[redacted]

Memorandum

Date: 12 November 2024
(Revised 27 January 2025)

To: [redacted], P.E., Navy Closure Task Force, Red Hill (NCTF-RH)

From: [redacted], P.E., S.E., F.ASCE

cc: CAPT Steven J. Stasick, P.E., PMP, [redacted], R.A., and [redacted] (NCTF-RH)
[redacted], PMP (Jacobs)
[redacted], Ph.D., P.E., [redacted], Ph.D., P.E., S.E., [redacted], Ph.D., P.E., and [redacted], P.E. ([redacted]

Project: 240838 – Red Hill Closure Support, Joint Base Pearl Harbor-Hickam, Honolulu, HI

Subject: Structural Considerations for Decommissioning of Surge Tanks at Underground Pump House, Joint Base Pearl Harbor-Hickam, Hawaii

1. INTRODUCTION

With the decommissioning of the Red Hill tanks, there is no operational use for the four underground surge tanks (hereinafter designated as the 'Surge Tanks') at the Underground Pumphouse (UGPH) at Joint Base Pearl Harbor-Hickam (JBPHH). The Surge Tanks functioned as holding tanks to allow for the rapid offloading of tankers until the fuel could be transferred by the pumps in the UGPH to the Red Hill tanks.

Rather than being used to overcome the elevation head to the Red Hill tanks, the future configuration of the UGPH will utilize pumps to compensate for the loss of gravity flow from the Red Hill tanks when issuing or receiving fuel between ships and the Upper Tank Farm (UTF).

Safely decommissioning the four Surge Tanks is a key objective within the Navy Closure Task Force-Red Hill (NCTF-RH) closure scope. In this memorandum, [redacted] presents the preferred decommissioning strategy, which is to close the Surge Tanks in place without filling them with any material. This recommendation is made in compliance with Hawaii Administrative Rules (HAR) Chapter 11-280.1, Environmental Protection Agency (EPA) Title 40 CFR (Code of Federal Regulations) Part 280.71, and the American Petroleum Institute

(API) codes of practice. The memorandum also provides a justification that ensures the long-term structural integrity of the tanks and future safety at the site, with supporting structural analysis provided in Appendix A and scans of the original drawings from the 1940s presented in Appendix B. Note that in order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.

Since the release of a previous version of this memorandum on 26 November 2024, comments from NCTF-RH, the EPA, and the Hawaii State Department of Health (DOH) have been received. Responses to these comments are provided in a new section of this memorandum, specifically created for this purpose (Section 5).

2. DECOMMISSIONING STRATEGY

The proposed strategy for decommissioning the Surge Tanks will include the following steps:

Step 1 – Pipe Cleaning, Air Gap, and Capping: The first step will be to clean, air gap, and cap the pipes at the Surge Tanks, as required. For the issue/receipt pipes that remain in place, valves will be removed and disposed of, with blind flanges installed near the nozzle between the Surge Tank walls and issue/receipt pipes to introduce an air gap. For example, at Surge Tank No. 2 (Figure 1), the pipes in the Surge Tank gallery tunnel can remain, but the contractor should provide a blind flange near the nozzle between the pipe and the tank wall after the completion of pipe cleaning. The structural stability of the piping system that remains should also be addressed by the contractor.

In addition, the issue/receipt piping will be air gapped within the UGPH as part of a pump and piping upgrade project supporting continued FLC (Fleet Logistics Center) operations. The remnant wall penetrations will be sealed. The result will be a segmentation of the Surge Tunnel from the UGPH, with future removal of the abandoned piping planned.

(b) (3) (A) [redacted]

Figure 1 – Interface between Surge Tank No. 2 and Connecting Pipe

Step 2 – Venting and Cleaning: The second step in Surge Tank decommissioning will be to vent and clean the tanks. To support this, the current ongoing Red Hill tank cleaning contractor will clean the four Surge Tanks to the same level as that being undertaken for the Red Hill tanks, following the same AMPP (Association for Materials Protection and Performance) standards and criteria applied to the Red Hill tanks. The tanks are basic in their configuration, with a single access manway, a bottom sloped to the piping, an issue/receipt line, a FOR nozzle, a stilling well, and an atmospheric vent line. The atmospheric vent is a retrofit of the tank overflow lines encased in concrete to their daylight on the hillside above the FORFAC area.

Step 3 – Closure-in-Place: The final step is to close the Surge Tanks in place. Closure-in-place, without the need to remove the tanks or fill the tanks with any material, is acceptable from the perspective of safety of the site, structural integrity, and environmental risk to drinking water, as described below.

3. COMPLIANCE WITH HAWAII DOH AND EPA CLOSURE REQUIREMENTS WITHOUT THE NEED TO REMOVE OR FILL THE TANKS

The EPA 40 CFR Part 280.71 “Permanent Closure and Changes-in-service” states that permanent closure of underground storage tanks (USTs) can include removal, filling with a harmless, chemically inactive solid, or closing in place in a manner approved by the implementing agency. The Hawaii DOH is the lead implementing agency for the UST program. The DOH allows closure in place without removal or filling in accordance with Hawaii Administrative Rules (HAR) Chapter 11-280.1-71(c)(2), which states that “To permanently close a UST or tank system, owners and operators must…Remove the UST or tank system from the ground, fill the UST or tank system with an inert solid material, or close the tank in place in a manner approved by the department.” HAR Chapter 11-280.1-75 and EPA 40 CFR Part 280.71 additionally list closure codes of practice, with the American Petroleum Institute (API) Recommended Practice (RP) 1604, “Closure of Underground Petroleum Storage Tanks,” being cited as the appropriate code of practice for closure. API 1604, Section 7.5 states that closure in place can be appropriate and that the intent of filling closed in place USTs is “to minimize any surface settling subsequent to the closure of the tank in place.”

4. JUSTIFICATION FOR NOT FILLING THE SURGE TANKS

The Surge Tanks at the UGPH do not need to be removed or filled for long-term structural integrity or to prevent surface settling because the Surge Tanks are field-constructed, robust, and not typical factory-built steel or fiberglass USTs. Steel and fiberglass storage tanks are not designed for significant surcharge loads, and the tank walls could degrade over time due to the surrounding soil and moisture conditions; therefore, they are often required to be filled with an inert solid and structurally stable material to prevent future formation of sinkholes, settlement issues or surface failures. However, the Surge Tanks differ significantly from typical USTs. Each of the four Surge Tanks has interior dimensions of [redacted] ft in diameter and [redacted] ft in height, constructed with a minimum [redacted]-in. thick reinforced concrete shell lined with a [redacted]-in. thick interior steel liner plate. The tanks were excavated from the volcanic tuff rock formation and share a combined integral [redacted]-ft thick heavily reinforced concrete roof which is also supported on the volcanic tuff rock formation (Figures 2 – 11). Structural drawings indicate the roof slabs were poured as a continuous roof slab rather than four circular roofs, as shown in Figures 7 – 9 below. The bottom reinforcement spacing in the central area of the roof slab is very dense at [redacted] in. on center (o.c.) (Figure 9), while the top layer of reinforcement ranges from [redacted] in. to [redacted] in. o.c. (Figure 8). In addition, a significant number of trussed reinforcement units (Figure 10) were also provided in the roof slab. The reason the roof slab is so robust is that it was designed to resist conventional weapons effects at the time of its construction. The typical Surge Tank bottom concrete slab is [redacted] in. thick reinforced with [redacted] in x [redacted] in [redacted] wire mesh (Figure 12). A [redacted]-in. thick steel bottom plate is provided.

The unique construction of the Surge Tanks ensures a level of durability and stability that far exceeds that of typical USTs. Unlike standard USTs, which are surrounded by soil and susceptible to risks such as deterioration, subsidence, and surface depression, the Surge Tanks at the UGPH are safeguarded against these issues by their robust design and construction. The exceptional stability provided by the combination of reinforced concrete, steel lining, and volcanic tuff rock formation excavation eliminates the long-term degradation risks typically associated with USTs.

Figure 2 – Excavation from the Volcanic Tuff Rock Formation During Construction

Figure 3 – Top of Completed [redacted] ft Thick Concrete Roof Slab over Four Surge Tanks

[redacted]

Figure 4a – Surge Tank Excavation Plan and Section Along Center Line

(Note: This figure shows general information only on the Surge Tank plan and elevation. Dimensions are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in x 36 in., minimum.)

[redacted]

Figure 4b – Excavation Section View for Each Surge Tank

(Note: This figure shows general information only on the Surge Tank sections. Dimensions are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

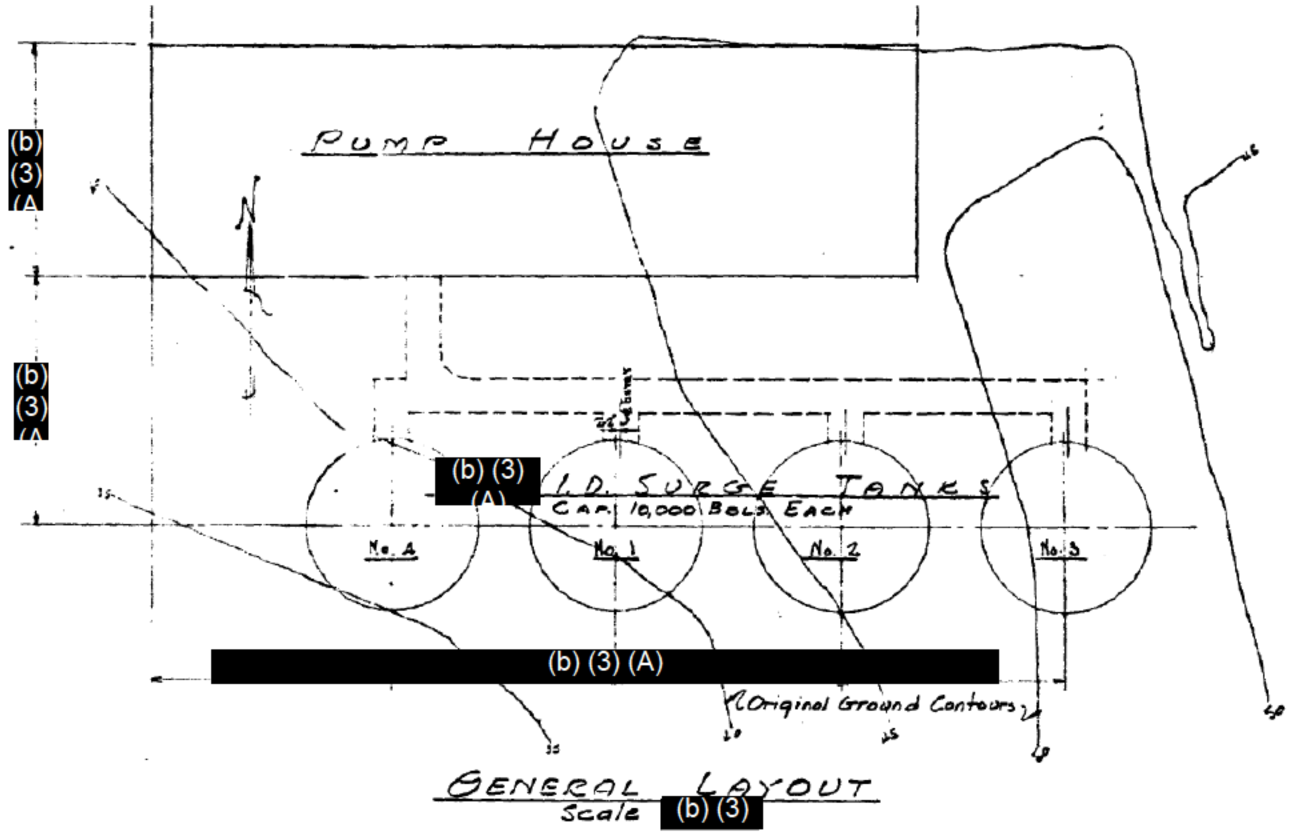

Figure 5 – Surge Tank General Layout

(Note: This figure shows general information only on the Surge Tank layout. Dimensions are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 6 – Surge Tank Dimensions and Original Design Assumptions

(Note: This figure shows general information only on the Surge Tank dimensions and design assumptions. Dimensions are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be fully legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 7 – Surge Tank Plan Layout of Combined Roof Slab

(Note: This figure shows general information only on the Surge Tank combined roof slab layout. Dimensions are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 8 – Typical Top Layer of Reinforcement Details of [redacted] ft Thick Concrete Roof
(Note: This figure shows general information only on a typical top layer of reinforcement in the roof of the Surge Tanks. Dimensions and details are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 9 – Typical Bottom Layer of Reinforcement Details of [redacted]t Thick Concrete Roof
(Note: This figure shows general information only on a typical bottom layer of reinforcement in the roof of the Surge Tanks. Dimensions and details are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 10 – Trussed Reinforcement Unit Details of [redacted]-ft Thick Concrete Roof

(Note: This figure shows general information only on typical trussed reinforcement units in the roof of the Surge Tanks. Dimensions and details are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 11 – Typical [redacted] in. Thick Wall Section Reinforcement Details

(Note: This figure shows general information only on a typical wall reinforcement in the Surge Tanks. Dimensions and details are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

[redacted]

Figure 12 – Typical [redacted] in. Thick Surge Tank Bottom Slab Reinforcement Details

(Note: This figure shows general information only on the typical bottom slab reinforcement in the Surge Tanks. Dimensions and details are not intended to be read from this figure. For a full-size version of the figure, see Appendix B. In order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.)

