Contents

1. INTRODUCTION

1.1 Objective

Northern Star (Pogo) LLC (NSR) is the operator of the Pogo gold mine, located 38 miles northeast of Delta Junction, Alaska.

The Pogo Mine Dry Stack Tailings Facility (DSTF) has been in operation since February 2006 and is permitted for a capacity of 20 million tons (Mt) of waste based on a design engineered by SRK (2012). Pogo anticipates achieving the DSTF permitted capacity by March 2024.

The DSTF was originally designed by AMEC (AMEC, 2004a), and the Operating, Maintenance and Surveillance (OMS) Manual was issued in January 2006 by AMEC as a guiding document for the construction of the DSTF. The OMS Manual was incorporated into the DSTF Construction and Maintenance Plan, which is updated at least every two years to include additional information such as field compaction testing, year-by-year plans, geotechnical investigations, stability evaluations, and any changes to the construction or operation of the DSTF.

NSR Pogo is currently planning to expand the DTSF beyond the 20 Mt capacity by staying below the elevation of the current diversion ditches. This allows approximately 3,281,000 cubic yards of waste rock and dewatered floatation tailings (approximately 4.9 Mt) to be deposited on the DTSF, resulting in a nominal capacity of 24.9 Mt. WSP will be engineering the expansion project.

The small-scale DSTF expansion project is an interim step towards a larger DSTF expansion that will allow for adequate preparation, data compilation and review, and, if required, additional geotechnical investigations in support of the larger permitting effort.

This Plan provides the steps required to construct and maintain the DSTF at the proposed 24.9 Mt nominal capacity. It should be noted that the water quality, hydrology, and geochemical monitoring plans are omitted from this Plan and are described in the *Pogo Mine Monitoring Plan*.

1.2 Document Control and Responsibility

The Environmental Manager is responsible for the preparation and administration of this Plan, as well as implementing the monitoring and inspection required. Any revisions or updates to DSTF management, construction, or maintenance should be noted in the Plan and submitted for approval by the Alaska Department of Natural Resources (ADNR) and the Alaska Department of Environmental Conservation (ADEC).

The Maintenance Manager is responsible for the construction of the DSTF. The site-specific Safe Work Procedure (SWP) *DSTF Tailings and Rock Placement* provides best practices for the placement and management of material in the DSTF.

1.3 Acronyms

AAC	Alaska Administrative Code
ADEC	Alaska Department of Environmental Conservation
ADF&G	Alaska Department of Fish & Game
ADNR	Alaska Department of Natural Resources
APDES	Alaska Pollutant Discharge Elimination System
ARD	Acid rock drainage
CIP	Carbon-in-pulp
CFS	Cubic feet per second
CSP	Corrugated steel pipe
CSR	Cyclic stress ratio
CRR	Cyclic resistance radio
DSTF	Dry Stack Tailings Facility
EDMS	Environmental Data Management System
EPZ	Embankment Placement Zone
fbgl	Feet below ground level
FoS	Factor of Safety
GPA	General Placement Area

HDPE	High-density polyethylene
LOM	Life of Mine
MDE	Maximum Design Earthquake
Mt	Million tons
NSR	Northern Star (Pogo) LLC
OMS	Operating, Maintenance and Surveillance
PGA	Peak Ground Acceleration
PPM	Parts per million
SWP	Safe Work Procedure
SPT	Standard Penetration Test
RTP	Recycled Tailings Pond
VWP	Vibrating Wire Piezometer
USGS	United States Geological Survey

2. FACILITY DESCRIPTIONS

2.1 Major Components

The major components of the DSTF include:

- Flow-Through Drains
- Starter Berm and Toe Berm
- Development Rock
- Shell Area
- Embankment Placement Zone (EPZ)
- General Placement Area (GPA)
- North and South Diversion Ditches

2.1.1 Flow-through Drains

All runoff in and around the DSTF area below the diversion ditches is directed to the Recycle Tailings Pond (RTP) by means of a network of drains. Flow-through drains are constructed in the existing stream valleys within the DSTF area to augment the existing drainage courses and allow them to pass runoff under the stack. The drains are extended upstream as the elevation of GPA rises. **Figure 1** shows the general configuration of the DSTF, **Figure 2** shows aerial view of Flow-Through Drains, and **Figure 3a** depicts the cross-section of the flow-through drains. The rock fill used in the flow-through drains is between 12 inch and 36 inch in size and covered with filter material to reduce the potential for fines from dewatered flotation tailings from migrating into and clogging the drain. The flow-through drain filter consists of two layers: Filter 1 and Filter 2. Sand is used for Filter 1, and gravel is used for Filter 2. The gradation requirements for the filters are shown in **Figure 3b**. The flow capacity of the flow-through drains were calculated to be approximately 120 times the daily average flow of 0.47 cfs (200 gpm) measured at the USGS gauge on Liese Creek. This capacity is estimated to be equivalent to a 1:10,000-year/24-hour storm event from the entire DSTF watershed including the area above the diversion ditches (AMEC,2004a).

A perimeter drain of green rock placed around the GPA has been used to convey runoff from the GPA to the flow-through drains. This is an evolution of the original facility's design to have a 1.5-ft thick, unfiltered erosion control / drainage blanket constructed using green rock or coarse colluvium soils constructed above the cleared and grubbed ground surface. To limit the potential flow of water into the foundation of the DSTF, the perimeter drain will stop being constructed for the interim expansion. Any existing springs or other surface flow will be connected to the perimeter drain or the flow-through drain prior to placement of compacted tailings.

