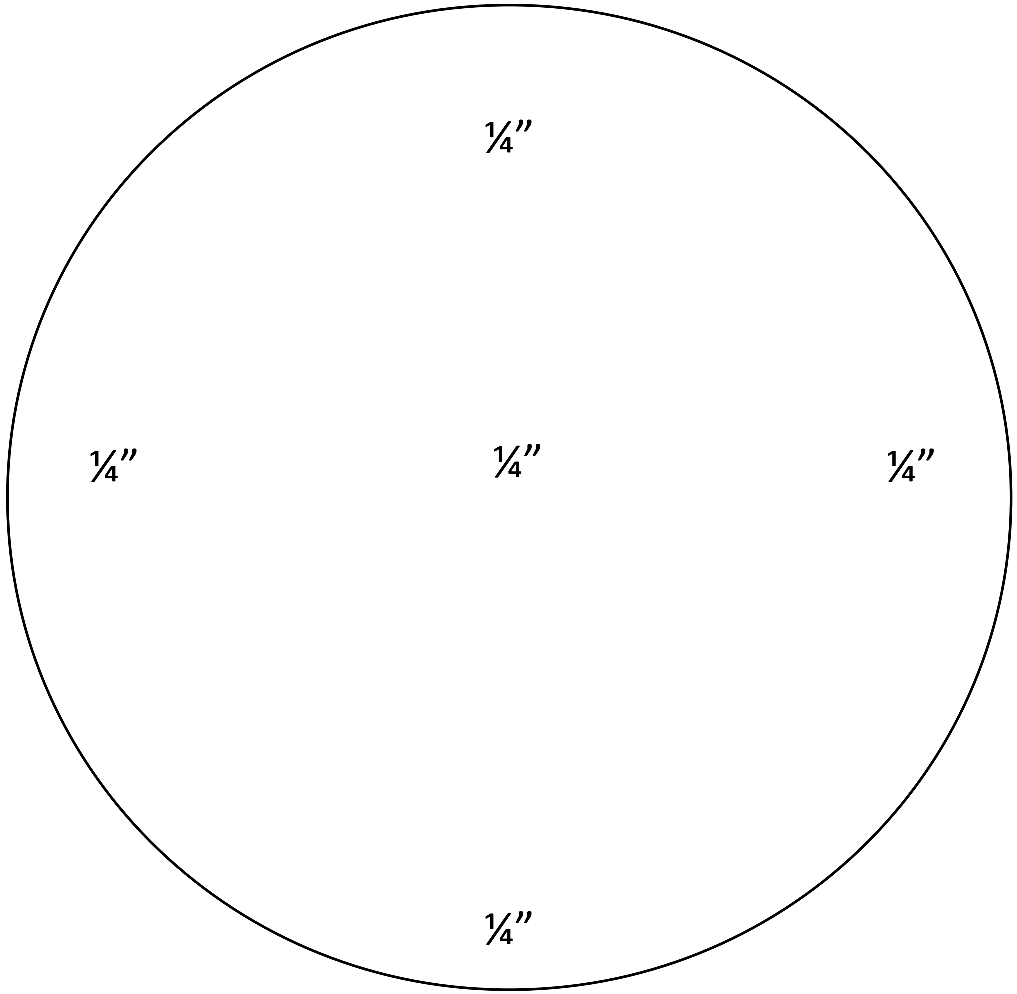
Coating Failure

10%

Recommendations

None at this time.

Sediment Depth



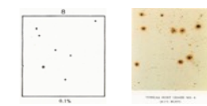
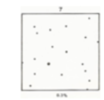
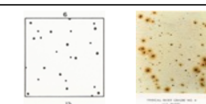
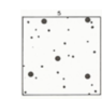
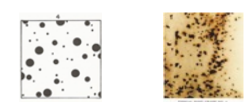
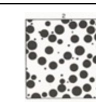
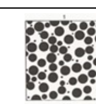
References

Standard Method of Evaluating Degree of Rusting on Painted Steel Surfaces – SSPC-Vis 2-82 & ASTM D 610-85 (1989)

The graphical representations show examples of area percentages, which may be helpful in rust grading. The use of photographical reference standards requires the following precautions:

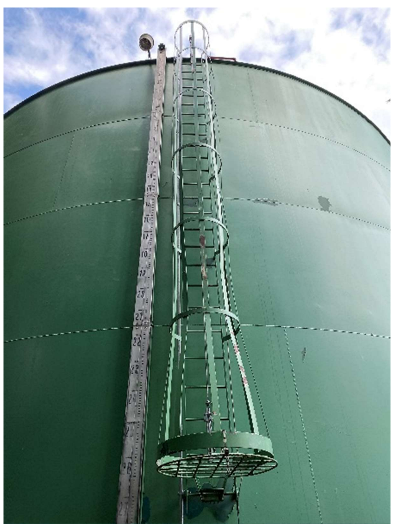
- ❖ Some finishes are stained by rust. This staining must not be confused with the actual rusting involved.
- ❖ Accumulated dirt or other material may make accurate determination of the degree of rusting difficult.
- ❖ Certain types of deposited dirt that contain iron or iron compounds may cause surface discoloration that should not be mistaken for corrosion.
- ❖ It must be realized that failure may vary over a given area and discretion must therefore be used in applying these reference standards.
- ❖ In evaluating surfaces, consideration shall be given to the color of the finish coating, since failures will be more apparent on a finish that shows color contrast with rust, such as white, than on a similar color, such as iron oxide finish.
- ❖ The photographic reference standards are not required for use of the rust-grade scale since the scale is based upon the percent of the area rusted and any method of assessing area rusted may be used to determine the rust grade.

A	Similar to European Scale of Degree of rusting for Anti-Corrosive Paints (1961) (Black & White)
B	Corresponds to SSPC Initial Surface Conditions E (0 - 0.1%) and BISRA (British Iron and Steel Research Association) 0.1%
C	Corresponds to SSPC Initial Surface Conditions F (0.1%-1%) and BISRA 1%
D	Corresponds to SSPC Initial Surface Conditions G (1 - 10%)
E	Rust grades below 4 are of no practical importance in grading performance of paints
F	Corresponds to SSPC Initial Surface Condition H (50 - 100%)

Rust Grades A	Description	Graphical Representation
10	No rusting or less than 0.01% of surface rusted	Unnecessary
9	Minute rusting less than 0.03% of surface rusted	
8B	Few isolated rust spots less than 0.1% of surface rusted	
7	Less than 0.3% of surface rusted	
6C	Extensive rust spots but less than 1% of surface rusted	
5	Rusting to the extent of 3% of surface rusted	
4D	Rusting to the extent of 10% of surface rusted	
3E	Approximately on sixth of the surface rusted 16%	
2	Approximately one third of the surface rusted 33%	
1	Approximately one half of the surface rusted 50%	

LAKE WHATCOM WATER & SEWER DISTRICT

DIVISION 22-1, DIVISION 30, AND GENEVA RESERVOIR COATING CONDITION ASSESSMENT



EVERGREEN COATING ENGINEERS

December 2022

INTRODUCTION

Lake Whatcom Water and Sewer District (District) contracted with Evergreen Coating Engineers, LLC. (ECE) to complete a condition assessment of three of the District's reservoirs: Division 22-1, Division 30, and the Geneva Reservoir. The field evaluation was conducted on September 14 and 15, 2022 by Lance Stevens, P.E., NACE CIP Level 3.

REVIEW OF EXISTING DOCUMENTATION

The District provided copies of dive inspections of all three reservoirs performed by H2O Solutions, Inc. on April 10, 2018 (H2O report). The reports were reviewed prior to the site visit. After the site visits were conducted, the District provided the "Reservoir Seismic Vulnerability Assessment Technical Report" prepared by BHC Consultants in December 2016 (BHC Report). The District provided the Option C Summary information regarding changes to the reservoir storage requirements as part of the Division 7 Reservoir being designed. Information from these reports is utilized in the Analysis section of this report.

SITE INVESTIGATION

The site inspection started with a floating inspection of the interior roof and general condition assessment of the exterior of the Division 22-1 Reservoir followed by the general condition assessment of the exterior of the Division 30 Reservoir. Six 20mm adhesion testing dollies were placed on each reservoir and coating samples taken. The second day began with a floating inspection of the interior roof, general condition assessment of the exterior, and coating sample grab and repair on the Geneva Reservoir. The adhesion tests were then performed on the Division 22-1 and 30 Reservoirs followed by the repair of the test and sample scars. Adhesion testing was not performed on the Geneva Reservoir due to the deteriorated condition of the exterior coating system. Coating thickness measurements were taken of the exterior coating system on the Division 22-1, Division 30, and Geneva reservoirs. Given the deteriorated nature of the interior coating system on each reservoir, per field discussion with Kristin Hemenway, interior coating thickness measurements were not taken.

Coating Adhesion Testing

There are two options for recoating a tank. The first option is for all of the coatings to be removed to bare steel and a new coating system applied. The second option is for the existing coatings to be cleaned, damaged areas repaired, and a new system applied over the old system. Not removing the existing system lowers project cost by eliminating the containment that must be constructed if the existing coatings are blasted off. From experience, the cost to blast clean a structure versus pressure wash and hand clean every rusted spot are about equal. It must be understood that applying a new system over an existing system, or top coating, does carry risk to the owner. Any issue that occurs with the existing coating system after top coating will not be warranted by the Contractor as there is likely an existing condition associated with the issue that is outside of his control. The issues can be delamination from stresses that are imparted to the existing system by the new coating system or sometimes from the solvents used in the new system which can attack the old coating system causing failures. There are two ways to help lessen these risks, but some risk does remain. The first way is adhesion testing and the second is to paint large patches of the new coating system on the existing system and give it time to field test the effects.

Adhesion testing is utilized to determine how tight the existing coating system is held to itself and to the structure. The purpose of the testing is to determine whether the existing coating system can withstand the weight of the new coatings as well as the stresses that will be imparted as the new coatings dry. The test is conducted by utilizing an epoxy adhesive to glue an aluminum dolly to the coating. Once the epoxy is cured, an adhesion tester is attached to the dolly and pressure is applied until the dolly is pulled from the surface or 3,500 psi is reached. If the coatings fail, they will fail in some combination of cohesive failure which is within the same layer of paint, and/or adhesive failure which is failure between layers of paint or between the paint and the substrate. The glue can also fail adhesively or cohesively but in either event it is noted as a percentage of glue failure. For this test, a Defelsko PosiTest AT-A Automatic S/N 17275 was utilized which has a hydraulic pump that automatically applies a smooth and continuous pull-off pressure which will provide the best result.

Six dollies were set on each tank with three placed on the roof and three placed on the first ring of the shell wall. The test results are provided in tabular format under the site visit description for each reservoir. Typically, results over 1,000 psi are acceptable and over 1,400 psi are preferred. It should be noted that these are values that Evergreen Coating Engineers recommends and industry values, depending upon the source, can be as low as 600-700 psi. We believe that the risk that the Owner carries in opting to top coat versus the savings involved should meet a higher standard than the industry minimums.

Evaluating Rust on Steel Surfaces

Rust grades utilized to describe the degree of rusting on surfaces are per SSPC-VIS 2: Standard Method of Evaluating Degree of Rusting on Painted Steel Surfaces. Table 1 contains the definitions the rust grades, percentage of rusting, and type of rusting. Photographs of the various percentages and types are located in the SSPC-VIS 2 Manual. Spot rusting refers to rusting where the bulk of the rusting is concentrated in a few localized areas of the painted surface. General rusting refers to various size rust spots that are randomly distributed across the surface. Pinpoint rusting refers to rust that is distributed across the surface as very small individual specks of rust.

Table 1: Scale and Description of Rust Grades per SSPC-VIS 2

Rust Grade	Percent of Surface Rusted	Photographic Standard[1]		
		Spot	General	Pinpoint
10	Less than or equal to 0.01%	NONE		
9	Greater than 0.01% to 0.03%	9-S	9-G	9-P
8	Greater than 0.03% to 0.1%	8-S	8-G	8-P
7	Greater than 0.1% to 0.3%	7-S	7-G	7-P
6	Greater than 0.3% to 1.0%	6-S	6-G	6-P
5	Greater than 1.0% to 3.0%	5-S	5-G	5-P
4	Greater than 3.0% to 10.0%	4-S	4-G	4-P
3	Greater than 10.0% to 16.0%	3-S	3-G	3-P
2	Greater than 16.0% to 33.0%	2-S	2-G	2-P
1	Greater than 33.0% to 50.0%	1-S	1-G	1-P
0	Greater than 50.0%	NONE		

[1]Photographic references are found in the SSPC-VIS 2 publication.

Testing for Total Metals in the Coating System

Samples were taken of the interior and exterior coating systems for each reservoir and tested by EPA Method 6010D (SW-846) for RCRA 8 Metals except for Mercury. Mercury is not a metal known to be found in coating systems and per Method 6010D, is not typically analyzed by this method. Results for lead, which is the primary metal of concern, are provided in the description for each reservoir and the full results are provide in Appendix A: Metals Testing Laboratory Results.

Division 22-1 Reservoir

The Division 22-1 Reservoir is a 50 feet diameter by 35 feet tall, 500,000 gallon, welded steel reservoir that was constructed in 1971 by Union Tank Works. The reservoir has one 24-inch by 18-inch elliptical manway and one round, 24-inch diameter rooftop access hatch for interior entry. The reservoir has a level gauge that faces the driveway and an exterior light that is mounted above the level gauge and ladder. A water sample stand and impressed current cathodic protection rectifier are also mounted near the base of the ladder. Photographs are provided in Appendix B: Division 22-1 Reservoir Photos.

The roof is accessed by a ladder with a ladder cage and safety climb device. The ladder cage ends flush with the rooftop and safety climb device only extends a couple feet above the roof making the transition onto the roof from the ladder difficult. For this reason, the District currently only allows access to the roof via manlift. The ladder and cage are not compliant with current WAC 296-876-600 due to the ladder rungs being closer than 7-inches to the shell wall, as well as the dimensions and flare of the cage not meeting the WAC requirements. Once on the roof, there is a fall restraint cable attached to an anchor near the vent for use in fall protection. There are five cathodic protection ports and one junction box for the connection of the reference anodes to the rectifier. There are two U-shaped railings marked as unsafe for tie-off use near the hatch.

The interior roof and area above the waterline were inspected by inflatable raft. The inspection equipment was deployed to the roof of the reservoir. A tarp was laid out on the roof, the raft was inflated, and all gear was disinfected utilizing a 200+ ppm bleach solution for approximately 15 minutes. The raft was deployed inside of the reservoir and the inspection was begun. The interior structure of the roof consists of one center column and dollar plate supporting radial C-channel rafters that connect to the side shell. The rafters are bolted to the dollar plate and are bolted to an angle bracket that is welded to the side shell. Many of the bolts are missing at the rafter to dollar plate connection. The rust grades of the interior components are provided in Table 2: Division 22-1 Interior and Exterior Surfaces Rust Grades.

Table 2: Division 22-1 Interior and Exterior Surfaces Rust Grades

Interior Surfaces	Rust Grade	Exterior Surfaces	Rust Grade
Roof Plates	0	Roof Plates	4-G
Rafters	0	Shell Wall	8-G
Shell Wall	6-S	Ladder and Cage	4-G
Center Column	5-P		
Ladder	3-G		
Overflow Pipe	5-P		
Inlet Pipe	4-P		

The interior shell wall of the reservoir was covered in rust staining but it did not appear that there was much corrosion on the wall above the waterline except at the rim angle where the shell wall connects to the roof plates. The roof plates, rafter angle brackets on the shell wall, rafters, and bolts connecting the rafters to the angle brackets have undergone significant corrosion.

The exterior shell wall has a significant number of coating repair patches distributed around the reservoir. Areas of delamination exist as well as areas of corrosion. The lower foot of the shell wall was covered in mildew and dirt around the reservoir but the areas above that appeared clean. The reservoir roof was heavily covered with lichens, dirt, and evergreen needles. Delamination of the coating system was observed in multiple locations all around the reservoir without a distinguishable pattern; however, the primer was still largely present. The roof vent is an older style “mushroom” vent and was covered with #24 mesh. The doubler plate for the vent riser does not sit flush with the roof and some type of filler material, maybe a foam or mastic, was used to seal the gaps. The hatch riser has corrosion over approximately one-third of the exterior surface area; however, the hatch lid appears to be in good condition.

The site around the reservoir is generally well kept. The ringwall sits a couple of inches above the surrounding grade on average although a few areas lower than that exist. The sill plate grout is in fair condition with some missing. There is a gravel driveway that is at least ten feet wide in good condition around the reservoir. There are trees on the east and west sides of the reservoir while the north and south sides are open. No tree limbs overhang or touch the reservoir but limbs do overhang the driveway. The site appears well drained.

The reservoir is in a developed neighborhood with houses immediately adjacent to the reservoir. The reservoir is not protected by fencing. The ladder is protected by a cage and cage guard. The cage could be bypassed for access to the roof without much difficulty. No intrusions alarms were noted on the reservoir.

The results of the adhesion testing are provided in Table 3: Division 22-1 Reservoir Adhesion Test Results below. Dollies 1, 2, and 3 were placed on the shell wall of the reservoir while Dollies 4, 5, and 6 were placed on the roof. The coating layers are as follows from the primer to the outermost coat, respectively: Tan primer, red intermediate, dark green finish coat, silver tie-coat, and light green top coat.

Table 3: Division 22-1 Reservoir Adhesion Test Results

Dolly No.	Max: 3,500 PSI	Failure %			Location of Failure[1]
		Adhesion %	Cohesive %	Glue %	
1	1662		7		B
		25			D/E
			68		F
2	1591			5	Y/Z
			15		B
			80		E
3	1592		100		B
4	661	5			C/D
		10			E/F
				15	Y/F
				25	Y/Z
		45			B/C
5	1152	5			C/D
				35	F/Y
		60			B/C
6	299	25			C/D
		75			B/C

[1] A = Substrate; B= Primer coat; C= Intermediate coat; D= Finish; E= Tie-Coat; F= Topcoat; Y= Adhesive; Z= Dolly

The interior coating system tested at 4,500 ppm for lead and the exterior coating system tested at 16,000 ppm for lead. Dry film thickness testing of the exterior coating system averaged 15.2 mils. As discussed with the District in the field, the interior coating system was not tested due to the condition of that coating system.

Division 30 Reservoir

The Division 30 Reservoir is a 25 feet diameter by 40 feet tall, 150,000 gallon, welded steel reservoir that was constructed in 1973 by Union Tank Works. The reservoir has one round 24-inch diameter manway and one 24-inch square rooftop access hatch for interior entry. The reservoir has a level gauge that faces the driveway and an exterior light that is mounted above the level gauge and ladder. A water sample stand is located at the top of the driveway. A galvanic cathodic protection rectifier, meter, and electrical cabinet are mounted near the base of the ladder. Photographs are provided in Appendix C: Division 30 Reservoir Photos.

The roof is accessed by a ladder and landing system with a ladder cage and safety climb device. There is one intermediate platform and the cage extends above the reservoir to the same height at the guardrails that extend out on either side from the cage. The safety climb device only extends a couple feet above the roof making the transition onto the roof from the ladder difficult. Once on the roof, there is a fall restraint cable attached to an anchor near the roof vent for use in fall protection. There are five cathodic protection ports and one junction box for the connection of the reference anodes to the rectifier.

The interior roof and area above the waterline were inspected from the access hatch. The interior ladder has a ladder cage which prevents the interior from being inspected from a raft. The roof is a self-supporting dome and therefore has no rafters or columns. The rust grades of the interior components are provided in Table 4: Division 30 Interior and Exterior Surfaces Rust Grades.

Table 4: Division 30 Interior and Exterior Surfaces Rust Grades

Interior Surfaces	Rust Grade	Exterior Surfaces	Rust Grade
Roof Plates	2-G	Roof Plates	5-S
Shell Wall	6-S	Shell Wall	5-S
Ladder	5-G	Ladder and Cage	7-S
Overflow Pipe	5-S		
Inlet Pipe	3-G		

The interior shell wall of the reservoir is undergoing corrosion mostly above the waterline although it is not significant at this time. There is a significant amount of rust staining present from corrosion on the roof plates. The roof plates are rusting over a significant portion of the roof but the corrosion appears to be light surface rusting at this time and not likely to require any structural repairs nor leave any significant pitting. The riser for the access hatch is heavily pitted, actively corroding, and should be cleaned and coated soon.

The exterior shell wall has a number of coating repair patches mostly on the lower half of the first ring of the reservoir. These may be the result of rock chips from mowing the area between the reservoir and driveway. Areas of delamination between the top and intermediate coats are occurring in that area as well. One area on the top ring to the left of the ladder has several large coating failures that are actively corroding. The lower foot of the shell wall has mildew growth but most of the rest of the shell wall was clean of growth. The exception is the backside of the reservoir where it is apparent the crew could not reach to complete the cleaning of the shell wall. In this area, active growth of mildew and moss is occurring. The reservoir roof was heavily covered with lichens, dirt, and evergreen needles; however, the coatings appeared to be fully intact with the exception of the doubler plate for the vent riser which was covered with surface rust. The vent is an older style "mushroom" vent and was covered with #24 mesh.

The site around the reservoir is generally well kept. The ringwall of the reservoir is mostly at grade level in the front and below grade around the back side of the reservoir. The sill plate grout is mostly missing. The reservoir site was dug into a hillside so there is an embankment on the backside of the reservoir with a heavily treed hillside ascending steeply from there. The trees surrounding the reservoir are mature and significantly taller thus the degree of debris on the roof. The site appears well drained.

The reservoir is in a developed neighborhood with houses in the general area of the reservoir. The reservoir is not protected by fencing. The ladder is protected by a cage and cage guard. The cage could be bypassed for access to the roof without much difficulty. No intrusions alarms were noted on the reservoir.

The results of the adhesion testing are provided in Table 5: Division 30 Reservoir Adhesion Test Results below. Dollies 1, 2, and 3 were placed on the shell wall of the reservoir while Dollies 4, 5, and 6 were placed on the roof. The coating layers are as follows from the primer to the outermost coat, respectively: Red primer, dark green finish coat, silver tie-coat, and light green top coat.

Table 5: Division 30 Reservoir Adhesion Test Results

Dolly No.	Max: 3,500 PSI	Failure %			Location of Failure[1]
		Adhesion %	Cohesive %	Glue %	
1	915			10	Y/Z
		30			D/E
			60		C
2	1,837			5	Y/Z
		15			D/E
				50	Y/E
		15			D/C
			15		C
3	945			40	Y/Z
				25	Y/E
		10			D/E
			25		C
4	1,082		60		B
			40		C
5	1,089	10			D/C
			40		C
			50		B
6	1,161		100		C

[1] A = Substrate; B= Primer coat; C= Finish; D= Tie-Coat; E= Topcoat; Y= Adhesive; Z= Dolly

The interior coating system tested at 18,000 ppm for lead and the exterior coating system tested at 11,000 ppm for lead. Dry film thickness testing of the exterior coating system averaged 8.9 mils while the interior tested at 9.7 mils.

Geneva Reservoir

The Geneva Reservoir is a 50 feet diameter by 32 feet tall, 500,000 gallon, welded steel reservoir that was constructed in 1979 by Reliable Steel Fabricators. The reservoir has one 30-inch manway and one 24-inch square rooftop access hatch for interior entry. The reservoir has a level gauge that faces the driveway and three exterior lights. One light is mounted above the level gauge and ladder and the other two are spaced around the reservoir. A water sample stand and an impressed cathodic protection rectifier are at the base of the reservoir. Photographs are provided in Appendix D: Geneva Reservoir Photos.

The roof is accessed by a galvanized ladder with a ladder cage and safety climb device. There is one intermediate platform and the cage extends above the reservoir to the same height at the guardrails that extend out on either side from the cage. Once on the roof, there is a fall restraint cable attached to an anchor near the roof vent for use in fall protection. There are seven cathodic protection ports and one junction box for the connection of the reference anodes to the rectifier.

The interior roof and area above the waterline were inspected by inflatable raft. The inspection equipment was deployed to the roof of the reservoir. A tarp was laid out on the roof, the raft was inflated, and all gear was disinfected utilizing a 200+ ppm bleach solution for approximately 15 minutes. The raft was deployed inside of the reservoir and the inspection was begun. The interior structure of the roof consists of one center column and dollar plate supporting radial C-channel rafters that connect to the side shell. The rafters are bolted to the dollar plate and are bolted to a tab that is welded to the side shell. Two bolts are missing at the rafter to dollar plate connection. The rust grades of the interior components are provided in Table 6: Geneva Interior and Exterior Surfaces Rust Grades.

Table 6: Geneva Interior and Exterior Surfaces Rust Grades

Interior Surfaces	Rust Grade	Exterior Surfaces	Rust Grade
Roof Plates	0	Roof Plates	0
Rafters	0	Shell Wall	4-S
Shell Wall	4-P	Ladder and Cage	10
Center Column	4-S		
Ladder	5-S		
Overflow Pipe	2-P		
Inlet Pipe	4-S		

The interior shell wall of the reservoir has a lot of rust staining but it did not appear that there was much corrosion on the wall above the waterline. The coatings are severely blistered and pinpoint rusting is starting to appear through some of the blisters. The roof plates, rafter tabs on the shell wall, rafters and dollar plate are covered with a mild to moderate surface corrosion.

The coatings on the exterior shell wall are largely intact even though they have lost significant color and gloss. Streaks of rust staining from the roof are found around the reservoir. The top ring has a number of scratches and other scars where the top coat was removed and the primer mostly remains but some corrosion has begun. Overall, the shell appears to still be protected other than minor corrosion in random locations. The reservoir roof was clean but most of the roof is covered with a light surface rust. The remaining top coat and primer are protecting less than 25% of the roof area. The roof vent appears to comply with DOH requirements and was screened with #24 mesh. The hatch riser has light to moderate surface corrosion over most of it.

The site around the reservoir is generally well kept. The ringwall generally sits 2- to 6-inches above the surrounding grade. The sill plate grout is in fair condition with some broken or missing. There is a gravel driveway that is at least ten feet wide in good condition around the reservoir. There are no trees close to the reservoir. The site appears well drained.

The reservoir is fenced in the same site as the maintenance building. The ladder is protected by a cage and cage guard. The cage could be bypassed for access to the roof without much difficulty. No intrusions alarms were noted on the reservoir.

Adhesion testing was not performed on the Geneva Reservoir due to the condition of the exterior coating system. The interior coating system tested at 26 ppm for lead and the exterior coating system tested at 200 ppm for lead. Dry film thickness testing of the exterior coating system averaged 4.1 mils. As discussed with the District in the field, the interior coating system was not tested due to the condition of that coating system.

ANALYSIS

The analyses of these reservoirs are intended to take the observations from this site investigation, dive reports from H2O Solutions, and the seismic assessment performed by BHC Consultants and provide the District with the current state of their reservoirs.

