September 8, 2022

Coburg 500K Gallon Steel Reservoir Seismic Evaluation

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PSE Project Number: 2101-0181

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PETERSON STRUCTURAL ENGINEERS

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1 Introduction

The purpose of the evaluation was to identify seismic vulnerability issues for the specified structures and determine potential upgrades required for current code compliance. The subject structures are to be evaluated according to current code criteria based on available as-built documents and information collected from field investigations.

2 Reservoir Analysis

2.1 Description

According to the plates present on each tank, the south tank was completed in 1975 and the north tank was completed in 1977, and both were manufactured by Pacific Tank and Construction Corporation of Portland Oregon. The tank plates did not indicate the design code used but based on the construction date both tanks were likely designed to the American Water Works Association (AWWA) D100-73 and the 1973 Uniform Building Code. No construction drawings were provided for review or verification of the design criteria or structural elements.

Based on observations and data collected on site, both tanks have the same overall geometry and structural design. Both are ground supported, welded steel non-standpipe tanks with a nominal capacity of 500,000 gallons and an inside dimeter of 52'-0". Both tanks consist of (4) wall shell plate courses with a nominal height to the top of the shell of 32'-0". The roof framing consisted of a center column and ring of (26) C-channel rafters. The overflow for both tanks is at approximately 31'-4", directly below the roof rafters which sit 8" deep.

Using an ultrasonic thickness gauge, the roof plate thickness was measured to be 0.226-inches on the north tank and 0.212-inches on the south tank. The roofs consist of steel plates which are lapped together and welded on their external seams, with the roof plate overhanging the wall shell plate by 2.25-inches on both tanks. The foundation is cast-in-place concrete that is 12-inches thick and extends 4.0-inches beyond the exterior shell wall. The concrete is assumed to be at least minimally reinforced but no testing was performed to verify this.

2.2 Visual Condition Assessment

PSE performed a site visit to observe the as-built current condition of each reservoir, including the interior as observable from the roof hatches. The site visit was performed on August 9th, 2022 and occurred while the water level of each tank was approximately 5-feet below the overflow. Condition observations are only reported for items that have the potential to affect the structural performance of the reservoir. Assessments of general maintenance, life safety, or coating issues are beyond the scope of this document.

2.2.1 North Tank

The general exterior condition of the north tank appeared to be in sound structural condition. No significant cracking or weathering of the ring footing concrete was observed. Some minor areas of corrosion were noted at the base of the shell, as well as some areas of flaking in the exterior coating but neither observation appears to present an immediate structural issue. The exterior surface of the roof was free of debris build up but appeared to have some minor areas of corrosion and flaking coating. The interior of the reservoir exhibited areas of corrosion staining at the typical operating level, roof plate seams, rafter flanges, and connection points but it is not a structural concern at this time.

2.2.2 South Tank

The exterior condition of the south tank generally matched that of the north tank. No notable structural concerns were observed with the visible portion of the footing or wall shell plates. Similar to the north tank, some minor areas of corrosion were noted in the bottom shell course, as well as some areas of flaking of the coating. The exterior surface of the roof was free of debris build up but appeared to have some minor areas of corrosion and flaking coating. The interior of the reservoir exhibited minor areas of corrosion staining, primarily occurring at the roof plate seams and bolted connections but it is not a structural concern at this time.

2.3 Approach and Assumptions

The AWWA D100-21 standard for "Welded Carbon Steel Tanks for Water Storage" as supplemented by the American Society of Civil Engineering (ASCE) 7-16 "Minimum Design Loads and Associated Criteria for Buildings and Other Structures" standard was used for evaluation of the reservoirs.

PSE assessed the adequacy of the structure for the following conditions:

1) Vertical loads (self-weight, hydrostatic, roof, snow)
2) Horizontal loads (wind, seismic, hydrostatic, hydrodynamic, slosh)

The following general criteria were used for evaluation. Wind criteria and seismic data were obtained from the American Society of Civil Engineering Hazards (ASCE 7 Hazards) resource based on the site coordinates. Snow loads were obtained from the Structural Engineers Association of Oregon (SEAO) snow load website and Oregon Structural Specialty Code (OSSC) designated minimums. For this assessment the following inputs were used for our analysis:

<u>General</u>:

- Coordinates: 44.1213, -123.0589
- Year Built: North Tank 1977; South Tank 1975
- Overflow Level: 31'-4"

<u>Materials</u>:

- Concrete: 2,500 PSI (assumed)
- Steel Roof & Plates: A36, Fy = 36 KSI (assumed)
- Steel Wall Plate: Courses 1-4 – A36, Fy = 36 KSI (assumed)
- Joint Efficiency = 85% (assumed)

<u>Seismic</u>:

- Risk Category IV (Essential Facility)
- Soil Site Class: D
- $S_S = 0.70$ $S_1 = 0.39$
  $S_{DS} = 0.58$ $S_{D1} = 0.42$

<u>Wind (per OSSC 2019)</u>:

- Risk Category IV (Essential Facility)
- Ultimate Wind Speed: 110 mph
- Exposure: C

<u>Snow</u>:

- Ground Snow Load: 10 psf
- Minimum Snow Load: 25 psf

<u>Soils:</u>

- Unknown – Pending geotechnical report

Where observations could not be made, PSE has assumed that the structures were built in accordance with the provided historic documentation and have used the values for steel strengths provided therein for our analysis. Where additional assumptions have been made, they are noted in the analysis and evaluation sections.