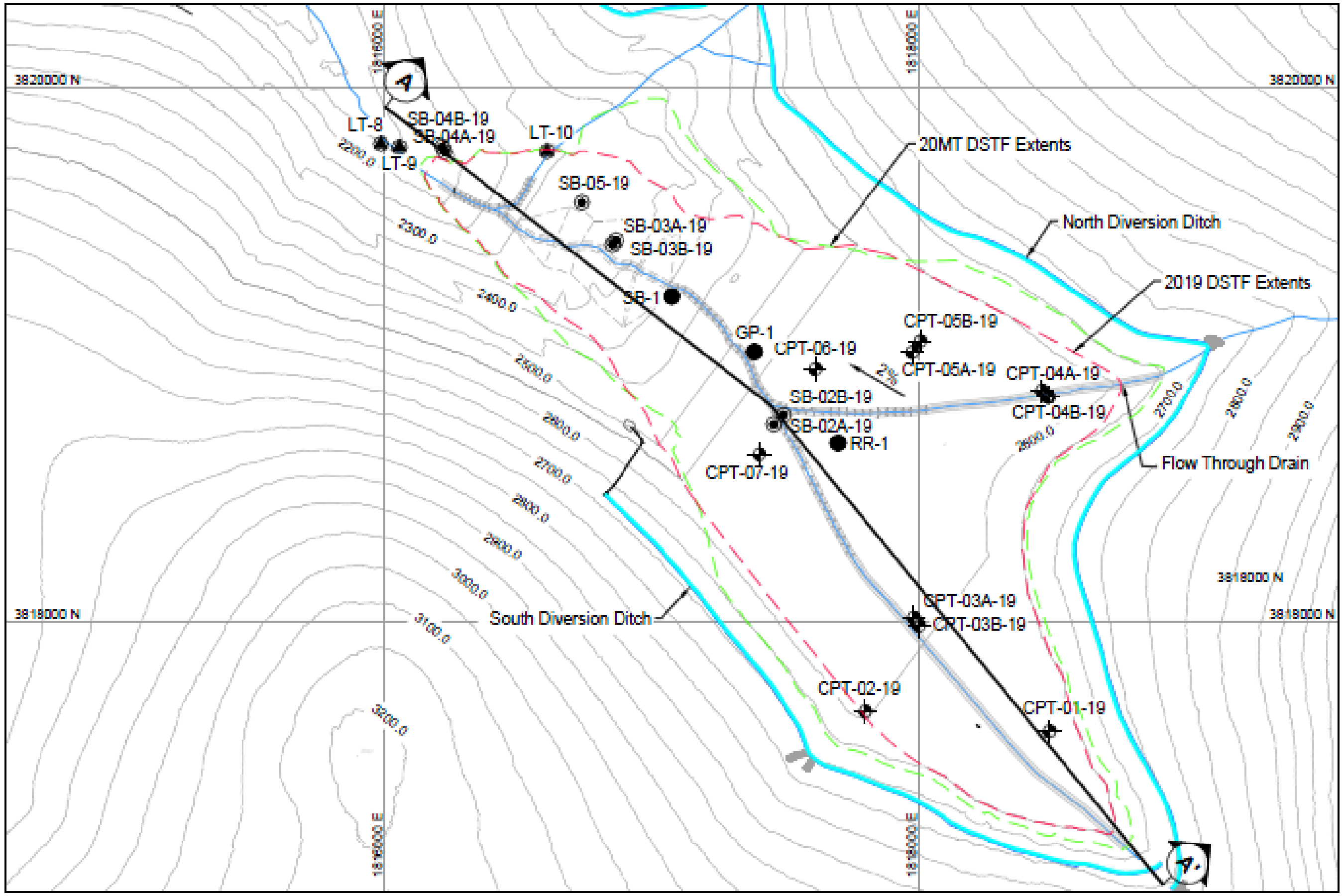
Figure 1: General Configuration of DSTF

LEGEND

- New Diversion Ditch
- Extent of 20MT Material Placement
- CPTs
- BORINGS
- SRK 2012 BORINGS
- AMEC 2000 BORINGS
- Shell
- Compacted Tailings
- Berm
- GPA Materials
- Flow-Through Drain

NOTES

1. Contour interval is 50ft.
2. Base contour data received from POGO mine site dated May 20, 2010.



PLAN VIEW
DRY STACK STORAGE AREA

Figure 2: Flow-Through Drain Locations

Figure 3a: Typical Cross Section of Flow-Through Drain

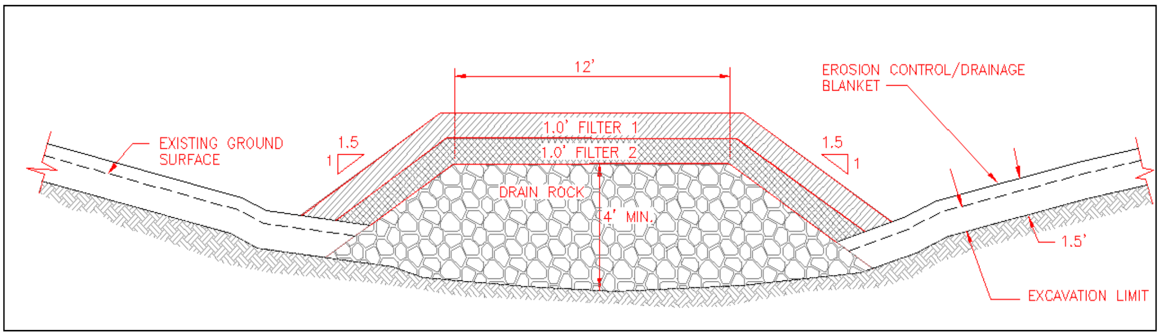
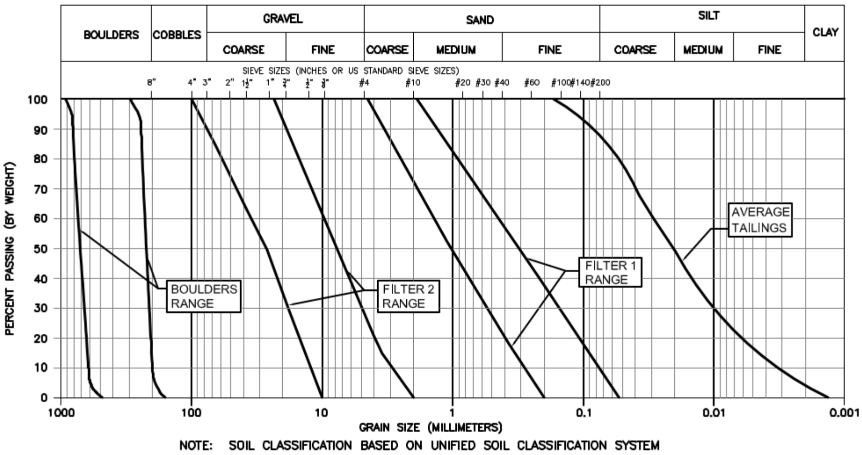


Figure 3b: Gradation Requirements for Flow-Through Drain Filters



2.1.2 Starter Berm and Toe Berm

The starter berm was constructed as the initial containment for the GPA with the material from nearby colluvium excavations. The toe berm, downstream of the starter berm was constructed of green rock and acts as a foundation of the shell area. The toe berm was extended to downstream before the construction of the second and third shell. The starter berm and toe berm are located directly upstream of the DRYTOE, shown in **Figure 2**.

2.1.3 Development Rock

Development rock includes mineralized (red) rock and non-mineralized (green) rock as described in Section 2.2.3. Green rock is used exclusively in shell and drain construction. Red rock may be encapsulated in the tailings to limit the oxidation of sulfide minerals. Red rock cells may be constructed in the shells, the EPZ or the GPA under the procedures described in Section 4.1.5.

2.1.4 Shell Area

There are three composite shells of compacted tailings forming the downstream section of the DSTF; shells are constructed with a 3:1 outer slope with an outer layer of green rock providing erosion resistance. The innermost Shell 1 contains a one-lane switchback road for access to monitoring locations, as well as a portion of the two-lane haul road as the DSTF raises. As the DSTF expands, Shell 1 raises with the height of the DSTF to buttress the GPA. Based on an excess of green rock production in the early years of the mine life, the 100-ft wide Shell 1 was initially constructed using only green rock. This width was continued and combined with an inner 100-ft wide zone of compacted flotation tailings through August 2021. Based on stability analyses and recommendations from the *AECOM 2021 Geotechnical Review Report* (AECOM, 2021), the Shell 1 green rock zone was reduced to 20 feet and the compacted tailings layer was reduced to 50 feet.

Construction began on Shell 2 and Shell 3 in 2010. Shells 2 and 3 are composite shells consisting of a 20 ft wide green rock layer with an interior of compacted flotation tailings. The width of the tailings in Shell 2 is 160 ft, for a total shell width of 180 ft. Shell 3 has a 130 ft layer of tailings for a total shell width of 150 ft. Shells 2 and 3 have not been raised since 2019 are not yet at design height for the 20Mt facility configuration as of May 2023. **Figure 4** shows the current and proposed shell configuration. Shells 2 and

3 are now planned to be raised monolithically; as a result, the intermediate zone of green rock between Shell 2 and Shell 3 will be eliminated as it will not be needed for erosion control nor is it necessary for the geotechnical stability of the DSTF. Furthermore, green rock will be reclaimed from the portion of the Shell 1 slope that will be buried by the combined Shells 2 and 3 for the same reason. The combined Shell 2/3 crest area provides sufficient space for red rock placement while maintaining the same offset criteria as for placement in the GPA. Therefore, red rock may be used to construct the interior sections of the combined Shell 2/3 as described in Section 4.1.5.