The degree of corrosion on steel surfaces are rated mild, moderate, and severe. Mild corrosion means that the surface is rusted but steel loss is negligible and pitting of the surface is not likely detrimental. Moderate corrosion means that steel loss is likely negligible; however, pitting of the surface is likely. Severe corrosion means that steel loss has occurred that may require repair and heavy pitting of the surface should be expected.

The cost of abrasive blast cleaning and the longevity of applied coating systems are significantly impacted by the degree of surface pitting and steel roughness caused by corrosion, particularly on interior surfaces. The standard for surface preparation is an SSPC SP-10 Near White Blast which requires all rust, coatings, or other materials to be removed from the surface and only 5% staining may remain. Pits and roughened steel can be very difficult to clean to that standard due to the variety of angles required to attack the surface and the very small crevices in which a tiny bit of rust may be. The degree that a surface is roughened, particularly on edges of steel or in cases of severe pitting, increase the likelihood of thin areas in the coating system, pockets where the coatings do not wet out the surface properly, or holidays. These weaknesses in the coating system combine to allow moisture to get to the substrate quicker and start the corrosion cycle over again.

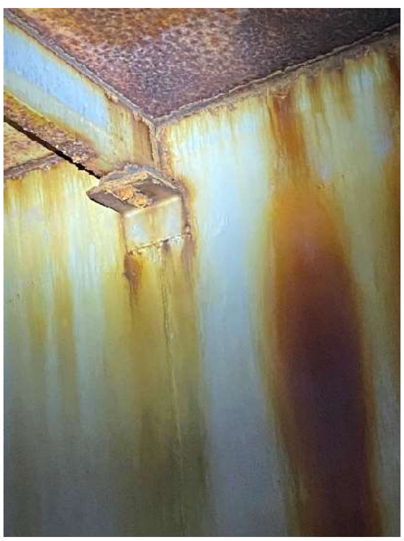
Seal welding is discussed relative to each reservoir and is highly recommended. Seal welding results in a tighter interior reservoir roof and eliminates locations that cannot be blast cleaned and coated. These areas include underneath the roof lap joints and between the rafters and roof plates. The coating system on a seal welded roof will last longer than one on a non-seal welded roof given an equally applied coating system. Examples of the damage to the roof plates from the inaccessible area between rafter and roof plate are included below where excess portions of the rafters were removed during the seal welding process. The steel loss in the deeper pits is more than half of the plate thickness of 1/4-inch. Additionally, there was steel loss on the rafter flange.

Two examples of the corrosion damage to the roof plates above the rafters of a 38-year-old reservoir.

Division 22-1 Reservoir

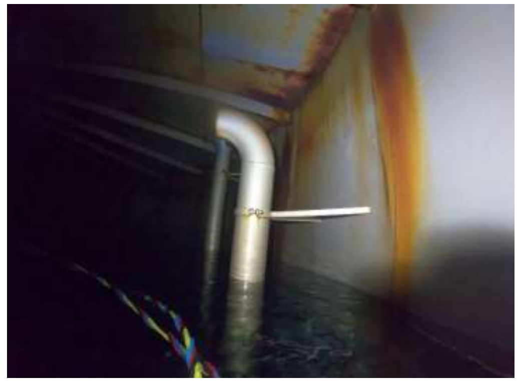
The exterior coating system has numerous repair patches on it and areas of delamination exist around the shell wall and roof. Some of the repair patches are likely due to rock chips but others are likely due to failures of the top coat that was applied over the original coating. A few areas of corrosion exist although most of these areas appears mild in nature. The organic growth on the roof is likely due to a very difficult environment to keep a reservoir clean. The area receives a lot of rainfall and has nearby trees that likely keep the roof covered in wet needles and debris. The adhesion test results were generally positive but two of the six dollies pulled well below the recommended minimum. Comparison to the H2O Solutions report show that on the backside of the reservoir a significant number of repair patches have been made since 2018.

The interior coating system can be broken down into two components above the waterline: The roof structure and the shell wall. The coatings on the roof and rafters have completely failed and aggressive corrosion is occurring. The flanges on the C Channel rafters are severely corroded in places and it can be assumed that the roof plates above the flanges are similarly corroded. The rafters are connected to the shell wall by angle brackets. Most of these brackets and the bolts that connect them to the rafters are moderately to severely corroded. The roof plates have light to moderate corrosion over most of the surface area. The interface between the roof plates and the shell wall at the rim angle also shows significant corrosion.



The cathodic protection system and coatings on the shell wall, while heavily stained, appear to be protecting the substrate. The H2O Solutions report showed minimal corrosion below the waterline even though the coating system was blistered throughout. Staining and corrosion above the waterline appears to have increased significantly since 2018.

Staining on the wall in 2022.



Staining on the wall in 2018.

The BHC report describes the seismic deficiencies of the Division 22-1 Reservoir and recommended that the reservoir be retrofitted with Option A, an external gravity ringwall collar with an estimated project cost of $367,000 in 2016. Using ENR Construction Cost Index (ENR CCI), the cost today is approximately $515,000 based upon the 2016 CCI of 10338, a Nov 2022 CCI of 13175, and an estimated 10% increase from 2022 to 2023 for a CCI of 14493 or an ENR CCI multiplier of 1.402. That option also included the following additional improvements: New 24- and 30-inch manways, level gauge, ladder, and flexible couplings.

As noted in the site investigation, the ladder system is not compliant with WAC 296-876-600 and should be removed and replaced. When replaced, the cage should be extended above the height of the reservoir roof and guardrails constructed out from either side of the cage to facilitate a safe area for crew to work around the access hatch and facilitate the transition from the ladder to the roof and back. Additionally, the site should be graded so that the ring wall sits 6-inches above the surrounding ground and the sill grout needs to be repaired. The District should also consider adding intrusion switches on the ladder guard and access hatch.

It is my opinion that the exterior coating system is not a good candidate to be cleaned and top coated. The coating system should be removed and replaced based upon several factors. First, the reservoir has already been top coated once and the risks of failure generally increase with the more coats of paint that are applied. Second, the two low adhesion test results along with general observations of random delaminations, show that weak areas in the coating exist. Finally, the organic growth on the roof has likely grown roots into the existing coating system and may have damaged it.

The interior of the reservoir has undergone significant corrosion. Abrasive blasting the interior will likely reveal many areas where repair to the structural steel will be required and will expose significant steel loss. Additionally, the remaining surface will be rough and pitted creating a short lifecycle for the coating system. The upper flanges on the rafters and the roof plates above them have likely degraded enough that without significant amounts of flat bar bridging, seal welding is not an option. Some of the lower flanges may also require repair. While the side shell appears to be in good condition, the roof and roof structure should be removed and replaced rather than rehabilitated. Replacing the roof could also provide the District with the opportunity to raise the height of the shell wall for improvement against seismic sloshing wave. The ability to add to the height of the shell wall is dependent upon the thicknesses of the existing shell wall. We recommend having this option evaluated by a structural engineer if desired by the District.

The District has three alternatives for this reservoir with costs provided in Table 7: Division 22-1 Reservoir Alternative Opinion of Probable Construction Costs.

1. Recoat the reservoir without seal welding and do not seismically upgrade it. This alternative is a stopgap measure meant to keep the reservoir in service until such time as the reservoir can be either demolished and rebuilt or fully rehabilitated. No appurtenance improvements are included in this alternative but the rafter angle brackets and structural deficiencies discovered during abrasive blasting would be repaired. I recommend an AWWA D102 ICS 5 coating system (zinc primer/epoxy/epoxy) for this alternative with an anticipated coating life of 8 to 12 years. For this alternative, I recommend spot repairing and managing the exterior coating system until the useful life of the new interior coating system is expended. The reservoir would remain seismically deficient.
2. Replace the roof, seismically upgrade, and recoat the reservoir. This alternative would include appurtenance upgrades and include seismic upgrades recommended in the BHC Report. Using an ICS 3 interior coating system and an OCS 4 exterior coating system would provide a coating life of approximately 25 to 30 years for each.
3. Demolish existing and construct new reservoir. This alternative would result in a brand new reservoir with anticipated coating lives of 25 to 30 years each with ICS 3 and OCS 4 systems.

Table 7: Division 22-1 Reservoir Alternative Opinion of Probable Construction Costs

Alternative	Total Project Cost
Alternative 1 – Recoat w/o upgrading the reservoir.	$ 640,000
Alternative 2 – Replace roof, seismically upgrade, and recoat reservoir.	$2,120,000
Alternative 3 – Demolish existing and construct new reservoir.	$2,100,000

Division 30 Reservoir

The exterior coating system has numerous repair patches on it and areas of delamination exist around the shell wall and roof. Some of the repair patches at ground level are likely due to rock chips and others are likely due to impacts. A few areas of corrosion exist on the top ring on the left side of the ladder. The organic growth on the roof is likely due to a very difficult environment to keep a reservoir clean. The area receives a lot of rainfall and has nearby trees that keep the roof covered in wet needles and debris. The adhesion test results were positive. Two of the six dollies pulled below the recommended minimum but barely so. Comparison to the H2O Solutions report shows that the top coat and tie coat have delaminated from the original finish coat in a significant number of areas since 2018.

The interior coating system can be broken down into two components above the waterline: The roof and the shell wall. The coatings on the roof plates have light corrosion over approximately 20 percent of the surface area which is approximately double the area in photos from the H2O report in 2018. The shell wall appears to have a little more corrosion. The H2O report indicated that blistering of the coatings below the water line was widespread. The cathodic protection system should be protecting the steel substrate below the water line.

The BHC report describes the seismic deficiencies of the Division 30 Reservoir and recommended that the reservoir be retrofitted with Option C, an anchored supplemental ringwall with an estimated project cost of $541,000 in 2016 or $758,000 in 2023 using the 1.402 ENR CCI multiplier calculated earlier. That option also included the following additional improvements: 8- and 10-inch flexible couplings.

The ground adjacent to the ringwall should be graded out and lowered so that the ringwall sits 6-inches above the ground. A small rock wall may need to be constructed around the back of the reservoir in order to lower the grade in that area. The sill grout needs to be cleaned and repaired. The District should also consider adding intrusion switches on the ladder guard and access hatch.

It is my opinion that the exterior coating system is currently a good candidate to be cleaned and top coated at this time. If the reservoir is not topcoated within the next 2-3 years, the reservoir should be adhesion tested again during design and reevaluated.

It is my opinion that the interior coatings of the reservoir have 3 to 5 years of life left at this time before steel loss starts to become more of a concern. Abrasive blasting the interior within the next 3 to 5 years will not likely reveal any significant issues or pitting.

The District has three alternatives for this reservoir:

1. Build a new reservoir. To be feasible, the reservoir would need to either be built on land adjacent to the existing reservoir or the existing reservoir would need to be demolished so that this reservoir can be constructed. A 26-foot diameter by 40-foot tall reservoir would provide sufficient storage and hydraulic pressure. It may be possible to clear a large enough area on the existing site to construct a reservoir of that size and then demolish the existing in order to provide working space around the structure. Alternatively, it may be possible to modify the pump station that supplies the Division 30 reservoir to work as a closed zone during construction.

   Constructing a concrete, Baker Silo-style reservoir is significantly cheaper than constructing a welded steel reservoir of the same volume or even seismically upgrading and recoating the existing reservoir. The concrete reservoir will also have a lower lifecycle cost than either the new or rehabilitated welded steel reservoir due to the cost to recoat the steel reservoir over time.
2. Recoat the reservoir and not seismically upgrade it. I would recommend recoating the interior with an AWWA D102 ICS 5 system and topcoating the exterior with an epoxy tie-coat and polyurethane finish coat that would result in a coating life of approximately 15 to 20 years. The reservoir would remain seismically deficient; however, it would preserve the steel of the reservoir. This option would require alternative storage while out of service for approximately two months.
3. Seismically upgrade and recoat the reservoir. This alternative would cause significant damage to the existing exterior coating system and thus require its full removal and replacement. I would recommend replacing the interior coatings with an AWWA D102 ICS 3 system and the exterior with an AWWA D102 OCS 4 system providing a coating life of approximately 25 to 30 years. The reservoir would be seismically stable. This option would require alternative storage while out of service for approximately four months.

Table 8: Division 30 Reservoir Alternative Opinion of Probable Construction Costs

Alternative	Total Project Cost
Alternative 1 – Construct new concrete reservoir.	$1,020,000
Alternative 2 – Recoat the reservoir without seismic upgrades.	$ 630,000
Alternative 3 – Seismically upgrade and recoat reservoir	$1,490,000

Geneva Reservoir

The exterior coating system is in poor condition at this time. The coatings on the roof are largely gone and no longer protecting the steel substrate. The coatings on the shell wall are still intact and protecting the substrate. The openness of the site is helping to keep the reservoir in better condition but corrosion on the roof will continue unabated during the rainy months. Comparison to the H2O Solutions report show that corrosion on the roof has progressed significantly since 2018 with the area actively rusting increasing perhaps 300- to 400-percent.

Roof condition in 2022.

Roof condition in 2018.

The interior coating system can be broken down into two components above the waterline: The roof structure and the shell wall. The coatings on the roof plates have completely failed and corrosion is occurring unabated. At this time, the corrosion largely appears to be mild to moderate surface corrosion. The coatings on the rafters are largely intact the rafters appear to be in fair condition with mostly light surface corrosion. The interface between the roof plates and the shell wall at the rim angle appears to be in good condition.

The impressed current cathodic protection system and coatings on the shell wall, while heavily stained, appear to be protecting the substrate. The H2O Solutions report showed minimal corrosion below the waterline even though the coating system was blistered throughout. Staining and corrosion above the waterline has increased significantly since 2018.

The BHC report describes the seismic deficiencies of the Geneva Reservoir and recommended that the reservoir be retrofitted with Option C, an anchored external ringwall with an estimated project cost of $505,000 in 2016 or $708,000 in 2023 using the 1.402 ENR CCI multiplier calculated earlier. That option also included the following additional improvements: 10- and 12-inch flexible couplings.

The site should be graded so that the ring wall sits 6-inches above the surrounding ground and the sill grout needs to be repaired. The District should also consider adding intrusion switches on the ladder guard and access hatch.

It is my opinion that the exterior coating system is not a good candidate to be cleaned and top coated. The coating system is non-existent on the roof and because of that, the entire exterior should be abrasive blast cleaned and coated with a new coating system. A new coating system with a fluoropolymer finish coat complying with AWWA D102 OCS 4 would likely provide an exterior coating system that would last 25-30 years.

It is my opinion that the interior coating system has completely failed and is in need of replacement as soon as possible to prevent steel loss from becoming problematic. The steel loss will not likely cause structural deficiencies for five or more years; however, the corrosion will continually roughen the surface and cause future coating systems to have a shorter lifespan. As of now, the corrosion appears to be surficial in nature but given the rate of change in the amount of corrosion since 2018, the degree of corrosion will likely accelerate.

The District has three alternatives for this reservoir with costs provided in Table 9: Geneva Reservoir Alternative Opinion of Probable Project Costs.

1. Recoat the reservoir and do not seismically upgrade or seal weld it. This alternative, if conducted in the next 3 to 4 years, should prevent the reservoir from deteriorating to the point of increasing lifecycle costs. I recommend an AWWA D102 ICS 3 system for the interior and an OCS 4 system for the exterior. These coatings should provide a coating life of approximately 25 to 30 years. Because the roof is not seal welded, corrosion between the rafters and roof plates and within the roof plate lap joints will continue unabated causing rust staining of the interior and replacement or repair of the roof when steel loss becomes too great in those areas. The reservoir would remain seismically deficient.

   The remaining life of the roof of the reservoir if it is not seal welded is unknown and can vary significantly depending upon a number of factors. The way to approximate the remaining life is to measure the steel thickness of the roof plates above the rafters and just inside of the roof lap joints from on top of the roof utilizing a steel thickness gage. A rate of corrosion can be estimated based upon the recorded steel loss and age of the structure. If the corrosion is found to be significant, areas can be permanently marked on the roof so that the rate of steel loss can be monitored utilizing repeatable measurements over time.
2. Seismically upgrade and recoat the reservoir. This alternative would include the seismic upgrades recommended in the BHC Report. I recommend AWWA D102 ICS 3 and OCS 4 system for the interior and exterior to provide a coating life of approximately 25 to 30 years. Seal welding the roof would stop the continuation of steel loss in inaccessible areas; however, the cost to seal weld would increase the project cost to approximately $2,000,000. Given that the cost of a new steel reservoir is approximately $2,100,000, if the District desires a seal welded reservoir, a new reservoir should be constructed.
3. Recoat the reservoir and lower the water level to reduce seismic upgrade requirements. This alternative was not explored thoroughly but based upon the information in the BHC Report and provided by the District in the “Meeting Minutes – Option C Summary”. This alternative would use surplus storage in the Division 22-1 Reservoirs to count against the required storage in the

Geneva Reservoir and allow the water level in Geneva to be lowered to 14-feet. Per the BHC Report, other than the addition of flexible couplings, this would make the seismic upgrades unnecessary and save significant costs. Costs for this alternative were not developed due to the uncertainty of piping and system upgrades that may be required in order to facilitate this alternative.

Table 9: Geneva Reservoir Alternative Opinion of Probable Project Costs

Alternative	Total Project Cost
Alternative 1 – Recoat without seismic upgrades	$ 920,000
Alternative 2 – Seismically upgrade and recoat reservoir	$1,780,000
Alternative 3 – Recoat the reservoir and lower the water level	N/A

Conclusions and Recommendations

Based upon the results of the condition assessment and a review of supporting documentation provided by the District, Evergreen Coating Engineers is presenting the following recommendations:

1. The Geneva Reservoir should be the District's first priority to recoat. Surprisingly, even with the degree of corrosion inside and out on the reservoir, the corrosion appears to have remained largely surficial and reservoir is still in good condition. This window of opportunity will not likely last long before the corrosion progresses and becomes more moderate and severe and thus increases the overall lifecycle costs of the reservoir by shortening the coating life of both coating systems.

2. The Division 22-1 Reservoir is likely beyond the point of economical repair. The cost to replace the roof, raise the shell wall, and seismically upgrade is approximately the same cost as to demolish and rebuild the reservoir. The condition of the angle brackets connecting the rafters to the shell wall are of concern and should be evaluated as soon as possible.

   At a minimum, the District should consider abrasive blast cleaning the interior roof plates, rafters, angle brackets, and the shell wall to a point below the high waterline to determine the extent of required repairs and apply a new coating system. The cathodic protection system would protect the steel below the waterline. While this option would only be slightly less expensive than the cost provided in Alternative 1 in Table 7, it would extend the life of the reservoir and provide the District with time to plan for its replacement.

3. If land is available or can be obtained to construct a new Baker Silo-style reservoir, the Division 30 Reservoir should be planned to be replaced rather than seismically upgraded. The lifecycle costs to upgrade and/or recoat the existing reservoir are too significant compared to constructing a new reservoir and the reservoir is already half way through its design life. Additionally, storage would have to be provided, or the zone would need to be operated as a closed zone, for the duration of the project which may prove difficult.

A minor project should be immediately undertaken address the corrosion on the interior of the roof access hatch riser and exterior shell wall of the Division 30 Reservoir. Repair of these areas will extend the life of the existing coating systems and prevent further steel loss.

APPENDIX A

METALS TESTING LABORATORY RESULTS

eurofins

Environment Testing America

ANALYTICAL REPORT

Eurofins Seattle
5755 8th Street East
Tacoma, WA 98424
Tel: (253)922-2310

Laboratory Job ID: 580-118122-1
Client Project/Site: Lake Whatcom Condition Assessment

For:
Evergreen Coating Engineers
6925 37th Ave SW
Seattle, Washington 98126

Attn: Lance Stevens

Authorized for release by:
10/9/2022 9:42:03 PM

Pauline Matlock, Project Manager
(253)922-2310
Pauline.Matlock@et.eurofinsus.com

This report has been electronically signed and authorized by the signatory. Electronic signature is intended to be the legally binding equivalent of a traditionally handwritten signature.

Results relate only to the items tested and the sample(s) as received by the laboratory.

Table of Contents

Job ID: 580-118122-1

Laboratory: Eurofins Seattle

Narrative

Job Narrative
580-118122-1

Comments
No additional comments.

Receipt
The samples were received on 9/21/2022 10:40 AM. Unless otherwise noted below, the samples arrived in good condition. The temperature of the cooler at receipt was 19.7º C.

Receipt Exceptions
The Chain-of-Custody (COC) was incomplete as received and/or improperly completed: There are no sample times on the COC. The default time of 00:01 has been used for these samples.

Insufficient sample volume was provided for these samples for all analyses requested. The Lead testing was prioritized per client comment on the COC, and there was not enough sample remaining for the Mercury testing, so that has been cancelled.

Metals
No analytical or quality issues were noted, other than those described in the Definitions/Glossary page.

Client: Evergreen Coating Engineers | Job ID: 580-118122-1
Project/Site: Lake Whatcom Condition Assessment

Qualifiers

Metals

Qualifier	Qualifier Description
J	Result is less than the RL but greater than or equal to the MDL and the concentration is an approximate value.

Glossary

Abbreviation	These commonly used abbreviations may or may not be present in this report.
¤	Listed under the "D" column to designate that the result is reported on a dry weight basis
%R	Percent Recovery
CFL	Contains Free Liquid
CFU	Colony Forming Unit
CNF	Contains No Free Liquid
DER	Duplicate Error Ratio (normalized absolute difference)
Dil Fac	Dilution Factor
DL	Detection Limit (DoD/DOE)
DL, RA, RE, IN	Indicates a Dilution, Re-analysis, Re-extraction, or additional Initial metals/anion analysis of the sample
DLC	Decision Level Concentration (Radiochemistry)
EDL	Estimated Detection Limit (Dioxin)
LOD	Limit of Detection (DoD/DOE)
LOQ	Limit of Quantitation (DoD/DOE)
MCL	EPA recommended "Maximum Contaminant Level"
MDA	Minimum Detectable Activity (Radiochemistry)
MDC	Minimum Detectable Concentration (Radiochemistry)
MDL	Method Detection Limit
ML	Minimum Level (Dioxin)
MPN	Most Probable Number
MQL	Method Quantitation Limit
NC	Not Calculated
ND	Not Detected at the reporting limit (or MDL or EDL if shown)
NEG	Negative / Absent
POS	Positive / Present
PQL	Practical Quantitation Limit
PRES	Presumptive
QC	Quality Control
RER	Relative Error Ratio (Radiochemistry)
RL	Reporting Limit or Requested Limit (Radiochemistry)
RPD	Relative Percent Difference, a measure of the relative difference between two points
TEF	Toxicity Equivalent Factor (Dioxin)
TEQ	Toxicity Equivalent Quotient (Dioxin)
TNTC	Too Numerous To Count

Client Sample ID: DIVISION 22 INT

Lab Sample ID: 580-118122-1

Date Collected: 09/14/22 00:01

Matrix: Solid

Date Received: 09/21/22 10:40

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	ND		14	1.2	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Barium	**79**		2.3	0.37	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Cadmium	**0.30**	**J**	4.7	0.23	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Chromium	**3000**		6.1	1.0	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Lead	**4500**		7.0	1.0	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Selenium	ND		23	1.9	mg/Kg		09/26/22 12:23	09/26/22 21:32	1
Silver	ND		12	2.6	mg/Kg		09/26/22 12:23	09/26/22 21:32	1

Client Sample Results

Client: Evergreen Coating Engineers
Job ID: 580-118122-1
Project/Site: Lake Whatcom Condition Assessment

Client Sample ID: DIVISION 22 EXT
Lab Sample ID: 580-118122-2

Date Collected: 09/14/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	ND		24	2.0	mg/Kg		09/26/22 12:23	09/26/22 21:36	1
Barium	**4700**		3.9	0.62	mg/Kg		09/26/22 12:23	09/26/22 21:36	1
Cadmium	**0.83**	**J**	7.9	0.39	mg/Kg		09/26/22 12:23	09/26/22 21:36	1
Chromium	**6900**		10	1.7	mg/Kg		09/26/22 12:23	09/26/22 21:36	1
Lead	**16000**		120	17	mg/Kg		09/26/22 12:23	09/27/22 17:02	10
Selenium	ND		39	3.1	mg/Kg		09/26/22 12:23	09/26/22 21:36	1
Silver	ND		20	4.4	mg/Kg		09/26/22 12:23	09/26/22 21:36	1

Client Sample ID: DIVISION 30 INT
Lab Sample ID: 580-118122-3
Date Collected: 09/14/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	2.7	J	30	2.5	mg/Kg		09/26/22 12:23	09/26/22 21:39	1
Barium	1300		5.0	0.78	mg/Kg		09/26/22 12:23	09/26/22 21:39	1
Cadmium	ND		9.9	0.49	mg/Kg		09/26/22 12:23	09/26/22 21:39	1
Chromium	36		13	2.1	mg/Kg		09/26/22 12:23	09/26/22 21:39	1
Lead	18000		150	22	mg/Kg		09/26/22 12:23	09/27/22 17:05	10
Selenium	ND		50	3.9	mg/Kg		09/26/22 12:23	09/26/22 21:39	1
Silver	ND		25	5.6	mg/Kg		09/26/22 12:23	09/26/22 21:39	1

Client Sample Results

Client: Evergreen Coating Engineers
Project/Site: Lake Whatcom Condition Assessment

Job ID: 580-118122-1

Client Sample ID: DIVISION 30 EXT
Date Collected: 09/15/22 00:01
Date Received: 09/21/22 10:40

Lab Sample ID: 580-118122-4
Matrix: Solid

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	**11**	**J**	49	4.0	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Barium	**770**		8.1	1.3	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Cadmium	ND		16	0.80	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Chromium	**2400**		21	3.5	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Lead	**11000**		24	3.6	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Selenium	ND		81	6.4	mg/Kg		09/26/22 12:23	09/26/22 21:43	1
Silver	ND		41	9.1	mg/Kg		09/26/22 12:23	09/26/22 21:43	1

Client Sample Results

Client: Evergreen Coating Engineers
Job ID: 580-118122-1
Project/Site: Lake Whatcom Condition Assessment

Client Sample ID: GENEVA INT
Lab Sample ID: 580-118122-5

Date Collected: 09/15/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	ND		13	1.0	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Barium	**450**		2.1	0.33	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Cadmium	**0.25**	**J**	4.2	0.21	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Chromium	**45**		5.4	0.91	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Lead	**26**		6.3	0.93	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Selenium	ND		21	1.7	mg/Kg		09/26/22 12:23	09/26/22 21:47	1
Silver	ND		10	2.3	mg/Kg		09/26/22 12:23	09/26/22 21:47	1

Client Sample Results

Client: Evergreen Coating Engineers
Project/Site: Lake Whatcom Condition Assessment

Job ID: 580-118122-1

Client Sample ID: GENEVA EXT
Date Collected: 09/15/22 00:01
Date Received: 09/21/22 10:40