Reservoir Shell – Material Thickness: Shell thicknesses were measured in field with an ultrasonic thickness gauge. The lower average plate thickness for each course was used in the analysis. The following table lists the shell thicknesses gathered during the site investigation.

Table 1 – Reservoir Plate Thickness

Reservoir Steel Thickness		
Component	North Tank Measured Value (in)	South Tank Measured Value (in)
Shell Course 1	0.286*	0.287
Shell Course 2	0.286	0.286*
Shell Course 3	0.291	0.272*
Shell Course 4	0.284*	0.285
Roof Plate	0.226	0.212*

*value used in analysis

2.3.1 Upcoming Code Change Discussion

The primary codes used for the reservoir analysis are ASCE 7 and AWWA D100. ASCE 7 is updated on a six-year cycle and the most recently published iteration is ASCE 7-22. However, the most recent version of AWWA D100 is D100-21, which still references ASCE 7-16. The changes reflected in ASCE 7-22 will not be adopted by AWWA until the next code cycle, but they will inevitably be adopted. As part of the evaluation, PSE reviewed the changes presented in ASCE 7-22 in order to conceptually discuss how they may impact the analysis of the reservoir in the future.

The ASCE 7 Hazards Tool references new ground snow load maps for ASCE 7-22 load generation and typically returns higher ground snow loads than those used in the original tank design. Higher ground snow load values will result in higher roof design loads and the existing roof framing may not be adequate to support these increases.

There have also been changes to elements of the seismic load development, specifically related to site classification. Largely the changes affect the determination of the Site Class of the project site, which subsequently affects the seismic design coefficients. The degree to which these changes affect individual project sites and their seismic design coefficients varies depending on the soil makeup and geotechnical test results. Based on the information provided by the ASCE 7 Hazards Tool for the project site, the seismic design coefficients for ASCE 7-16 are slightly unconservative when compared to those generated according to ASCE 7-22 for the same Site Class and the existing structure may have more or increased deficiencies when evaluated according to the ASCE 7-22 requirements. The ASCE 7-22 coefficients are approximately 7% higher than the ASCE 7-16 values used for the following analysis and would propagate through the analysis results.

2.4 Structural Analysis Results

2.4.1 Gravity Analysis

Roof Framing: The roof plate thickness and layout are adequate under current code for flexure stress requirements.

While the roof rafters did not appear to be continuously attached to the roof plate, D100-21 allows for a rafter with a depth less than 15-inches to be evaluated as being continuously braced. If we consider the rafters to be continuously braced per the AWWA code, the rafters are adequate for flexure and shear loading. The maximum deflection is also within the allowable serviceability limits. The roof rafters are anticipated to meet the requirements for ASCE 7-22 in terms of strength but may exceed the deflection limits.

The center column is assumed to be a 6-inch diameter Sch 40 pipe section based on scaled photos and typically specified member sizes. Using A53-B pipe steel, analysis shows the column has adequate capacity for current code gravity loads.

Reservoir Shell – Hydrostatic Stress: At the current overflow height of 31.33-feet from the tank base, the bottom course is overstressed by 16.0%. This analysis is based on steel plate thicknesses as measured in the field and assumed material strengths. Since the static stress equations have not changed significantly since the tank was designed, this result could indicate that the shell plate has a higher yield strength than assumed or that the tank was designed using Section 14 "Alternative Design Basis for Standpipes and Reservoirs" of the Current AWWA D100-21, which is equivalent to Appendix C of the previous iteration of D100 that was adopted at the time of the tank's construction.

The tank was also analyzed for a maximum operating height of 28.08-feet from the tank base which resulted in the bottom shell courses being overstressed by 4.1% for static loading. This operating height would accommodate the full slosh wave height and prevent it from impacting the roof. See the following Lateral Analysis section for further discussion of the slosh wave analysis.

Foundations: The foundation widths could not be verified but were assumed to be equal to twice the distance that they extended out beyond the wall shell. Assuming an 8-inch-wide ring footing, the static bearing pressure was 3,370-psf for the current overflow height (31.33-feet) and 3,248-psf for the lower operating height (28.08-feet). The AWWA requires a factor of safety of 3.0 for static loads, which will need to be verified by the geotechnical engineer.

2.4.2 Lateral Analysis

Reservoir Shell – Seismic Hydrodynamic Stress: For short duration loads, the code allows for the tensile capacity of the steel to be increased by 33%. Without using the increased allowances provided by Section 14/Appendix C, the bottom course is overstressed by 8.9% when analyzed with the water level at the overflow height (31.33-feet). When considering the lower maximum operating level analyzed (28.08-feet), the bottom course was adequate for the dynamic loading. The shell is adequate for vertical longitudinal stresses due to lateral loading conditions for both water levels considered.