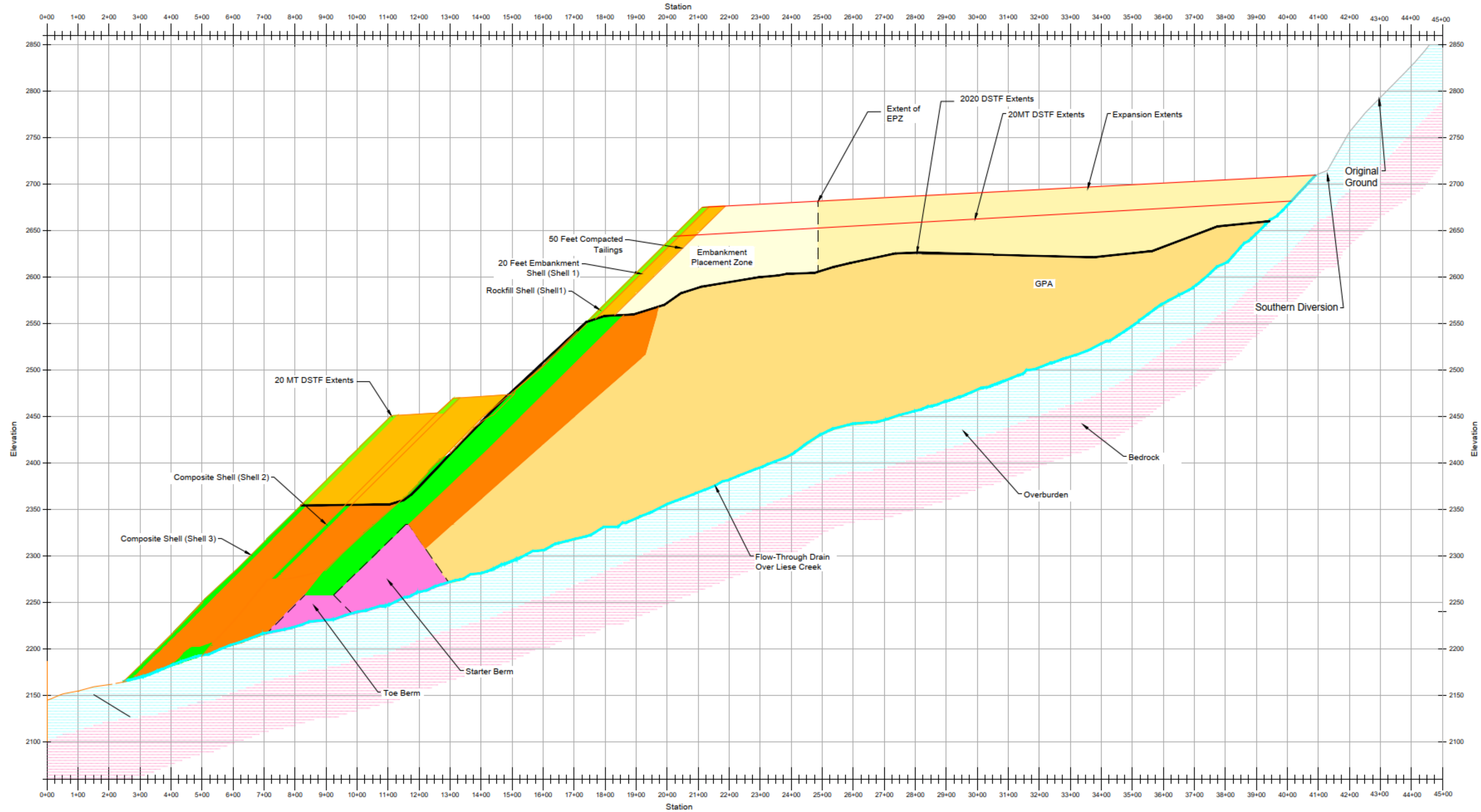
2.1.5 Embankment Placement Zone (EPZ)

Combined with narrowing the width of the green rock and compacted flotation tailings for future raises of Shell1, AECOM (2021) delineated an Embankment Placement Zone (EPZ) between Shell1 and the GPA to provide the required DSTF performance. The EPZ is an intermediate zone of compacted flotation tailings and mineralized rock with specific QA/QC requirements to enhance stability for the DSTF. The EPZ is shown on the **Figure 4** DSTF cross-section. The EPZ is currently approximately 500 ft wide and will be 460 ft wide at the top of the permitted 20 Mt design. The EPZ will further narrow to approximately 370 ft at the top of the proposed interim expansion. The target geotechnical criteria for compacted tailings in EPZ are described in Section 5.1. Red rock cells may be constructed in the EPZ as described in Section 4.1.5.

2.1.6 General Placement Area (GPA)

The GPA is the tailings and red rock co-disposal area upstream of the EPZ and consists of the majority of the DSTF volume. Tailings placed in the GPA are not required to meet target geotechnical criteria including moisture content and density, although the compaction procedures are generally the same as dewatered flotation tailings placed in the shell or EPZ. Red rock cells may be constructed in the GPA as described in Section 4.1.5. High moisture content tailings may be placed in the GPA in discrete cells contained by compacted tailings.

Figure 4: DSTF Cross Section



2.1.7 Diversion Ditches

A full description of the diversion ditches is provided in the *RTP Operations and Maintenance Manual*. The diversion ditches are designed to intercept a one in 200-year, 24-hour precipitation event (4.6 inches within 24 hours). The ditches are sized to convey flow with a minimum one foot of freeboard. The estimated design flow calculated by SRK is 78 cfs at Flume #2 (north diversion ditch), 24 cfs at the New South Flume, and 34 cfs at Flume #1 (south diversion ditch) (SRK 2013b). Flow intercepted by the diversion ditches is discharged into Liese Creek downstream of the RTP Dam as non-contact water.

2.2 Environmental Management

2.2.1 Water Management

The RTP Dam serves as the impoundment where mine-contacted water can be stored prior to recycling or subsequent treatment and discharge to the environment. The RTP Dam impounds runoff from the DSTF, captures natural flows from the catchment area below the limits of diversion ditch and the DSTF, and collects various plant site contact runoff water. Runoff down gradient of the diversion ditch and DSTF seepage are considered "mine-contacted." Pogo's *RTP Operating and Maintenance Manual* further describes water management which enters the RTP from the DSTF.

2.2.2 Sedimentation Control

Flotation tailings erosion translates into a sediment load in the RTP, thus specific sedimentation control measures are used to limit erosion:

- The lower slope of each shell is covered with green rock.
- The materials deposited on the DSTF are compacted as soon as possible.

2.2.3 Development Rock Characterization

Development rock is classified as "mineralized" if it contains >600 parts per million (ppm) arsenic or >0.5% sulfur. Mineralized development rock (red rock) is segregated for long-term storage because of the potential for generating acid rock drainage (ARD) and/or neutral arsenic leaching due to weathering. The *2020 Pogo Mine Monitoring Plan* provides detailed information regarding development rock segregation and tracking procedures.

It is assumed that development rock placed and compacted will have a dry in-place density of approximately 125 lb/ft^3. No geotechnical laboratory test was carried out using the development rock. The geotechnical characteristics of the development rock were estimated based on typical published values and engineering judgment for use in design.

2.2.4 Dust Control

Tailings have the potential to create dust, especially after they have been frozen and subsequently desiccated by the sun. Best management practices are used to control dust during dry stack operations such as compacting the tailings, controlling traffic on the compacted flotation tailings, and limiting the use of equipment to active placement area(s) only. Summer moisture from rainfall assists in keeping the surface moisture content within an acceptable range although prolonged periods of warm weather with low humidity may require additional controls.

3. CONSTRUCTION DESIGN CRITERIA

3.1 Placement Schedule

Table 1 shows the most recent placement schedule. The schedule is based on as-built survey data and the life of mine plan adopted in May 2021. Major assumptions used for placement estimates are as follows:

Assumed material dry densities:

Dewatered flotation tailings (compacted): 105 lb/ft^3 or 19.0 ft^3/ton; and

Waste rock (compacted): 125 lb/ft^3 or 16.0 ft^3/ton,

Tails in paste fill: 59.7 lb/ft^3 or 33.5 ft^3/ton at 63% solids and 7.2% cement,

Approximately 50% of green waste rock (35% of total waste rock) is utilized annually around the mine site (i.e., not at the DSTF) for road construction, underground projects, and where practicable.

Remaining DSTF volume and placement rates for tailings and waste rock are calculated monthly based on mill throughput data, haul truck load data, and/or WingtraOne Drone Surveys. Based on the most recent projections, it is estimated that the DSTF will reach the 20Mt permitted capacity in October 2023.

Approximately 4.9 Mt of additional compacted tailings and waste rock can be deposited on the DSTF GPA, EPZ, and Shell 1, 2 and 3 areas as an interim expansion while staying below the current diversion ditch elevations, allowing continued operations into approximately 2025 without further modification of the DSTF.

Table 1: Material Placement Schedule at the DSTF

Year	2021	2022	2023	2024	2025	2026
Production						
Ore Milled	994,611	1,343,966	1,366,903	1,449,987	1,449,974	1,449,988
Waste Rock Excavated	350,400	571,750	737,089	811,661	875,892	817,794
Tailings Backfilled in Paste	282,088	413,920	408,350	333,559	339,799	327,083
Material Placed at DSTF						
Tailings	704,329	896,004	936,716	1,116,428	1,110,175	1,122,905
Waste Rock	227,760	371,638	479,108	527,580	569,330	531,566
Total	932,089	1,267,641	1,415,824	1,644,008	1,679,505	1,654,471
Cumulative Tonnage at DSTF						
Tailings	704,329	1,600,333	2,537,049	3,653,477	4,763,652	5,886,557
Waste Rock	227,760	599,398	1,078,505	1,606,085	2,175,415	2,706,981
Total Material	932,089	2,199,730	3,615,554	5,259,562	6,939,067	8,593,538
Total in DSTF	16,997,932	18,265,573	19,681,397	21,325,405	23,004,910	24,659,381

3.2 Tailings Characterization

Several tests have been completed to characterize the DSTF tailings. **Table 2** summarizes the geotechnical properties for lab tests that have been conducted since 2009.