Lab Sample ID: 580-118122-6
Matrix: Solid

Method: SW846 6010D - Metals (ICP)

Analyte	Result	Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	**5.4**	**J**	25	2.0	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Barium	**3200**		4.1	0.65	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Cadmium	**0.70**	**J**	8.2	0.40	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Chromium	**20**		11	1.8	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Lead	**200**		12	1.8	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Selenium	ND		41	3.2	mg/Kg		09/26/22 12:23	09/26/22 21:50	1
Silver	ND		20	4.6	mg/Kg		09/26/22 12:23	09/26/22 21:50	1

Client: Evergreen Coating Engineers
Project/Site: Lake Whatcom Condition Assessment

Job ID: 580-118122-1

Method: 6010D - Metals (ICP)

Lab Sample ID: MB 580-405003/20-A
Matrix: Solid
Analysis Batch: 405108

Client Sample ID: Method Blank
Prep Type: Total/NA
Prep Batch: 405003

Analyte	MB Result	MB Qualifier	RL	MDL	Unit	D	Prepared	Analyzed	Dil Fac
Arsenic	ND		3.0	0.25	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Barium	ND		0.50	0.079	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Cadmium	ND		1.0	0.049	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Chromium	ND		1.3	0.22	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Lead	ND		1.5	0.22	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Selenium	ND		5.0	0.40	mg/Kg		09/26/22 12:23	09/26/22 20:24	1
Silver	ND		2.5	0.56	mg/Kg		09/26/22 12:23	09/26/22 20:24	1

Lab Sample ID: LCS 580-405003/21-A
Matrix: Solid
Analysis Batch: 405108

Client Sample ID: Lab Control Sample
Prep Type: Total/NA
Prep Batch: 405003

Analyte	Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec Limits
Arsenic	50.0	48.0		mg/Kg		96	80 - 120
Barium	50.0	47.3		mg/Kg		95	80 - 120
Cadmium	50.0	46.6		mg/Kg		93	80 - 120
Chromium	50.0	46.8		mg/Kg		94	80 - 120
Lead	50.0	49.9		mg/Kg		100	80 - 120
Selenium	50.0	49.3		mg/Kg		99	80 - 120
Silver	50.0	48.8		mg/Kg		98	80 - 120

Lab Sample ID: LCSD 580-405003/22-A
Matrix: Solid
Analysis Batch: 405108

Client Sample ID: Lab Control Sample Dup
Prep Type: Total/NA
Prep Batch: 405003

Analyte	Spike Added	LCSD Result	LCSD Qualifier	Unit	D	%Rec	%Rec Limits	RPD	RPD Limit
Arsenic	50.0	48.1		mg/Kg		96	80 - 120	0	20
Barium	50.0	47.4		mg/Kg		95	80 - 120	0	20
Cadmium	50.0	46.6		mg/Kg		93	80 - 120	0	20
Chromium	50.0	46.8		mg/Kg		94	80 - 120	0	20
Lead	50.0	49.7		mg/Kg		99	80 - 120	0	20
Selenium	50.0	49.4		mg/Kg		99	80 - 120	0	20
Silver	50.0	48.4		mg/Kg		97	80 - 120	1	20

Lab Chronicle

Client: Evergreen Coating Engineers
Project/Site: Lake Whatcom Condition Assessment

Job ID: 580-118122-1

Client Sample ID: DIVISION 22 INT

Lab Sample ID: 580-118122-1
Date Collected: 09/14/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:32

Client Sample ID: DIVISION 22 EXT

Lab Sample ID: 580-118122-2
Date Collected: 09/14/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:36
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		10	405288	JLS	EET SEA	09/27/22 17:02

Client Sample ID: DIVISION 30 INT

Lab Sample ID: 580-118122-3
Date Collected: 09/14/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:39
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		10	405288	JLS	EET SEA	09/27/22 17:05

Client Sample ID: DIVISION 30 EXT

Lab Sample ID: 580-118122-4
Date Collected: 09/15/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:43

Client Sample ID: GENEVA INT

Lab Sample ID: 580-118122-5
Date Collected: 09/15/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:47

Client Sample ID: GENEVA EXT

Lab Sample ID: 580-118122-6
Date Collected: 09/15/22 00:01
Matrix: Solid
Date Received: 09/21/22 10:40

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Analyst	Lab	Prepared or Analyzed
Total/NA	Prep	3050B			405003	ABP	EET SEA	09/26/22 12:23
Total/NA	Analysis	6010D		1	405108	JLS	EET SEA	09/26/22 21:50

Laboratory References:

EET SEA = Eurofins Seattle, 5755 8th Street East, Tacoma, WA 98424, TEL (253)922-2310

Accreditation/Certification Summary

Client: Evergreen Coating Engineers Job ID: 580-118122-1
Project/Site: Lake Whatcom Condition Assessment

Laboratory: Eurofins Seattle

The accreditations/certifications listed below are applicable to this report.

Authority	Program	Identification Number	Expiration Date
Washington	State	C788	07-13-23

Sample Summary

Client: Evergreen Coating Engineers | Job ID: 580-118122-1
Project/Site: Lake Whatcom Condition Assessment

Lab Sample ID	Client Sample ID	Matrix	Collected	Received
580-118122-1	DIVISION 22 INT	Solid	09/14/22 00:01	09/21/22 10:40
580-118122-2	DIVISION 22 EXT	Solid	09/14/22 00:01	09/21/22 10:40
580-118122-3	DIVISION 30 INT	Solid	09/14/22 00:01	09/21/22 10:40
580-118122-4	DIVISION 30 EXT	Solid	09/15/22 00:01	09/21/22 10:40
580-118122-5	GENEVA INT	Solid	09/15/22 00:01	09/21/22 10:40
580-118122-6	GENEVA EXT	Solid	09/15/22 00:01	09/21/22 10:40

Eurofins TestAmerica, Seattle
5755 8th Street East
Tacoma, WA 98424

Chain of Custody Record

eurofins | Environment Testing TestAmerica

Regulatory Program: ☐ DW ☐ NPDES ☑ RCRA ☐ Other:

TestAmerica Laboratories, Inc. d/b/a Eurofins TestAmerica

Client Contact	**Project Manager: Lance Stevens**
Evergreen Coating Engineers	Email: lance@coating-engineers.com
6925 37th Ave SW	**Tel/Fax: 206.450.6243**
Seattle, WA 98126	**Analysis Turnaround Time**
206.450.6243	☐ CALENDAR DAYS ☐ WORKING DAYS
(xxx) xxx-xxxx FAX	TAT if different from Below ______
Project Name: Lake Whatcom Condition Assessment	☐ 2 weeks
Site: Varies	☐ 1 week
P O # N/A	☐ 2 days
	☐ 1 day

Site Contact: | **Date:**
Lab Contact: | **Carrier:**

COC No: ______
______ of ______ COCs
TALS Project #:
Sampler:
For Lab Use Only:
Walk-in Client:
Lab Sampling:
Job / SDG No.:

Sample Identification	Sample Date	Sample Time	Sample Type (C=Comp, G=Grab)	Matrix	# of Cont.	Filtered Sample (Y / N)	Perform MS / MSD (Y / N)	Sample Specific Notes:
Division 22 Int	9/14/22	NA				n	n	Test for RCRA 8 Total Metals Lead is the contaminant of highest concern
Division 22 Ext	9/14/22	NA						
Division 30 Int	9/14/22	NA						
Division 30 Ext	9/15/22	NA						
Geneva Int	9/15/22	NA						
Geneva Ext	9/15/22	NA						

580-118122 Chain of Custody

Preservation Used: 1= Ice, 2= HCl; 3= H2SO4; 4=HNO3; 5=NaOH; 6= Other ______

Possible Hazard Identification:
Are any samples from a listed EPA Hazardous Waste? Please List any EPA Waste Codes for the sample in the Comments Section if the lab is to dispose of the sample.
☐ Non-Hazard ☐ Flammable ☐ Skin Irritant ☐ Poison B ☐ Unknown

Sample Disposal (A fee may be assessed if samples are retained longer than 1 month)
☐ Return to Client ☐ Disposal by Lab ☐ Archive for ______ Months

Special Instructions/QC Requirements & Comments: RCRA 8 Total metals test. Lead is the contaminant of highest concern. Eliminate mercury testing first if sample size is too small.

19.7 /19.1 IR8/EnJ/None/None/ MAI

Custody Seals Intact: ☐ Yes ☐ No	Custody Seal No.:		Cooler Temp. (°C): Obs'd: ______ Corr'd: ______ Therm ID No.: ______		
Relinquished by:	Company:	Date/Time:	Received by: [signature]	Company: EETN	Date/Time: 9/21/22 10:40
Relinquished by:	Company:	Date/Time:	Received by:	Company:	Date/Time:
Relinquished by:	Company:	Date/Time:	Received in Laboratory by:	Company:	Date/Time:

Form No. CA-C-WI-002, Rev. 4.33, dated 5/4/2020

Login Sample Receipt Checklist

Client: Evergreen Coating Engineers   Job Number: 580-118122-1

Login Number: 118122   **List Source: Eurofins Seattle**
List Number: 1
Creator: Vallelunga, Diana L

Question	Answer	Comment
Radioactivity wasn't checked or is </= background as measured by a survey meter.	N/A	
The cooler's custody seal, if present, is intact.	True	
Sample custody seals, if present, are intact.	N/A	
The cooler or samples do not appear to have been compromised or tampered with.	True	
Samples were received on ice.	True	
Cooler Temperature is acceptable.	True	
Cooler Temperature is recorded.	True	
COC is present.	True	
COC is filled out in ink and legible.	True	
COC is filled out with all pertinent information.	True	
Is the Field Sampler's name present on COC?	True	
There are no discrepancies between the containers received and the COC.	True	
Samples are received within Holding Time (excluding tests with immediate HTs)	True	
Sample containers have legible labels.	True	
Containers are not broken or leaking.	True	
Sample collection date/times are provided.	True	
Appropriate sample containers are used.	True	
Sample bottles are completely filled.	True	
Sample Preservation Verified.	True	
There is sufficient vol. for all requested analyses, incl. any requested MS/MSDs	True	
Containers requiring zero headspace have no headspace or bubble is <6mm (1/4").	N/A	
Multiphasic samples are not present.	True	
Samples do not require splitting or compositing.	True	
Residual Chlorine Checked.	N/A	

APPENDIX B

DIVISION 22 RESERVOIR PHOTOS

General condition.

General condition.

Inlet and Overflow

General condition.

Severe corrosion of the upper flange.

Severe corrosion of the upper and lower flanges.


Overflow.


Inlet pipe.


Corrosion on upper and lower flanges.


Corroded angle bracket, rim angle, and rafter.

Severe corrosion on upper and lower flanges.

Bolts to the angle bracket have corroded away.

Corroded angle bracket, rim angle, and rafter.

Severe corrosion of the roof plates and rafters.

Rafter corrosion.

Rafter and roof plate corrosion.

Rafter and roof plate corrosion.

Rafter and roof plate corrosion.

Rafter corrosion at the dollar plate.

Rafter and roof plate corrosion.

General exterior.

General exterior.

Delamination and repair areas.

Delamination and repair areas.

Delamination and repair areas.

Delamination and repair areas.

General exterior.

Ladder, cage, and level gauge.


Access hatch.


General roof condition.


General roof condition.


General roof condition.


General roof condition.


Ladder transition area.

Roof vent and anchor.

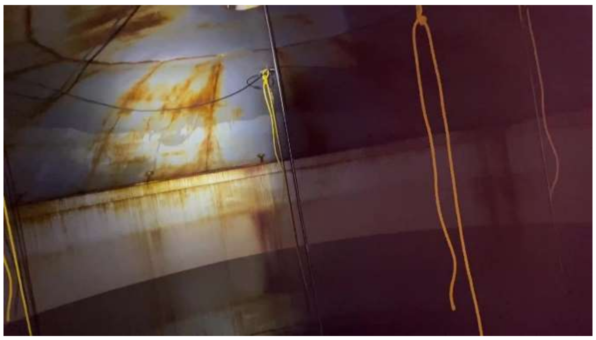
Nameplate.

APPENDIX C

DIVISION 30 RESERVOIR PHOTOS

Mild corrosion and rust staining on the shell wall.

General rusting of the roof plates.

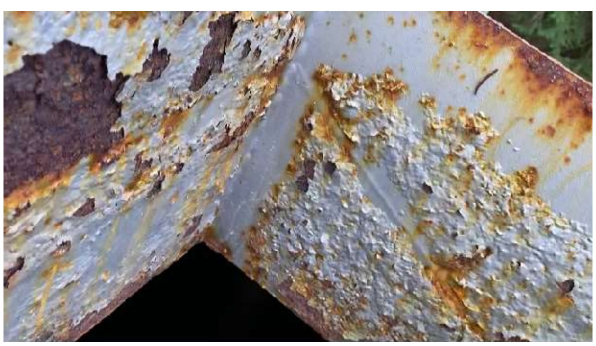


General rusting of the roof plates.

General rusting of the roof plates.

General rusting of the roof plates.

Overflow.



Moderate corrosion on the access hatch riser.

Moderate corrosion on the access hatch riser.

Moderate corrosion under the hatch riser. **Moderate corrosion under the hatch riser.**

General exterior. **General exterior.**

General exterior.

Ringwall below grade.

Ringwall below grade.

Moss growth on the shell wall.

General exterior.

Driveway.

Access hatch and delamination areas.

Shell wall delamination.

Mildew growth and ringwall below grade.

Grade behind reservoir.

General exterior.

Ladder transition area.

Access hatch.

Roof vent and anchor.

General roof condition.

General roof condition.

General roof condition.

Nameplate.

APPENDIX D

GENEVA RESERVOIR PHOTOS


General condition.

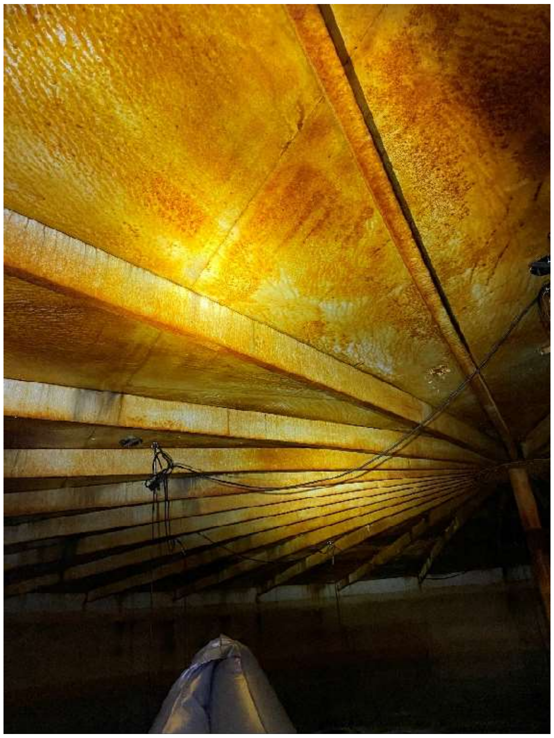

General condition.

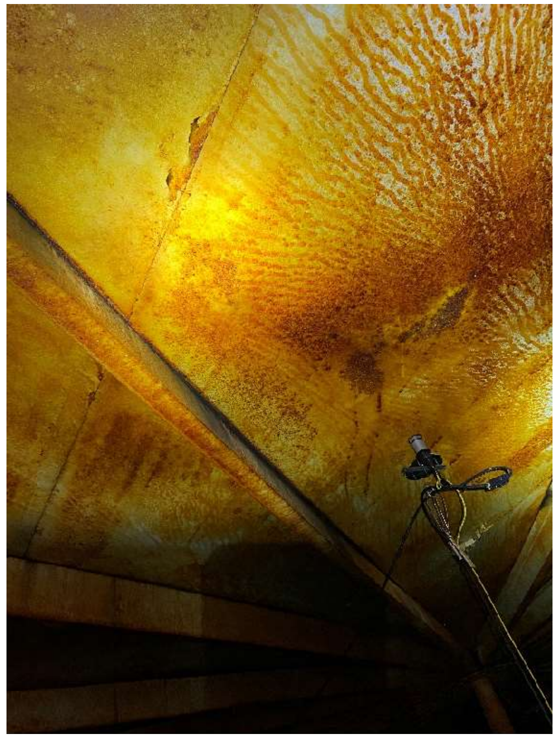

Inlet and Overflow


General condition.


Mild to moderate general corrosion of the roof plates.


Overflow.



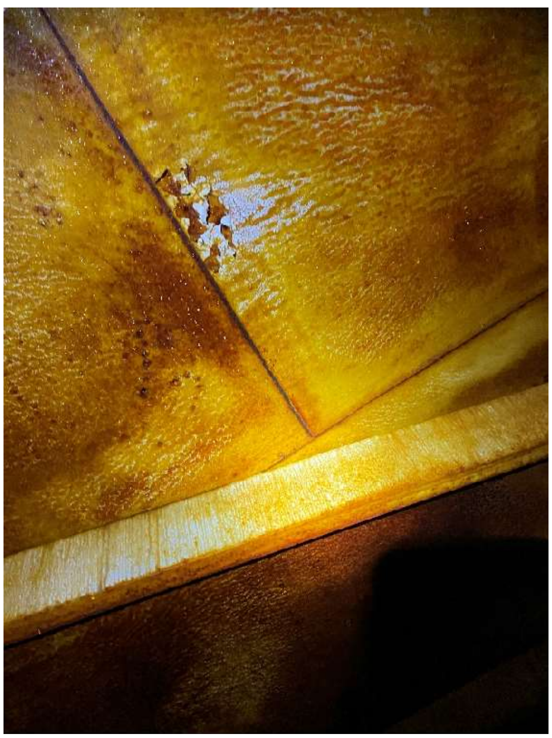


Mild corrosion of the roof plates and rafters. **Mild corrosion of the roof plates and rafters.**

Coatings on the shell wall heavily blistered. **Moderate corrosion on the rafter and roof plate.**

Dollar plate.

Dollar plate.



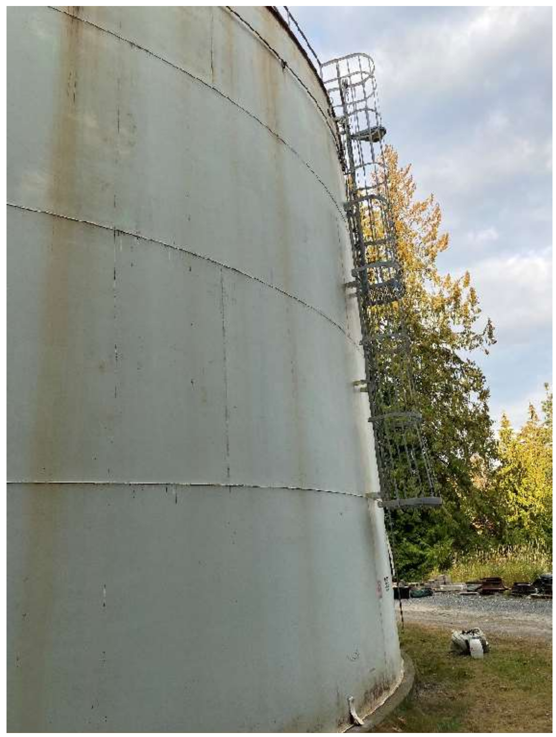
Roofplate and rafters.

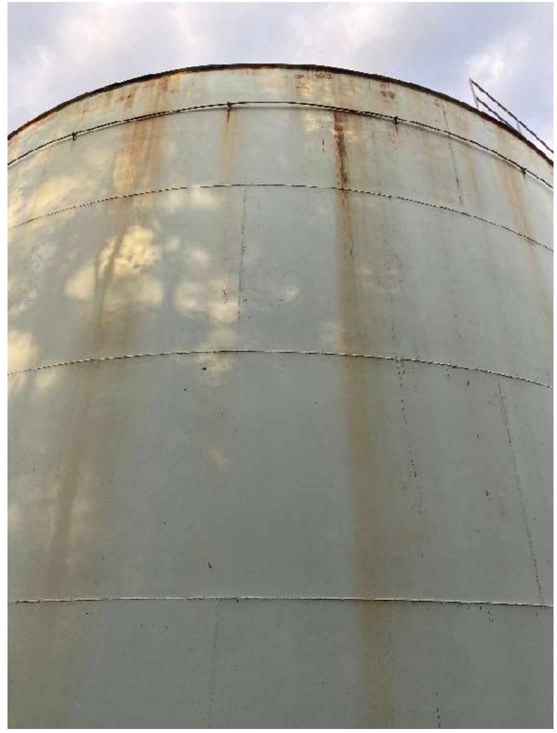
Rafter and rafter tab.


Inlet.


Dollar plate.


General exterior.


General exterior.

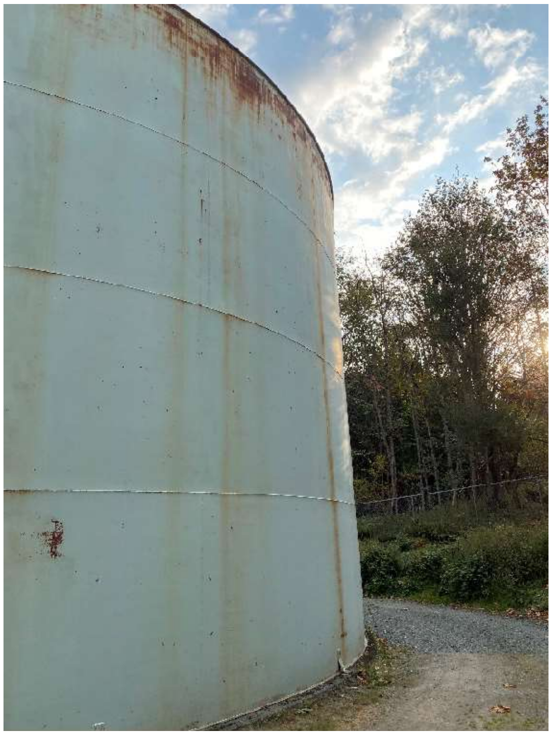


General exterior.

Corrosion beginning at the top of the shell wall.

General exterior.

Corrosion beginning at the top of the shell wall.



Corrosion beginning at the top of the shell wall.

General exterior.

Manway.

Nameplate.

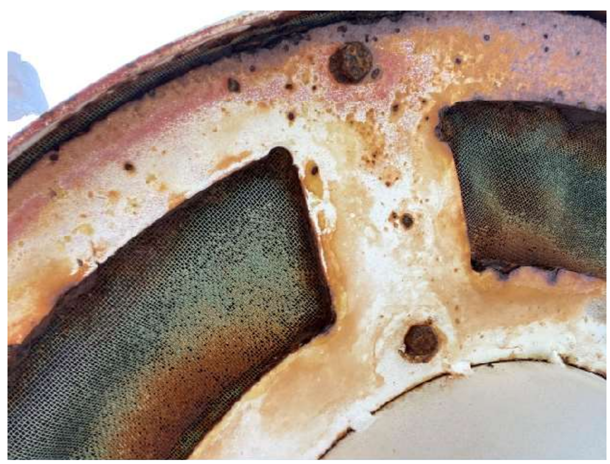

General roof condition.


General roof condition.


General roof condition.


Ladder transition area.


Roof vent and anchor.


Underside of roof vent.