As noted in the drawings, the combined [redacted]-ft thick reinforced concrete roof slab is partially supported on volcanic tuff rock formation (Figures 2 – 9). This rock formation is structurally capable of supporting the reinforced concrete roof even in the absence of the [redacted]-in. thick tank walls. Although there is no apparent geotechnical data available for the rock formation, a photograph from the time of construction, as shown in Figure 2, demonstrates a stable excavation without the need for shoring. Nevertheless, a simplified structural analysis has been conducted for the closure-in-place of the Surge Tanks to further demonstrate that the tanks remain structurally sound for the proposed long-term closure approach. Calculations are provided in Appendix A and are based on very conservative assumptions, namely:

- The primary support for the concrete roof is provided by the [redacted]-in. thick concrete tank walls rather than the underlying volcanic tuff rock formation.

- The excavated volcanic tuff surrounding the Surge Tanks exerts lateral pressure on the exterior of the tank walls, similar to the behavior of regular soil, as opposed to excavated stable rock.

- The lateral pressure corresponding to the maximum pressure for any soil type listed in Table 1610.1 of IBC 2021 has been assumed (whereas, in reality, the rock formation around the tank walls would exert a much smaller lateral pressure, if any at all).

- A characteristic compressive strength of 3,000 psi for concrete and a characteristic yield strength of 40,000 psi for steel reinforcement have been assumed. These values are typical of those at the time of construction. The concrete compressive strength is likely higher than the value assumed.

Gravity loads (self-weight of the tank wall and roof slab, weight of the soil above the tank, uniform live load surcharge at the surface, and vehicular loading at the surface) and lateral earth pressure loads on the tank wall were considered in the analysis, as were buoyancy and hydrostatic pressure on the tank walls, representing an unlikely scenario where the water table rises above the tanks. Future earthquake loading was not explicitly considered in the analysis in Appendix A. However, earthquake loads were addressed for the main Red Hill underground storage tanks (see Simpson Gumpertz & Heger, 'Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks,' which has been provided as Enclosure 1 to Supplement 2 of the Department of the Navy's Tank Closure Plan at the Red Hill Bulk Fuel Storage Facility). In that report, the adequacy of the Red Hill tanks for earthquake loading was demonstrated, and similar good performance for the Surge Tanks would be expected. Please also see the response to Comment 7 in Section 5. An allowance for potential future deterioration has also been made in order to address the long-term performance of the Surge Tanks.

The analysis results indicate that the structural demands on the Surge Tanks, when closed in place, remain within their capacity. Therefore, the Surge Tanks can be considered structurally sound and safe for in-place closure under the loads considered. See Appendix A for analysis details.

5. RESPONSE TO COMMENTS

Since the release of an earlier version of this memorandum on 26 November 2024, comments from NCTF-RH, the EPA, and the Hawaii DOH have been received. Responses to these comments are provided in this section. Comments are in no particular order.

1. **Calculations. Please provide a complete package of calculations that support the structural findings.** Calculations are provided in Appendix A.

2. **Load Factors. Provide the appropriate load factoring for the calculations and corresponding strength reduction factors for the different loads and capacity calculations for the demand-to-capacity ratios.** The calculations that were provided have always used strength reduction factors. However, since this is not a new design but rather an evaluation of existing tanks, unfactored loads were originally used as the uncertainty in the loading is minimal. For example, the densities of concrete and soil contributing to the dead load on the tanks are reasonably well established in codes. Similarly, the dimensions of the structural elements, such as the tank wall and roof slab thicknesses, are documented in the construction drawings with no reason for variation. However, the calculations in Appendix A now include load factors. Please see Appendix A for updated calculations.

3. **Future Conditions. Provide a statement about the current observed conditions and the estimated future conditions.** Access to the interior (or exterior) of the Surge Tanks, apart from what can be observed in the Surge Tank gallery, was not possible. However, concrete durability was assessed in the earlier report on the Red Hill tanks (see Simpson Gumpertz & Heger, Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks,' which has been provided as Enclosure 1 to Supplement 2 of the Department of the Navy's Tank Closure Plan at the Red Hill Bulk Fuel Storage Facility). A similar evaluation of likely deterioration mechanisms is presented in Appendix A and the reader is referred to the earlier Red Hill report for an in-depth discussion of concrete deterioration mechanisms, not all of which are applicable to the Surge Tanks.

   To summarize the material in Appendix A, it is estimated that a minimum of 250 years would be required for carbonation to reach the depth of the embedded reinforcing steel in the Surge Tanks, i.e., approximately 165 years from now since the Surge Tanks are already approximately 85 years old. Once the carbonation reaches the reinforcement depth and if the concrete has already cracked, it may take approximately an additional 30 to 40 years under moderate carbonation-induced corrosion rates (i.e., an average of 35 years) for a 10% loss in cross-sectional area of the reinforcement due to carbonation induced corrosion. The total being 165 years plus 35 years, or a total of approximately 200 years from now.

   Although a 10% loss of the cross-sectional area in the reinforcement corresponds to the condition of the roof slab approximately 200 years from now, such loss would not make the support of the objects on top of the surge tanks untenable. However, it is not recommended that permanent structures be built on top of the Surge Tanks without further in-depth analysis considering specific building loads, since the Surge Tanks were not originally designed in this manner. Section 16 of Appendix A presents calculations that recommend that any loads placed above the [redacted] ft of soil presently on top of the Surge Tanks be limited to no more than an additional 200 psf (two hundred pounds per square foot) without performing additional detailed structural analysis.

Chloride-related corrosion is not considered to be a high risk as the likelihood of frequent water infiltration through the tank is limited based on the groundwater depth and chloride concentrations at nearby wells. Please see Appendix A for additional details. However, some infrequent groundwater infiltration was observed in a Fitness-for-Service report dated 2004. Accordingly, it is recommended to consider keeping valves on the FOR nozzles to allow for any future draining, if needed.

Similarly, significant deterioration of the steel liner in the Surge Tanks is not anticipated within the next 50 years, based on corrosion rates likely to occur within the sealed and abandoned Surge Tanks. The estimated corrosion rates discussed in Section 6 of Appendix A are consistent with the range of observed corrosion rates in a condition assessment report dated 2020.

Appendix A also contains calculations that include an allowance for future corrosion of reinforcement and liner plate when determining the capacity of the Surge Tanks.

4. **Maintenance. Regarding the observed current and future conditions for the tanks: Are there recommendations for expected maintenance and are these maintenance actions taken under consideration?** As is the case for the Red Hill tanks (see Simpson Gumpertz & Heger, 'Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks,' which has been provided as Enclosure 1 to Supplement 2 of the Department of the Navy's Tank Closure Plan at the Red Hill Bulk Fuel Storage Facility), it is recommended that the Navy perform a visual review of the Surge Tanks should the site experience a future major earthquake (one with a peak ground acceleration at the site on the order of 0.2 g) to identify sustained damage. A drone or other means of safe access (such as providing internal tank scaffolding) can facilitate this initial visual assessment. Some means of ventilating the tanks before any inspection and maintenance is recommended. There is no need for an extensive inspection and maintenance program for the tanks.

5. **December 2022 Jacobs Report Versus December 2024 [redacted] Memorandum. There is a change in direction from the Jacobs report provided in December 2022 to the present report.** The prior recommendation to fill the Surge Tanks in the earlier 2022 report was provided by others, whereas in the present memorandum, [redacted] has evaluated alternatives that meet the requirements for long-term closure and are both code-compliant from a structural perspective and cost-effective.

6. **Readability of Images. Address issues with the readability of the images in the report. Consider enlarging the images. In addition, the figures on pp. 4 – 9 are hard to read. Request clearer, legible copies of these figures.** The figures provided in this memorandum show general information only. Dimensions and details are not intended to be read from these figures. For a full-size version of the figures, see Appendix B. However, in order to be legible, drawings in Appendix B may need to be printed at 24 in. x 36 in., minimum.

7. **Earthquake Response. Please include what ASCE 7 version calculations are based on and the basis for the earthquake and acceleration considerations.** Earthquake loads have not explicitly been considered in the analysis of the Surge Tanks. However, earthquake loads have been addressed for the main Red Hill underground storage tanks (see Simpson Gumpertz & Heger, 'Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks,' which has been provided as Enclosure 1 to Supplement 2 of the Department of the Navy's Tank Closure Plan at the Red Hill Bulk Fuel Storage Facility). In that report, the adequacy of the Red Hill tanks for earthquake loading was demonstrated. Similarly good performance for the Surge Tanks can be expected, considering the similar seismic demands on a shorter structure with a broader aspect ratio.

   The Red Hill tanks were previously evaluated for the following two levels of earthquakes:

   - Code-defined Design Basis Earthquake (DBE).
   - Conservative earthquake with 10,000-year mean recurrence interval (MRI).

   The DBE is also referred to as the Design Earthquake (DE) in the American Society of Civil Engineers (ASCE) Standard, ASCE 7-22, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures." It is defined as two-thirds of the Maximum Considered Earthquake (MCE), which is associated with a 2,475-year MRI.

   For the DBE, both Risk Category II (with importance factor, I_e = 1.0) and Risk Category III (with I_e = 1.25) were considered. Risk categories are classifications within the building code that depend on the risk associated with the unacceptable performance of the structure. ASCE 7-22 Risk Categories are defined in Table 1.5-1 of ASCE 7-22. Note that Risk Category I (the lowest risk category) applies to buildings and other structures that represent a low risk to human life. Since the Red Hill tanks and the Surge Tanks will be permanently closed in place and will not be occupied structures, Risk Category I could also apply.

   The 10,000-year MRI event is an extreme earthquake event that is typically considered only for critical infrastructure such as nuclear power plants.

The zero period acceleration (ZPA) response spectrum ordinates (also known as the peak ground acceleration (PGA)) for the DBE, MCE, and 10,000-year MRI events for the Red Hill site are 0.13g, 0.20g, and 0.42g, respectively.

8. **Degradation over Time. The degradation of the tanks over the next decades to centuries is unknown and should be evaluated.** Please see the response to Comment 3.