Table 2: Geotechnical Properties of Compacted Flotation Tailings

Parameters	Properties	Testing Method	Information Source
Maximum Dry Density	111 lb/ft^3	ASTM D698-12 Method A Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort	2021 Standard Proctor Tests
Optimum moisture	15.5 %	ASTM D698-12 Method A Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort	2021 Standard Proctor Tests
Shear Strength (Saturated)	Effective Friction Angle 34.4 degree(1) Cohesion - 63 psf	Triaxial Compression Test (CU- Test) (ASTM D-4767)	Golder Associates (2009)
Shear Strength (Saturated)	Effective Friction Angle 34.4 - 35 degree(2) Cohesion - 0.7 psf	Triaxial Compression Test (CU- Test) (ASTM D-4767)	SRK (2014)
Shear Strength (Drained)	Effective Friction Angle 35 degree(3)	Triaxial Compression Test (CU-Test) (ASTM D-4767)	AECOM (2019)
Direct Shear Strength (90% Compaction)	Friction Angle - 37 degree Cohesion – 140 psf	Direct Shear Test (ASTM D-3080)	2011 Compaction Test
Direct Shear Strength (95% Compaction)	Friction Angle - 39 degree Cohesion – 90 psf		
Direct Shear Strength (100% Compaction)	Friction Angle - 41 degree Cohesion – 60 psf		
Hydraulic Conductivity (saturated)	1E-07 m/s	Tri-axial Saturated Hydraulic Conductivity (ASTM D-5084-90) Flexible Wall Permeability (ASTM D-5084-Method C)	2011 Compaction Test

Specific Gravity	2.56	ASTM D854-06	2011 Compaction Test
Optimum Moisture Content	15% - 16%	Standard Proctor (ASTM D-698)	2011 Compaction Test

Notes:

1) Dry densities of specimens for triaxial tests were 101 – 102 pcf (93 – 94% of maximum dry density).
2) Triaxial testing indicated the following with respect to excess pore pressure generation in tailings (SRK, 2014b):
   i. For low confining pressures (near 5 psi) the samples under triaxial compression generally seemed to preserve volume with little to no contraction, dilation, or generation of excess pore pressure; and
   ii. At higher confining pressures (over 120 psi), the soil under triaxial compression generally showed an initial contractive behavior (i.e., increasing excess pore pressure) for axial deformations between 2% and 5%, with dilatant behavior (i.e., decreasing excess pore pressure) for higher deformations.
3) Drained friction angle from triaxial tests varied from 35 to 41 degrees. Used the lowest value (35 deg) for the analyses.

3.3 Structural Stability Evaluation

WSP has evaluated the structural stability of the interim expansion based on review of data from past AECOM, SRK, and AMEC reports and NSR monitoring. The stability analyses have reviewed the sensitivity of the DSTF to potential undrained strength response and liquefaction (strength loss) of the GPA materials, earthquake loading conditions, and an elevated phreatic surface.

3.3.1 Design Criteria

Stability analysis of embankment slopes requires assessment of the structure's ability to withstand the effects of self-weight (static) and earthquake loading conditions. Earthquake loading is evaluated using a pseudo-static analysis assuming that liquefaction of the GPA materials does not occur. An additional analysis is performed to evaluate the stability of the DSTF if liquefaction of those materials does occur. Limit-equilibrium analyses and the method of slices have been used to evaluate the stability of the DSTF under these conditions.

The minimum acceptable stability factor of safety under static loading conditions is 1.5 and under post-liquefaction conditions is 1.1 following guidance from ICOLD (2022). There are no design criteria related to maximum deformations of the DSTF; however, potential movements should not cause significant impact to the RTP.

3.3.2 Seismic Analysis Parameters

Seismic design criteria were developed for the Pogo site during completion of the project's Feasibility Study (Teck-Pogo, 2004) and reiterated in the RTP Dam Design Report (AMEC, 2004b). A magnitude M8.0 earthquake and a peak ground acceleration (PGA) of 0.2 g (i.e., 20% of acceleration due to gravity) for an event with a recurrence interval of 2,475 years represents the Maximum Design Earthquake (MDE) for the project (AMEC, 2004b). The current USGS Unified Hazard Tool for the site (Dynamic: Alaska 2007 (2.1.2) edition) indicates a PGA of 0.18 for the 2,475-recurrence interval event; however, a Magnitude 9.2 event is the largest contributor to the probabilistic earthquake hazard. Based on the relatively thin mantle of overburden soils above the bedrock, the PGA value that is based on a Site Class on the B/C boundary has not been adjusted.

3.3.3 Material Strength Parameters

AMEC (AMEC, 2004a) modelled the shells with moderate shear strength and GPA with no shear strength, whereas SRK (SRK, 2011a; SRK, 2014b) modelled the shells and GPA with moderate shear strengths due to operational compaction of GPA.

AMEC (AMEC, 2004a) reduced the laboratory-obtained shear strength (tangent of effective friction angle) by 20% for use in the slope stability analysis to simulate a "direct shear stress path". SRK (SRK, 2011a) utilized a 20% reduction in effective friction angle to evaluate sensitivity of the slope stability analysis to shear strength.

ADNR questioned the methodology for the shear strength reduction of AMEC (AMEC, 2004a) and considered the effective friction angle reduction of SRK (SRK, 2011a) to be arbitrary. In response to these concerns, NSR collected geotechnical parameters and samples from sonic boreholes drilled in the DSTF for laboratory index and shear strength test. In 2019, AECOM performed geotechnical field tests and laboratory tests on compacted tailings to provide up to date information on the bulk unit weight, saturated unit weight, and friction angle. In 2021, AECOM further reviewed the geotechnical design parameters based on additional moisture content data from the compaction testing and the 2020 Geotechnical Investigation.

To analyze the stability of the interim expansion, WSP generally used the material properties developed by AECOM. The shear strength (τ) of materials that respond to loading without developing excess pore pressures are defined using a Mohr-Coulomb strength model as function of the effective vertical stress (σ_v'): $\tau = \sigma_v' \times \tan(\phi')$, where ϕ' is the drained strength friction angle. The GPA tailings could behave in an undrained fashion even under relatively slow rates of loading; for these scenarios, undrained strength (s_u) is calculated as a ratio of the initial effective vertical stress. For seismic displacement and pseudo-static analysis, reduced shear strength values were considered for the tailings based on commonly recommended practice to reduce the strength of materials that are susceptible to development of excess pore pressures during cyclic loading. The potential for the GPA tailings to liquefy is currently uncertain; accordingly, a residual strength was estimated using engineering judgement based on the material type and typical standard penetration test blow counts for the tailings and modelled using the undrained strength ratio method. The bedrock has sufficient strength to not be a factor in the stability of the DSTF. Because the red rock in the EPZ and GPA is heterogeneously distributed, resulting in the potential for sliding surfaces to preferentially follow continuous paths through tailings, the EPZ and GPA are modelled as homogenous tailings. The proposed presence of red rock in the raise of Shell 2 and Shell 3 is not significant in terms of geotechnical stability, these raises are similarly modelled as homogeneous tailings for simplicity and because they could potentially be built only using compacted tailings.

***Table 4 stability analysis material properties.* Material**	**Unit Weight (pcf)**	**Strength Model(s)**
Compacted Tailings	127	$\phi' = 34°$
GPA Tailings	127	$\phi' = 34°$ $s_u/\sigma_v' = 0.4$ $s_{u-pseudo-static}/\sigma_v' = 0.32$ $s_{u-post-liquefaction}/\sigma_v' = 0.24$
Green Rock Shell	125	$\phi' = 38°$
Overburden Soil	125	$\phi' = 32°$
Starter Dam and Toe Berm	125	$\phi' = 32°$

Notes:

1) Moist unit weight of compacted tailings and GPA was obtained from the 2012, 2019, and 2020 investigation field and laboratory tests and the monitoring data over the years 2009-2012 and 2018-2019.
2) Drained friction angle of compacted tailings and GPA from triaxial tests performed on samples from 2012, 2019, and 2020 investigations varied predominantly from 34 to 38 degrees.
3) The shear strength parameters for GPA, Rock Shell, Flow-through Drain, Starter Berm and Toe Berm, Overburden, and Bedrock used a combination of measured field and laboratory data from 2012, 2019, and 2020 investigations and previous studies (AMEC 2004b; SRK 2014) along with published data on similar soils.