APPENDIX E

DRY FILM THICKNESS TEST RESULTS

Division 22 Exterior

Created:	2022-09-14 11:52:12
PosiTector Body S/N:	853527
Probe Type:	PosiTector 6000 FNDS
Probe S/N:	390538

Calibration

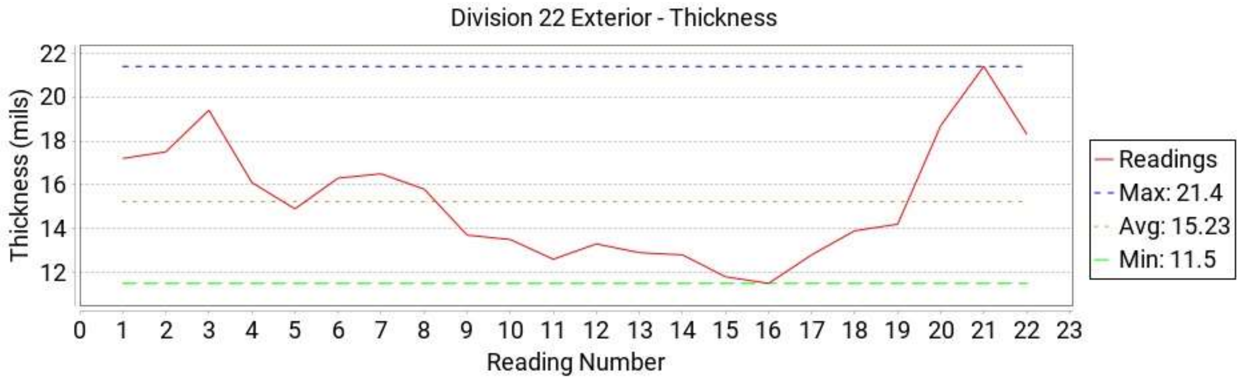
Cal Name:	Cal 1

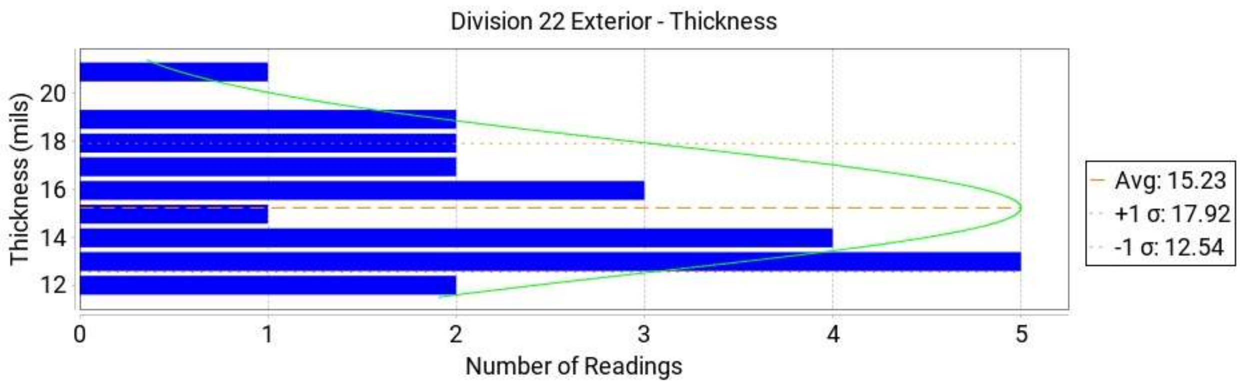
Summary

	#	$\bar{x}$	σ	⤓	⤒
Coating Thickness (mils)	22	15.23	2.69	11.5	21.4

Readings

#	Thickness (mils)	Time
		2022-09-14
1	17.2	11:52:15
2	17.5	11:52:17
3	19.4	11:52:19
4	16.1	11:52:26
5	14.9	11:52:27
6	16.3	11:52:29
7	16.5	11:52:37
8	15.8	11:52:38
9	13.7	11:52:45
10	13.5	11:52:47
11	12.6	11:52:48
12	13.3	11:52:54
13	12.9	11:52:56
14	12.8	11:52:57
15	11.8	11:53:05
16	11.5	11:53:07
17	12.8	11:53:08
18	13.9	11:53:15
19	14.2	11:53:17
20	18.7	11:53:23
21	21.4	11:53:24
22	18.3	11:53:26


Division 22 Exterior - Thickness
Thickness (mils)
22
20
18
16
14
12
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23
Reading Number
Readings
Max: 21.4
Avg: 15.23
Min: 11.5


Division 22 Exterior - Thickness
Thickness (mils)
20
18
16
14
12
0 1 2 3 4 5
Number of Readings
Avg: 15.23
+1 σ: 17.92
-1 σ: 12.54

Division 30 Interior

Created:	2022-09-14 13:42:25
PosiTector Body S/N:	853527
Probe Type:	PosiTector 6000 FNDS
Probe S/N:	390538

Calibration

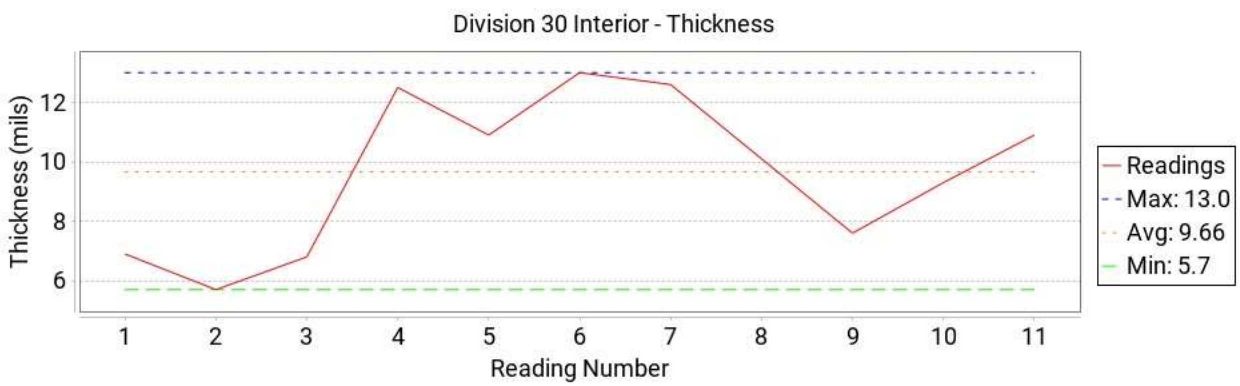
Cal Name:	Cal 1

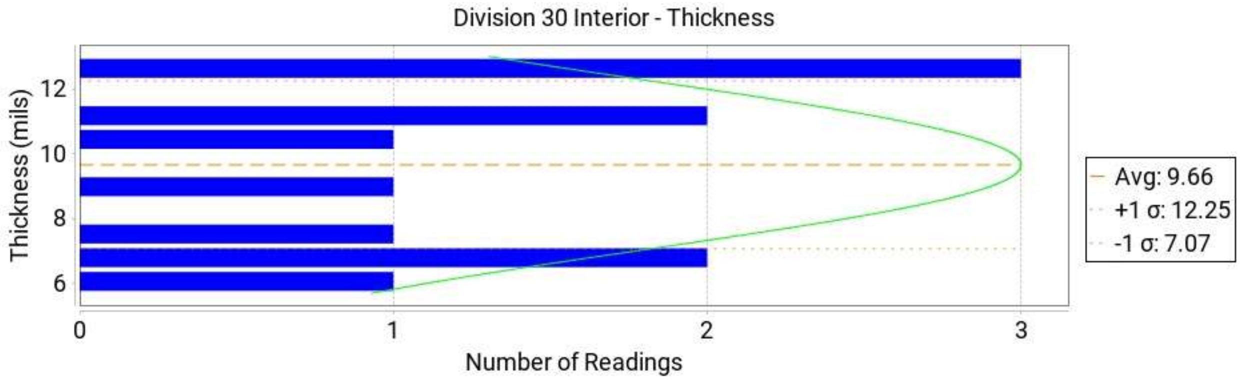
Summary

	#	x̄	σ	↓	↑
Coating Thickness (mils)	11	9.66	2.59	5.7	13.0

Readings

#	Thickness (mils)	Time
		2022-09-14
1	6.9	13:42:39
2	5.7	13:42:41
3	6.8	13:42:42
4	12.5	13:42:55
5	10.9	13:42:57
6	13.0	13:42:59
7	12.6	13:43:02
8	10.1	13:43:14
9	7.6	13:43:16
10	9.3	13:43:19
11	10.9	13:43:23





Division 30 Interior - Thickness
Thickness (mils)
12
10
8
6
0
1
2
3
Number of Readings
Avg: 9.66
+1 σ: 12.25
-1 σ: 7.07

Division 30 Exterior

Created:	2022-09-14 12:40:09
PosiTector Body S/N:	853527
Probe Type:	PosiTector 6000 FNDS
Probe S/N:	390538

Calibration

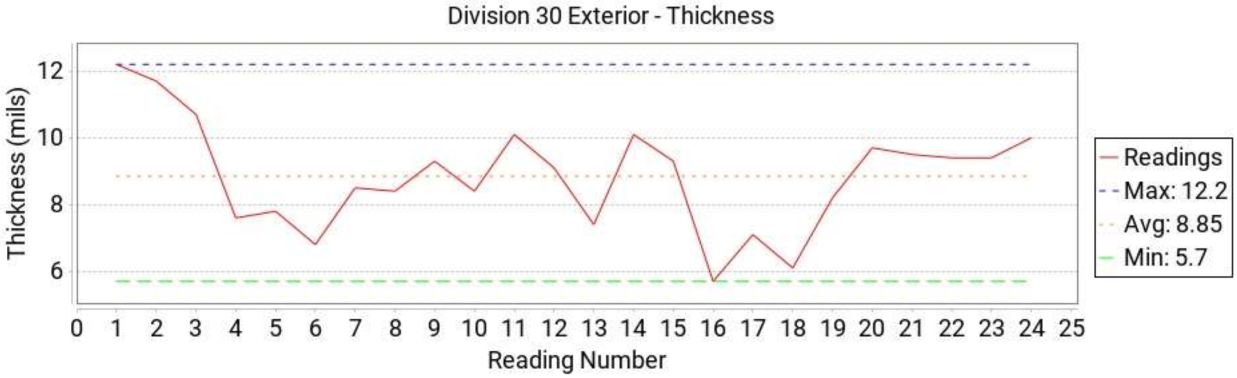
Cal Name:	Cal 1

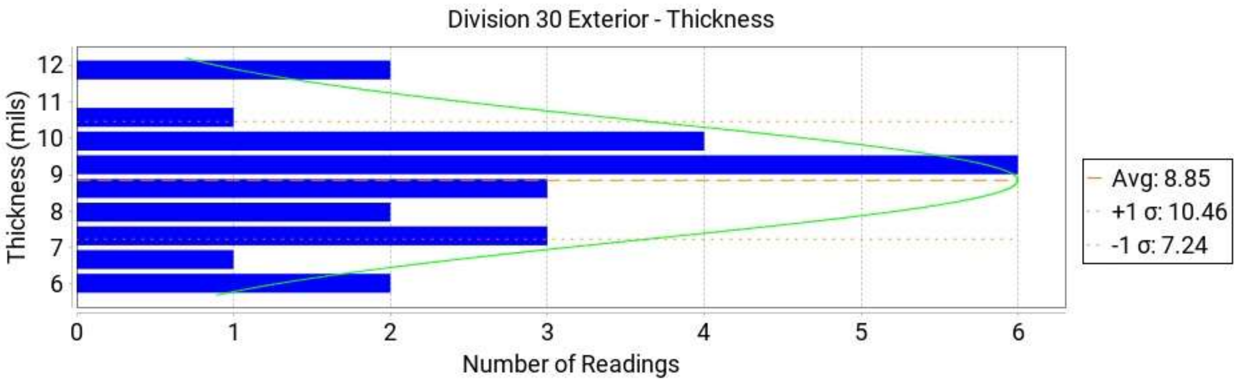
Summary

	#	$\bar{x}$	σ	⭳	⭱
Coating Thickness (mils)	24	8.85	1.61	5.7	12.2

Readings

#	Thickness (mils)	Time
		2022-09-14
1	12.2	12:40:13
2	11.7	12:40:14
3	10.7	12:40:16
4	7.6	12:40:20
5	7.8	12:40:21
6	6.8	12:40:23
7	8.5	12:40:26
8	8.4	12:40:27
9	9.3	12:40:29
10	8.4	12:40:33
11	10.1	12:40:34
12	9.1	12:40:36
13	7.4	12:40:41
14	10.1	12:40:43
15	9.3	12:40:44
16	5.7	12:40:51
17	7.1	12:40:53
18	6.1	12:40:54
19	8.2	12:41:00
20	9.7	12:41:02
21	9.5	12:41:03
22	9.4	12:41:08
23	9.4	12:41:10
24	10.0	12:41:11


Division 30 Exterior - Thickness
Thickness (mils)
Reading Number
Readings
Max: 12.2
Avg: 8.85
Min: 5.7


Division 30 Exterior - Thickness
Thickness (mils)
Number of Readings
Avg: 8.85
+1 σ: 10.46
-1 σ: 7.24

Geneva Exterior

Created:	2022-09-15 11:35:16
PosiTector Body S/N:	853527
Probe Type:	PosiTector 6000 FNDS
Probe S/N:	390538

Calibration

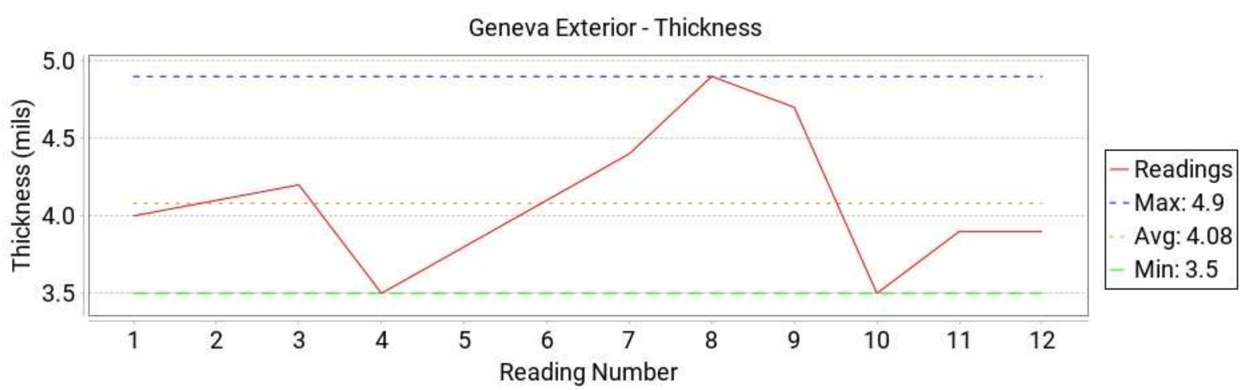
Cal Name:	Cal 1

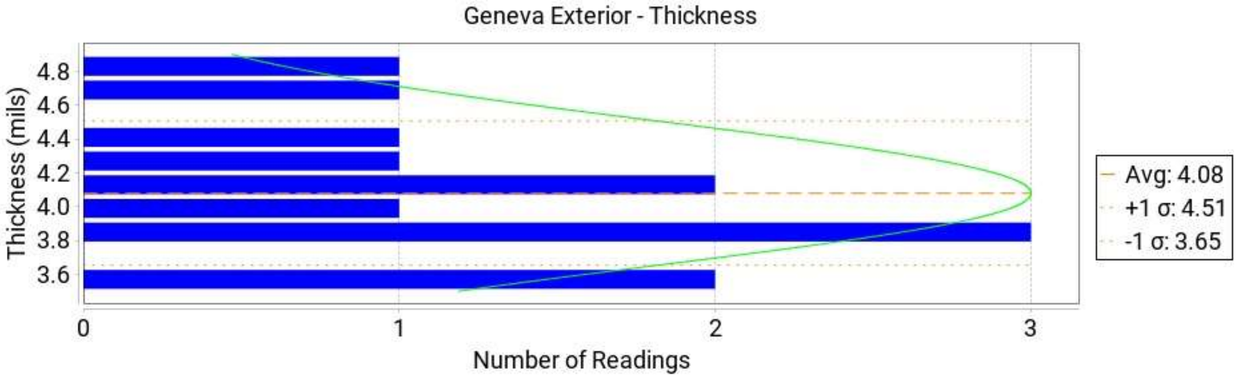
Summary

	#	$\bar{x}$	σ	⭳	⭱
Coating Thickness (mils)	12	4.08	0.43	3.5	4.9

Readings

#	Thickness (mils)	Time
		2022-09-15
1	4.0	11:35:30
2	4.1	11:35:32
3	4.2	11:35:34
4	3.5	11:35:38
5	3.8	11:35:40
6	4.1	11:35:42
7	4.4	11:35:58
8	4.9	11:36:00
9	4.7	11:36:01
10	3.5	11:36:13
11	3.9	11:36:15
12	3.9	11:36:17





Geneva Exterior - Thickness
Thickness (mils)
4.8
4.6
4.4
4.2
4.0
3.8
3.6
0
1
2
3
Number of Readings
Avg: 4.08
+1 σ: 4.51
-1 σ: 3.65

APPENDIX F

ADHESION TEST RESULTS

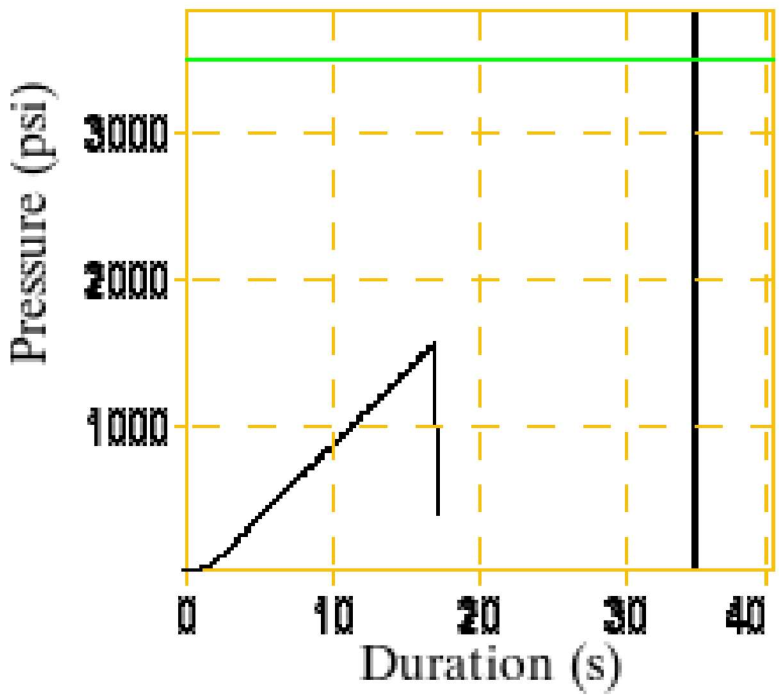
Division 22

Created:	2012-02-21 23:54:02
PosiTest AT-A S/N:	17275

Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
1	1591 3500	17.1 0.0/0.0	20	100	Pulled	X 23:58:34

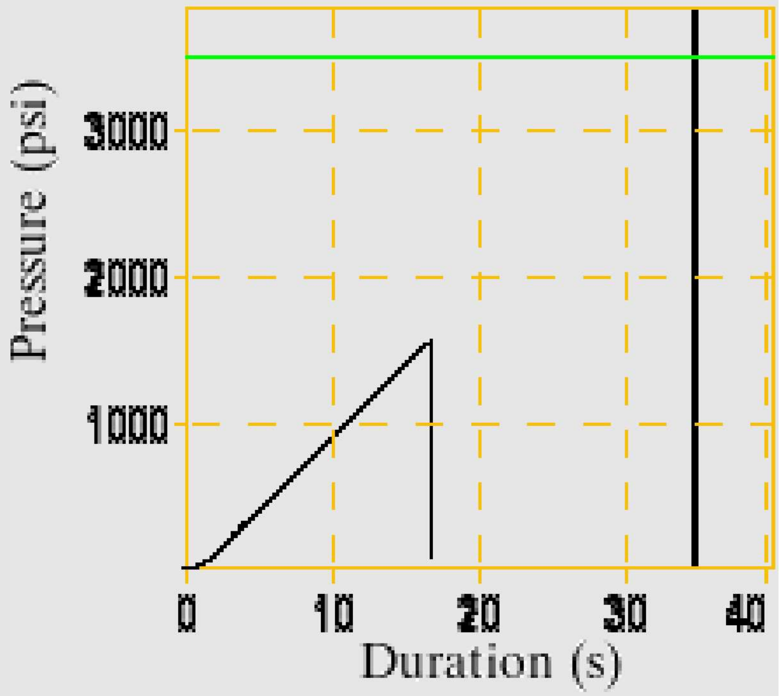
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 22 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
2	1592 3500	16.7 0.0/0.0	20	100	Pulled	X 00:03:23

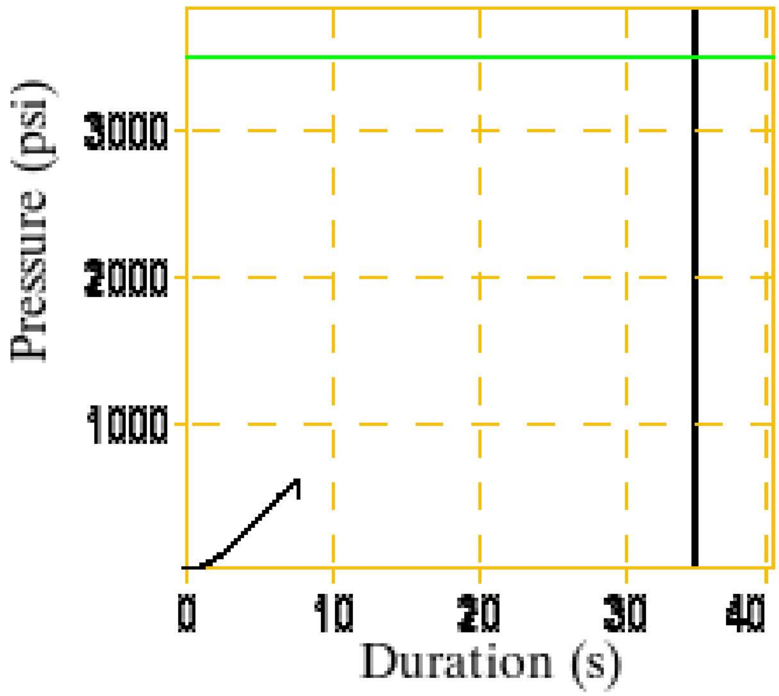
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 22 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
3	661 3500	7.8 0.0/0.0	20	100	Pulled	X 00:37:20

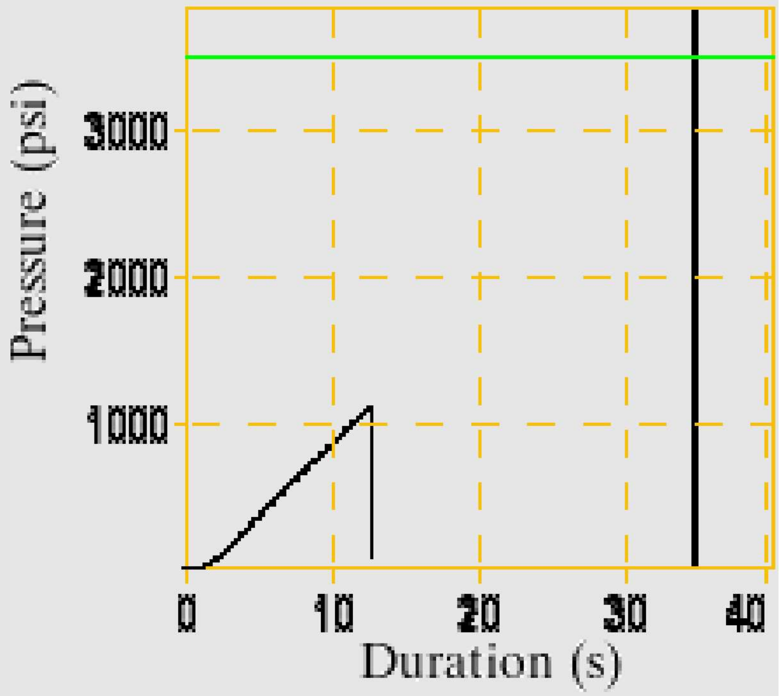
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 22 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
4	1152 3500	12.7 0.0/0.0	20	100	Pulled	X 00:39:26

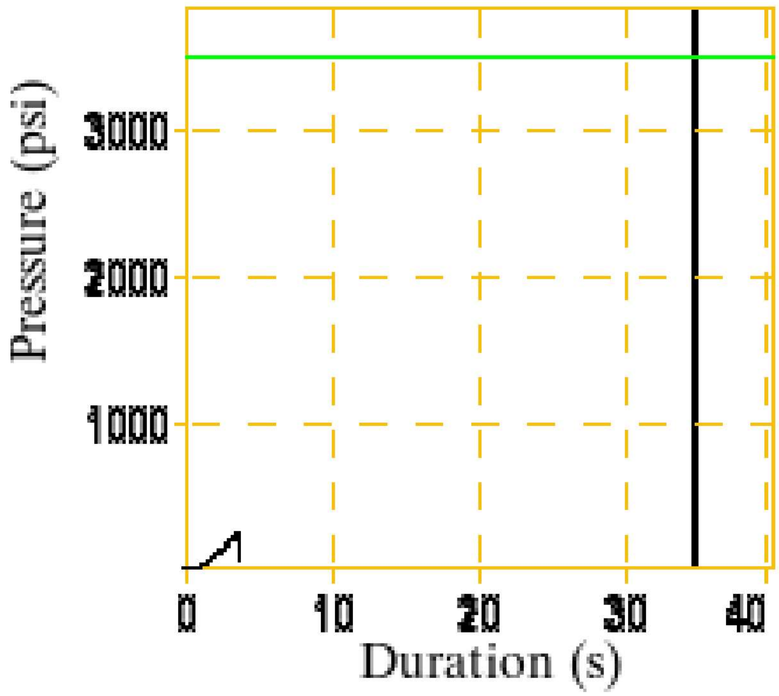
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0

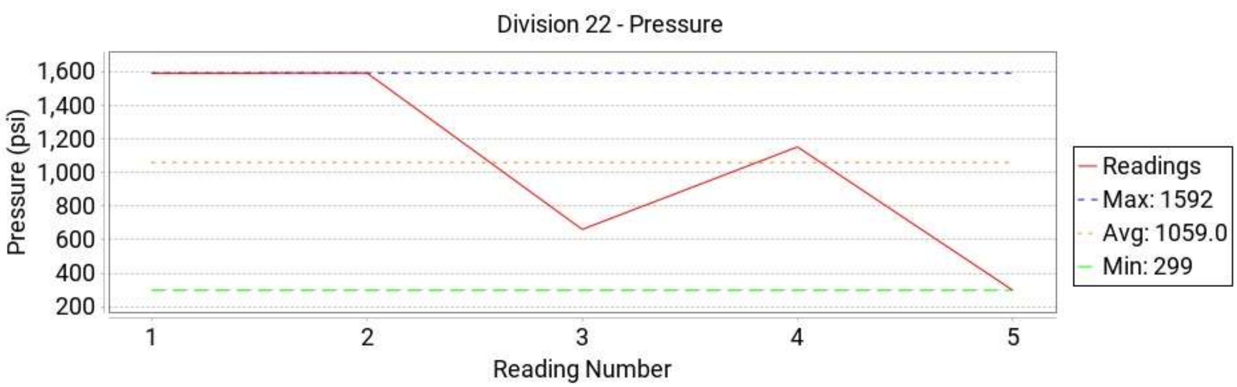


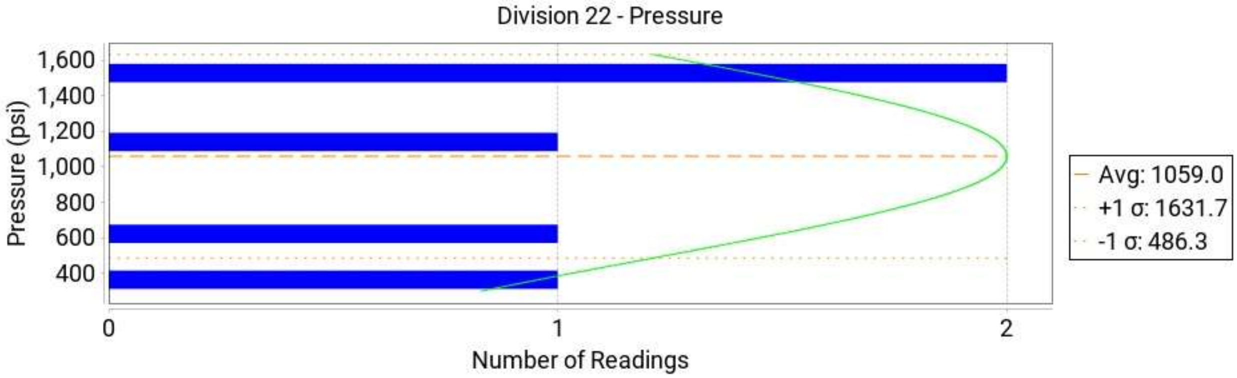
Division 22 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
5	299 3500	3.8 0.0/0.0	20	100	Pulled	X 00:40:44

Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0






Division 22 - Pressure
1,600
1,400
1,200
1,000
800
600
400
Pressure (psi)
0
1
2
Number of Readings
Avg: 1059.0
+1 σ: 1631.7
-1 σ: 486.3