9. **Risk of Flooding and Buoyancy of Empty Tanks. The drainage conditions of the site are unknown, including potential damage that may occur in the event of maximum rainfall and flooding events. The tech memo should evaluate the risk of flooding, the buoyancy of the tanks, and the potential to "float" the concrete foundation. Similarly, the risk of collapse due to hydrostatic pressure on an empty tank should also be addressed.** Since the tanks sit approximately at the level of the UGPH, floating of the tanks is not considered to be a significant risk since if submerging of the tanks were to occur, much of JBPHH would need to be underwater. From monitoring wells located near Hotel Pier in Joint Base Pearl Harbor-Hickam (https://health.hawaii.gov/ust/ files/2021/11/DFT-Hotel-Pier-Plume-Delineation.pdf), the groundwater table is approximately at 0.3 ft to 0.9 ft above sea level. The bottom of the Surge Tanks is at an elevation of [redacted] ft per the drawings (Drawing # 3UF-S9, also see Figure 6 and Appendix B of this memo). Therefore, the bottom floor slab of each Surge Tank is approximately [redacted] ft above sea level. Consequently, buoyancy is not considered to be a credible risk. However, calculations in Appendix A consider both buoyancy of the tanks and hydrostatic pressure on the tank walls.

10. **Explosive Gases. Vapors from past spills could potentially diffuse back into the empty space of the tank volume and accumulate. Although this scenario is unlikely, the potential for explosive gases entering the tanks is possible and should be evaluated.** Similar to Comment 9, this is not regarded as a credible scenario. Upon the completion of the decommissioning of the Surge Tanks, diffusion through a [redacted]-ft thick reinforced concrete roof or a [redacted]-in. thick reinforced concrete wall that is lined with a [redacted]-in. thick steel plate or [redacted]-in. thick reinforced concrete floor topped with [redacted]-in. thick bottom plate is remote. The steel liner and steel base plate serve as an impermeable barrier that prevents the intrusion of external vapors into the tank volume from the sides and bottom of the tanks. Steel is inherently non-porous and creates a continuous seal against vapor diffusion. The combination of the impermeable [redacted]-in. thick steel liner and the [redacted]-in. thick reinforced concrete walls, as well as [redacted]-in. thick bottom plate and [redacted]-in. thick reinforced concrete floor slab create a dual-layer barrier, effectively isolating the internal tank space from the surrounding environment. Over the life of the Surge Tanks, this unique design has demonstrated its

effectiveness in containing liquids and vapors with no history of compromise. During the decommissioning process, the surge tanks will be thoroughly cleaned and inspected to confirm the absence of any residual liquid or vapor, consistent with the approved cleaning methodology for closure, further mitigating any potential for vapor accumulation.

Modern environmental concrete structures are typically designed using ACI 350 series standards to ensure leak tightness against internal and external load cases. These standards require the use of special concrete mix and limits on rebar stresses and liners so that the stored material or gases do not leak out of the tank. ACI 350.4R-12, Section 4.6 indicates that steel liners can be used as a means of gas-proofing where required. The surge tanks were constructed in the 1940s, but the steel liners and steel bottom plate of Surge Tanks can serve as means of gas-proofing as indicated in ACI 350.4R-12.

Considering the design and physical characteristics of the Surge Tanks, particularly the gas-proof properties of the steel liners and bottom plates, the risk of explosive gas accumulation can be concluded to be minimal.

Notwithstanding this, should the Surge Tanks be entered sometime in the future and well after closure-in-place, they could be treated as confined spaces, and proper safety protocols should be followed. Note that these tanks have been previously degassed to facilitate clean, inspect, and repair (CIR) services during their operation. That degassing process is considered the worst-case gaseous condition within the tanks, given that fuel residual covers the inside of the tanks in that state. However, prior to closure, the Surge Tanks will be cleaned similarly to the Red Hill tanks, i.e., they will be made safe for human entrance to facilitate the cleaning process. Therefore, when closed, the gaseous state will be such that it is not hazardous to personnel. Nothing will be stored in the tanks to facilitate an ignition source, i.e., the space will be intrinsically safe without an ignition source, and therefore, in the future, given the very remote scenario of gases accumulating in the tanks, no explosive event is expected.

11. **Monitoring and Inspection. How will the Navy ensure the ongoing integrity of the tanks?** Please see the response to Comment 4.

12. **Future Land Use. Documentation guidelines should be established so that the presence of the tanks is known by all parties and cannot be lost in the recordkeeping process after decades of time and/or transfer of ownership.** The calculations in Appendix A establish structural requirements for loading from future land use above the tanks. Please see Appendix A. It is recommended that future loads placed above the [redacted] of soil presently on top of the Surge Tanks be limited to no more than an additional 200 psf (two hundred pounds per square foot) without performing

additional detailed structural analysis. For comparison purposes, one foot of soil typically weighs approximately 120 psf, a reasonably heavy modular building, approximately 10 ft tall, weighs approximately 70 psf, and the standard live loading in an office building is 50 psf.

13. **Future Use of Attached Piping. P. 2 recommends the pipes be cleaned, air gapped, and capped. However, we understand that the fuel piping from the underground pump house to the Surge Tanks that will no longer be used will be removed. This should be included in the memo, as structural considerations may be required for the remaining active fuel piping.** Per NCTF-RH, the direction by NAVFAC is to remove piping that is no longer in use. The removal of the piping is captured in Section 2, Step 1, with revised wording, which indicates that future removal of the abandoned piping is planned. In addition, per NCTF-RH, the remaining piping that supports the Navy's mission will remain in place and any modifications to the pipe support structures will be addressed on an as-needed basis.

14. **Surge Tank Reinforcement. In the Surge Tank bottoms and roofs, what size is the rebar? How many rows of rebar are there?** A description of the pertinent Surge Tank roof, wall, and bottom slab reinforcement is provided in Section 4. See Appendix B for drawings.

15. **Relationship between [redacted] and Jacobs. We recommend clarifying the relationship between [redacted] and Jacobs and the different scopes of information used to support the recommendations made in this memo versus the 12/20/22 Report, Red Hill Tank Closure Plan Analysis of Alternatives & Concept Design to Close in Place.** The Jacobs/B&V Joint Venture (Jacobs) Engineering was awarded this project and is acting as the Project Manager. [redacted] is subcontracted to Jacobs as a specialty structural engineering consultant to provide structural investigation, analysis, reporting, and design services. This memorandum and future designs are to be reviewed and approved by Jacobs. See also the response to Comment 5 on the different conclusions of the Jacobs December 2022 report and the present memorandum.

6. CONCLUSIONS

The recommended decommissioning strategy for the Surge Tanks at the UGPH is to close them in place without the need to remove them or the addition of any fill material. This approach is supported by the tanks' robust construction and inherent structural integrity, as validated by the engineering calculations and analysis captured in this memo, demonstrating both long-term stability and safety. The strategy is fully compliant with Hawaii DOH HAR Chapter 11-280.1, EPA regulations (EPA 40 CFR Part 280.71), and API standards (API RP 1604), given the sound condition of the tanks and the absence of environmental risks associated with leaving them empty.

It is recommended that the Surge Tanks be closed in place, contingent upon final approval from pertinent regulatory authorities and stakeholders. There is no need for an extensive inspection and maintenance program for the tanks. However, it is recommended to consider keeping valves on the FOR nozzles to allow for any future draining if needed.

There are no recommendations for future maintenance of the tanks other than to possibly enter them for inspection following major seismic events. As is the case for the Red Hill tanks, it is recommended that the Navy perform a visual review of the Surge Tanks should the site experience a future major earthquake (one with a peak ground acceleration at the site on the order of 0.2 g) to identify sustained damage. A drone or other means of safe access (such as providing internal tank scaffolding) can facilitate this initial visual assessment. Some means of ventilating the tanks before any inspection and maintenance is recommended.

Regarding future land use above the Surge Tanks, it is recommended that any loads placed above the [redacted] ft of soil that is presently on top of the Surge Tanks be limited to no more than an additional 200 psf (two hundred pounds per square foot) without performing additional detailed structural analysis. For comparison purposes, one foot of soil typically weighs approximately 120 psf, a reasonably heavy modular building, approximately 10 ft tall, weighs approximately 70 psf, and the standard live loading in an office building is 50 psf.

APPENDIX A
Calculations for Closure-in-Place of Surge Tanks

3	Revised to Include Information from a 2020 Condition Assessment Report	01/27/2024	[redacted]	[redacted]	[redacted]
2	Revised to Address Client, EPA, and DOH Comments	12/13/2024	[redacted]	[redacted]	[redacted]
1	Revised to Address Client Comments	11/26/2024	[redacted]	[redacted]	[redacted]
0	Original Issue	11/12/2024	[redacted]	[redacted]	[redacted]
Rev.	**Description**	**Date**	**Prepared by**	**Checked by**	**Approved by**

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CLIENT Navy Closure Task Force, Red Hill [NCTF-RH]

BY [redacted]

SUBJECT Calculations for Closure in-Place of Surge Tanks

CHECKED BY [redacted]

1. Surge Tank details

Tank radius (in-to-in)	radius_tank := [redacted] ft
Tank wall height	tank_wall_ht := [redacted] ft
Tank wall thickness	tank_wall_thickness := [redacted] ft
Roof slab thickness	thickness_of_roof_slab := [redacted] ft
Depth of soil surcharge above tank	soil_surcharge_ht := [redacted] ft
Bottom slab thickness	thickness_of_bot_slab := [redacted] in
Uniform live load at surface	uni_live_load := [redacted] psf
Vehicular wheel loads	veh_load := [redacted] kip
Tank wall foundation width	width_tank_wall_found := [redacted] ft
Tank wall foundation thickness	tank_wall_found_thk := [redacted] ft

Note that two Surge Tank roof slabs have [redacted] ft radius and the other two Surge Tank roof slabs have [redacted] ft radius.

Average radius of roof slab avg_rad_of_roof_slab := [redacted] ft

The tanks were constructed by excavating the soil/rock and constructed of reinforced concrete. The tank wall is [redacted] in. thick with a [redacted] in. thick shell plate on the interior side. The [redacted] ft. thick roof slab is shared by the four Surge Tanks.

[redacted]

[redacted]

2. Codes, standards, and references

ASCE 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures

ACI 318-19: Building Code Requirements for Structural Concrete

AISC 360-16: Specification for Structural Steel Buildings

ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings

IBC 2021: International Building Code

Terzaghi, K., Peck, R. B., & Mesri, G. (1996). Soil Mechanics in Engineering Practice. (3 ed.) Wiley

Sánchez, M. J, Saura, G. P, Torres, M. J., Chinchón-Payá, S., & Rebolledo, R. N. Variation of Corrosion Rate, Vcorr, during the Carbonation-Induced Corrosion Propagation Period in Reinforced Concrete Elements. Materials (Basel). 2023 Dec 24;17(1):101. doi: 10.3390/ma17010101. PMID: 38203955; PMCID: PMC10779882

Young, W. C., Budynas, R. G., & Roark, R. J. (2002). Roark's formulas for stress and strain. 7th ed. / New York ; London, McGraw-Hill

Neville, A.M., 2012, Properties of Concrete, Trans-Atlantic Publications, Inc., 5th Edition

Cramer, S.D. and Covino Jr., B.S. (2006) ASM Handbook Vol. 13C Corrosion: Environments and Industries. ASM International, Ohio https://doi.org/10.31399/asm.hb.v13c.9781627081849

3. Materials

Detailed material properties were not shown in the available drawings. Based on the time of construction (1940s), we used a nominal compressive strength at 28 days of 3,000 psi for concrete, ASTM A15 Gr. 40 for reinforcing steel (minimum yield stress of 40 ksi), and ASTM A9 for liner plate (minimum yield stress of 33 ksi). These material properties fall within the suggested lower-bound material properties to use based on the time of construction of the facility per Table 9-1, Table 10-2 and Table 10-3 of ASCE 41-17.