3.3.4 Phreatic Surface

One significant difference among stability analyses conducted by AMEC, SRK, and AECOM was the assumed phreatic surface:

1) AMEC (AMEC 2004a) assumed a phreatic surface 10 ft below the original ground surface;
2) SRK (SRK 2011b) performed a sensitivity analysis, using the AMEC phreatic surface, a phreatic surface at the original ground surface, and a phreatic surface within the DSTF up to 50 ft above the original ground surface.
3) SRK (SRK 2014b) assumed a phreatic surface based on the following observations:
    i. The SB-1 deep vibrating wire piezometer (VWP) has consistently reported positive pore pressures since shortly after installation in October 2012; pore pressures measured through October 2013 have ranged up to 6 psi, indicating a maximum recorded phreatic surface elevation of 2,317.5 ft. In addition, wet material was encountered in the bottom 5 ft of the SB-1 borehole during drilling in October 2012.
    ii. Water discharges from the flow-through drain at the toe of the DSTF; therefore, the phreatic surface was assumed to project from the measured elevation in SB-1 (at the starter berm) downgradient to the top of the flow-through drain at the DSTF toe.

iii. Water enters the flow-through drain upgradient of the DSTF. Furthermore, the deep VWP in GP-1 and RR-1 reported negative pore pressures or pore pressures near 1 psi. Therefore, the phreatic surface was assumed to project from the measured elevation in SB-1 upgradient to the flow-through drain and follow the top of the drain upgradient to the highest elevation on the DSTF section.

iv. Given these observations, the phreatic surface at SB-1 was set to 2,330 ft for this analysis, which corresponds to the crest of the starter berm (from data supplied by Pogo) and is approximately 12 ft higher than the maximum measured pore pressure in SB-1, as of October 22, 2013.

4) AECOM (AECOM 2019) used static water levels identified in the 2012 SRK investigation for the slope stability analyses during the 2019 and 2020 geotechnical investigations.

Given the uncertainty with the future location of the phreatic surface in the GPA materials, WSP's stability analyses for the interim raise consider a potential elevated surface in the lower portion of the DSTF.

3.3.5 Stability Analysis

SRK performed a slope stability analysis in 2014 using the computer program SLIDE (Version 5.026).

The results of the slope stability analysis are summarized in **Table 5** and show that the predicted stability of the critical cross-section satisfies the minimum allowable FoS for both static (1.5) and pseudo static (1.1) conditions. **Table 5** shows the lowest FoS resulting from the different material parameters listed in **Table 4** and seismic/excess pore pressure parameters. Results of the analysis show minimal sensitivity of the pseudo static model to vertical acceleration or excess pore pressure, i.e., less than 5% difference in FoS relative to scenarios with horizontal acceleration only and drained conditions (SRK 2014b).

Table 5: Results of DSTF Slope Stability Evaluations, SRK 2014

Section A-A'	Circular Failure Surface		Noncircular Failure Surface	
	FoS -Static	FoS - Seismic	FoS - Static	FoS -Seismic
Circular Failure	1.77	1.22	--	--
Block Failure Plane 1	--	--	2.40	1.72
Block Failure Plane 2	--	--	2.14	1.56
Block Failure Plane 3	--	--	2.02	1.47
Block Failure Plane 4	--	--	2.21	1.50

AECOM (AECOM 2019) completed updated stability analysis for the following configurations:

- 2019 condition
- 2019 condition on the bench and 20 MTon extents at the top of the DSTF

Stability analyses show that the calculated FS meets or exceeds the required FS for static and pseudo-static. The approach was consistent with AMEC (2004a) and SRK (2014). Results of the slope stability analyses are provided in **Table 6**.

Table 6: Results of DSTF Slope Stability Evaluations, AECOM 2019

Analysis Case	FoS -Static	FoS – Pseudo-static
2019 condition	1.82	1.29
2019 condition on the bench and 20 Mt extents at the top of the DSTF	1.82	1.27

Additional stability analyses were completed by AECOM for the following configurations, both drained and undrained:

- Current Permitted Capacity (20 Mt)
- Current Permitted Capacity (20 Mt) with reduced 20 ft Shell 1 non-mineralized layer

The results show that the stability analysis for all configurations meet or exceed the required FS for shallow and global.

Table 7: Results of DSTF Slope Stability Evaluations, AECOM 2020-2021

Analysis Case	Drained		Undrained	
	FoS -Shallow	FoS – Global	FoS - Shallow	FoS – Global
Current Permitted Capacity (20 Mt)	1.733[1]	2.443	1.733	2.093
Current Permitted Capacity (20 Mt) with reduced 20 ft Shell 1 non-mineralized layer	1.71[1]	2.34	1.59[1]	1.98

Notes:

1) Lowest reported FoSs for static and shallow yield coefficient for all three cases are at the toe of the DSTF.

WSP's analysis of the interim expansion condition resulting in FoS for static drained, static undrained, and post-liquefaction analyses of 1.7, 1.7, and 1.3, respectively. The estimated median seismic displacement of the DSTF is about 3 inches. The analyses demonstrate that the proposed limited raise of the DSTF meets or exceeds geotechnical engineering design criteria and remains safe and stable with no effect on the RTP.

3.3.6 Liquefaction Analysis

AMEC (2004) evaluated the liquefaction susceptibility of the foundation soils using Youd and Idriss's (1997) simplified procedure. SRK (SRK 2014b) also conducted a liquefaction analysis using the updated simplified procedure published by Youd *et al* (2001). The simplified procedure to evaluate the liquefaction resistance of soils requires two variables: (1) the seismic demand on a soil layer, termed the cyclic stress ratio (CSR); and (2) the capacity of the soil to resist liquefaction, termed the cyclic resistance ratio (CRR). The FoS against liquefaction can be obtained by dividing CRR by CSR. CSR is a function of peak horizontal acceleration at the ground surface, total vertical overburden stress, effective vertical overburden stress, and the sample depth.

The simplified procedures using standard penetrating test (SPT) data were adopted to determine CRR in the liquefaction analysis for the Pogo DSTF materials. The potential for liquefaction can exist only when loose, granular soil is saturated or close to being saturated. AMEC evaluated data from 36 SPTs completed across eight boreholes from 1998 and 2000 in the creek bottom and south facing slopes that tended to have more sand-type soils. The data was plotted on a chart from the Youd and Idriss (1997) report and were all in the area of the chart that indicates resistance to liquefaction. Among the soil samples collected from the three boreholes drilled in the 2012, only one sample was below the established water table and was therefore used for liquefaction analysis. The result of the SRK liquefaction analysis indicated a FoS of 2.3 against liquefaction. AMEC (2004) and SRK (2014b) concluded that liquefaction of the DSTF foundation materials during the Maximum Design Earthquake (MDE) is considered unlikely.

AECOM (AECOM 2019) conducted a liquefaction analysis for the DSTF tailings and overburden soils in 2019 using LiqSVs V.2.0.1.8 software. Seismic liquefaction of the DSTF tailings was performed on subsurface data from boring SB-1 and SB-03B-19, which was drilled in the 2019 geotechnical investigation. The subsurface data from SB-03B-19 between 75 and 100 ft bgs was used. Both seismic and static liquefaction of the DSTF tailings is considered unlikely. The potential for liquefaction of the underlying overburden soils was reviewed as part of evaluating the overall stability and safety of the DSTF. The overburden material information from SB-03B-19 below 110 feet bgs was used for static and seismic liquefaction review of the underlying overburden soils. Based on the available information, static liquefaction was deemed unlikely. Based on field observations, overburden samples are not likely prone to seismic liquefaction; however, AECOM recommended the seismic liquefaction analyses for the underlying overburden soils be updated with additional information.