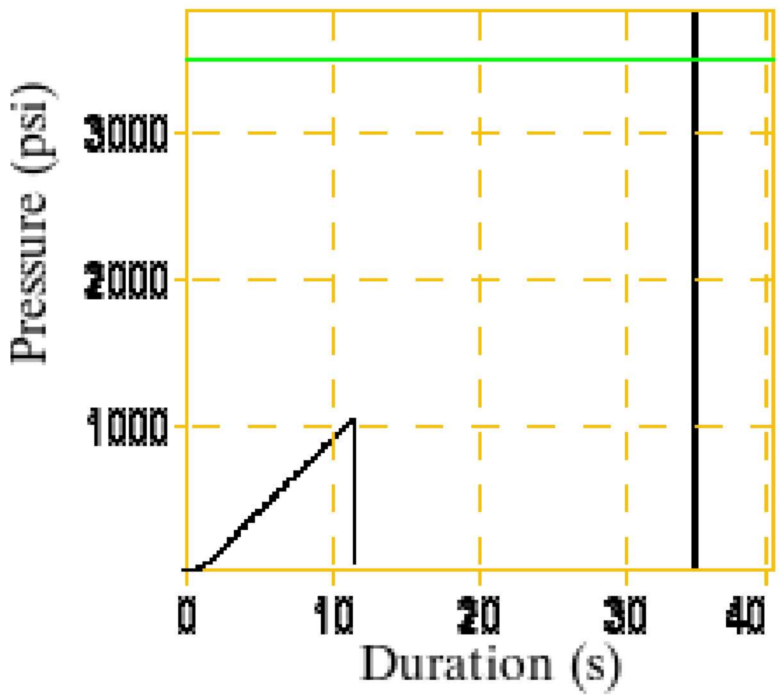
Division 30

Created: 2012-02-22 01:57:50
PosiTest AT-A S/N: 17275

Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
1	1082 3500	11.5 0.0/0.0	20	100	Pulled	X 01:58:30

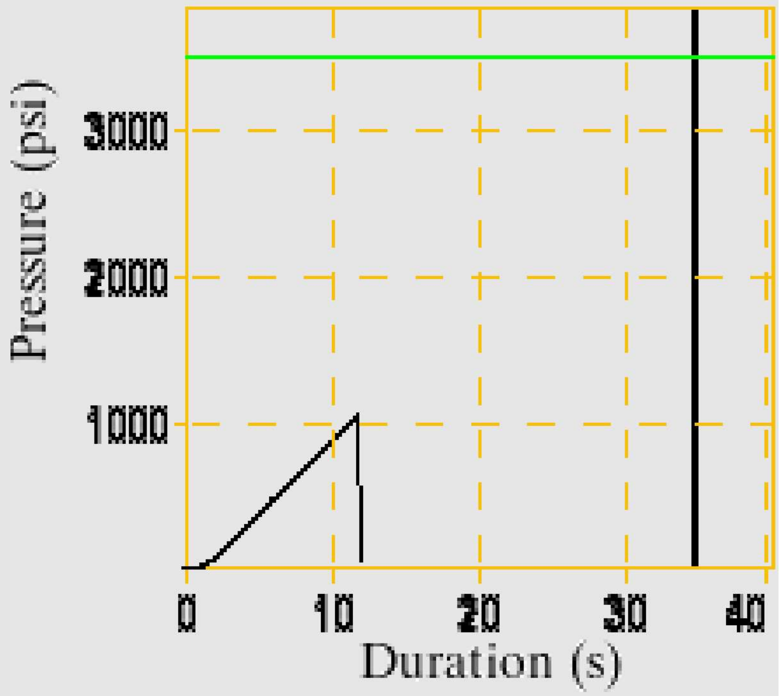
Glue Y: 0 Y/Z Interface: 0
Layer 1: B 0 B/Y Interface: 0
Substrate: A 0 A/B Interface: 0



Division 30 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
2	1089 3500	11.9 0.0/0.0	20	100	Pulled	X 01:59:54

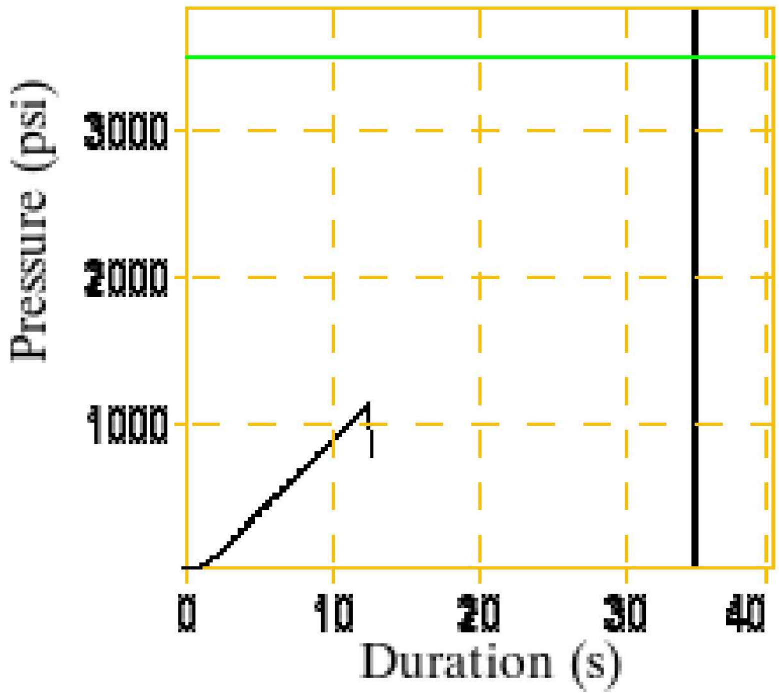
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 30 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
3	1161 3500	12.6 0.0/0.0	20	100	Pulled	X 02:01:13

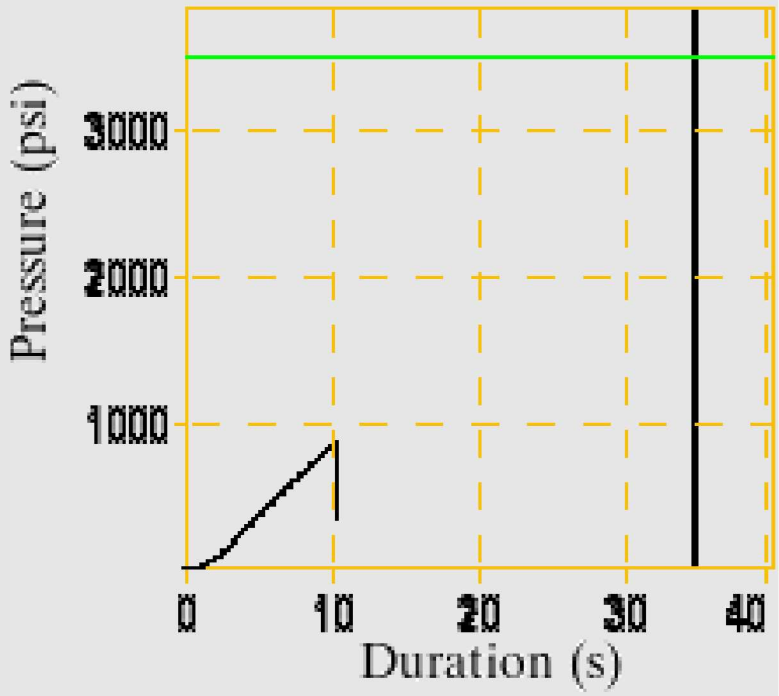
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 30 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
4	915 3500	10.3 0.0/0.0	20	100	Pulled	X 02:24:19

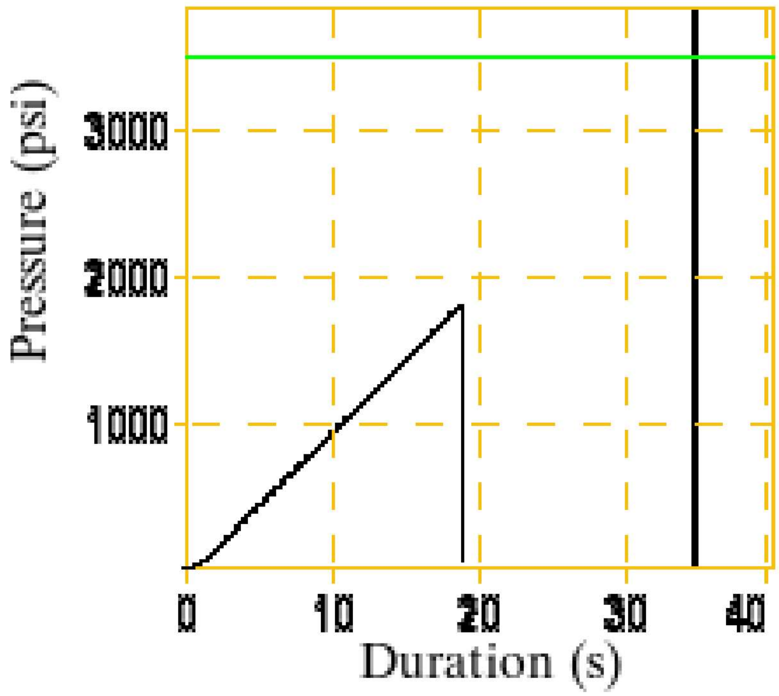
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0



Division 30 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
5	1837 3500	18.9 0.0/0.0	20	100	Pulled	X 02:26:20

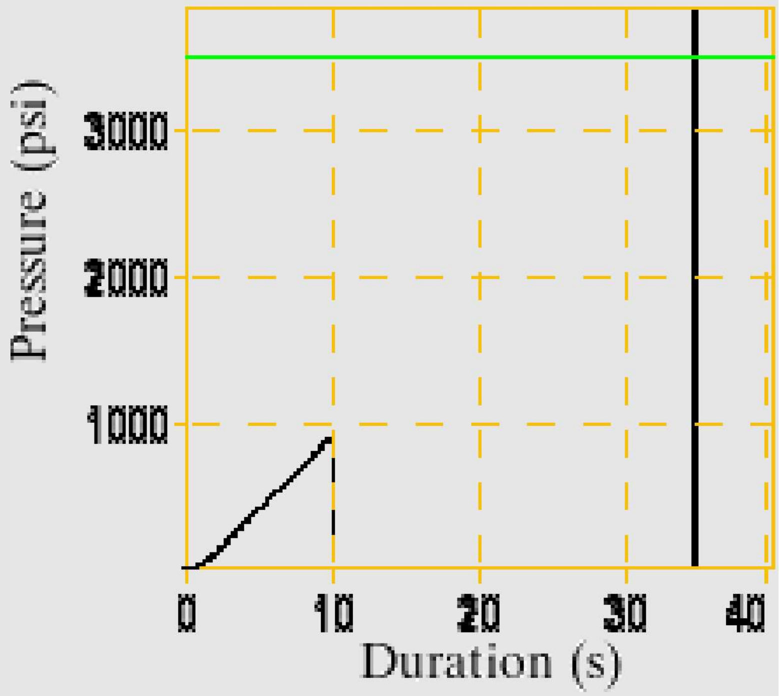
Glue Y:	0	Y/Z Interface:	0
Layer 1: B	0	B/Y Interface:	0
Substrate: A	0	A/B Interface:	0

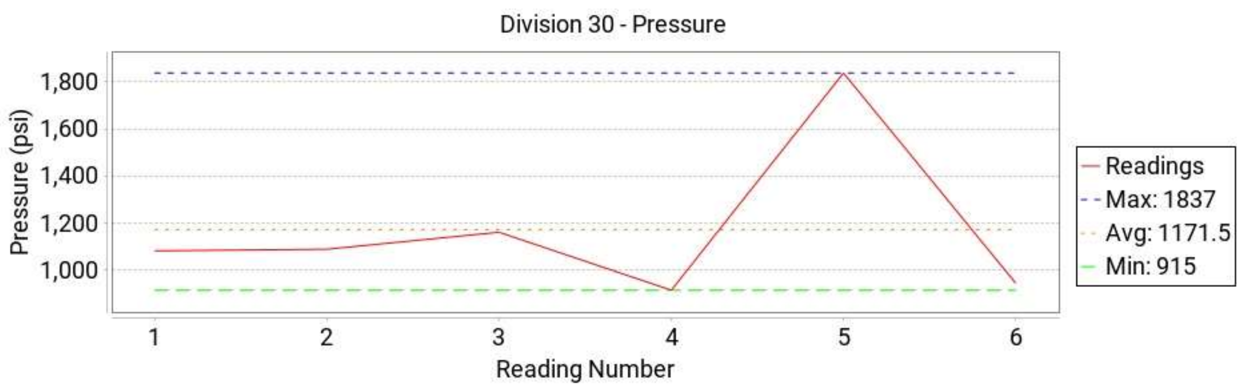


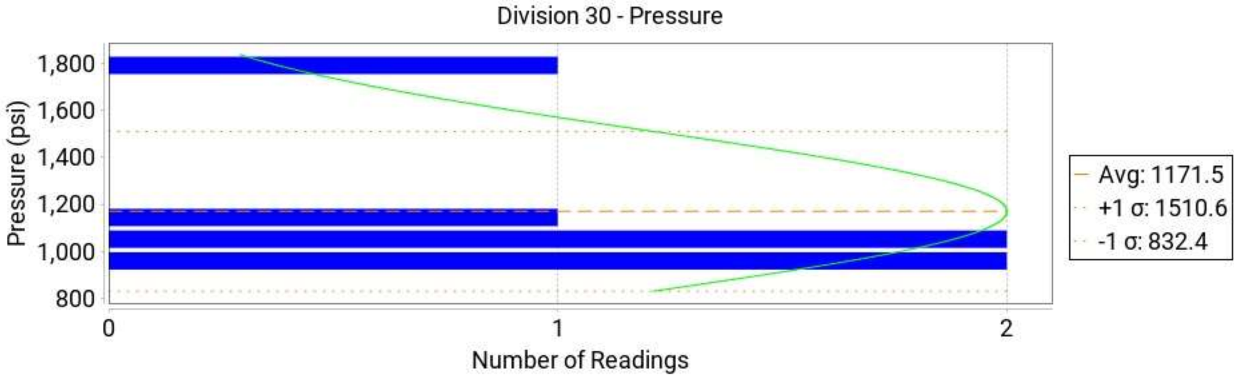
Division 30 Readings

#	Pressure Limit (psi)	Duration Hold Time (s)	Dolly (mm)	Rate (psi/s)	Result	Pass/Fail Time
6	945 3500	10.1 0.0/0.0	20	100	Pulled	X 02:28:51

Glue Y: 0 Y/Z Interface: 0
Layer 1: B 0 B/Y Interface: 0
Substrate: A 0 A/B Interface: 0






Division 30 - Pressure
1,800
1,600
1,400
1,200
1,000
800
Pressure (psi)
0
1
2
Number of Readings
Avg: 1171.5
+1 σ: 1510.6
-1 σ: 832.4

APPENDIX G

OPINIONS OF PROBABLE PROJECT COSTS

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 22-1
PRELIMINARY COST ESTIMATE
Alternative 1: Recoat without Upgrading the Reservoir
October 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$15,000	$15,000
2.	Mobilization and Demobilization	1	LS	$29,250	$29,250
3.	Angle Bracket Replacement	25	EA	$500	$12,500
4.	Miscellaneuos Metal Repair	25	LF	$500	$12,500
5.	Interior Recoating	1	LS	$250,000	$250,000
6.	Exterior Coating Spot Repairs	1	LS	$40,000	$40,000
7.	Removal of Mill Scale	2,000	SF	$4	$8,000
8.	Surface Restoration	1	LS	$2,000	$2,000
	Subtotal				$369,250
	Contingency @ 30%				$110,775
	Construction Subtotal				$480,025
	Sales Tax at 8.6%				$41,282
	Engineering Design, CM, and Inspection @ 25%				$120,006
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$640,000**

NOTES:

1.) No seismic or appurtenance upgrades included.

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 22-1
PRELIMINARY COST ESTIMATE
Alternative 2: Replace Roof, Seismically Upgrade, and Recoat Reservoir
December 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$74,070	$74,070
3.	Ladder, Landing, and Guardrail	1	LS	$45,000	$45,000
4.	Manway	1	EA	$20,000	$20,000
5.	Roof Vent	1	LS	$30,000	$30,000
6.	New Reservoir Roof and Side Shell Extension	1	LS	$400,000	$400,000
7.	Foundation Seal Grout Replacement	1	LS	$7,000	$7,000
8.	Interior Recoating	1	LS	$135,000	$135,000
9.	Exterior Recoating	1	LS	$98,000	$98,000
10.	Reservoir Containment	1	LS	$60,000	$60,000
11.	Removal of Mill Scale	2,000	SF	$4	$8,000
12.	Level Gauge Board	1	LS	$10,000	$10,000
13.	Surface Restoration	1	LS	$10,000	$10,000
	Subtotal				$922,070
	Contingency @ 30%				$276,621
	Construction Subtotal				$1,198,691
	Sales Tax at 8.6%				$103,087
	Engineering Design, CM, and Inspection @ 25%				$299,673
	Seismic Upgrade Total Project Costs				$515,000
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$2,120,000**

NOTES:

1.) Assumes built in Reservoir No. 1 location.

2.) Seismic upgrade costs in the BHC Report included engineering, contingency, and tax at unknown rates so the total provided in their report is included without additional markup.

3.) Interior and Exterior Recoating costs are only for side shell and bottom of reservoir. Coating costs are included in roof cost.

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 22-1
PRELIMINARY COST ESTIMATE
Alternative 3: Demolish and Construct New Reservoir
October 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$97,650	$97,650
3.	Temporary Erosion and Sediment Control	1	LS	$10,000	$10,000
4.	Site Earthwork	1	LS	$20,000	$20,000
5.	Demolition of Existing Reservoir and Foundation	1	LS	$80,000	$80,000
6.	500,000 Gallon Steel Reservoir and Foundation	1	LS	$825,000	$825,000
7.	Site Piping	1	LS	$45,000	$45,000
8.	Electrical, Telemetry, and Instrumentation	1	LS	$60,000	$60,000
9.	Cathodic Protection	1	LS	$35,000	$35,000
10.	Surface Restoration	1	LS	$10,000	$10,000
	Subtotal				$1,207,650
	Contingency @ 30%				$362,295
	Construction Subtotal				$1,569,945
	Sales Tax at 8.6%				$135,015
	Engineering Design, CM, and Inspection @ 25%				$392,486
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$2,100,000**

NOTES:

1.) Assumes built in Reservoir No. 1 location.

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 30
PRELIMINARY COST ESTIMATE
Alternative 1: Construct a New Concrete Reservoir
October 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$46,350	$46,350
3.	Temporary Erosion and Sediment Control	1	LS	$10,000	$10,000
4.	Site Earthwork	1	LS	$60,000	$60,000
5.	Demolition of Existing Reservoir and Foundation	1	LS	$80,000	$80,000
6.	158,000 Gal. Concrete Reservoir and Foundation	1	LS	$250,000	$250,000
7.	Site Piping	1	LS	$45,000	$45,000
8.	Electrical, Telemetry, and Instrumentation	1	LS	$60,000	$60,000
9.	Surface Restoration	1	LS	$10,000	$10,000
	Subtotal				$586,350
	Contingency @ 30%				$175,905
	Construction Subtotal				$762,255
	Sales Tax at 8.6%				$65,554
	Engineering Design, CM, and Inspection @ 25%				$190,564
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$1,020,000**

NOTES:
1.) Assumes reservoir is constructed in same location as existing reservoir.
2.) 26' Diam. x 40' Tall Baker Silo-style reservoir.

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 30
PRELIMINARY COST ESTIMATE
Alternative 2: Recoat the Reservoir Without Seismic Upgrades
October 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$26,910	$26,910
3.	Manway	1	EA	$20,000	$20,000
4.	Roof Vent	1	LS	$30,000	$30,000
5.	Foundation Seal Grout Replacement	1	LS	$7,000	$7,000
6.	Interior Recoating	1	LS	$86,000	$86,000
7.	Exterior Recoating	1	LS	$76,000	$76,000
8.	Reservoir Containment	1	LS	$60,000	$60,000
9.	Removal of Mill Scale	2,000	SF	$4	$8,000
10.	Level Gauge Board	1	LS	$10,000	$10,000
11.	Surface Restoration	1	LS	$2,000	$2,000
	Subtotal				$350,910
	Contingency @ 30%				$105,273
	Construction Subtotal				$456,183
	Sales Tax at 8.6%				$39,232
	Engineering Design, CM, and Inspection @ 30%				$136,855
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$630,000**

NOTES:

1.) A new manway would be required to recoat the interior of the reservoir.

LAKE WHATCOM WATER & SEWER DISTRICT
DIVISION 30
PRELIMINARY COST ESTIMATE
Alternative 3: Seismically Upgrade and Recoat the Reservoir
December 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$31,680	$31,680
3.	Circumferential Guardrail	1	LS	$45,000	$45,000
4.	Manway	1	EA	$20,000	$20,000
5.	Roof Vent	1	LS	$30,000	$30,000
6.	Foundation Seal Grout Replacement	1	LS	$7,000	$7,000
7.	Interior Recoating	1	LS	$86,000	$86,000
8.	Exterior Recoating	1	LS	$76,000	$76,000
9.	Reservoir Containment	1	LS	$60,000	$60,000
10.	Removal of Mill Scale	2,000	SF	$4	$8,000
11.	Level Gauge Board	1	LS	$10,000	$10,000
12.	Surface Restoration	1	LS	$10,000	$10,000
	Subtotal				$408,680
	Contingency @ 30%				$122,604
	Construction Subtotal				$531,284
	Sales Tax at 8.6%				$45,690
	Engineering Design, CM, and Inspection @ 30%				$159,385
	Seismic Upgrade Total Project Costs				$758,000
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$1,490,000**

NOTES:

1.) Seismic upgrade costs in the BHC Report included engineering, contingency, and tax at unknown rates so the total provided in their report is included without additional markup.

LAKE WHATCOM WATER & SEWER DISTRICT
GENEVA RESERVOIR
PRELIMINARY COST ESTIMATE
Alternative 1: Reservoir Recoat Without Seismic Upgrades
October 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$41,760	$41,760
3.	Manway	1	EA	$20,000	$20,000
4.	Roof Vent	1	LS	$30,000	$30,000
5.	Foundation Seal Grout Replacement	1	LS	$7,000	$7,000
6.	Interior Recoating	1	LS	$187,000	$187,000
7.	Exterior Recoating	1	LS	$146,000	$146,000
8.	Reservoir Containment	1	LS	$54,000	$54,000
9.	Removal of Mill Scale	2,000	SF	$4	$8,000
10.	Level Gauge Board	1	LS	$10,000	$10,000
11.	Surface Restoration	1	LS	$2,000	$2,000
	Subtotal				$530,760
	Contingency @ 30%				$159,228
	Construction Subtotal				$689,988
	Sales Tax at 8.8%				$60,719
	Engineering Design, CM, and Inspection @ 25%				$172,497
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$920,000**

NOTES:

1.) Manway is optional.

LAKE WHATCOM WATER & SEWER DISTRICT
GENEVA RESERVOIR
PRELIMINARY COST ESTIMATE
Alternative 2: Seismically Upgrade and Recoat the Reservoir
December 2022

NO.	ITEM	QUANTITY		UNIT PRICE	AMOUNT
1.	Minor Change	1	LS	$25,000	$25,000
2.	Mobilization and Demobilization	1	LS	$47,070	$47,070
3.	Circumferential Guardrail	1	LS	$45,000	$45,000
4.	Manway	1	EA	$20,000	$20,000
5.	Roof Vent	1	LS	$30,000	$30,000
6.	Foundation Seal Grout Replacement	1	LS	$7,000	$7,000
7.	Interior Recoating	1	LS	$187,000	$187,000
8.	Exterior Recoating	1	LS	$146,000	$146,000
9.	Reservoir Containment	1	LS	$60,000	$60,000
10.	Removal of Mill Scale	2,000	SF	$4	$8,000
11.	Level Gauge Board	1	LS	$10,000	$10,000
12.	Surface Restoration	1	LS	$10,000	$10,000
	Subtotal				$595,070
	Contingency @ 30%				$178,521
	Construction Subtotal				$773,591
	Sales Tax at 8.8%				$68,076
	Engineering Design, CM, and Inspection @ 30%				$232,077
	Seismic Upgrade Total Project Costs				$708,000
	TOTAL ESTIMATED PROJECT COST (ROUNDED):				**$1,780,000**

NOTES:

1.) Seismic upgrade costs in the BHC Report included engineering, contingency, and tax at unknown rates so the total provided in their report is included without additional markup.

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

December 2016

BHC CONSULTANTS

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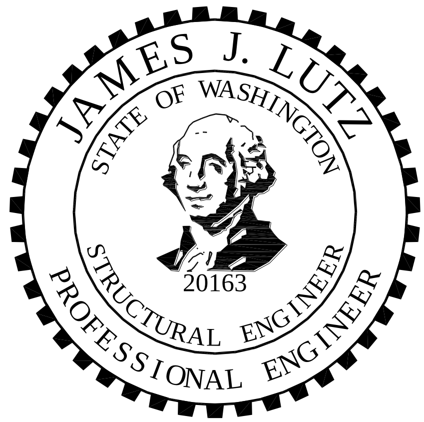
Professional of Record Certification

The report was prepared by Jim Lutz, P.E., S.E. under the direct supervision of Jim Gross, P.E.

Jim Gross, P.E.
Project Manager



Jim Lutz, S.E.
Structural Engineer

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B.3 Division 7 Reservoir Calculations
B.4 Division 30 Reservoir Calculations
B.5 SVWTP Reservoir Calculations

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.