Wind Girder: For wind loading against the wall shell, an intermediate wind girder is not required for either tank.

Freeboard/Slosh: Current criteria for determining the slosh wave height is largely controlled by the geometry of the tank. For these tanks, the slosh wave height was found to be 3.2-feet when operated at the overflow level of 31.33-feet. The available freeboard is based on the distance from the top of the tank overflow to the lowest level of the roof framing. The current overflow is set approximately flush with the bottom flange of the roof rafters, resulting in 0-feet of freeboard. The roof rafters and roof plate are inadequate for the load imparted by the slosh wave and the impact of the wave could result in failure of the roof and potential loss of tank contents. However, at the lower maximum operating level analyzed (28.08-feet), while the slosh wave height would still be 3.2-feet, it would not Impact the roof due to having adequate freeboard.

Overturning and Anchorage:

The reservoir self-weight and friction is adequate to resist sliding loads for both operating levels that were analyzed. However, the overturning results vary based on the water level. For the full overflow level, the self-weight is inadequate for overturning resistance and mechanical anchors would be required to resist the uplift. For the lower maximum operating level (28.08-feet), uplift would still occur, but the shell would be stable and mechanical anchors would not be required. Note, even though the shell is stable in this lower operating scenario, the uplift could result in larger deformations than accounted for in the piping design.

Foundations: Assuming an 8-inch-wide ring footing, the short-term bearing pressure was 9,754-psf for the current overflow height (31.33-feet) and 6,739-psf for the lower operating height (28.08-feet). The AWWA requires a factor of safety of 2.25 for short-term loads, which will need to be verified by the geotechnical engineer.

2.5 Recommendations

Overall, both tanks are currently in good structural condition. However, due to the age of the tanks and advancement of seismic load requirements there are multiple elements that are inadequate for current code load requirements based on the overflow water level. There are several options that could be evaluated to potentially bring the reservoir into partial or full compliance with current code.

Shell Recommendations

The bottom course of the shell was found to be inadequate for both hydrostatic and hydrodynamic loading conditions when considering the overflow level. The recommended solution is to add shell stiffeners to the bottom course, enforce a lower maximum operating level, or a combination of both options. Enforcing a maximum operating level of 28.08-feet, which allows adequate space for the slosh wave to dissipate, would reduce the loads to within the capacity of the bottom shell course for dynamic cases but stiffeners would still be required for static stresses. The lower operating level could be enforced by physically lowering the overflow pipe, or it could be enforced operationally.

If the desire is to maintain the current overflow level, further investigation should be performed to determine if the tanks were designed to Section 14/Appendix C, which would result in higher allowable capacities for the shell stresses.

Foundation

The foundation is presumed to be adequate for static conditions based on the absence of signs of settlement on site. The geotechnical engineer should verify if the bearing pressures provided are within their assessment of the site soils. If the site bearing capacities are lower than the applied pressures, further reduction of the operating level may be required to reduce foundation pressures within allowable limits or the foundation could be expanded.

Anchorage

If the desire is to maintain the current overflow level, mechanical anchors will be required and consequently a foundation expansion to accommodate those anchors. However, if a lower maximum operating level of 28.08-feet is enforced, mechanical anchors are not required, and subsequently a foundation expansion would not be required to accommodate them. Note, a foundation expansion may still be required to accommodate short-term bearing pressures, this should be verified with the geotechnical engineer.

Freeboard/Slosh

At the current overflow level there is inadequate freeboard to accommodate the slosh wave, which will result in a slosh load impacting the bottom of the roof. The roof plate and rafters are inadequate for the load from the slosh wave. In order to meet AWWA freeboard requirements, the operating water level would need to be reduced to a height of 28.08-feet, which would accommodate the full height of the slosh wave and prevent damage to the roof during a seismic event. The max water height could be enforced operationally, or the overflow could be physically lowered to the new max water height. Failure to increase the freeboard could result in damage to the roof and potential loss of contents.

3 Limitations and Disclaimer

This evaluation is limited to information obtained during a visual structural assessment and review of provided historic documentation. While PSE made every attempt to be as thorough as possible in the assessment, not every member, connection, or component can be visually assessed or evaluated.

The information presented within this report represents the opinion of a Structural Engineer registered in the State of Oregon. Following any major earthquakes, damage, modifications, upgrades, change of use/operation, and/or other substantial changes, the results herein should be reviewed, and the structures reassessed.

4 Endorsement

This report was prepared by Edward Ling, PE, SE or under his direct supervision while an employee of Peterson Structural Engineers. All work is original and represents the opinion of a Structural Engineer registered in the State of Oregon.