Concrete density $\text{conc\_density} := 150\text{pcf}$ (normal weight concrete)

Soil density $\text{soil\_density} := 135\text{pcf}$ (assumed dense gravel, highest density of different soil groups)

Concrete characteristic compressive strength

$fc := 3\text{ksi}$

(Section 19.2.2.1 of ACI 318-19)

Modulus of elasticity of concrete $E := 57000 \cdot \left(3000 \frac{\text{psi}}{\text{psi}}\right)^{0.5} \cdot 1\text{psi} = 3.122 \times 10^{6} \cdot \text{psi}$

Reinforcement yield strength $fy := 40\text{ksi}$

Concrete poisson's ratio $mu := 0.2$

Steel liner yield strength $\text{fy\_liner} := 33\text{ksi}$

4. Demands on the roof slab

Self-weight of the roof slab (assuming the roof slab spans between the walls and conservatively neglecting the portion of the slab supported by the soil/rock outside of the tank wall)

$$\text{weight\_of\_roof\_slab} := \frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \cdot \text{thickness\_of\_roof\_slab} \cdot \text{conc\_density} =$$ [redacted] kip

Weight of the surcharge soil over roof slab

$$\text{weight\_of\_soil} := \text{soil\_density} \cdot \text{soil\_surcharge\_ht} \cdot \left[\frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \right] =$$ [redacted] kip

Weight of the 100 psf uniform load at the surface

$$\text{uniform\_live\_load} := \text{uni\_live\_load} \cdot \left[\frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \right] =$$ [redacted] $\cdot$ kip

Assume a total of 72 kips (2*32 kips + 8 kips, representing an HL93 truck) of vehicular load acting at the surface which becomes a uniform load distributed over a larger area 6 ft below the surface

$$\text{vehicular\_load} := 72\text{kip}$$

Factored pressure at the top of the roof slab for 1.2D+1.6L (load combination per Section 2.3.1 of ASCE 7-22), where D=dead load and L=live load

$$\text{pr\_roof\_slab\_1} := \frac{(1.2 \cdot \text{weight\_of\_roof\_slab} + 1.2 \cdot \text{weight\_of\_soil} + 1.6\text{uniform\_live\_load} + 1.6\text{vehicular\_load})}{\left[\frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \right]} =$$ [redacted] psf

Factored pressure at the top of the roof slab for 1.4D (load combination per Section 2.3.1 of ASCE 7-22)

$$\text{pr\_roof\_slab\_2} := \frac{(1.4 \cdot \text{weight\_of\_roof\_slab} + 1.4 \cdot \text{weight\_of\_soil})}{\left[\frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \right]} =$$ [redacted] psf

$$\text{pr\_roof\_slab} := \max(\text{pr\_roof\_slab\_1}, \text{pr\_roof\_slab\_2}) =$$ [redacted] $\cdot$ psf

We conservatively assume the roof slab is simply supported at tank walls.

(b) (4) [redacted]

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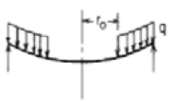
We use Roark's formula to calculate the stresses in the roof slab using the loads calculated above

TABLE 11.2 Formulas for flat circular plates of constant thickness

NOTATION: W = total applied load (force); w = unit line load (force per unit of circumferential length); q = load per unit area; M_o = unit applied line moment loading (force-length per unit of circumferential length); θ_o = externally applied change in radial slope (radians); y_o = externally applied radial step in the vertical deflection (length); y = vertical deflection of plate (length); θ = radial slope of plate; M_r = unit radial bending moment; M_t = unit tangential bending moment; Q = unit shear force (force per unit of circumferential length); E = modulus of elasticity (force per unit area); ν = Poisson's ratio; γ = temperature coefficient of expansion (unit strain per degree); a = outer radius; b = inner radius for annular plate; t = plate thickness; r = radial location of quantity being evaluated; r_o = radial location of unit line loading or start of a distributed load. F_1 to F_9 and G_1 to G_{19} are the several functions of the radial location r. C_1 to C_9 are plate constants dependent upon the ratio a/b. L_1 to L_{19} are loading constants dependent upon the ratio a/r_o. When used as subscripts, r and t refer to radial and tangential directions, respectively. When used as subscripts, a, b, and o refer to an evaluation of the quantity subscripted at the outer edge, inner edge, and the position of the loading or start of distributed loading, respectively. When used as a subscript, c refers to an evaluation of the quantity subscripted at the center of the plate.

Positive signs are associated with the several quantities in the following manner: Deflections y and y_o are positive upward; slopes θ and θ_o are positive when the deflection y increases positively as r increases; moments M_r, M_t, and M_o are positive when creating compression on the top surface; and the shear force Q is positive when acting upward on the inner edge of a given annular section

Bending stresses can be found from the moments M_r and M_t by the expression $\sigma = 6M/t^2$. The plate constant $D = Et^3/12(1-\nu^2)$. The singularity function brackets $\langle\ \rangle$ indicate that the expression contained within the brackets must be equated to zero unless $r > r_o$, after which they are treated as any other brackets. Note that Q_b, Q_a, M_{rb}, and M_{ra} are reactions, not loads. They exist only when necessary edge restraints are provided.

Case no., loading, load terms	Edge restraint	Boundary values	Special cases
10. Uniformly distributed pressure from r_o to a  $LT_y = \frac{-qr^4}{D}G_{11}$ $LT_\theta = \frac{-qr^3}{D}G_{14}$ $LT_M = -qr^2G_{17}$ $LT_Q = \frac{-q}{2r}(r^2 - r_o^2)\langle r - r_o\rangle^0$	10a. Simply supported	$y_a = 0, \quad M_{ra} = 0$ $y_c = \frac{-qa^4}{2D}\left(\frac{L_{17}}{1+\nu} - 2L_{11}\right)$ $M_c = qa^2 L_{17}$ $\theta_a = \frac{q}{8Da(1+\nu)}(a^2 - r_o^2)^2$ $Q_a = \frac{-q}{2a}(a^2 - r_o^2)$	$y = K_y \frac{qa^4}{D}, \quad \theta = K_\theta \frac{qa^3}{D}, \quad M = K_M qa^2$

r_o/a	0.0	0.2	0.4	0.6	0.8
K_{y_c}	−0.06370	−0.05767	−0.04221	−0.02303	−0.00677
K_{θ_a}	0.09615	0.08862	0.06785	0.03939	0.01246
K_{M_c}	0.20625	0.17540	0.11972	0.06215	0.01776

Note: If $r_o = 0$, $G_{11} = \frac{1}{64}$, $G_{14} = \frac{1}{16}$, $G_{17} = \frac{(3+\nu)}{16}$

$y_c = \frac{-qa^4(5+\nu)}{64D(1+\nu)}, \quad M_c = \frac{qa^2(3+\nu)}{16}, \quad \theta_a = \frac{qa^3}{8D(1+\nu)}$

Moment at the center of the roof slab

$$Mc := pr\_roof\_slab \cdot (radius\_tank)^2 \cdot \frac{(3 + mu)}{16} = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Moment at the center of the roof slab in the radial direction

$$Mr\_c := Mc = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Moment at the center of the roof slab in the circumferential direction

$$Mt\_c := (mu \cdot Mr\_c) = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Shear per unit circumferential length in the roof slab near tank wall

$$Qend := \frac{pr\_roof\_slab}{2 \cdot radius\_tank} \cdot (radius\_tank)^2 = \text{[redacted]} \cdot \frac{kip}{ft}$$

Roof slab overhang on exterior of the wall

$$overhang := \text{[redacted]} ft$$

Moment in the roof slab at the tank wall due to overhang

$$Mr\_overhang := pr\_roof\_slab \cdot \frac{overhang^2}{2} = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Effective moment demand at the center of the roof slab in the radial direction

$$Mr\_c - Mr\_overhang = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

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5. Capacity of the roof slab and demand-to-capacity ratios

Below is the bottom reinforcement plan in the roof slab. The bottom reinforcement in the central area of the roof slab is [redacted]" dia reinforcement at [redacted]" on center in both the directions.

db_roof_slab := [redacted]in

sp_roof_slab_bot_reinf := [redacted]in

[redacted]

Area of bottom steel reinforcement in the radial direction per unit width along the circumferential direction (note the same bottom reinforcement is provided in the circumferential direction per unit width along the radial direction)

$$\text{Ast\_roof\_slab\_bottom\_reinf} := \frac{1 \cdot 3.14}{4} \cdot \text{db\_roof\_slab}^2 \cdot \frac{1\text{ft}}{\text{sp\_roof\_slab\_bot\_reinf}} \cdot \frac{1}{1\text{ft}} = [\text{redacted}] \cdot \frac{\text{in}^2}{\text{ft}}$$

Compression block depth in the roof slab for calculation of the flexural strength

$$a\_conc := Ast\_roof\_slab\_bottom\_reinf \cdot \frac{fy}{0.85 \cdot fc \cdot 12 \frac{in}{ft}} = [redacted] \cdot in$$

Effective depth of the roof slab considering [redacted] in. of cover

$$d\_roof\_slab := thickness\_of\_roof\_slab - [redacted] in = [redacted] \cdot in$$

Depth of the neutral axis from the extreme fiber compression

$$c\_roof\_slab := \frac{a\_conc}{beta} = [redacted] \cdot in$$

$beta := 0.85$ (for 3000 psi concrete, Table 22.2.2.4.3 of ACI 318-19)

Calculating the tensile strain in the bottom reinforcement using strain compatibility

$$epsilon\_tension\_roof\_slab := 0.003 \cdot \frac{(d\_roof\_slab - c\_roof\_slab)}{c\_roof\_slab} = 0.[redacted]$$

Since the tensile strain in the bottom reinforcement is greater than 0.005, strength reduction factor in flexure per Table 21.2.1 of ACI 318-19

$$phi\_flexure := 0.9$$

Strength of the roof slab in flexure at the center of the slab per ACI 318-19

$$phi\_Mn := phi\_flexure \cdot fy \cdot Ast\_roof\_slab\_bottom\_reinf \cdot \left(d\_roof\_slab - \frac{a\_conc}{2}\right) = [redacted] \cdot kip \cdot \frac{ft}{ft}$$

Demand to capacity ratio for flexure in the roof slab near center in the radial direction

$$DCR\_roof\_slab\_flexure := \frac{Mr\_c - Mr\_overhang}{phi\_Mn} = [redacted]$$

Demand to capacity ratio for flexure in the roof slab near center in the circumferential direction

$$DCR\_roof\_slab\_flexure\_2 := \frac{Mt\_c}{phi\_Mn} = [redacted]$$

Trussed units marked T2 at [redacted]" spacing are provided along the E-W direction in the roof slab near tank walls in Tanks 1 and 2. Similarly trussed units marked T1 at [redacted]" spacing are provided along the E-W direction in the roof slab near tank walls in Tank 4. In Tank 3, a combination of trussed units marked T4 and T12 at [redacted]" spacing along the E-W direction are provided in the roof slab near tank walls.

Below shows the trussed units. All the trusses units have [redacted]" dia inclined shear reinforcement spaced at [redacted]" on spacing.

[redacted]

Min. shear reinforcement per unit width required in slabs per Section 7.6.3.3 of ACI 318-19

$$\text{Av\_min} := \frac{\text{psi}}{\text{fy}} \cdot \max\left(0.75 \cdot \sqrt{\frac{\text{fc}}{\text{psi}}} \cdot 1\,\text{ft}, 50 \cdot 1\,\text{ft}\right) = [\text{redacted}] \cdot \frac{\text{in}^2}{\text{ft}}$$

Provided shear reinforcement in the roof slab near the east and west ends of the roof slab near tank wall is based on [redacted]" dia reinforcement at 8" spacing per unit width along the circumferential direction.

$$\text{db\_shear} := \frac{[\text{redacted}]}{[\text{redacted}]}\,\text{in} \qquad \text{spacing\_shear\_reinf} := [\text{redacted}]\,\text{in} \qquad \text{spacing\_trussed\_units} := [\text{redacted}]\,\text{in}$$

$$\text{Av\_provided\_EW} := \frac{3.14}{4} \cdot (\text{db\_shear})^2 \cdot \frac{\text{d\_roof\_slab}}{\text{spacing\_shear\_reinf}} \cdot \frac{1}{1\,\text{ft}} = [\text{redacted}] \cdot \frac{\text{in}^2}{\text{ft}}$$

Provided shear reinforcement in the roof slab near the north and south ends of the roof slab near tank wall

$$Av\_provided\_NS := \frac{3.14}{4} \cdot (db\_shear)^2 \cdot \frac{d\_roof\_slab}{spacing\_trussed\_units} \cdot \frac{1ft}{spacing\_shear\_reinf} \cdot \frac{1}{1ft} = \text{[redacted]} \cdot \frac{in^2}{ft}$$

Since the provided shear reinforcement exceeds the minimum shear reinforcement, the size effect need not be considered per ACI 318-19.