The 2020 AECOM Geotechnical Investigation (AECOM 2020) included additional subsurface boreholes to analyze for both static and seismic types of liquefaction. It was determined that liquefaction of the DSTF as a result of an MDE is considered unlikely. There could be localized saturated zones of tailings susceptible to seismic liquefaction; however, these zones are isolated and contained and are not of concern with respect to liquefaction. Static liquefaction of the underlying overburden materials is also not likely to be a concern, because the materials are predominantly coarse grained and dense to very dense based on SPT N-value.

While liquefaction of the GPA tailings has been considered unlikely, WSP has evaluated the post-liquefaction stability of the DSTF to evaluate the sensitivity of the structure to this condition. As discussed in the previous section, the resulting FoS of 1.3 exceeded the design criteria value of 1.1.

Prior to completion of Shell 2 and 3, transient conditions exist where relatively minor deformation may occur from the Maximum Design Earthquake. Section 7 describes contingency plans for seismic events during construction and operation of the DSTF.

3.4 Compaction Testing

In order to evaluate the effectiveness of compaction and to establish appropriate compaction procedures, testing is routinely conducted. Major findings from the compaction testing conducted in March 2011 are summarized below. Additional information can be found in **APPENDIX II.**

Dewatered flotation tailings can be placed in the DSTF within the limits of both GPA and Shell during winter conditions once the appropriate construction procedures are consistently followed.

Adequate shear strength which exceeds the design criteria can be developed in the dewatered flotation tailings at 90% Standard Proctor compaction.

To achieve 90% Standard Proctor compaction effort during winter/freezing conditions, dewatered flotation tailings should be spread within three days of placement and compacted with a minimum of four passes using a 12-ton compactor.

4. CONSTRUCTION PROCEDURES

4.1 Top Portion of DSTF

Materials may be placed on the GPA year-round, but material placed in the shells and EPZ must meet QA/QC criteria that are difficult to accomplish during winter or extremely wet conditions. This section describes the construction procedures for the GPA, EPZ, and Shell 1 and associated structures.

4.1.1 Shell 1 Construction

Shell 1 has been constructed using non-mineralized rock since the commencement of operation. It was historically constructed with a 100 ft width of non-mineralized rock and a 100 ft layer of compacted tailings. Based on stability analyses completed by AECOM (AECOM, 2021), the rock width was decreased to a width of 20 ft and the tailings width to 50 ft. The shell is constructed on a 3:1 slope. Non-mineralized rock is dumped and spread into 3-ft loose lift. The lift is then compacted with three passes of a D7 Dozer. Shell 1 should be constructed during times of the year as described below in Section 4.2.1. Red rock cells may not be constructed in Shell 1.

A temporary single lane haul road may be constructed on the slope of Shell 1.

4.1.2 Embankment Placement Zone

The placement and compaction of flotation tailings and red rock in the EPZ follow the methods described in Section 4.1.4 and 4.1.5 The target geotechnical criteria for compacted tailings in the EPZ are described in Section 5.1.

4.1.3 Flow-through Drain and Perimeter Preparation

The flow-through drain along the creek will be extended upward as necessary. The specifications of the flow-through drain are described in Section 2.1.1.

4.1.4 Dewatered Flotation Tailings Placement

The trees, shrubs, and topsoil along the perimeter of DSTF are removed prior to placement of flotation tailings. The dewatered flotation tailings are dumped 15 ft apart, then spread into a maximum 12 in loose lift. For trafficability of the surface, the GPA will be compacted with methods similar to those used on the shells.

Operation during Winter Conditions

During winter season (October to May), some additional work may be required:

Windrows of dewatered floatation tailings must be spread and compacted within three days; and

The placement area needs to be regularly cleared to prevent build-up of snow and ice.

Operation in Wet Conditions

During rainy periods, the dewatered floatation tailings may become difficult to compact if water is allowed to infiltrate. In order to reduce the adverse effect on compaction, the following actions may be taken:

Keep tailings placement area as small as possible;

Prior to placement of tailings in this small area, the saturated and softened surface will be scraped off;

If the tailings cannot be compacted immediately, then they will not be spread at all, but left in a pile. If the tailings remain in a pile, the rain will generally only penetrate the outer shell of the pile;

Tailings with moisture contents that exceeds three percent of optimum may be placed in the GPA in discreet cells within compacted tailings or used to cover red rock cells, and

Once dewatered floatation tailings placement in the area is complete, the tailings surface will be smooth, free of water traps, and graded to allow water to run off the surface.

4.1.5 Mineralized Rock Placement

Mineralized (red) rock cells may be placed in shell zones, EPZ and GPA any time under the offset and encapsulation criteria as described here. Mineralized rock requires encapsulation in the dewatered flotation tailings with the following procedures applied:

The mineralized rock may not be placed within 50 ft from the perimeter of DSTF,

The mineralized rock is placed in piles and spread into 3-foot loose lifts and compacted; and

Once three lifts are placed, the mineralized rock will be covered with two one-foot thick lifts of compacted, dewatered flotation tailings before placing another lift of mineralized rock. In the GPA, mineralized rock cells may be covered with a single, minimum two-foot layer of high-moisture content tailings.

4.2 Shell 2 and 3 Area

This section describes the construction procedures for Shell 2 and Shell 3.

4.2.1 Construction Period

Shell construction should preferentially occur and be prioritized during the warmer months of the year. Placement during freezing conditions should be considered on a case by case basis and specifically approved by the design engineer. Development rock may generally be placeable during freezing conditions provided that the fines content (dry weight passing the US No. 200 sieve) is less than 5%, fill temperatures remain above 30 degrees, fill is placed expeditiously and exposure of uncompacted fill to freezing temperatures is limited, snow, ice, or frozen materials are not entrained into the fill, and fill subgrades have not been impacted by snow and/or ice. Current and forecasted temperatures should be considered when depositing windrows and scheduling dozing and compaction.

4.2.2 Flow-Through Drain and Toe Berm

The flow-through drain and toe berm for the Shell 2 and Shell 3 have already been constructed. In case an additional shell will be constructed, the flow-through drain and toe berm will be extended. The specifications of the flow-through drain are described in Section 2.1.1.

The toe berm is constructed using non-mineralized rock and acts as a foundation for the shell.

4.2.3 Shell 2/3 Construction Procedures

Shell 2 and Shell 3 are composite shells consisting of compacted dewatered flotation tailings, mineralized rock entombed within the tailings, and non-mineralized rock placed on the slope surface of the shells. These shells serve as limited elevation buttresses on the outward slope of Shell 1. Shell 2 and Shell 3 will typically be raised concurrently as a Shell 2/3 monolith. The construction procedures for Shell 2/3 are as follows:

A 20 ft layer with 3:1 slope of non-mineralized rock is used as the outside layer of the Shell 3 to limit erosion. The non-mineralized rock in Shell 2 is not extended into the Shell 2/3 monolith. Non-mineralized rock is dumped on the slope side of the shells and then spread into 3-ft loose lift. Compaction then proceeds with a minimum of three passes of a D7 dozer. Non-mineralized rock may be reclaimed from the face of Shell 1 in areas that will be backfilled with compacted floatation tailings. Excavations into the face of Shell 1 shall be no longer than 50 ft in length and 20 ft into the shell face (horizontally) and slopes shall be maintained shallower than the angle of repose. If multiple excavations are concurrently developed, they shall be spaced at least 50 ft apart. Excavations must be backfilled with compacted flotation tailings before seasonal freezing conditions occur.

The dewatered flotation tailings are placed on the interior of the non-mineralized rock crest. They are placed 15-ft apart within the crest, and then spread into maximum 12-inch loose lift. Compaction then proceeds with a minimum of six passes of a smooth drum roller having a minimum 12-ton equivalent weight. Though adequate shear strength can be developed in the dewatered flotation tailings with a minimum of four passes compaction, six passes compaction is applied for Shell construction to minimize the variability of operation. The target geotechnical criteria for compacted tailings in Shell 2/3 are described in Section 5.1.

Red rock may be placed in Shell2/3 as described in Section 4.1.5. Red rock must be fully encapsulated by tailings and may not contact zones of green rock to avoid hydraulic connection.

The total combined width of Shell 2 and Shell 3 is at least 330 ft and will be wider where green rock is reclaimed from Shell 1.

Operation in Wet Conditions

During rainy periods, the dewatered flotation tailings and non-mineralized rock may become difficult to compact to achieve the target density. In order to minimize the adverse effect on compaction, the following actions may be taken:

Prior to placement of dewatered floatation tailings, the saturated and softened surface will be scraped off;

Windrows of dewatered flotation tailings and non-mineralized rock have to be dozed down and compacted as soon as possible; and

If the amount of rainfall begins to reach extreme levels (more than 0.5 inches in 24 hours), placement of dewatered flotation tailings in the shell area may be suspended.