5 Appendix

5.1 Appendix A – North Tank Photos

Photo 1 –Tank Plate

Photo 2 – 500,000 Gallon Standpipe Tank

Photo 3 – Tank Exterior Upper

Photo 4 – Tank Exterior Lower

Photo 5 – Tank Exterior Hatch

Photo 6 – Tank Exterior Coating Deterioration

Photo 7 – Tank Exterior Coating Deterioration

Photo 8 – Tank Exterior Roof

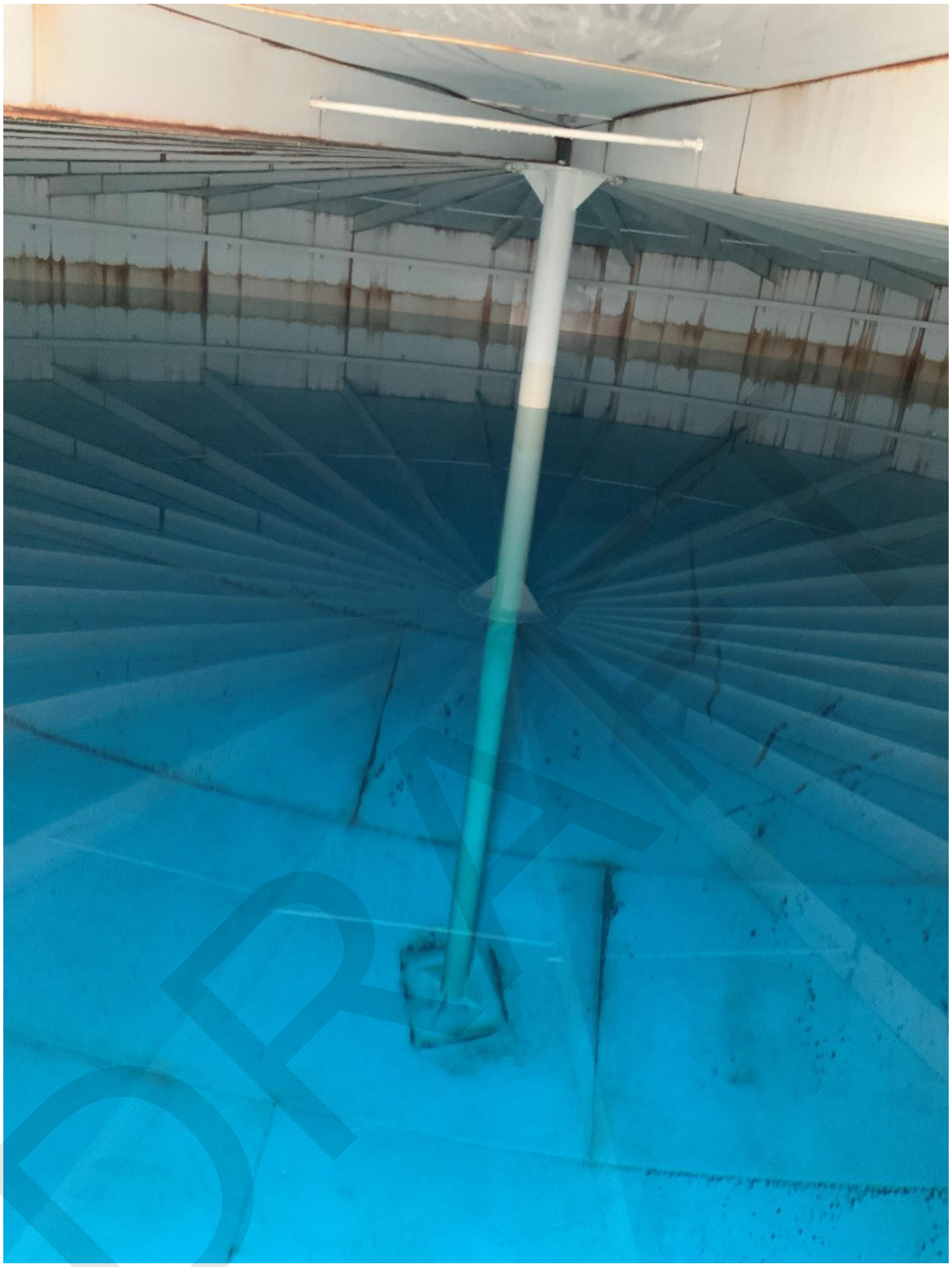

Photo 9 – Tank Interior Center Column

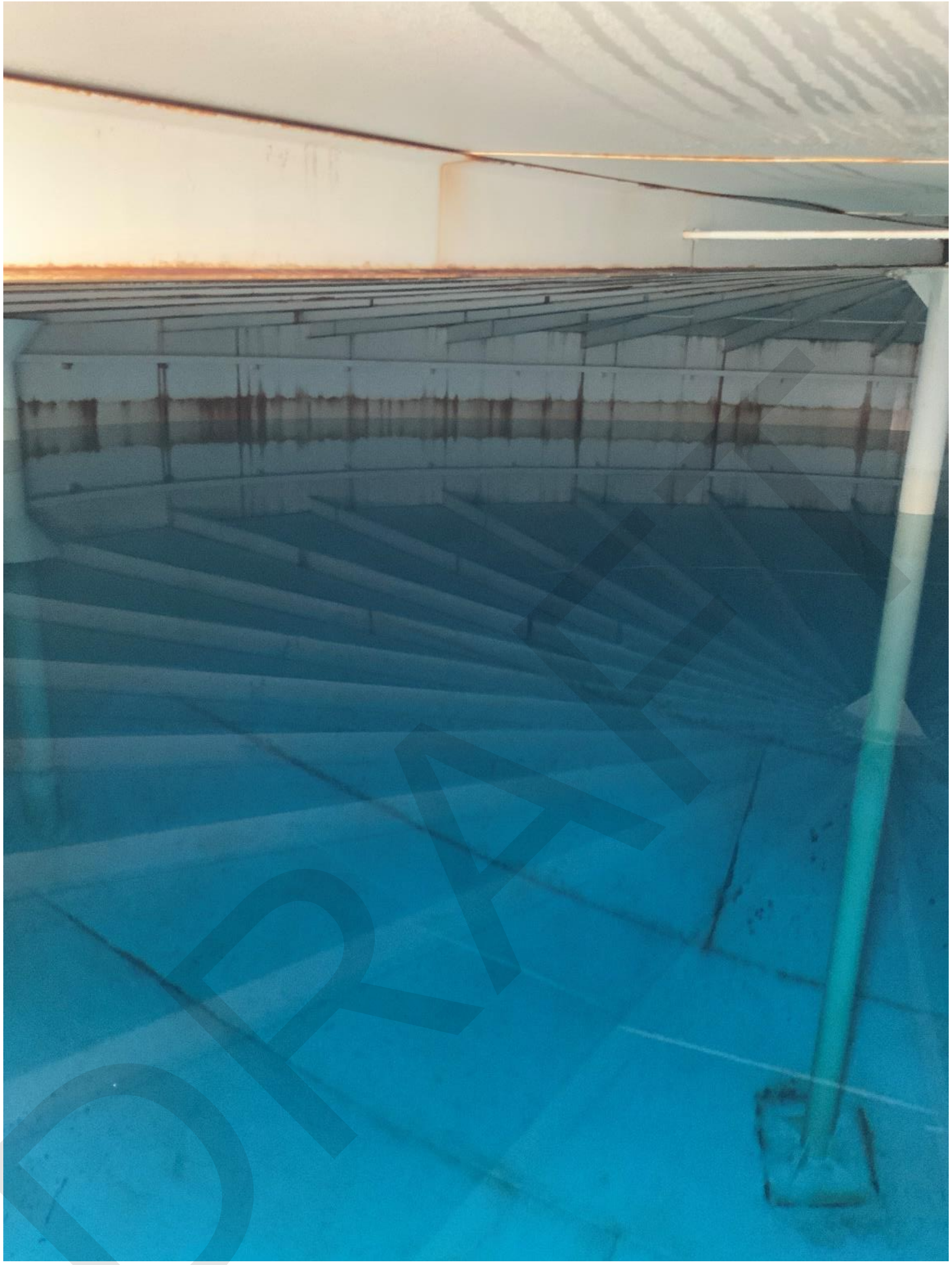

Photo 10 – Tank Interior Roof Framing

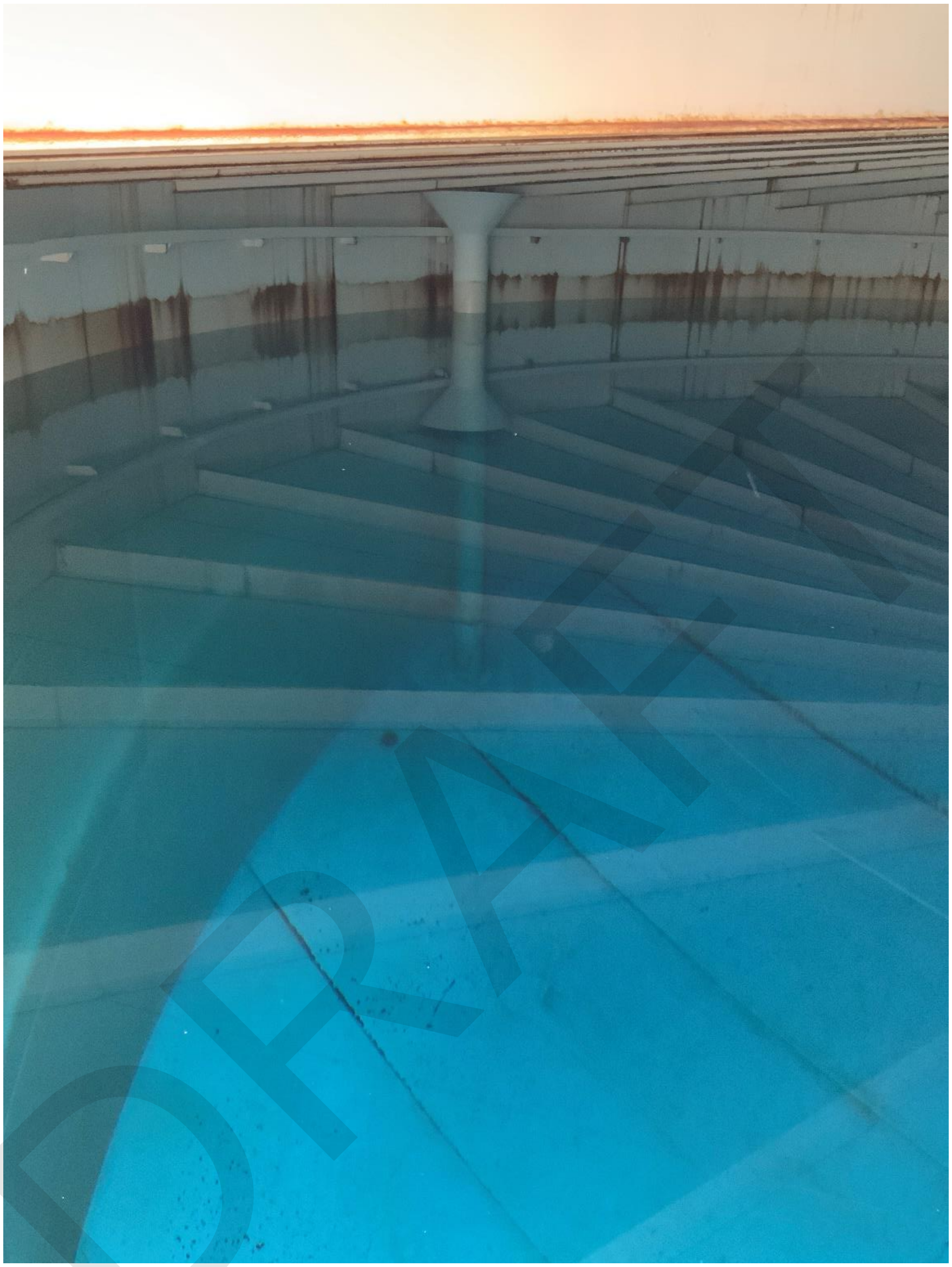