Nominal one way shear strength provided by concrete per Section 22.5.5 of ACI 318-19.

$$Vc := 2psi \left(\frac{fc}{psi}\right)^{0.5} \cdot d\_roof\_slab = \text{[redacted]} \cdot \frac{kip}{ft}$$

Nominal shear strength provided by shear reinforcement per Section 22.5.8.5.4 of ACI 318-19.

$$Vs := fy \cdot Av\_provided\_EW \cdot (\sin(45deg) + \cos(45deg)) = \text{[redacted]} \cdot \frac{kip}{ft}$$

Capacity of the roof slab in shear per unit length along the circumferential direction per ACI 318-19

$$phi\_shear := 0.75$$ (Table 21.2.1 of ACI 318-19)

$$phi\_Vn := phi\_shear \cdot (Vc + Vs) = \text{[redacted]} \cdot \frac{kip}{ft}$$ (Section 22.5.1.1 of ACI 318-19)

Demand to capacity ratio for shear in the roof slab

$$DCR\_slab\_shear := \frac{Qend}{phi\_Vn} = \text{[redacted]}$$

6. Estimated future conditions of the Surge Tanks

In Section 7 of SGH's May 2023 Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks, we evaluated the long-term risk of concrete and steel deterioration of buried storage tanks (https://health.hawaii.gov/about/files/2023/06/Enclosure-1-Structural-Assessment-2023-May-31-0915.pdf). For detailed information about the long-term durability of concrete, we refer the reader to this online resource.

Although the Red Hill Underground Fuel Storage Tanks and the Surge Tanks are not identical, nor are they directly adjacent to each other, the mechanisms of concrete deterioration for buried concrete are similar based on the literature, and therefore our Red Hill study on steel lined concrete deterioration can be applied to the Surge Tanks. In our 2023 report we cited peer reviewed literature, codes, and military and industry standards relevant to evaluating concrete deterioration. From this extensive study we determined that the most likely cause of concrete degradation, given the probable conditions in the closed Red Hill tanks, would be carbonation of the concrete reaching the embedded reinforcement. Although carbonation is the controlling deterioration mechanism, we understand that chlorides are often understood as a driving mechanism for deterioration, and therefore we address both chloride and carbonation penetration in the discussion below.

First, the rate and depth of chloride penetration into concrete depends on the amount of chlorides at the surface of the concrete and the concrete's permeability. To evaluate chloride penetration into the concrete of the Surge Tanks we look at 1) the chloride concentration needed to initiate reinforcement corrosion 2) determination of which element would be likely to deteriorate first 3) the likely sources of chloride exposure at the Surge Tanks.

- Water with a chloride content of less than 1,000 ppm is acceptable for mixing water in conventionally reinforced concrete (PCA 2011, AASHTO 2020). This allowable chloride content is taken as our conservative threshold for reinforcement corrosion.

- From the drawings (14th N.D. Drwg. OA-N24-174, Y&D No. 294126), the Surge Tanks have [redacted] in. of concrete cover over [redacted] in. diameter at [redacted] in. on center reinforcement within a 12 in. thick minimum tank wall. Should deterioration occur, the thinnest member, i.e., the tank wall, would be the first element to deteriorate. We evaluate this element for chloride penetration risk.

- Given the tanks will be abandoned-in-place and not filled, environmental exposure will occur from the outside of the tanks inward. The Surge Tanks were constructed within excavated rock, with the base of the excavations at elevation [redacted] ft. In May 2021, NAVFAC issued a DRAFT report and shared this with the DOH. This report includes data from monitoring wells near Hotel Pier in Joint Base Pearl Harbor-Hickam (https://health.hawaii.gov/ust/files/2021/11/DFT-Hotel-Pier-Plume-Delineation.pdf). These monitoring wells show groundwater elevations at [redacted] ft to [redacted] ft above sea level. The Surge Tank bottoms, and thus the tank walls, are, at a minimum, approximately [redacted] ft above this groundwater table at the closeby Hotel Pier and are therefore not permanently submerged or surrounded by fresh or salt water.

- A DOH March 2022 technical review of the Navy's groundwater flow model notes that the chloride concentration at the Halawa shaft is 152 mg/L (equivalent to 70 ppm, based on a sodium chloride density of 2.16 mg/L), (https://www.epa.gov/system/files/documents/2022-03/epa-hdoh-groundwater-flow-model-report-disapproval-with-attachments-2022-03-17.pdf). This concentration is well below the conservative threshold of 1,000 ppm.

- Groundwater monitoring well NMW33, from October 2023 through October 2024 had chloride concentrations that varied between 250 and 280 ppm (https://app.powerbi.com/view?r=eyJrIjoiNmJkZTE5MGUtZjFmNC00YjVkLWIxN2QtZjE1MDBlM2U1OTU1IiwidCI6ImUyYzE5MDhiLTI2NzItNGE0Ni05M2ZkLTdmMDhkYTExNjZiNSIsImMiOjJ9). It is important to note that this well is located in an above ground storage tank area, to the east of Joint Base Pearl Harbor Hickam, and therefore is not indicative of typical wells in the region. However, we use this well as potentially indicative of conditions near the underground pump house. This range of chloride concentration is well below the conservative threshold of 1,000 ppm.

Given the distance of the Surge Tanks above the groundwater table, and given the chloride concentration of the water if it were adjacent to the concrete is far below that which is tolerable for mixing water in concrete as measured at a nearby well, we determine that there is no apparent source of chlorides that will increase the chloride content within the concrete above the corrosion threshold at the reinforcement depth. Therefore, corrosion due to chloride penetration is not deemed a risk considering known conditions at the Surge Tanks.

Next, we review carbonation in concrete. Carbonation is a process of atmospheric or water soluble CO_2 reacting with concrete and reducing the natural pH. Steel corrodes in the presence of moisture and low pH environments (less than or equal to 9 pH), therefore as the carbonation front reaches the reinforcement layer, the natural protective layer around the embedded steel is removed and the steel can begin to corrode. The depth of carbonation over time can be modeled as $D = Kt^{0.5}$, where K can be 3 to 4 mm/year for low-quality concretes (water to cement ratio ~ 0.6) in the air, based on work by A. M. Neville (Neville, A.M., 2012, Properties of Concrete, Trans-Atlantic Publications, Inc., 5th Edition). From this we estimate a timeframe on the order of 250 years for the carbonation front to reach the embedded steel reinforcement, assuming 2.5 in. (63.5 mm) of concrete cover.

Given the Surge Tanks are already approximately 85 years old, we estimate that carbonation may reach the reinforcement in another 165 years. This does not mean that the reinforcement will be 100% compromised in 165 years, rather it means in approximately 165 years, the low pH environment will allow for the commencement of reinforcement corrosion should moisture also be present at the reinforcement depth. We do not expect a reduction in reinforced concrete capacity due to reinforcement corrosion from chlorides or carbonation in the near-term.

Finally, conditions inside the abandoned-in-place and empty Surge Tanks will be similar to the conditions inside the abandoned-in-place and empty Red Hill Tanks. The Metals Handbook Volume 13C: Corrosion: Environments and Industries provides guidance on atmospheric corrosion rates. Given that the steel liner in the Surge Tanks will not be exposed to exterior conditions in its abandoned state, referencing the corrosion rate in the tropical marine climate of Hawaii is not appropriate. Rather, we consider atmospheric conditions in the Surge Tanks to likely be similar to a more Northern rural environment with a corrosion rate between 0.5 to 1 mil per year. The Surge Tank drawings specify a [redacted] in. thick steel plate liner ([redacted] mil). This means it will take approximately 125 years for atmospheric conditions to lead to corrosion of half the steel liner thickness. Note that the estimated corrosion rate of 1 mil (0.001 in.) per year for the liner plate is consistent with the range of observed corrosion rates documented in the Surge Tank 1 (Facility No. 12224) Engineering Review and Suitability for Service Evaluation, Final Condition Assessment Report (Pre-Repair), dated October 2, 2020, prepared by Enterprise Engineering, Inc., which ranged from negligible in three shell courses to 0.00127 in. per year in one shell course.

Considering both concrete and steel liner deterioration, we do not expect any significant deterioration of the concrete within the next 165 years or the steel liner within the next 50 years.

[redacted]

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7. Capacity of the roof slab considering potential future deteriorated condition

We have only been in the Surge Tank gallery and have not had access to the interior (or exterior) of the Surge Tanks apart from what can be observed in the Surge Tank gallery. However, we assessed concrete durability in our earlier report on the Red Hill tanks (see Simpson Gumpertz & Heger, 'Long-Term Structural Integrity Assessment of the Red Hill Underground Storage Tanks,' which has been provided as Enclosure1 to Supplement 2 of the Department of the Navy's Tank Closure Plan at the Red Hill Bulk Fuel Storage Facility). We present a similar evaluation of likely deterioration mechanisms in Section 6 and refer the reader to our May 2023 Red Hill report for an in-depth discussion of concrete deterioration mechanisms. To summarize here, we estimate that a minimum of 250 years would be required for carbonation to reach the depth of the embedded reinforcing steel in the Surge Tanks. Given the Surge Tanks are 85 years old, this indicates that the carbonation front will take 165 more years to reach the reinforcement depth. We do not consider chloride related corrosion to be a risk as there is no credible source of chlorides around the Surge Tanks, based on ground water depth and chloride concentrations at nearby wells. Similarly, we do not anticipate any significant deterioration of the steel liner in the Surge Tanks within the next 50 years.

Any potential surface cracks in the concrete in the roof slab would not affect the concrete strength since the cracks will close in the compression zones and in the tensile zones, the concrete tension strength is neglected in the evaluation. However, the cracks can lead to moisture ingress and result in corrosion of the reinforcement. To consider potential future deteriorated condition where the reinforcement in the roof slab is corroded, we estimate the capacity of the roof slab assuming the reinforcement in the bottom layer at the center of the slab has corroded and the effective cross-sectional area of this reinforcement is reduced by 10%. Sánchez et. al. estimated and measured average carbonation-induced corrosion rates for embeded reinforcement considering carbonation propagation until an unacceptable loss of diameter after the concrete has already cracked to be 33 to 40 micrometer/year. The 10% loss of cross-sectional area in the 1 in. dia. reinforcement corelates to approximately 30 to 40 years from the time carbonation propagates through cover and reaches the reinforcement. In this section, we calculate the roof slab strength after approximately 200 years from now (165 years for the carbonation to reach to the reinforcement + 35 years for the reinforcement to experience 10% loss in cross-sectional area due to carbonation-induced corrosion).