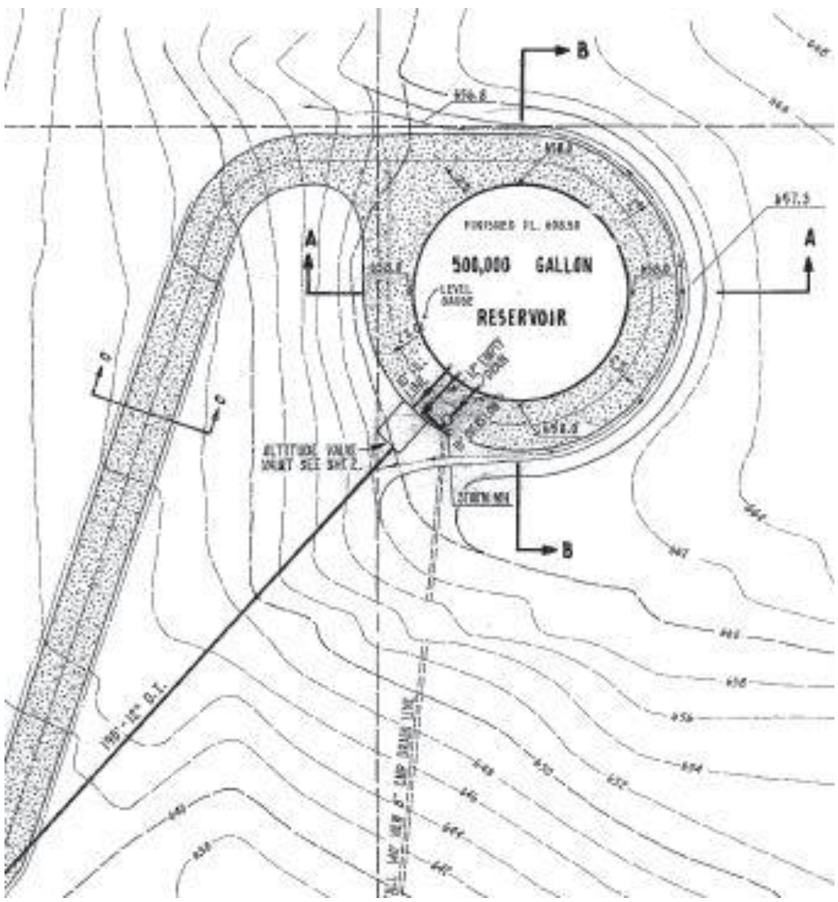


Figure 1 Site Plan from Original Design Drawings

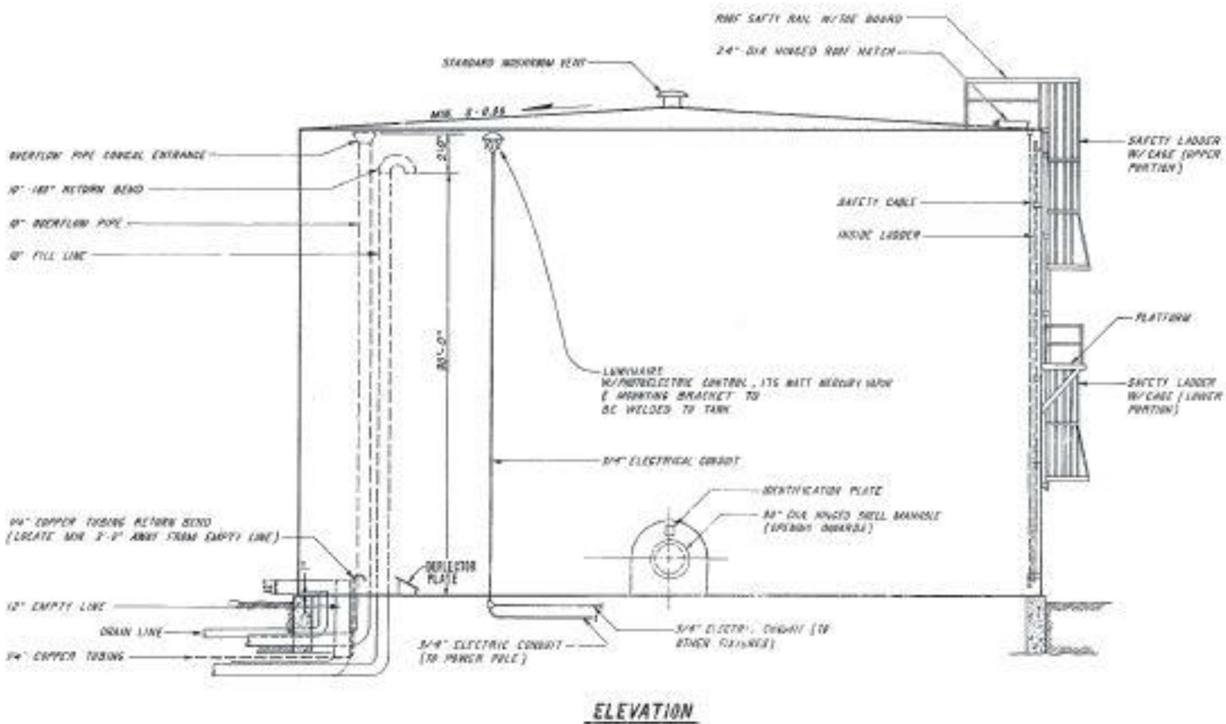


Figure 2 Elevation View from Original Design Drawings

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.

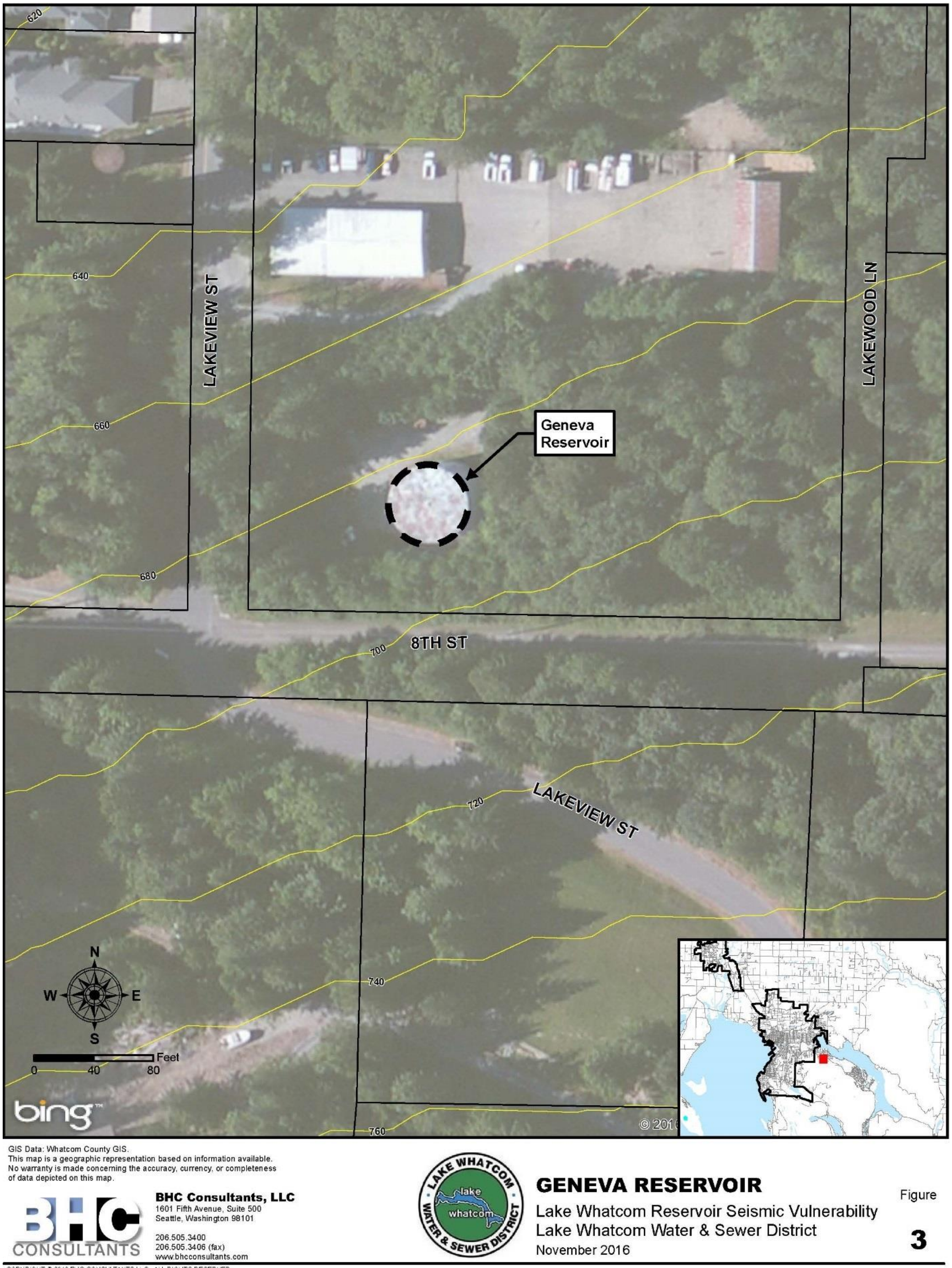


Figure 3 Geneva Reservoir

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.

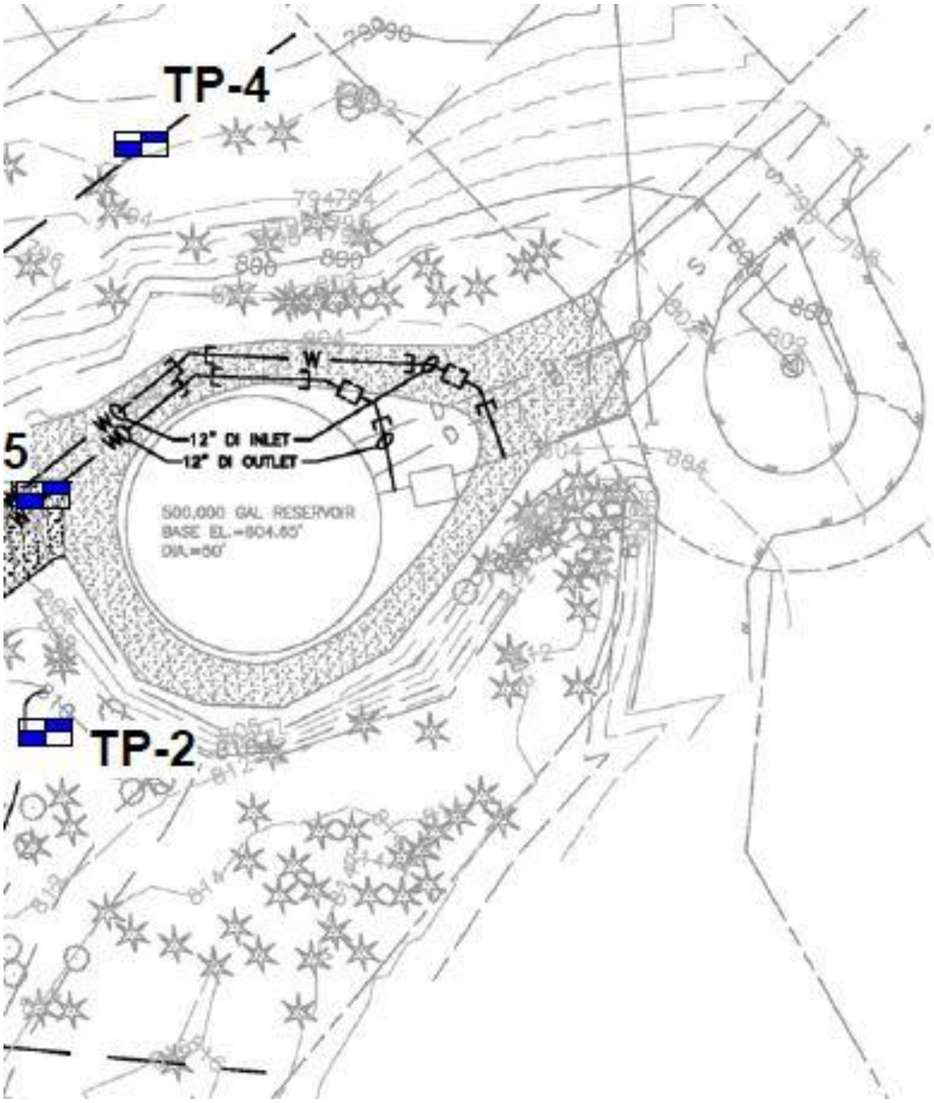


Figure 7 Division 22 Reservoir Site Plan from PanGeo Report

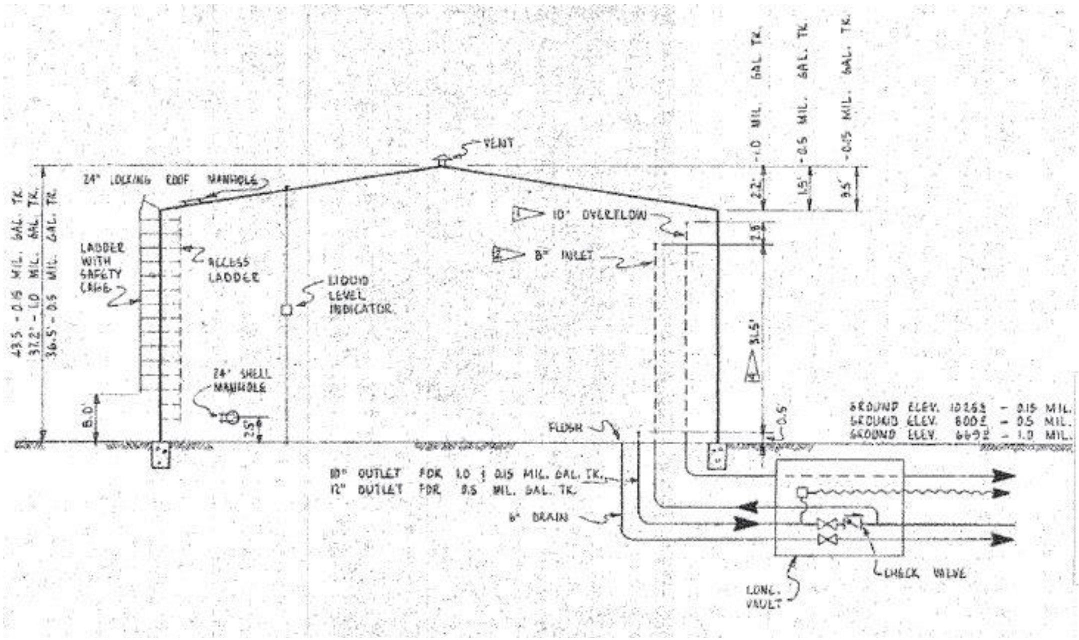


Figure 8 Division 22 Reservoir Elevation View from Original Design Drawings

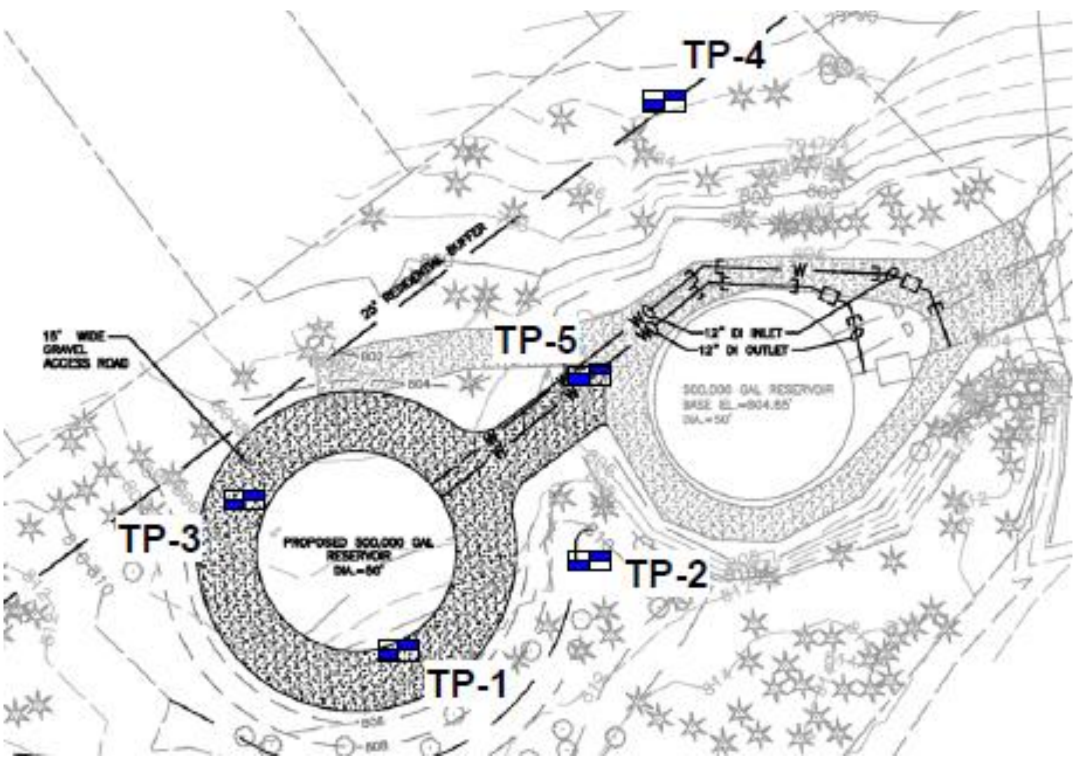


Figure 9 Division 22 Reservoir – Proposed Second Tank, from Pan Geo Report

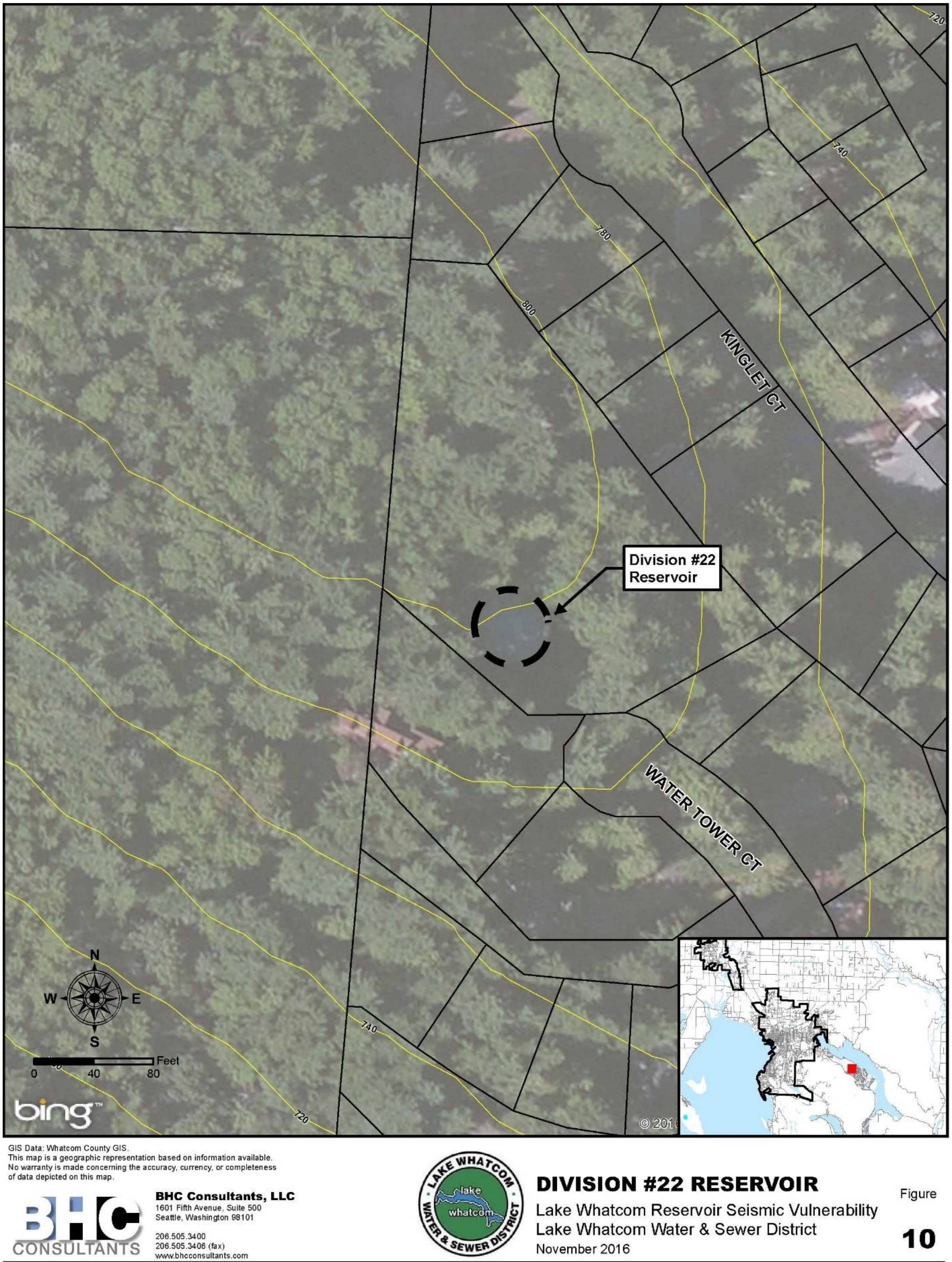


Figure 10 Division 22 Reservoir

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.

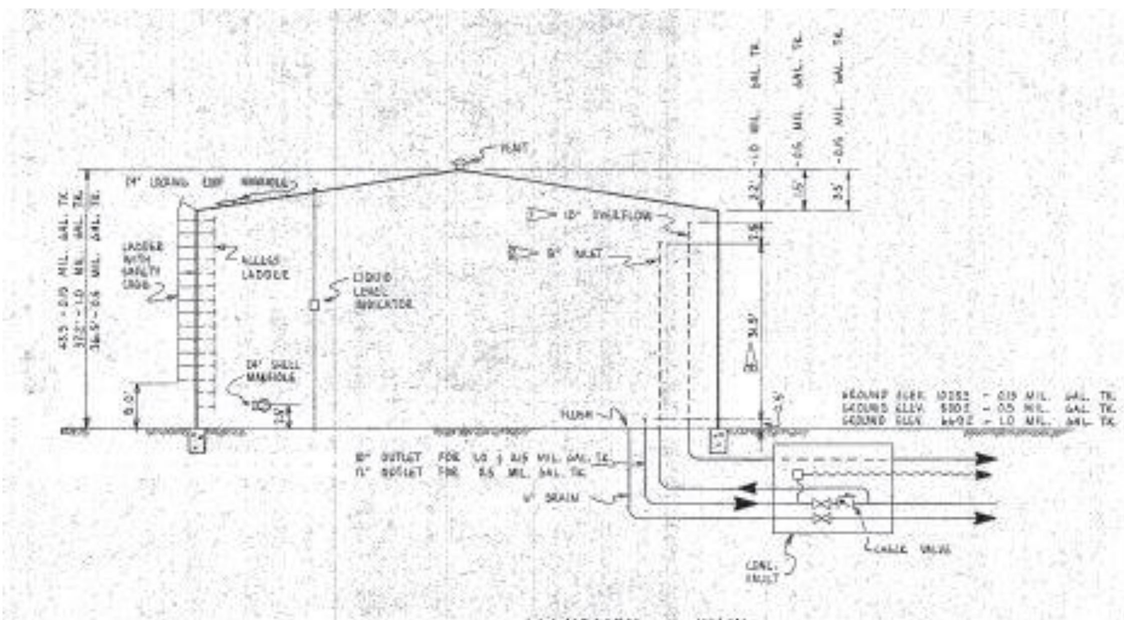


Figure 16 Elevation View from Original Design Drawings

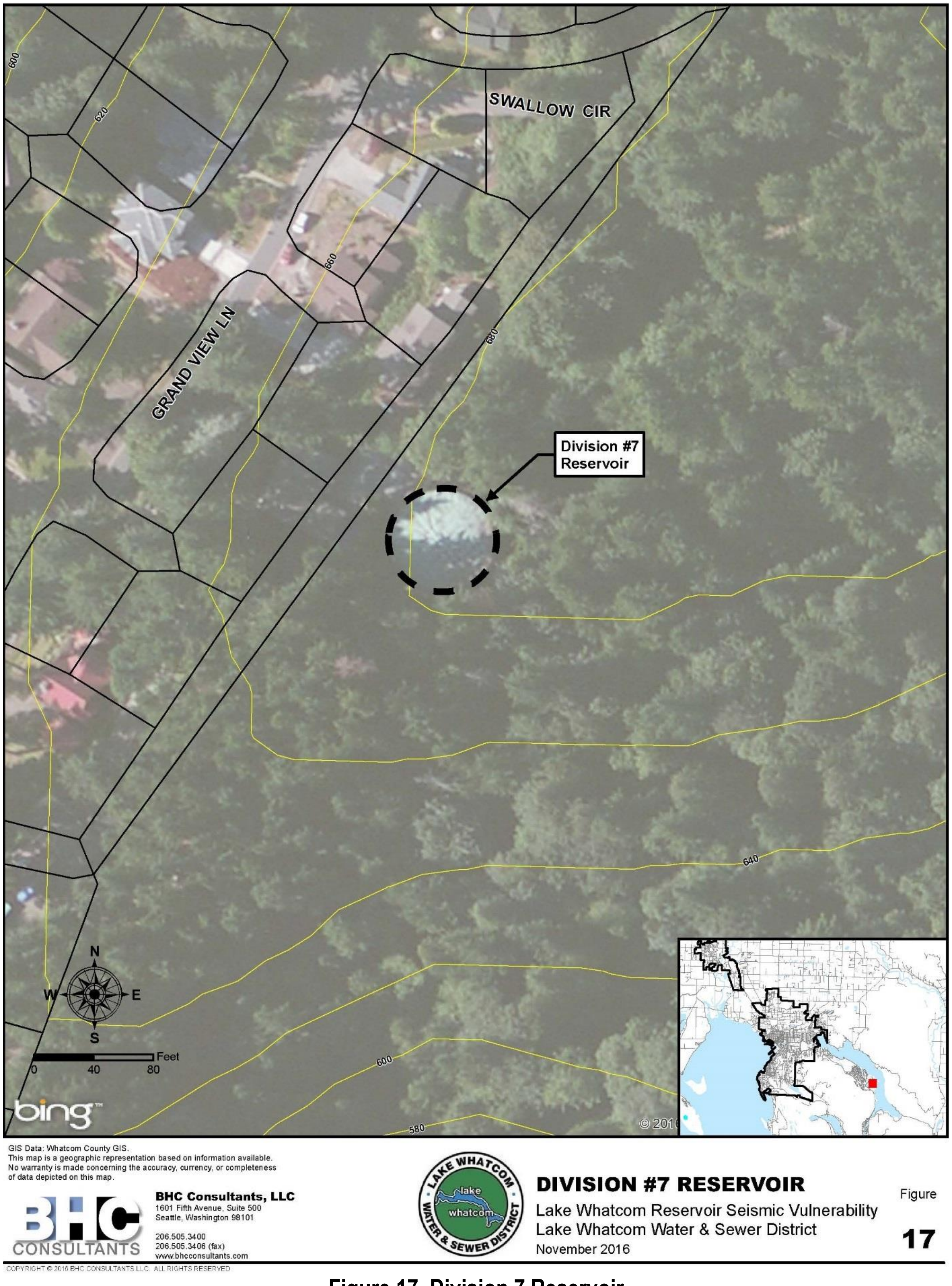


Figure 17 Division 7 Reservoir

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.

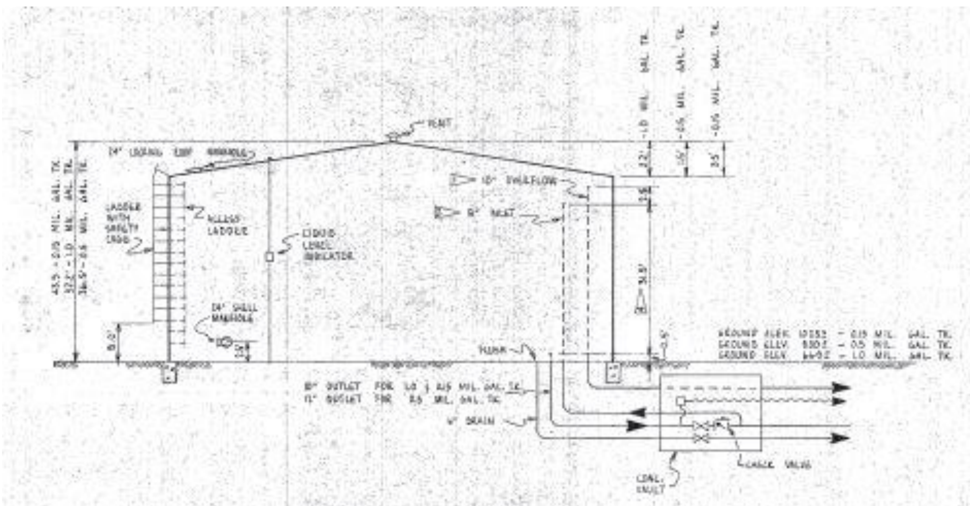


Figure 22 Elevation View from Original Design Drawings

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.