Photo 11 – Tank Interior Overflow Pipe

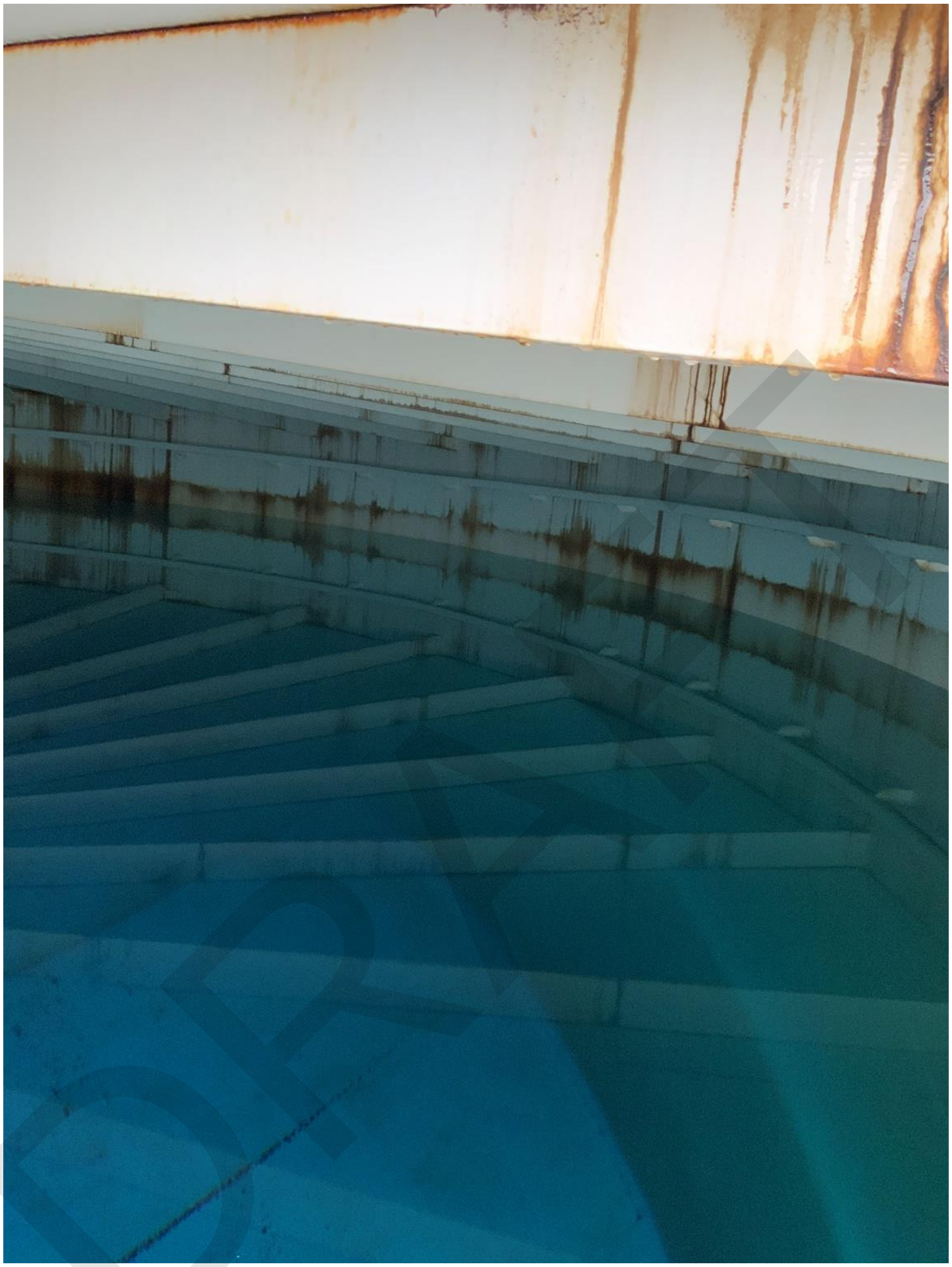

Photo 12 – Tank Interior Roof Framing Corrosion

Photo 13 – Tank Interior Roof Framing Corrosion

5.2 Appendix B – South Tank Photos

Photo 14 –Tank Plate

Photo 15 – 500,000 Gallon Standpipe Tank