5. MONITORING

5.1 Geotechnical Monitoring

The compaction of dewatered flotation tailings at the shells, EPZ, and GPA is important for overall stability of the DSTF and to provide the required volume capacity. The construction procedures for the GPA and shells aim to compact the dewatered flotation tailings to achieve a nominal 90% Standard Proctor of the dry density to secure the designed shear strength.

The QA/QC program for the DSTF is shown in **Table 8**. The location of nuclear densometer readings and grab samples are documented using the Troxler E-Gauge GPS, and included with the data collected for the QC program.

Table 8: Geotechnical Monitoring Items

QA/ QC	Test Description	ASTM Method	Test Frequency	Test Procedures	Target
Quality Control Program	In-Place Density & Water Content	D6938/ D7698 or D2937	Each lift and at least 1 test per 2,000 cubic yards	Performed before placement of any subsequent lift of material. Tests should be performed on the DSTF tailings and evenly spaced at a minimum of 50 feet. The tests should be performed to the full depth of the lift. Standard Proctor test results.	Minimum 90% of Standard Proctor maximum dry density
Quality Assurance Program [1]	In-Place Density & Water Content	D6938	Every 6 months (once during summer conditions and once during winter conditions)	Seven tests should be performed on the DSTF tailings and evenly spaced . The tests should be performed to the full depth of the lift.	Minimum 90% of Standard Proctor maximum dry density
	Standard Proctor	D698		Perform three tests. Grab samples should be collected from at least three equally spaced locations from each test area.	N/A
	Moisture Content	D2216		Perform three tests. Grab samples should be collected from at least three equally spaced locations from each test area.	N/A

	Grain Size Distribution	D422-63(2007)$^{\epsilon 2}$		Perform three tests. Grab samples should be collected from at least three equally spaced locations from each test area.	Verify Tailings Consistency

(1) If QC is performed by a third party testing firm, repeating tests for QA is not necessary, however the QC firm must also complete the three laboratory tests.

5.2 Drone Surveys

A detailed drone survey of the DSTF will be conducted quarterly when the ground is not covered in snow. The survey should document elevation and horizontal extent at each end of the front of the working area, as well as the intersection of the DSTF with the North and South forks of Liese Creek.

5.3 Vibrating Wire Piezometers

In October of 2012, a subsurface investigation of the DSTF was performed to evaluate the geotechnical, thermal, hydrogeological, and geochemical characteristics of the facility (SRK, 2014c). Three sonic boreholes (SB-1, GP-1, and RR-1) were vertically drilled in the following locations:

Immediately up-gradient of the starter berm (SB-1),

In a portion of the GPA where tailings was expected to comprise a significant fraction of the stratigraphy (GP-1); and,

In a portion of the GPA where mineralized red rock was expected to comprise a significant portion of the stratigraphy (RR-1).

In October of 2019, AECOM performed a geotechnical investigation on the DSTF that included installation of two more piezometers (AECOM, 2019). These piezometers are in the following locations:

- Immediately up-gradient of the non-mineralized rock shell between GP-1 and RR-1 (SB-02A-19)
- In the center of Shell 2 (SB-05-19)

An additional geotechnical investigation was completed in June 2020 that included installation of three piezometers and one thermistor (AECOM, 2020). All instrumentation was upgraded to a wireless monitoring system in 2020. These instruments are in the following locations:

General placement area (SB-01-20, SB-03-20)

In front of the Drytoe into native overburden (SB-04-20 and SB-04-20 Thermistor)

RST vibrating wire piezometers (VWP) are installed in each borehole to evaluate the presence and extent of saturated zones within the DSTF and to monitor changes in pore pressure. DSTF temperatures are also measured using thermistors located within each VWP sensor. An additional five sensor thermistor is installed downstream of the Drytoe. The installation depth of each sensor is shown in **Figure 5** and presented in **Table 9**.

Instrument data is downloaded at least quarterly, and datalogger batteries and signal are checked at this time.

Table 9: Summary of Vibrating Wire Piezometer Installation

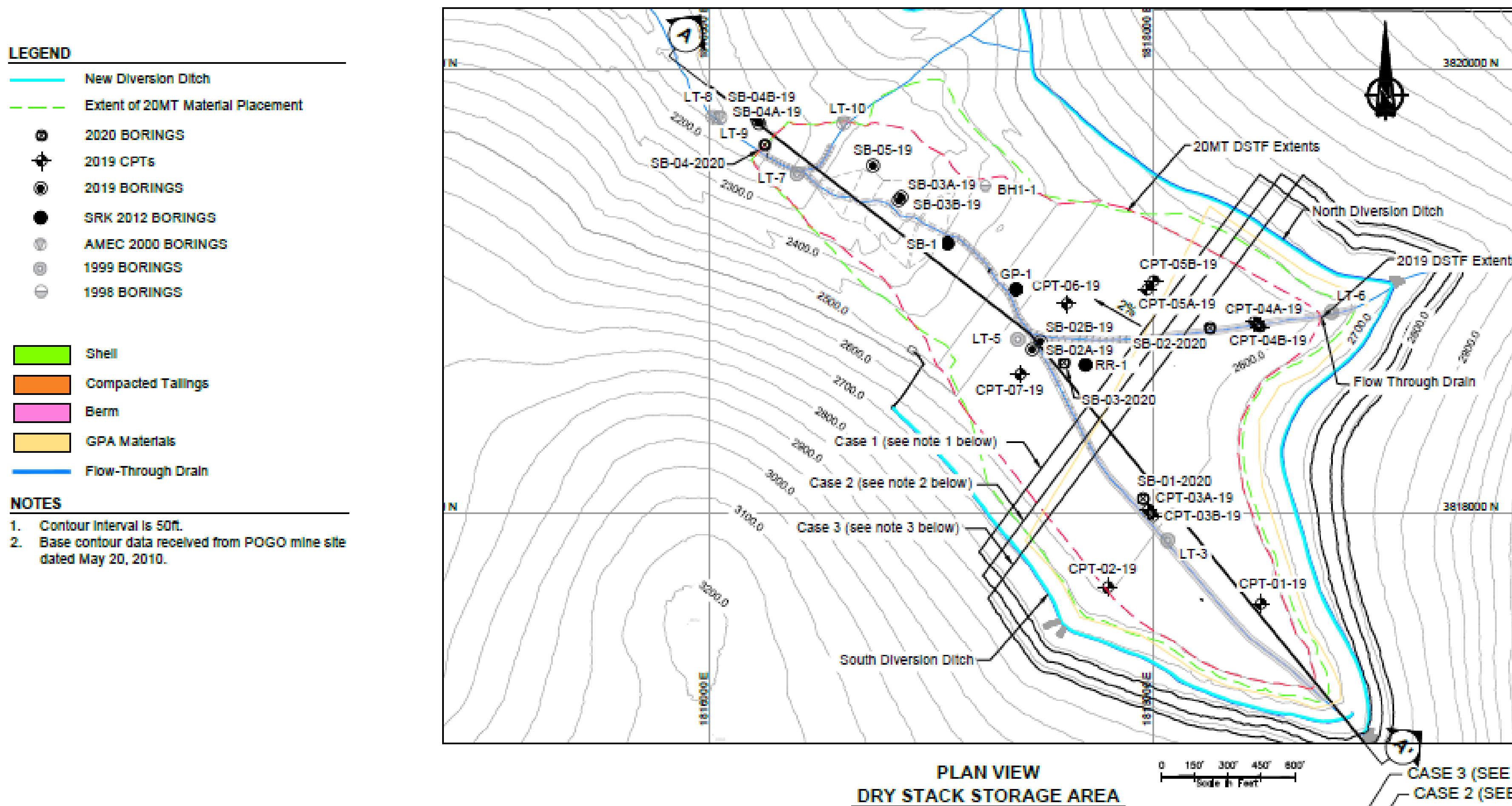
Drill Hole	Instrument ID	Description	Depth from Collar (ft)	Elevation (ft)
SB1	SB1-VW2	Shallow	25	2383
	SB1-VW1	Deep	104.5	2303.5
GP1	GP1-VW1	Shallow	63	2423
	GP2-VW2	Deep	137	2349
RR1	RR1-VW5	Shallow	2	2507
	RR1-VW3	Mid	61	2448
	RR1-VW4	Deep	94	2415
SB-02B-19	SB-02B-19 Piezo	N/A	43	2521
SB-05-19	SB-05-19 Piezo	N/A	75	2277
SB-01A-2020	SB-01-2020 Piezo	N/A	173	2454
SB-02A-2020	SB-02-2020 Piezo	N/A	120	2480