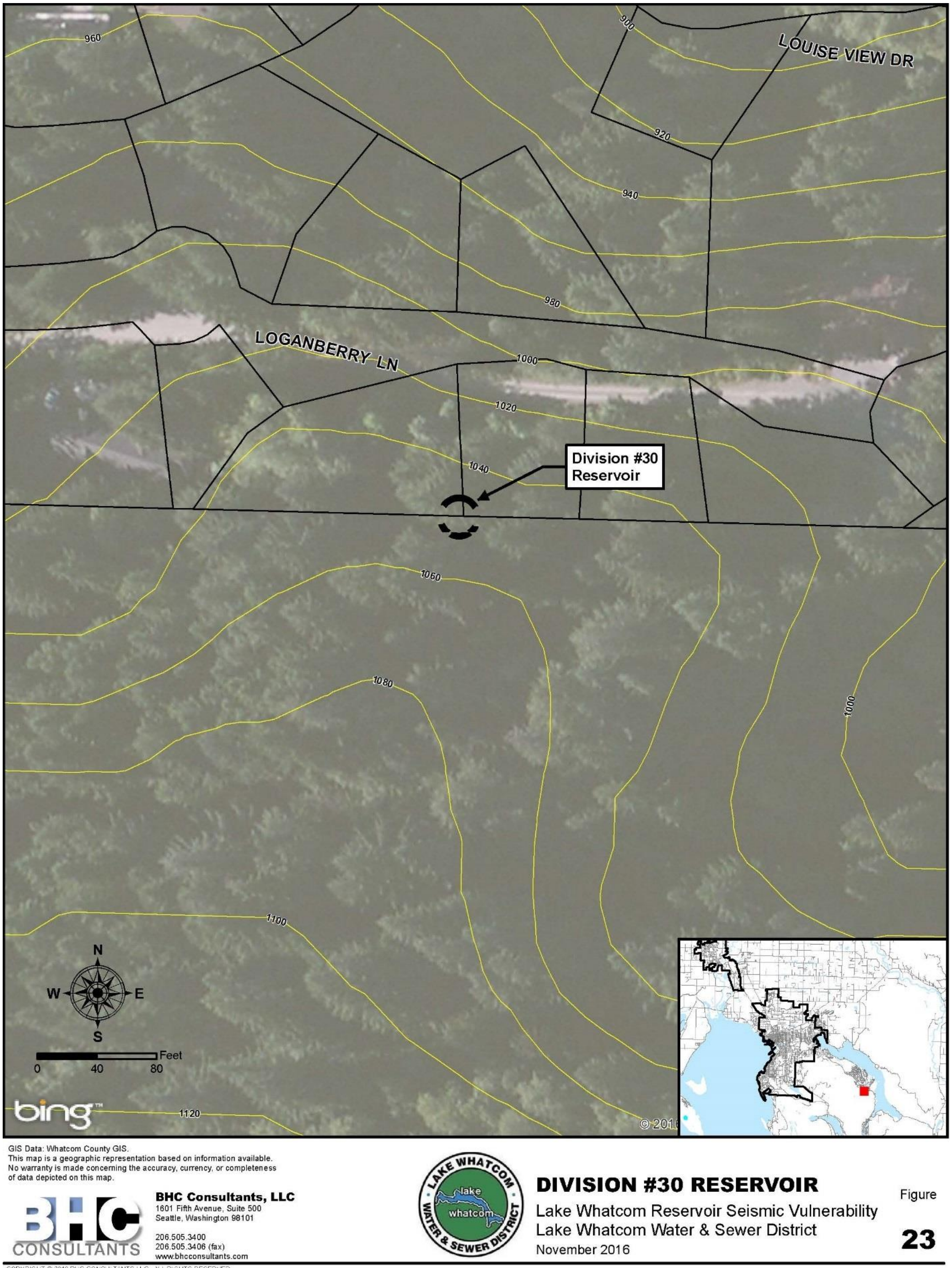


Figure 23 Division 30 Reservoir

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

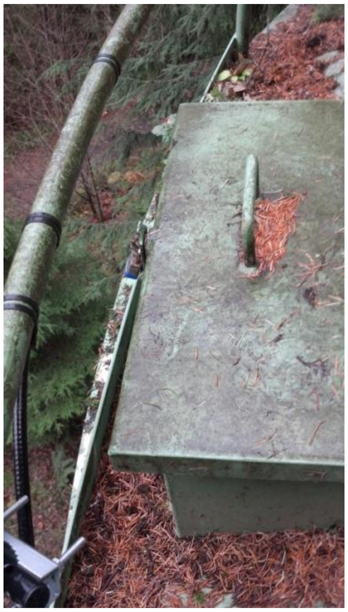
The measured diameter of the tank is 25 feet 5 inches, and the shell height is 40 feet 4.5 inches. The overflow elevation (record) is 6 inches below the top of shell, for a gross top capacity of 151,390 gallons compared to a nominal capacity of 150,000 gallons. The tank is anchored with 12 strap anchors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

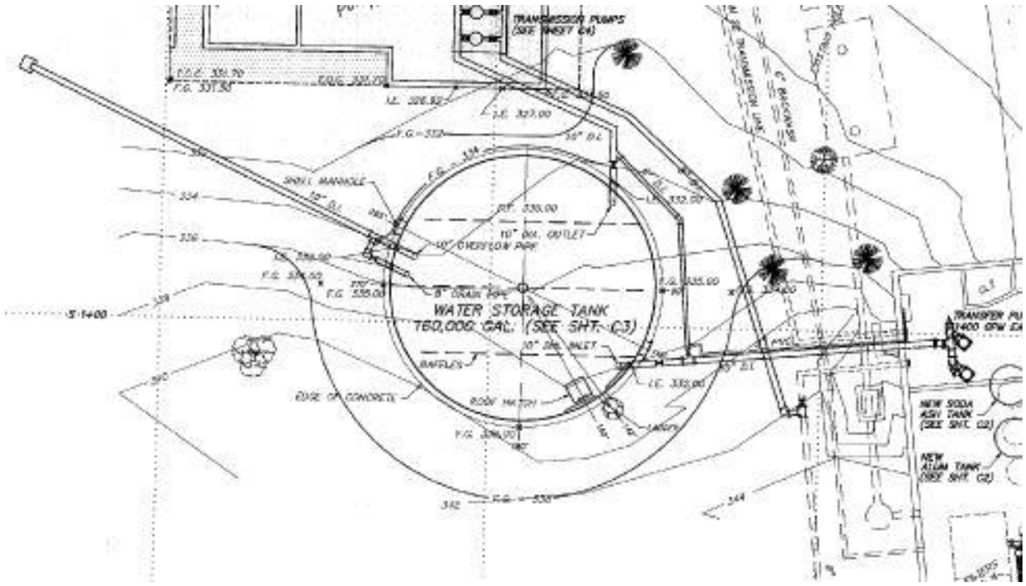


Figure 29 Site Plan from Original Design Drawings

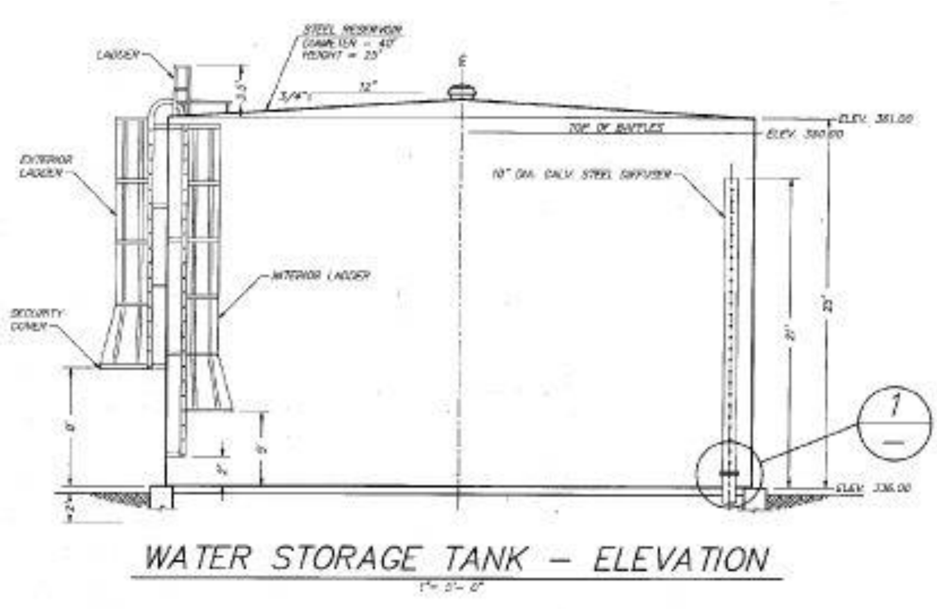


Figure 30 Elevation View with Inlet Diffuser, Wilson Engineering, 1992

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.

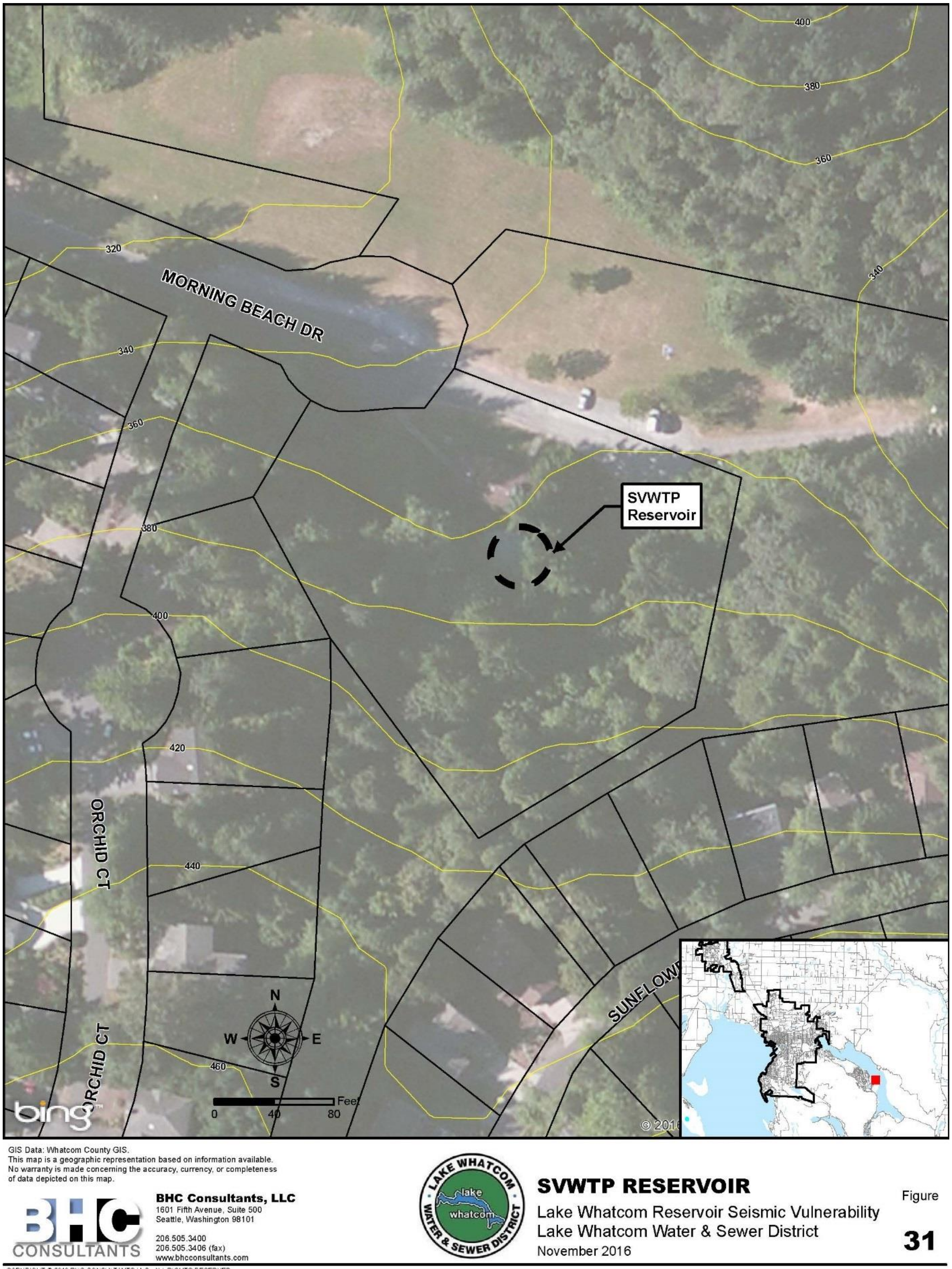
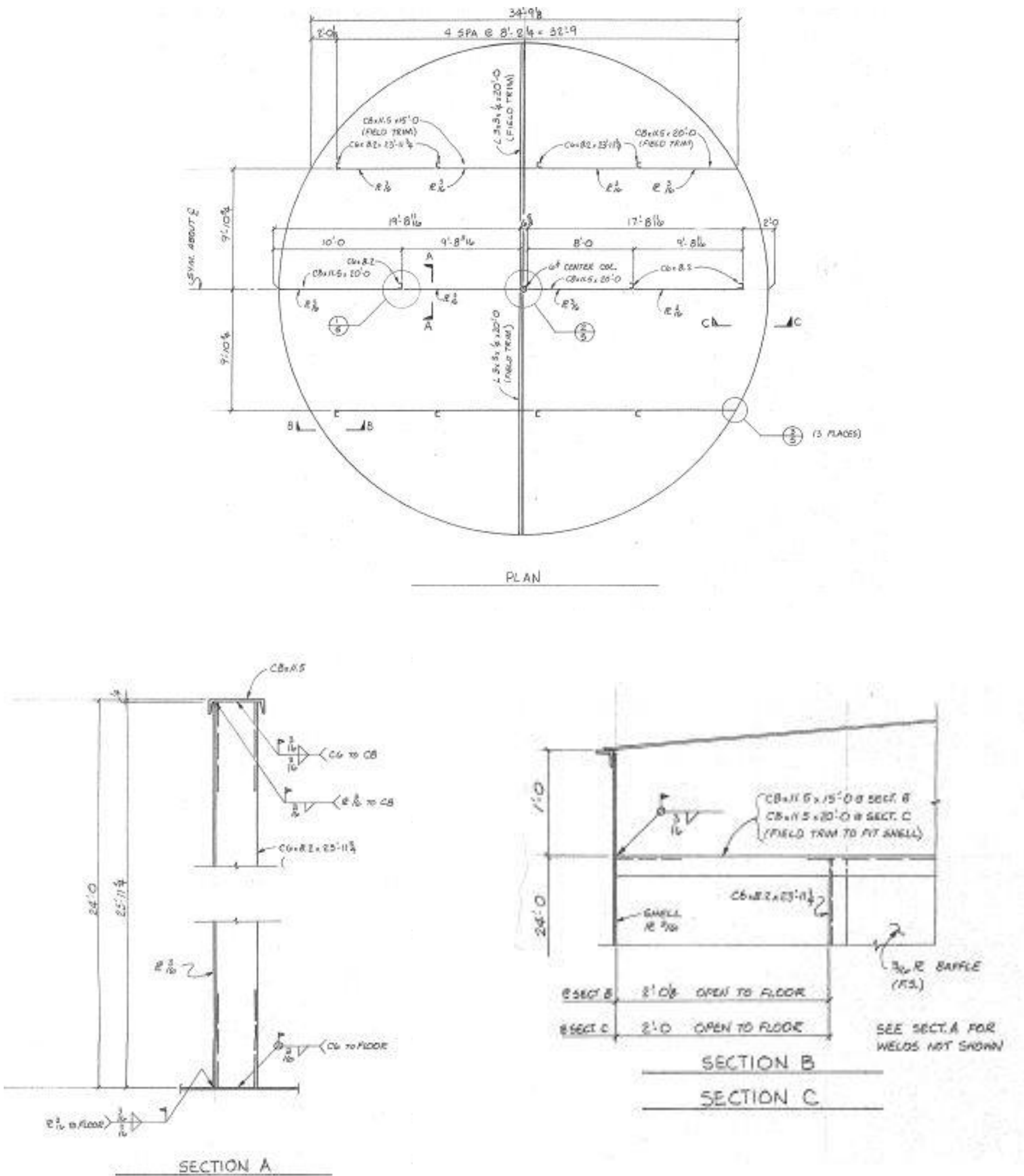


Figure 31 SVWTP Reservoir

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
Anchors			
Anchor spacing	9 ft. 8 in	≤ 10 ft.	OK
Predicted/Allowable Stress Ratio (anchor bolt)	1.05	≤ 1.0	Say OK,

Table 12 – Seismic Load vs AWWA D100 Allowable – SVWTP Reservoir

Effect of Baffles Ignored			
	Analysis	**AWWA Requirement**	**Result**
			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.

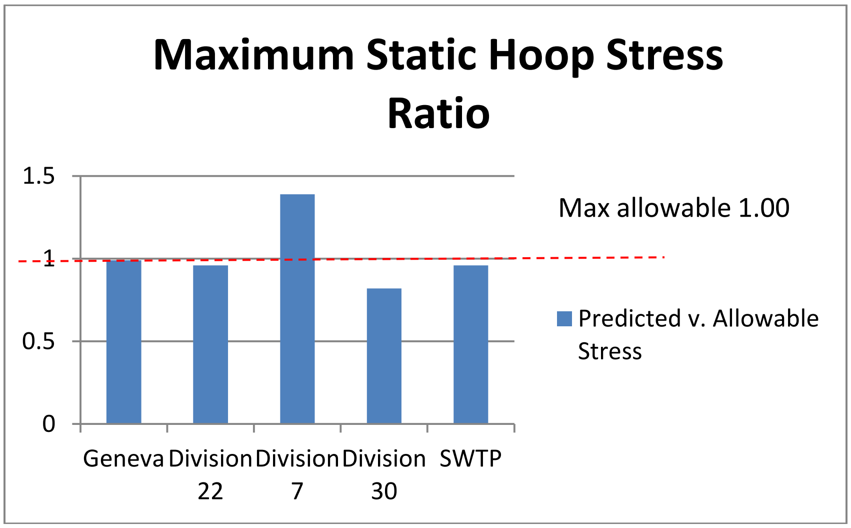


Figure 38 Maximum Static Hoop Stress Ratio

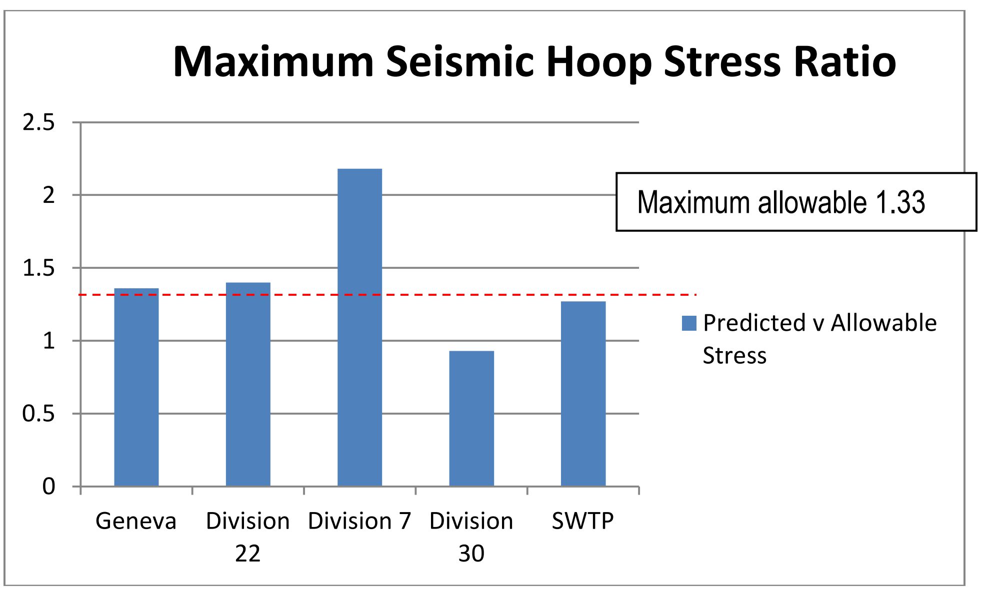


Figure 39 Maximum Seismic Hoop Stress

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.

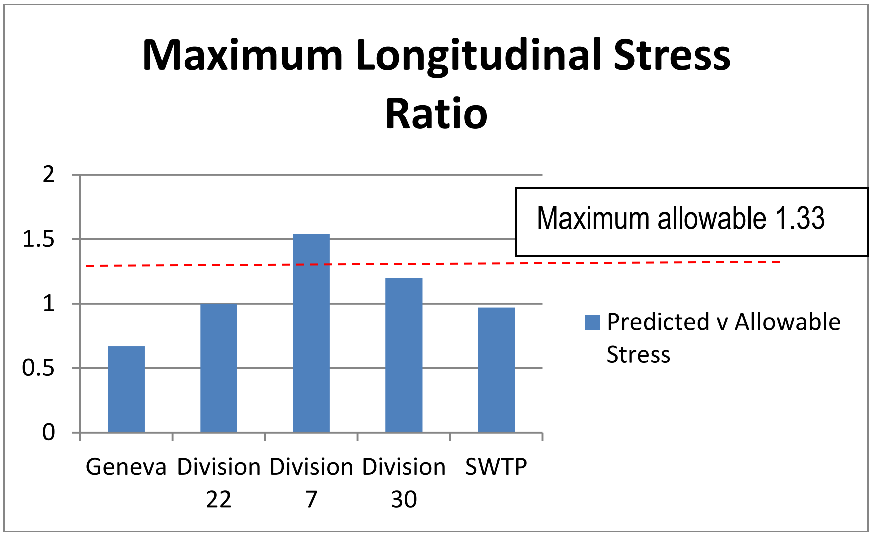


Figure 40 Maximum Longitudinal Stress Ratio

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.

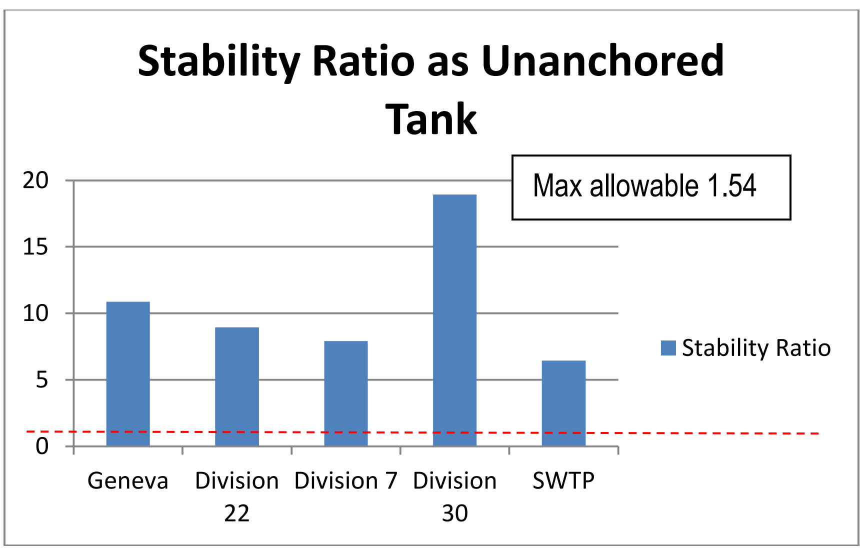


Figure 41 Stability Ratio as Unanchored Tank

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.

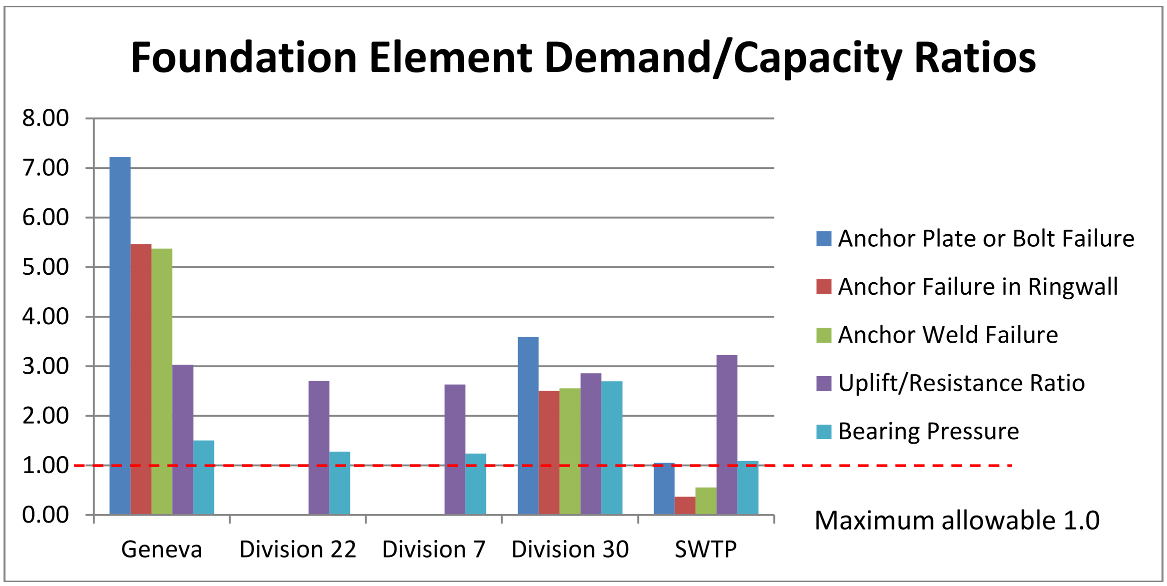


Figure 42 Foundation Element Demand/Capacity Ratios

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).