Photo 16 – Tank Exterior Upper

Photo 17 – Tank Exterior Lower

Photo 18 – Tank Exterior Hatch

Photo 19 – Tank Exterior Coating Deterioration

Photo 20 – Tank Exterior Coating Deterioration

Photo 21 – Tank Exterior Roof

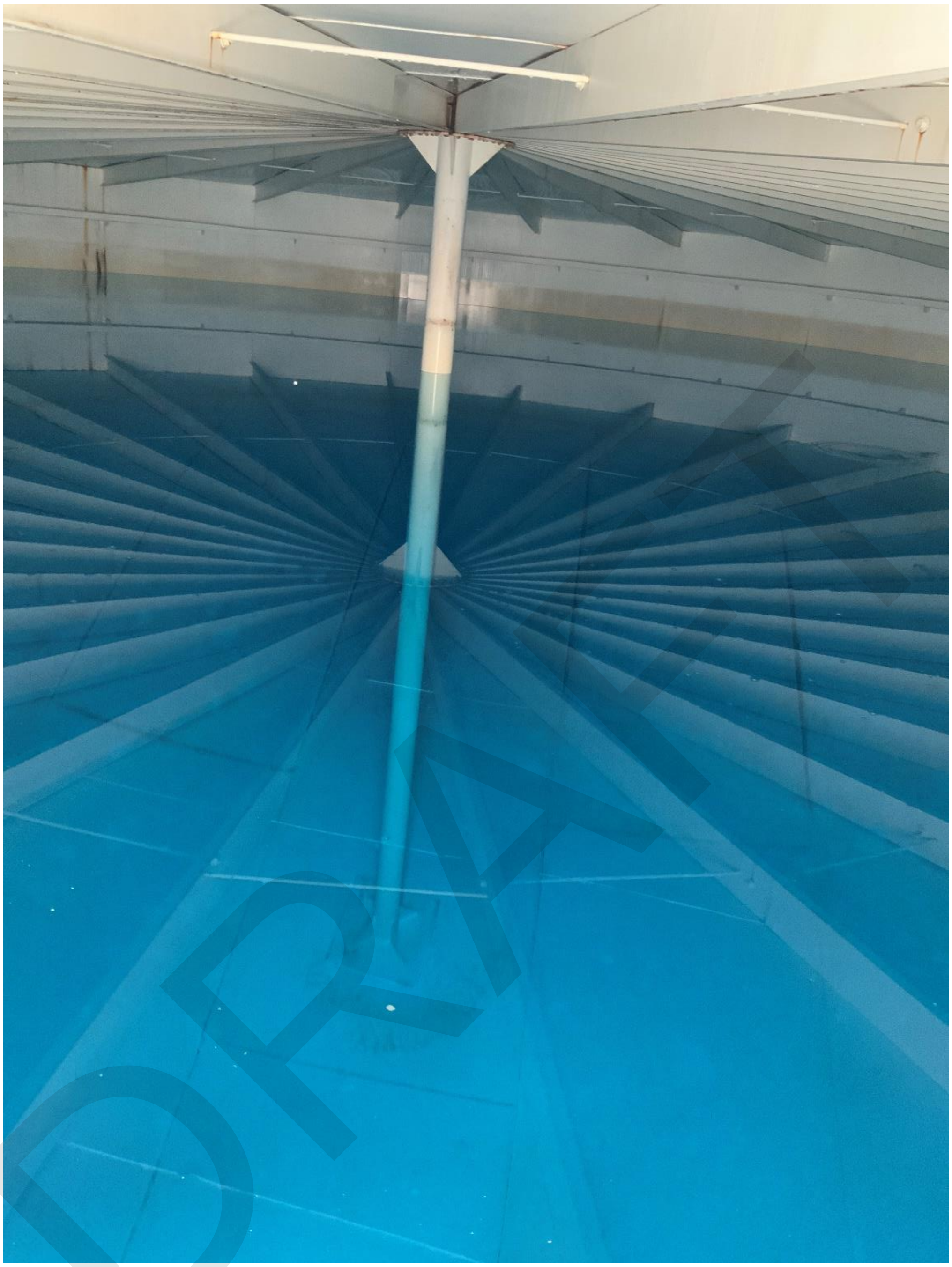