Area of corroded bottom steel reinforcement in the radial direction per unit width along the circumferential direction (note the same bottom reinforcement is provided in the circumferential direction per unit width along the radial direction)

$$Ast\_roof\_slab\_bottom\_reinf\_cor := \frac{[redacted]}{[redacted]} \cdot db\_roof\_slab^2 \cdot \frac{1ft}{sp\_roof\_slab\_bot\_reinf} \cdot \frac{1}{1ft} = [redacted] \cdot \frac{in^2}{ft}$$

Compression block depth in the roof slab for calculation of the flexural strength

$$a\_conc\_cor := Ast\_roof\_slab\_bottom\_reinf\_cor \cdot \frac{fy}{0.85 \cdot fc \cdot 12 \frac{in}{ft}} = [redacted] \cdot in$$

Effective depth of the roof slab considering 2.5 in. of cover

$$\text{d\_roof\_slab} := \text{thickness\_of\_roof\_slab} - 2.5\text{in} = \text{[redacted]} \cdot \text{in}$$

Depth of the neutral axis from the extreme fiber compression

$$\text{beta} := 0.85$$ (for 3000 psi concrete, Table 22.2.2.4.3 of ACI 318-19)

$$\text{c\_roof\_slab\_cor} := \frac{\text{a\_conc\_cor}}{\text{beta}} = \text{[redacted]} \cdot \text{in}$$

Calculating the tensile strain in the bottom reinforcement using strain compatibility

$$\text{epsilon\_tension\_roof\_slab\_cor} := 0.003 \cdot \frac{(\text{d\_roof\_slab} - \text{c\_roof\_slab\_cor})}{\text{c\_roof\_slab\_cor}} = \text{[redacted]}$$

Since the tensile strain in the bottom reinforcement is greater than 0.005, strength reduction factor in flexure per Table 21.2.1 of ACI 318-19

$$\text{phi\_flexure\_cor} := 0.9$$

Strength of the roof slab in flexure at the center of the slab per ACI 318-19

$$\text{phi\_Mn\_cor} := \text{phi\_flexure\_cor} \cdot \text{fy} \cdot \text{Ast\_roof\_slab\_bottom\_reinf\_cor} \cdot \left(\text{d\_roof\_slab} - \frac{\text{a\_conc\_cor}}{2}\right) = \text{[redacted]} \cdot \text{kip} \cdot \frac{\text{ft}}{\text{ft}}$$

Demand to capacity ratio for flexure in the roof slab near center in the radial direction

$$\text{DCR\_roof\_slab\_flexure\_cor} := \frac{\text{Mr\_c} - \text{Mr\_overhang}}{\text{phi\_Mn\_cor}} = \text{[redacted]}$$

Demand to capacity ratio for flexure in the roof slab near center in the circumferential direction

$$\text{DCR\_roof\_slab\_flexure\_2\_cor} := \frac{\text{Mt\_c}}{\text{phi\_Mn\_cor}} = \text{[redacted]}$$

Similarly, to consider potential future deteriorated condition where the reinforcement in the roof slab is corroded, we estimate the capacity of the roof slab assuming the shear reinforcement in the roof slab near the tank walls has corroded and the effective cross-sectional area of this reinforcement is reduced by 10%.

Min. shear reinforcement per unit width required in slabs per Section 7.6.3.3 of ACI 318-19

$$\text{Av\_min} := \frac{\text{psi}}{\text{fy}} \cdot \max\left(0.75 \cdot \sqrt{\frac{\text{fc}}{\text{psi}}} \cdot 1\text{ft}, 50 \cdot 1\text{ft}\right) = \text{[redacted]} \cdot \frac{\text{in}^2}{\text{ft}}$$

Provided (corroded) shear reinforcement in the roof slab near the east and west ends of the roof slab near tank wall is based on [redacted]" dia reinforcement at [redacted]" spacing per unit width along the circumferential direction.

$$\text{db\_shear} := \text{[redacted]}\,\text{in} \qquad \text{spacing\_shear\_reinf} := \text{[redacted]}\,\text{in} \qquad \text{spacing\_trussed\_units} := \text{[redacted]}\,\text{in}$$

$$\text{Av\_provided\_EW\_cor} := \text{[redacted]} \cdot (\text{db\_shear})^2 \cdot \frac{\text{d\_roof\_slab}}{\text{spacing\_shear\_reinf}} \cdot \frac{1}{1\text{ft}} = \text{[redacted]} \cdot \frac{\text{in}^2}{\text{ft}}$$

Provided (corroded) shear reinforcement in the roof slab near the north and south ends of the roof slab near tank wall

$$\text{Av\_provided\_NS\_cor} := \frac{0.9 \cdot 3.14}{4} \cdot (\text{db\_shear})^2 \cdot \frac{\text{d\_roof\_slab}}{\text{spacing\_trussed\_units}} \cdot \frac{1\text{ft}}{\text{spacing\_shear\_reinf}} \cdot \frac{1}{1\text{ft}} = \text{[redacted]} \cdot \frac{\text{in}^2}{\text{ft}}$$

Since the provided shear reinforcement exceeds the minimum shear reinforcement, the size effect need not be considered per ACI 318-19.

Nominal one way shear strength provided by concrete per Section 22.5.5 of ACI 318-19.

$$\text{Vc} := 2\text{psi}\left(\frac{\text{fc}}{\text{psi}}\right)^{0.5} \cdot \text{d\_roof\_slab} = \text{[redacted]} \cdot \frac{\text{kip}}{\text{ft}}$$

Nominal shear strength provided by corroded shear reinforcement per Section 22.5.8.5.4 of ACI 318-19.

$$\text{Vs\_cor} := \text{fy} \cdot \text{Av\_provided\_EW\_cor} \cdot (\sin(45\text{deg}) + \cos(45\text{deg})) = \text{[redacted]} \cdot \frac{\text{kip}}{\text{ft}}$$

Capacity of the roof slab in shear per unit length along the circumferential direction per ACI 318-19

$$\text{phi\_shear} := 0.75 \qquad \text{(Table 21.2.1 of ACI 318-19)}$$

$$\text{phi\_Vn\_cor} := \text{phi\_shear} \cdot (\text{Vc} + \text{Vs\_cor}) = 170.239 \cdot \frac{\text{kip}}{\text{ft}} \qquad \text{(Section 22.5.1.1 of ACI 318-19)}$$

$$\text{DCR\_slab\_shear\_cor} := \frac{\text{Qend}}{\text{phi\_Vn\_cor}} = 0.211$$

8. Deflection of the roof slab

Unfactored pressure at the top of the roof slab

$$pr\_roof\_slab\_3 := \frac{(1.\cdot weight\_of\_roof\_slab + 1.\cdot weight\_of\_soil + 1.uniform\_live\_load + 1.vehicular\_load)}{\left[\frac{3.14}{4}\cdot(2\cdot radius\_tank)^2\right]} = \text{[redacted]}\cdot psf$$

Plate constant per Roark's formula

$$D := E\cdot\frac{thickness\_of\_roof\_slab^3}{12\cdot\left(1 - mu^2\right)} = \text{[redacted]}\cdot kip\cdot ft$$

Factor for the deflection estimate per Roark's formula

$$Ky\_c := 0.06370$$

Deflection using the Roark's formula

$$y\_c := Ky\_c\cdot\left[pr\_roof\_slab\_3\cdot\frac{(radius\_tank)^4}{D}\right] = \text{[redacted]}\cdot in$$

Assuming the cracked moment of inertia of the roof slab is one third of the uncracked section

$$y\_c\_cracked := \frac{y\_c}{0.33} = \text{[redacted]}\cdot in$$

Long-term deflection factor of 2 to consider time-dependent deflection (Table 24.2.4.1.3 of ACI 318-19)

$$y\_c\_cracked\_long\_term := 2\cdot y\_c\_cracked = \text{[redacted]}\cdot in$$

Span to deflection ratio

$$\frac{2\cdot radius\_tank}{y\_c\_cracked\_long\_term} = \text{[redacted]}$$

The span to deflection ratio is large (881).

9. Tank roof slab evaluation conclusions

Table below summarizes the demands calculated following load combinations of ASCE 7-22 and capacities of the roof slab in flexure and shear calculated following ACI 318-19. Note that we calculated capacities for the as-designed condition and also for the potential future deteriorated condition. We considered surface cracks in the concrete and corrosion of the reinforcement corresponding to 10% loss of the cross-sectional area of the reinforcement. The table also shows the calculated long-term deflection of the roof slab following load combinations of ASCE 7-22 and accounting for the time-dependent deflection resulting from creep and shrinkage of concrete in the roof slab per ACI 318-19.

		As-Designed Condition		Potential Future Condition with Assumed Deterioration	
	Demand	Capacity	Demand-to-Capacity Ratio	Capacity	Demand-to-Capacity Ratio
Moment in the radial direction	[redacted]				
Moment in the circumferential direction					
Shear in the radial direction near tank wall					

	Long-Term Deflection	Span-to-Deflection Ratio
Maximum deflection at the center of the roof slab	[redacted]	

This calculation demonstrates the Surge Tank roof slab is structurally safe for closure in-place.

10. Tank wall details

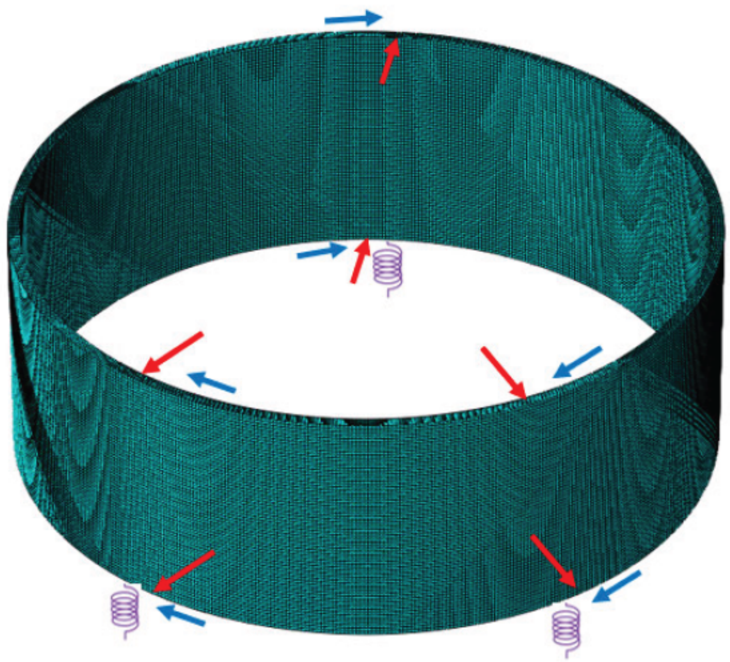
The [redacted] in. thick tank wall has two curtains of reinforcement in both directions. Both the vertical and hoop (horizontal) reinforcement in the exterior and interior layers are [redacted] in. dia at [redacted] in. on center. The reinforcement detailing at the base of the wall indicates a pinned boundary condition with the ringwall foundation. We did not find any details showing the connection of the tank wall to the roof slab. We assumed pinned boundary condition at the top of the wall in our evaluation.

[redacted]

11. Tank wall finite element (FE) model and loads

We developed a very simple finite element (FE) model of the [redacted] ft [redacted] in. tall wall in ABAQUS. ABAQUS is a general purpose, FE analysis method software developed by Dassault Systems. Our simple finite element model only includes the concrete wall, i.e., no roof slab or bottom slab was modeled and nor was the reinforcement or liner plate. We used solid elements to model the tank wall. We modeled elastic concrete material behavior with a modulus of elasticity of [redacted] ksi as calculated in Section 3 of this calculation.

We constrained radial and circumferential displacements of the nodes at the top of the wall to represent the boundary condition provided by the roof slab. We constrained radial and circumferential displacements of the nodes at the bottom of the wall to model with pinned boundary condition. We also modeled very stiff compression only springs in the vertical direction at the nodes at the base of the wall to represent the boundary condition provided by the wall foundation. Figure below shows our simple finite element model with boundary conditions.



We consider vertical loads on the tank wall due to uniform surface live load, vehicular load at the surface, weight of the [redacted] ft. soil surcharge, weight of the [redacted] ft. thick roof slab, and self-weight of the tank wall.