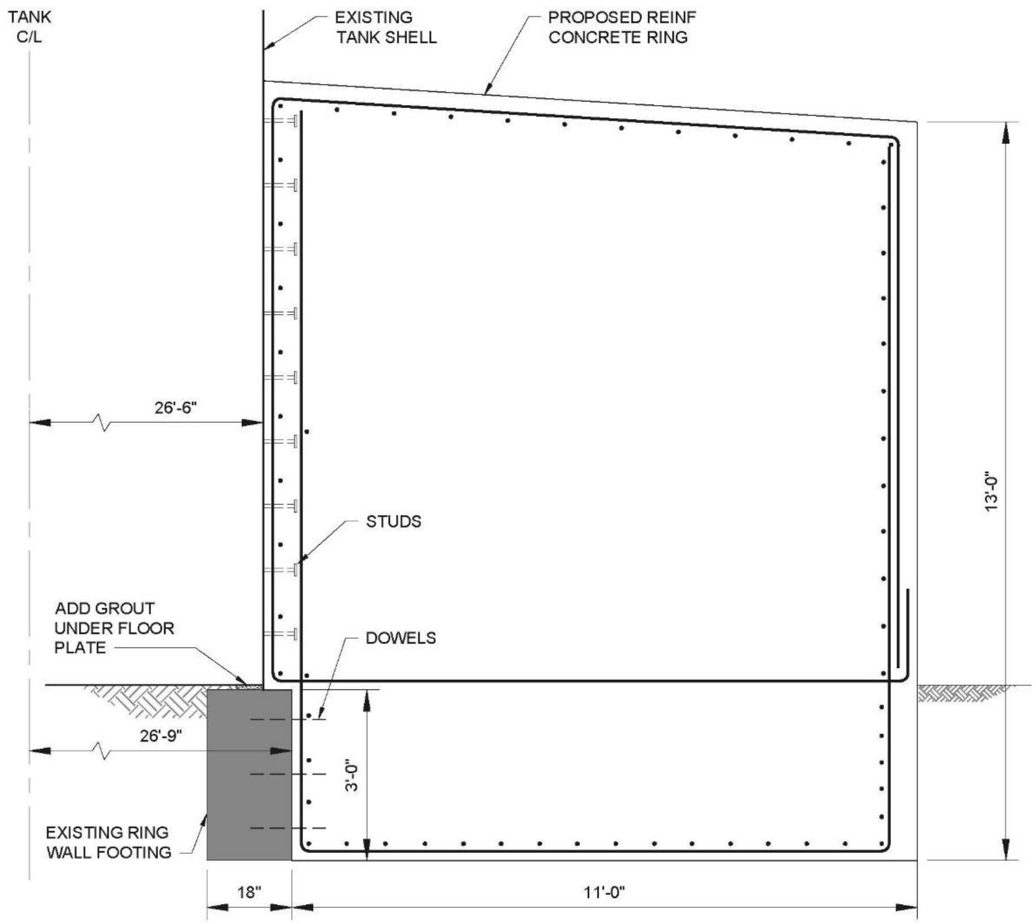


Figure 43 External Ringwall Above and Below Grade

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.

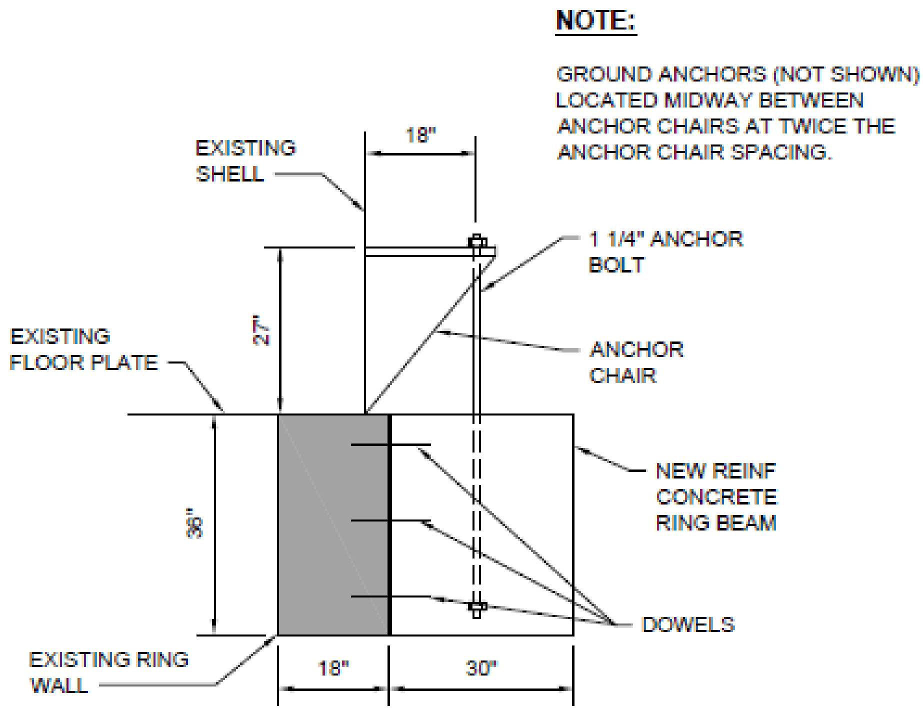


Figure 44 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

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.

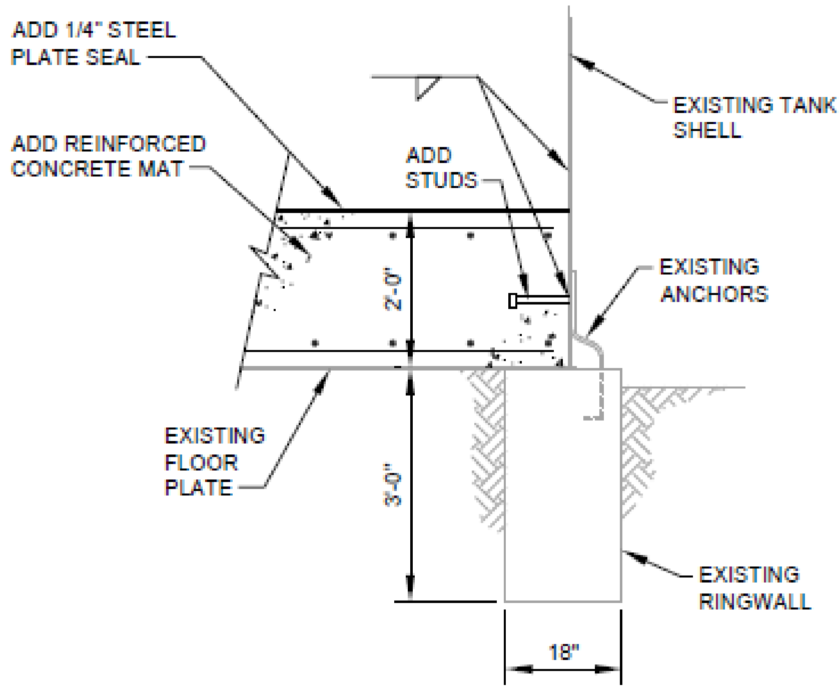


Figure 45 Supplemental Internal Bottom Mat Attached to Shell with Studs

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.

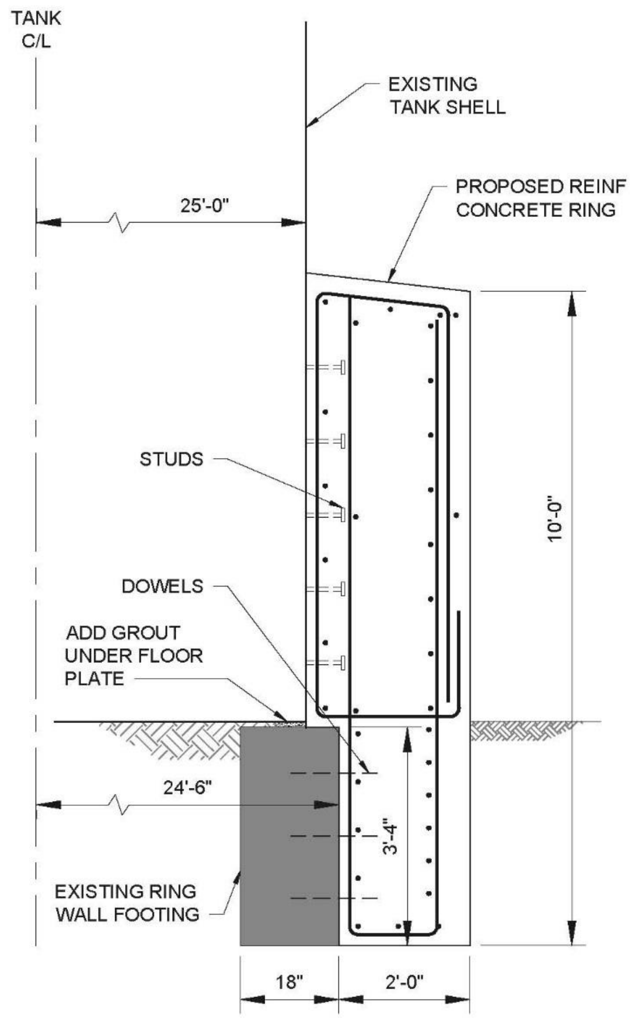


Figure 46 External Ringwall Above and Below Grade

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.

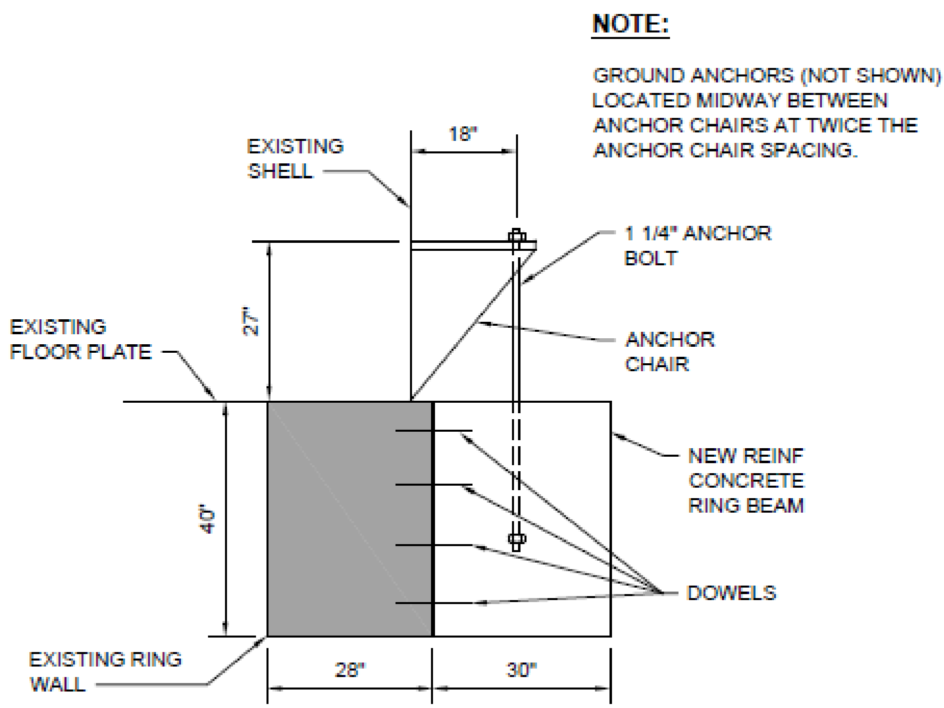


Figure 47 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

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.

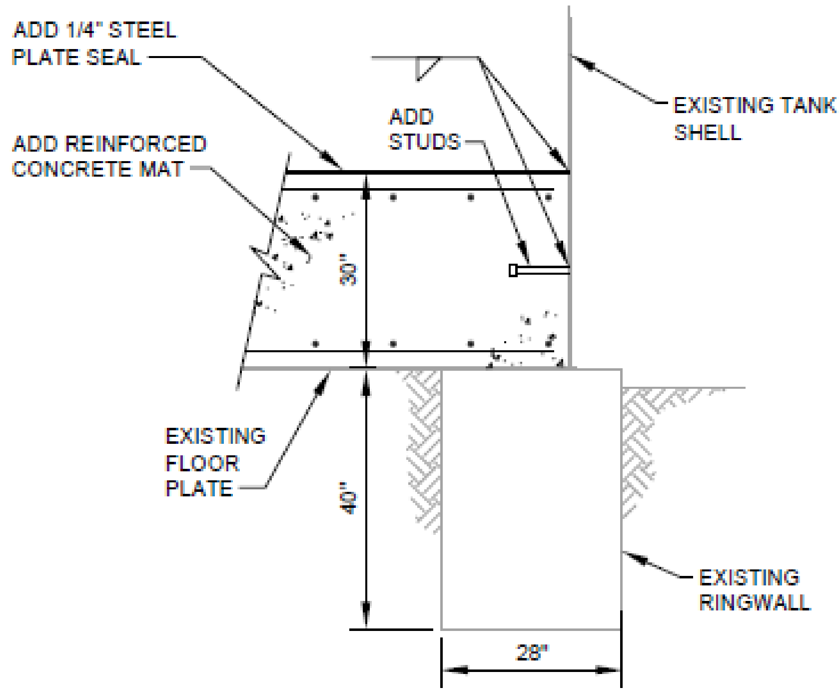


Figure 48 Supplemental Internal Bottom Mat Attached to Shell with Studs

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)

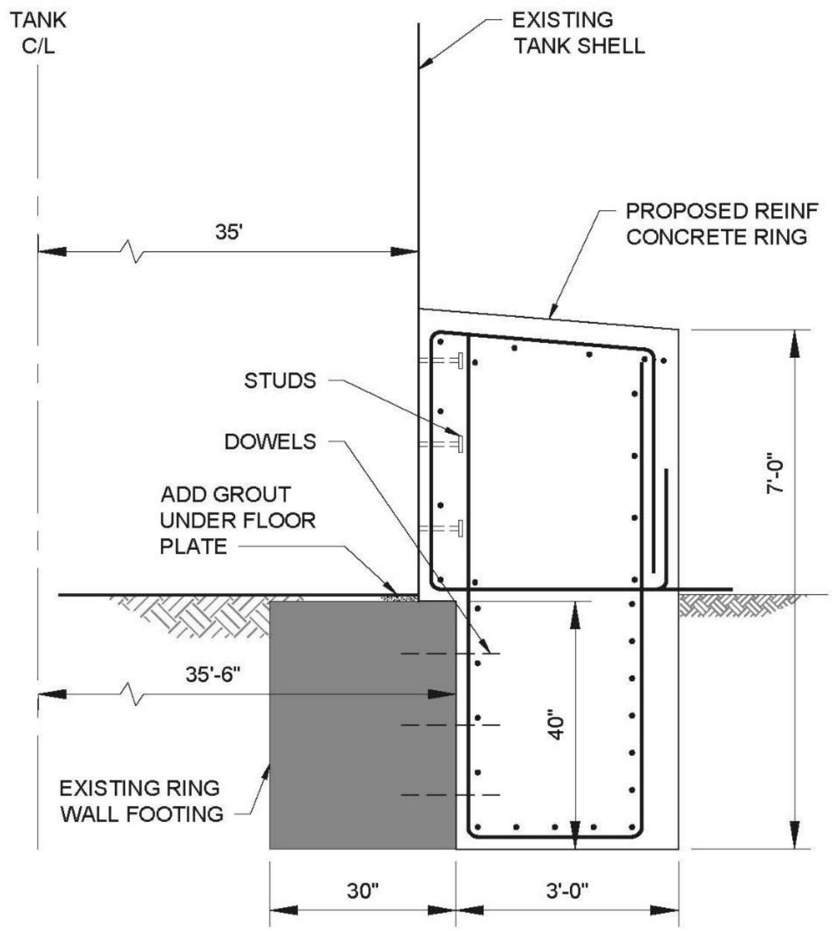


Figure 49 Alternate A – Division 7 Reservoir

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.

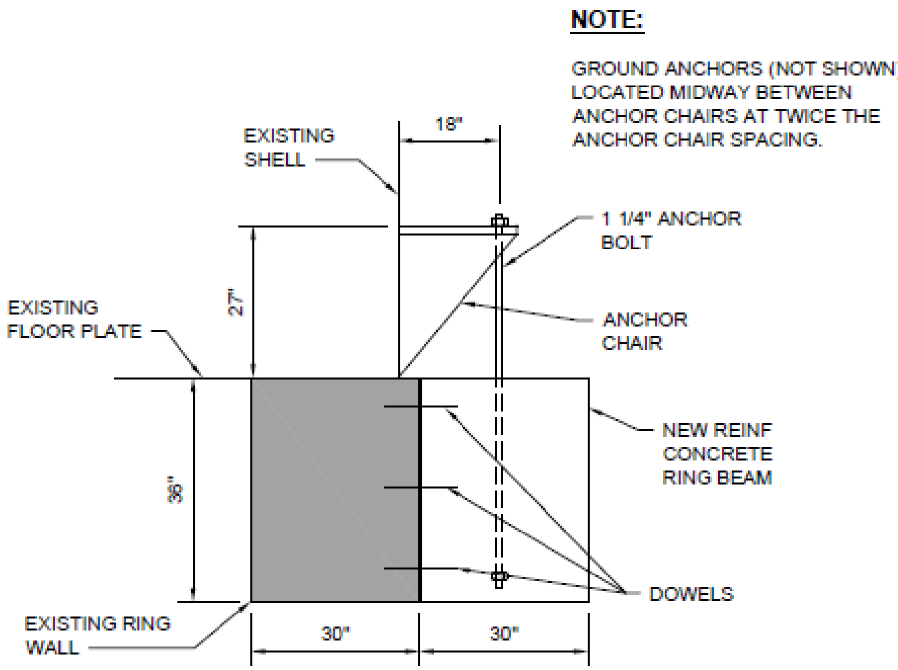


Figure 50 Alternate C – Division 7 Reservoir

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.

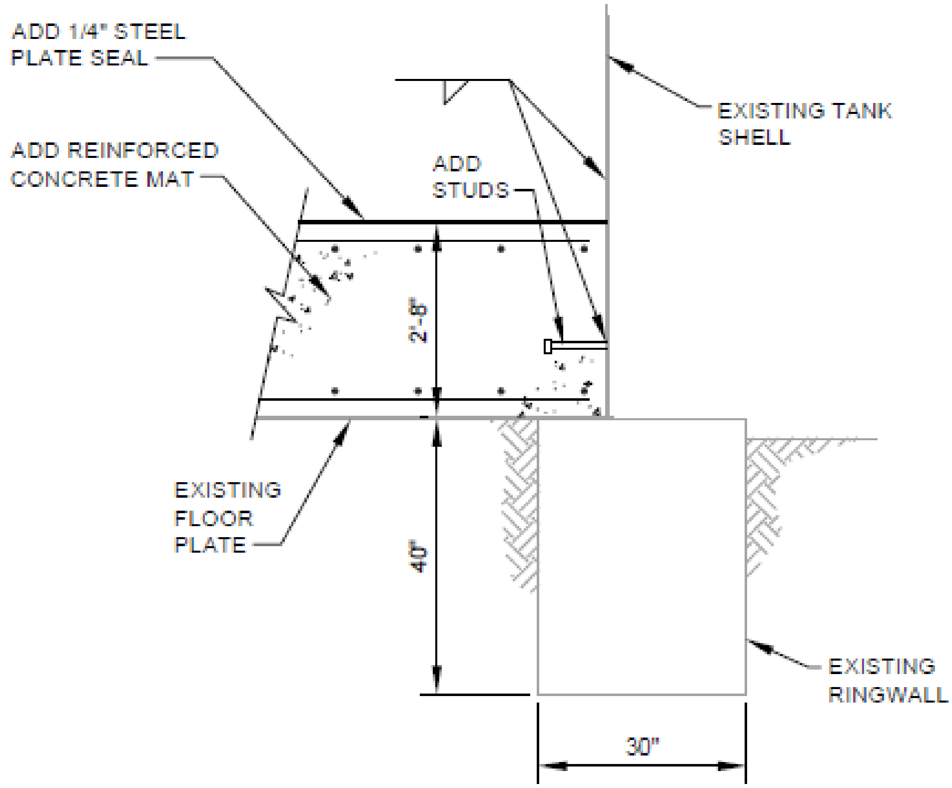


Figure 51 Supplemental Internal Bottom Mat Attached to Shell with Studs

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).

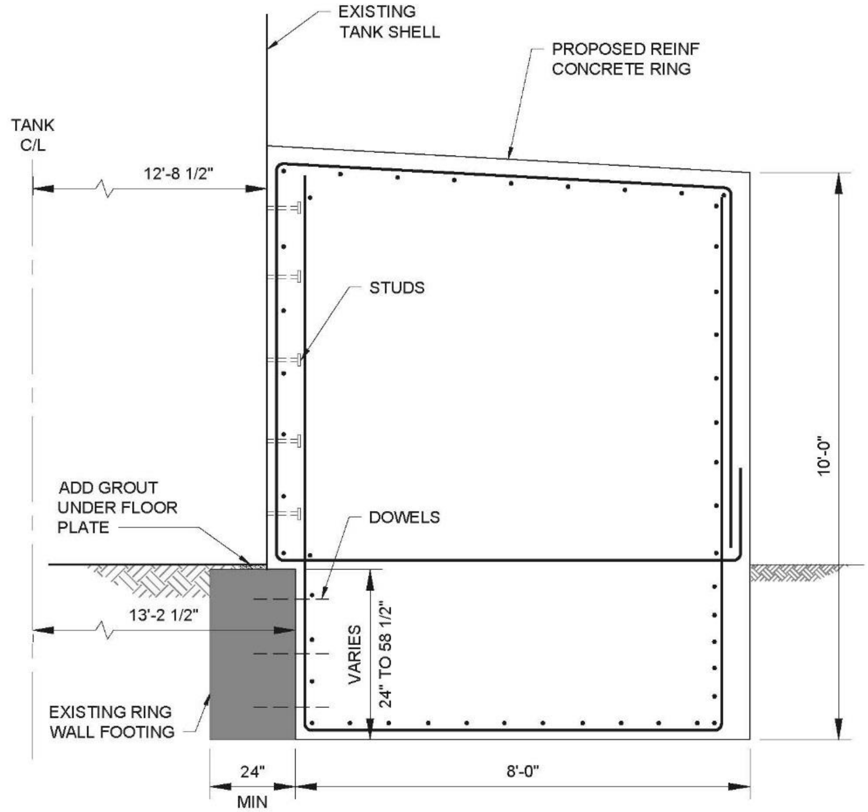


Figure 52 External Ringwall Above and Below Grade

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.

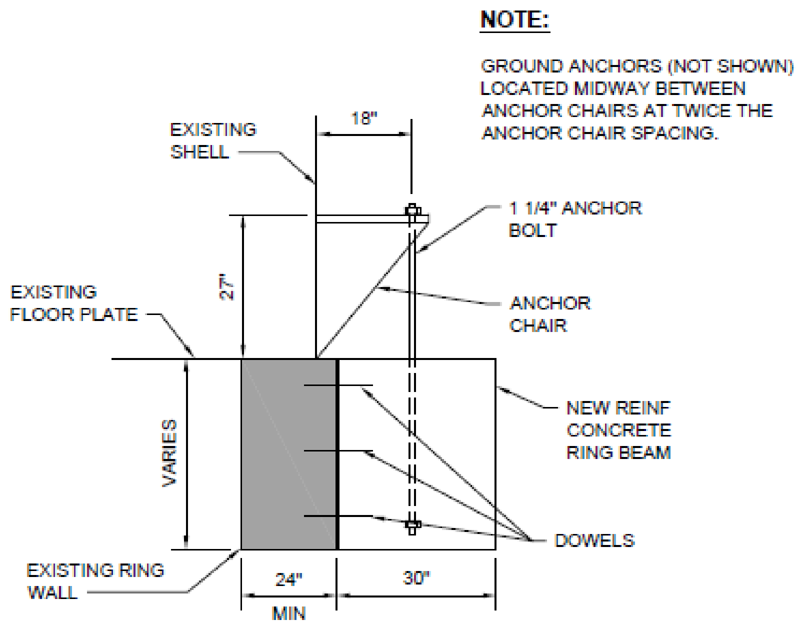


Figure 53 Supplementary External Ringwall with Anchor Bolts and Ground Anchors

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.

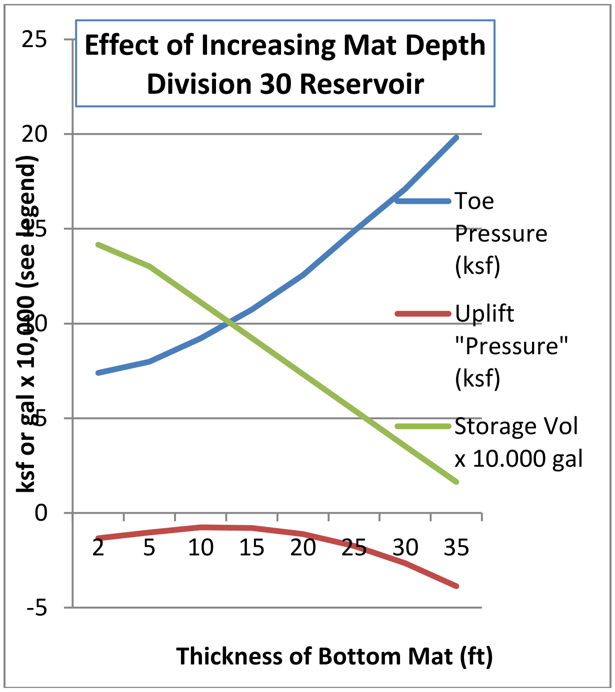


Figure 54 Effect of Increasing Mat Depth

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).

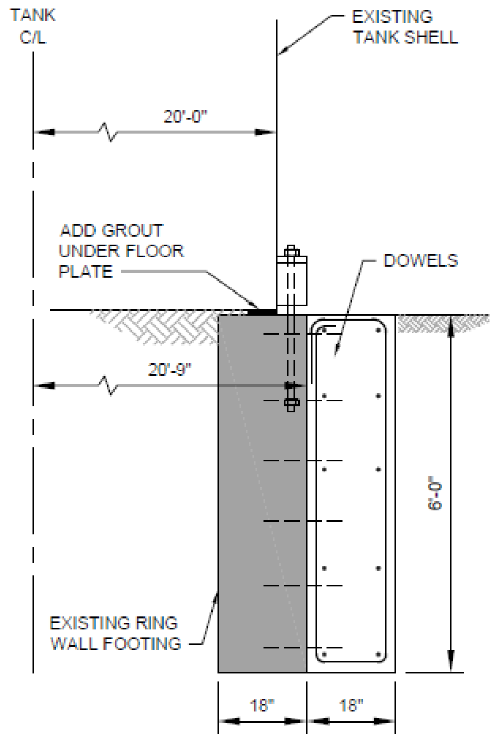


Figure 55 Added Ballast to Existing Ringwall

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

APPENDIX A.2

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APPENDIX B.1

GENEVA RESERVOIR CALCULATIONS

APPENDIX B.2

DIVISION 22 RESERVOIR CALCULATIONS

APPENDIX B.3

DIVISION 7 RESERVOIR CALCULATIONS

APPENDIX B.4

DIVISION 30 RESERVOIR CALCULATIONS

APPENDIX B.5

SVWTP RESERVOIR CALCULATIONS