Photo 22 – Tank Interior Center Column

Photo 23 – Tank Interior Roof Framing

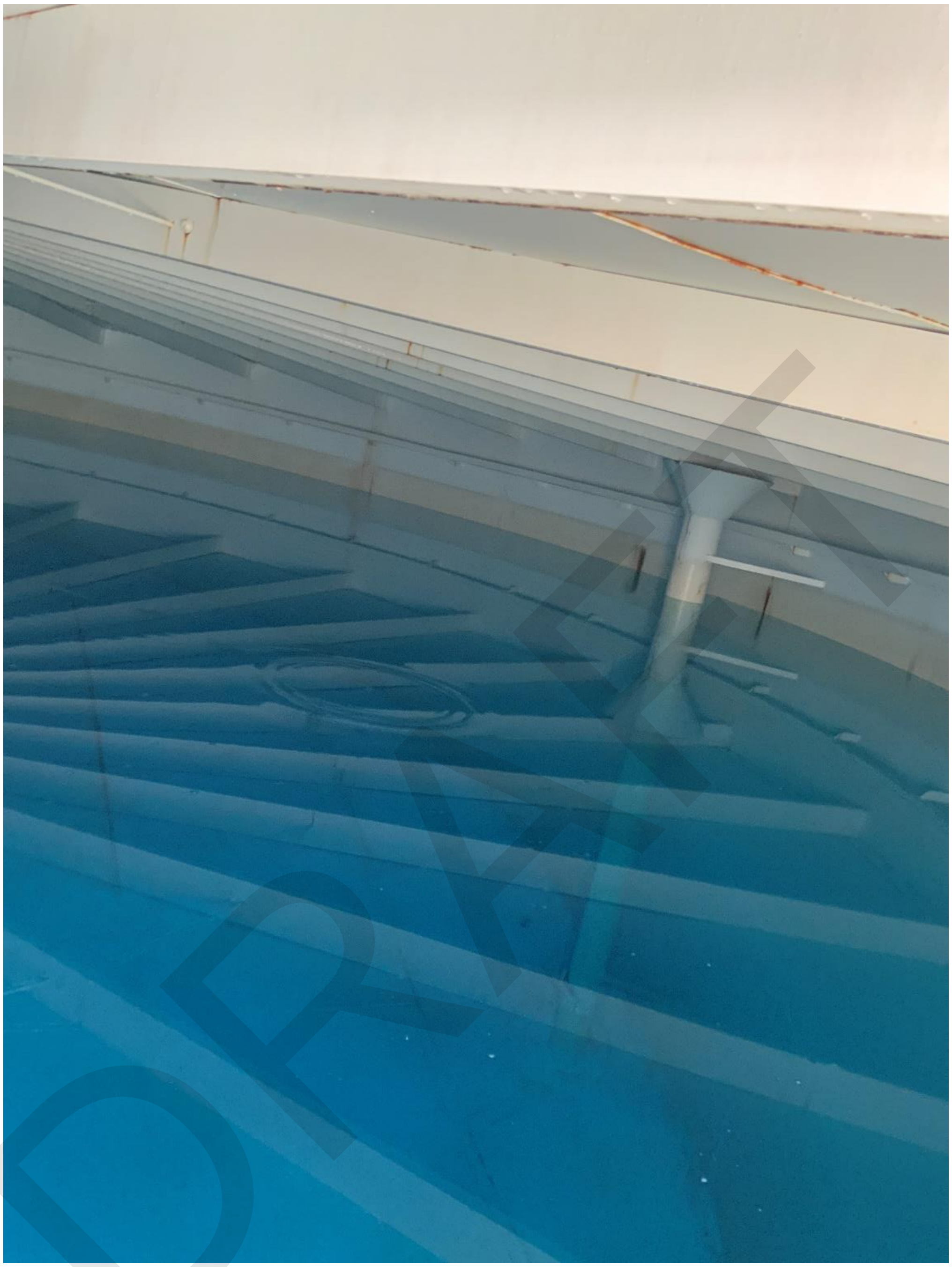

Photo 24 – Tank Interior Overflow Pipe

Photo 25 – Tank Interior Roof Framing Corrosion

Photo 26 – Tank Interior Roof Framing Corrosion