[redacted]

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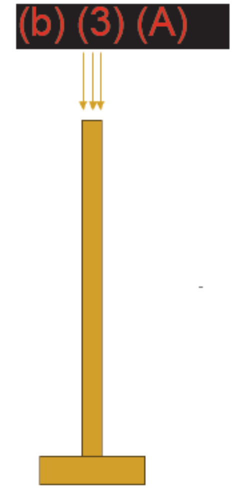
Factored downward pressure at the top of the tank wall for 1.4D (load combination per Section 2.3.1 of ASCE 7-22)

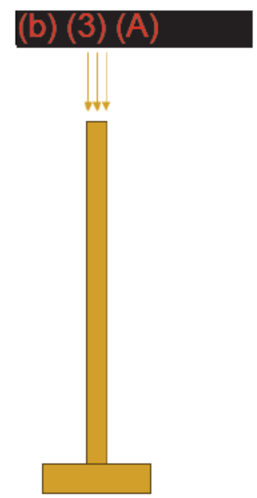
$$\text{pr\_at\_top\_of\_wall\_1} := \frac{(1.4\cdot\text{weight\_of\_roof\_slab} + 1.4\text{weight\_of\_soil})}{\left[\frac{3.14}{4}\cdot\left[\left[2\cdot(\text{radius\_tank} + \text{tank\_wall\_thickness})\right]^2 - (2\cdot\text{radius\_tank})^2\right]\right]} = \text{[redacted]}\cdot\text{psi}$$



Factored downward pressure at the top of the tank wall for 1.2D+1.6L (load combination per Section 2.3.1 of ASCE 7-22)

$$\text{pr\_at\_top\_of\_wall\_2} := \frac{(1.2\cdot\text{weight\_of\_roof\_slab} + 1.2\text{weight\_of\_soil} + 1.6\text{uniform\_live\_load} + 1.6\text{vehicular\_load})}{\left[\frac{3.14}{4}\cdot\left[\left[2\cdot(\text{radius\_tank} + \text{tank\_wall\_thickness})\right]^2 - (2\cdot\text{radius\_tank})^2\right]\right]} = \text{[redacted]}\cdot\text{psi}$$

$$\text{pr\_at\_top\_of\_wall} := \max(\text{pr\_at\_top\_of\_wall\_1}, \text{pr\_at\_top\_of\_wall\_2}) = \text{[redacted]}\cdot\text{psi}$$



Factored downward pressure at the top of the tank wall for 0.9D (load combination per Section 2.3.1 of ASCE 7-22)

$$\text{pr\_at\_top\_of\_wall\_3} := \frac{(0.9\cdot\text{weight\_of\_roof\_slab} + 0.9\text{weight\_of\_soil})}{\left[\frac{3.14}{4}\cdot\left[\left[2\cdot(\text{radius\_tank} + \text{tank\_wall\_thickness})\right]^2 - (2\cdot\text{radius\_tank})^2\right]\right]} = \text{[redacted]}\cdot\text{psi}$$

We also consider the lateral earth pressure from the soil/rock around the tank. We use saturated soil pressure when calculating the lateral pressure on the tank wall. The saturated soil pressure includes soil pressure based on the dry density and hydrostatic pressure on the tank wall. Note the assumption of the saturated soil corresponds to an event when the water table rises to the top of the tank, which is an unlikely event.

We also conservatively use the maximum lateral soil pressure (100 psf per foot of depth) from Table 1610.1 of IBC 2021 for any type of soil.

TABLE 1610.1 LATERAL SOIL LOAD

DESCRIPTION OF BACKFILL MATERIAL[c]	UNIFIED SOIL CLASSIFICATION	DESIGN LATERAL SOIL LOAD[a] (pound per square foot per foot of depth)	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	30	60
Poorly graded clean gravels; gravel-sand mixes	GP	30	60
Silty gravels, poorly graded gravel-sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly sand mixes	SW	30	60
Poorly graded clean sands; sand-gravel mixes	SP	30	60
Silty sands, poorly graded sand-silt mixes	SM	45	60
Sand-silt clay mix with plastic fines	SM-SC	45	100
Clayey sands, poorly graded sand-clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

For SI: 1 pound per square foot per foot of depth = 0.157 kPa/m, 1 foot = 304.8 mm.

a. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.

b. Unsuitable as backfill material.

c. The definition and classification of soil materials shall be in accordance with ASTM D2487.

[redacted]

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[redacted]

At-rest pressure coefficient

$$Ko := \frac{\frac{100psf}{1ft}}{soil\_density} = \text{[redacted]}$$

Density of water

$$water\_density := 1000 \frac{kg}{m^3} = \text{[redacted]} \cdot \frac{lbm}{ft^3}$$

Lateral earth pressure (Terzaghi et al., 1996) from soil/rock around the tank (ignoring the presence of the tunnel, which exists adjacent to a potion of the circumference of the tank)

Lateral pressure at top of the wall considering a load factor of 1.6 per Section 2.3.1 of ASCE 7-22

$$lat\_pr\_top\_wall := 1.6 \cdot \begin{bmatrix} Ko \cdot uni\_live\_load + Ko \cdot soil\_density \cdot (thickness\_of\_roof\_slab + soil\_surcharge\_ht) \ldots \\ + water\_density \cdot g \cdot (thickness\_of\_roof\_slab + soil\_surcharge\_ht) \end{bmatrix} = \text{[redacted]} \cdot psi$$

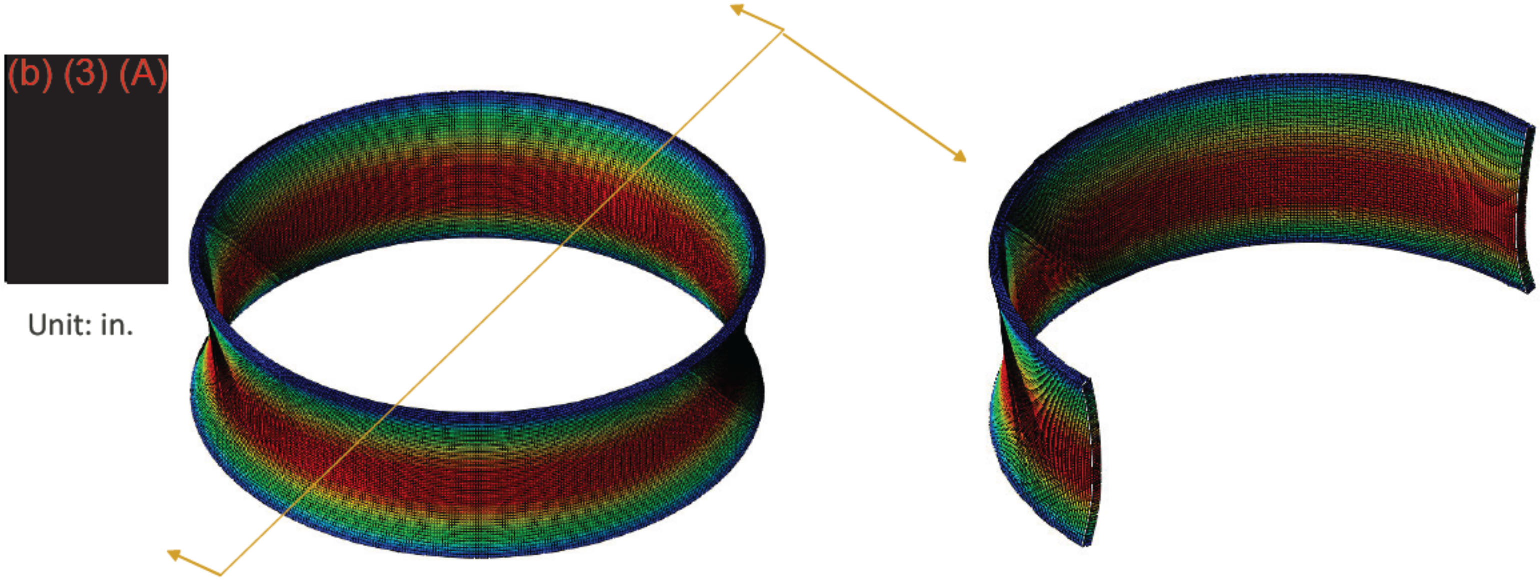
Lateral pressure at bottom of the wall considering a load factor of 1.6 per Section 2.3.1 of ASCE 7-22

$$lat\_pr\_bot\_wall := 1.6 \cdot \begin{bmatrix} Ko \cdot uni\_live\_load + Ko \cdot soil\_density \cdot (thickness\_of\_roof\_slab + soil\_surcharge\_ht + tank\_wall\_ht) \ldots \\ + water\_density \cdot g \cdot (thickness\_of\_roof\_slab + soil\_surcharge\_ht + tank\_wall\_ht) \end{bmatrix} = \text{[redacted]} \cdot psi$$

Note we used at-rest lateral pressure because our calculated wall displacements are smaller than that required to mobilize the soil movement for certain types of soils.

12. Tank wall FE analysis results, demands, capacities, and stress ratios

Figure below shows the deflected shape of the tank wall when subjected to the combined factored vertical downward pressure and lateral pressure for a load combination of [redacted] D+ [redacted] H. The maximum displacement in the tank wall is approximately [redacted] in. The maximum displacement is primarily in the radial direction. Since the tank wall is in compression in the radial and circumferential directions, even the assumption of a cracked wall section would result in a similar displacement. The maximum displacement in the tank wall for the load combination of [redacted] D+ [redacted] H is also [redacted] in.



Maximum displacement of the tank wall

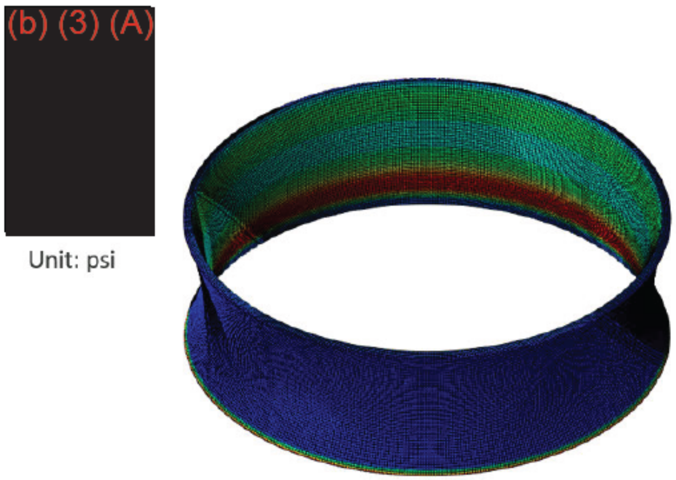
tank_wall_disp := [redacted] n

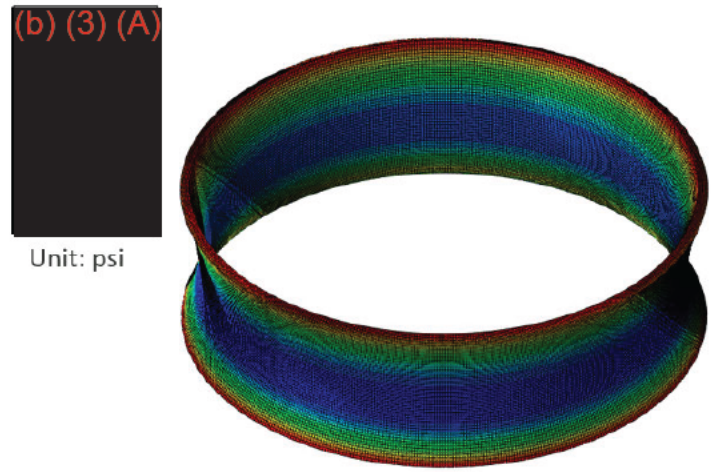
Span to deflection ratio

$$\frac{tank\_wall\_ht}{tank\_wall\_disp} =$$ [redacted]

The span to deflection ratio of the tank wall is large (1613).

Figure below shows the calculated maximum and minimum principal stresses in the tank wall "d" distance away from the boundary conditions. Note the "d" distance is equal to the tank wall thickness, which is [redacted]t. The maximum principal stresses are governed by [redacted]D[redacted]H and the minimum principal stresses are governed by [redacted]D+[redacted]H load combination. The calculated maximum principal stresses indicate tensile stresses on the order of 387 psi located on the interior of the tank wall in the lower half of the tank. The calculated tensile stresses are a result of the bending of the wall. The calculated minimum principal stresses indicate compressive stresses on the order of 1457 psi located on the exterior face of the tank.





Maximum calculated compressive stress in the tank wall

max_compressive_stress_wall := [redacted] psi

Maximum calculated tensile stress in the tank wall

max_tensile_stress_wall := [redacted] psi

Four inches of concrete of the [redacted] in. thick section is in tension at the location of the maximum principal stress. The average tensile stress across this 4 in. thick concrete at the interior of the tank wall at the location of the max. tensile stress predicted from FE analysis

avg_tensile_stress_across_4in_thick_interiror := mean(387psi, 165psi) = [redacted] · psi

tensile_force_demand_per_foot := avg_tensile_stress_across_4in_thick_interiror · 4in = [redacted] · $\frac{\text{kip}}{\text{ft}}$

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CHECKED BY [redacted]

Maximum design compressive stress per ACI 318-19

$$\text{phi\_compression\_conc} := 0.65$$ (Table 21.2.1 of ACI 318-19)

$$\text{max\_design\_comp\_stress\_wall} := \text{phi\_compression\_conc} \cdot \text{fc} = 1.95 \times 10^{3} \cdot \text{psi}$$

Modulus of rupture stress per Section 19.2.3.1 of ACI 318-19

$$\text{modulus\_of\_rupture} := 7.5 \cdot \left(\frac{\text{fc}}{\text{psi}}\right)^{0.5} \cdot 1\text{psi} = 410.792\ \text{psi}$$

Since the calculated maximum tensile stress ([redacted] psi) is smaller than modulus of rupture (411 psi), the concrete is not predicted to crack under considered loading. In addition, the location of the maximum tensile stresses is on the interior face of the wall, where [redacted] in. thick liner plate is present to resist any developed tension.

The capacity of the [redacted] in. thick liner plate in tension

$$\text{liner\_thickness} := \frac{[redacted]}{[redacted]}\text{in}$$

$$\text{phi\_tension\_liner} := 0.9$$ (AISC 360-16)

$$\text{liner\_plate\_capacity} := \text{phi\_tension\_liner} \cdot \text{fy\_liner} \cdot \text{liner\_thickness} = [redacted] \cdot \frac{\text{kip}}{\text{ft}}$$

Stress ratio check for compression

$$\text{stress\_ratio\_comp\_wall} := \frac{\text{max\_compressive\_stress\_wall}}{\text{max\_design\_comp\_stress\_wall}} = [redacted]$$

Stress ratio check for tension

$$\text{stress\_ratio\_tension\_wall} := \frac{\text{tensile\_force\_demand\_per\_foot}}{\text{liner\_plate\_capacity}} = [redacted]$$

Shear failure mode is not credible due to smaller shear stresses and the large area resisting shear loading.

13. Tank wall evaluation considering potential future deteriorated condition

As demonstrated in Section 12 of this calculation, the as-designed tank wall's capacity is higher than the demands posed by the factored loads considered. The primary stresses in the tank wall are compressive in nature. Any potential surface cracks in the concrete tank wall would not affect the concrete strength in compression since the cracks will close in compression. Since the stress ratio in tension is small (0.15), the tank walls would be structurally capable of resisting the tensile demands even with heavily corroded liner plate. Future deteriorated conditions are not a concern for the tank walls.

14. Tank wall evaluation conclusions

Table below summarizes the demands calculated following load combinations of ASCE 7-22 and capacities of the tank wall following ACI 318-19 and the liner plate following AISC 360-16 for the as-designed condition. As discussed in Section 13 of this calculation, because of the nature of the stresses (largely compressive in nature in the wall and localized small tensile stresses on the interior face of the tank wall), even in potential future deteriorated condition, the tank walls would be structurally capable of resisting the demands considered.

		As-Designed Condition	
	Demand	Capacity	Demand-to-Capacity Ratio
Maximum compressive stress	[redacted]		
Maximum tensile force			

	Radial Deflection	Span-to-Deflection Ratio
Maximum deflection in the tank wall	[redacted]	

This calculation demonstrates the Surge Tank walls are structurally safe for closure in-place.

15. Surge Tank buoyancy check

This check demonstrates the safety margin of the Surge Tanks against uplift (or floating) in a severe flooding event. Note the likelihood of such a flooding event is highly unlikely.

Density of water

$$\text{water\_density} := 1000 \cdot \frac{\text{kg}}{\text{m}^3} = 62.428 \cdot \frac{\text{lbm}}{\text{ft}^3}$$

Volume of a tank

$$\text{V\_tank} := 3.14 \cdot (\text{radius\_tank})^2 \cdot \text{tank\_wall\_ht} = \text{[redacted]} \cdot \text{ft}^3$$

Buoyancy force on a tank

$$\text{buoyancy\_F} := \text{water\_density} \cdot g \cdot \text{V\_tank} = \text{[redacted]} \cdot \text{kip}$$

Weight of roof slab considering average roof slab radius of four Surge Tanks

$$\text{weight\_of\_avg\_roof\_slab} := \frac{3.14}{4} \cdot (2 \cdot \text{avg\_rad\_of\_roof\_slab})^2 \cdot \text{thickness\_of\_roof\_slab} \cdot \text{conc\_density} = \text{[redacted]} \cdot \text{kip}$$

Weight of fully saturated soil considering average roof slab radius of four Surge Tanks

$$\text{weight\_of\_saturated\_soil} := [\text{soil\_density} - (\text{water\_density} \cdot g)] \cdot \text{soil\_surcharge\_ht} \cdot \left[\frac{3.14}{4} \cdot (2 \cdot \text{avg\_rad\_of\_roof\_slab})^2\right] = \text{[redacted]} \cdot \text{kip}$$

Weight of bottom slab

$$\text{weight\_of\_bot\_slab} := \frac{3.14}{4} \cdot (2 \cdot \text{radius\_tank})^2 \cdot \text{thickness\_of\_bot\_slab} \cdot \text{conc\_density} = \text{[redacted]} \cdot \text{kip}$$

Weight of wall

$$\text{weight\_of\_wall} := 3.14 \cdot (2 \cdot \text{radius\_tank}) \cdot \text{tank\_wall\_ht} \cdot \text{tank\_wall\_thickness} \cdot \text{conc\_density} = \text{[redacted]} \cdot \text{kip}$$

Weight of wall foundation

$$\text{weight\_of\_wall\_foundation} := 3.14 \cdot (2 \cdot \text{radius\_tank}) \cdot \text{width\_tank\_wall\_found} \cdot \text{tank\_wall\_found\_thk} \cdot \text{conc\_density} = \text{[redacted]} \cdot \text{kip}$$

Weight of tank and the saturated soil surcharge resisting buoyancy force

weight_resisting_bouancy_W := weight_of_saturated_soil + weight_of_avg_roof_slab + weight_of_wall ... + weight_of_bot_slab + weight_of_wall_foundation = [redacted] ·kip

Factor of safety against buoyancy

$$\text{FOS\_buoyancy} := \frac{\text{weight\_resisting\_bouancy\_W}}{\text{buoyancy\_F}} = 1.924$$

The calculated factor of safety against buoyancy is 1.92. Section 1807.2.3 in IBC 2021 specifies a minimum factor of safety of 1.5 for foundation stability.

16. Loading for future land use

The flexure in the roof slab was the governing limit state in the roof slab and the tank walls.

We calculate the surface loading above the Surge Tanks that can be introduced in future depending on the future land use without performing any further structural evaluation of the Surge Tanks.

Future additional surface loading considered as dead load

$$\text{future\_additional\_uni\_dead\_load} := 200\text{psf}$$

Factored pressure at the top of the roof slab for 1.4D (load combination per Section 2.3.1 of ASCE 7-22)

$$\text{pr\_roof\_slab\_future\_add\_1} := \frac{(1.4\cdot\text{weight\_of\_roof\_slab} + 1.4\cdot\text{weight\_of\_soil})}{\left[\frac{3.14}{4}\cdot(2\cdot\text{radius\_tank})^2\right]} + (1.4\cdot\text{future\_additional\_uni\_dead\_load}) = \text{[redacted]}\cdot\text{psf}$$

Future additional surface loading considered as live load

$$\text{future\_additional\_uni\_live\_load} := 250\text{psf}$$

Existing live load

$$\text{existing\_live\_load} := \frac{(\text{uniform\_live\_load} + \text{vehicular\_load})}{\left[\frac{3.14}{4}\cdot(2\cdot\text{radius\_tank})^2\right]} = \text{[redacted]}\cdot\text{psf}$$

Factored pressure at the top of the roof slab for 1.2D+1.6L (load combination per Section 2.3.1 of ASCE 7-22)

$$\text{pr\_roof\_slab\_future\_add\_2} := \frac{\left(\begin{array}{l}1.2\cdot\text{weight\_of\_roof\_slab} + 1.2\cdot\text{weight\_of\_soil} \ldots \\ + 1.6\text{uniform\_live\_load} + 1.6\text{vehicular\_load}\end{array}\right)}{\left[\frac{3.14}{4}\cdot(2\cdot\text{radius\_tank})^2\right]} + (1.6\cdot\text{future\_additional\_uni\_live\_load}) = \text{[redacted]}\cdot\text{psf}$$

$$\text{pr\_roof\_slab\_future\_add} := \max(\text{pr\_roof\_slab\_future\_add\_1}, \text{pr\_roof\_slab\_future\_add\_2}) = \text{[redacted]}\cdot\text{psf}$$

Moment at the center of the roof slab using roark's formula

$$\text{Mc\_future\_add} := \text{pr\_roof\_slab\_future\_add}\cdot(\text{radius\_tank})^2\cdot\frac{(3 + \text{mu})}{16} = \text{[redacted]}\cdot\text{kip}\cdot\frac{\text{ft}}{\text{ft}}$$

Moment at the center of the roof slab in the radial direction

$$Mr\_c\_future\_add := Mc\_future\_add = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Roof slab overhang on exterior of the wall

$$overhang := \text{[redacted]}\, ft$$

Moment in the roof slab at the tank wall due to overhang

$$Mr\_overhang\_future\_add := pr\_roof\_slab\_future\_add \cdot \frac{overhang^2}{2} = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

$$Mr\_c\_future\_add - Mr\_overhang\_future\_add = \text{[redacted]} \cdot kip \cdot \frac{ft}{ft}$$

Demand to capacity ratio for flexure in the roof slab near center in the radial direction

$$DCR\_roof\_slab\_flexure\_cor\_future\_add := \frac{Mr\_c\_future\_add - Mr\_overhang\_future\_add}{phi\_Mn\_cor} = 1$$

We recommend that any future load to be placed on the [redacted] ft. of soil presently on top of the tanks be limited to no more than [redacted] psf without performing any additional detailed structural analysis. For reference a typical modular building weighs approximately [redacted] psf, an office live load specified in ASCE 7-22 is 50 psf.

17. Overall conclusions

This calculation demonstrates the Surge Tanks are structurally safe for closure in-place. We used ASCE 7-22 and IBC 2021 for the calculation of demands and we used ACI 318-19 (concrete) and AISC 360-16 (liner plate) to calculate the capacities. We also considered potential future deterioration (corresponding to approximately 200 years from now) of the concrete and carbonation-induced corrosion of reinforcement in our calculation. The Surge Tanks are structurally capable of resisting the loads considered with the assumed deteriorated conditions. We considered self-weight of the tank, weight of the [redacted] ft. soil surcharge, uniform live load of 100 psf and vehicular loads at the surface. We also considered saturated soil pressure (including hydrostatic pressure) on the tank walls and checked the tanks for buoyancy.

We recommend that any future load to be placed on the [redacted] ft. of soil presently on top of the tanks be limited to no more than [redacted] psf without performing any additional detailed structural analysis. For reference a typical modular building weighs approximately [redacted] psf, an office live load specified in ASCE 7-22 is 50 psf.

APPENDIX B
Drawings

The figures provided earlier in this memorandum show general information only. Dimensions and details are not intended to be read from the figures. For a full-size version of the figures, see the drawings provided in this appendix. In order to be legible, drawings in this appendix may need to be printed at 24 in. x 36 in., minimum.

(b) (3) (A)

[redacted]

3x

(b) (3) (A)

[redacted]

13x

(6)

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

23x

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

3x

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]

(b) (3) (A)

[redacted]