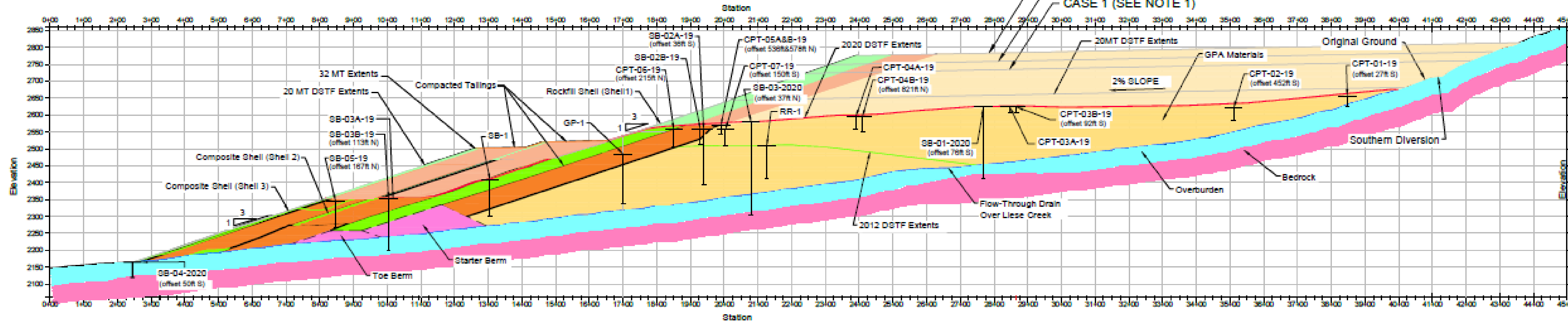
SB-03-2020	SB-03-2020 Piezo	N/A	105	2482
SB-04-2020	SB-04-2020 Piezo	N/A	35	2149
	SB-04-2020 Thermistor	5 sensors	0-38	2146-2184

5.4 Reporting

The results of the monitoring described in this section will be compiled and retained for future reviews and permitting as required.

Figure 5: As-Built Sonic Borehole and Vibrating Wire Piezometer Locations (AECOM, 2020)





2020 NORTHERN STAR GEOTECHNICAL BORING AS BUILT LOCATIONS

Boring Id	Easting/X	Northing/Y	Ground Elevation (ft)	Boring Depth (ft)	Bottom Elevation (ft)
SB-01A-20 [1]	1818002.458	3817949.762	2626.8	210.0	2416.8
SB-01B-20	1817997.458	3817949.762	2626.76	24.5	2602.3
SB-02A-20 [1]	1818261.416	3818832.732	2599.8	165.0	2434.8
SB-02B-20	1818256.416	3818832.732	2599.7	31.5	2568.25
SB-03-20 [1]	1817617.633	3818686.142	2586.7	265.0	2321.7
SB-04-20 [1]	1816248.981	3819663.413	2183.9	45.0	2138.9

2019 AECOM GEOTECHNICAL CPT & BORING AS BUILT LOCATIONS

CPT/Boring Id	Easting/X	Northing/Y	Ground Elevation (ft)	Boring/CPT Depth (ft)	Bottom Elevation (ft)
CPT-01-19	1818486.335	3817590.694	2657.2	40.0	2617.2
CPT-02-19	1817796.384	3817665.285	2621.6	35.0	2586.6
CPT-03A-19	1817988.325	3818002.223	2626.5	20.9	2605.7
CPT-03B-19	1817999.719	3817985.786	2626.5	20.6	2605.9
CPT-04A-19	1818470.647	3818857.074	2596.3	36.0	2560.3
CPT-04B-19 [2]	1818482.251	3818840.784	2596.3	57.0	2539.3
CPT-05A-19	1817973.776	3819008.832	2573.9	26.6	2547.3
CPT-05B-19	1818003.983	3819049.469	2570.4	27.0	2543.4
CPT-06-19	1817610.974	3818945.597	2556.9	49.3	2507.7
CPT-07-19	1817401.027	3818626.457	2556.9	49.6	2507.3
SB-02A-19 [2]	1817454.703	3818738.623	2557.8	161.5	2396.3
SB-02B-19	1817491.640	3818774.174	2557.8	45.0	2512.8
SB-03A-19	1816850.759	3819411.385	2352.8	29.0	2323.8
SB-03B-19	1816848.946	3819409.351	2352.8	151.0	2201.8
SB-04A-19	1816222.228	3819758.826	2187.9	12.5	2175.4
SB-04B-19	1816230.024	3819759.475	2197.9	33.0	2164.9
SB-05-19	1816736.496	3819568.109	2345.2	76.5	2268.7

NOTE:
1. PIEZOMETER & THERMISTOR INSTALLED. SEE 2020 GEOTECHNICAL REPORT FOR DETAILS.
2. PIEZOMETER INSTALLED. SEE 2019 GEOTECHNICAL REPORT FOR DETAILS.

2012 SRK GEOTECHNICAL BORING AS BUILT LOCATIONS

CPT/Boring Id	Easting/X	Northing/Y	Ground Elevation (ft)	Boring/CPT Depth (ft)	Bottom Elevation (ft)
SB-1	3819217.00	1817073.00	2408.0	106.5	2301.5
GP-1	3819010.00	1817382.00	2486.0	147.0	2301.5
RR-1	3818869.00	1817698.00	2509.0	97.0	2412.0

NOTES
1. Case 1: Below target expansion of 32 MTon Extent (approximately 75 feet vertical offset from 20 MTon DSTF Extent)
2. Case 2: Target expansion of 32 MTon Extent (approximately 100 feet vertical offset from 20 MTon DSTF Extent)
3. Case 3: Above target expansion of 32 MTon Extent (approximately125 feet vertical offset from 20 MTon DSTF Extent)

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FILE NAME: 2020 DSTF Geotech Borings.dwg

NORTHERN STAR RESOURCES LIMITED

Dry Stack Tailings Facility
2020 Geotech Investigation

DATE:	APPROVED:	DRAWING:
Nov 2020	MS	03

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6. INSPECTION

6.1 Weekly Inspection

Environmental personnel will conduct visual inspection of the DSTF on a weekly basis. Environmental personnel will look for any unusual physical conditions paying particular attention to:

- Any ponding of water on DSTF,
- Water flow into and out of the DSTF,
- Evidence of deformation on the slope of the shell; and
- Evidence of excessive erosion or seepage of the slope of the shell.

The results of inspections will be documented using the designated form (see **APPENDIX I**). If any unusual situation is found, it will be reported to the Maintenance and Environmental Managers. The Environmental Department retains records for monitoring activities described in this document. Data are retained in the environmental G:/ drive under the monitoring subfolders, or in Pogo's INX InControl database.

6.2 Daily Inspection

Surface personnel conduct a visual inspection of the DSTF on a daily basis. Surface operators check for unusual cracks, bulging, signs of settlement, seepage, erosion, and wildlife interaction. The results of these inspections are recorded in the Dry Stack Daily Inspection Log on NSR's server.

6.3 Upset Condition Inspection

The DSTF will be inspected by Environmental personnel after extreme rainfall (two inches within 24 hours) or an appreciable earthquake (reported in the area or felt by site personnel). A contingency plan for a seismic event is included in Section 7.

6.4 Diversion Ditch Inspection

The North and South Diversion ditches are inspected monthly when conditions allow (See **APPENDIX I** for inspection form). In addition to the monthly inspection, Environmental personnel will walk the ditches to look for failures annually. This inspection will take place in early summer.

7. CONTINGENCY PLAN

The stability of the DSTF was evaluated by WSP for the MDE. Portions of the DSTF may deform up to 4.5 feet under the MDE prior to completion of Shell 2/3 to the design configuration, however, no adverse consequences are expected under this low-probability event. In the case of a seismic event that is felt on site, the DSTF will be visually inspected including the GPA, the exposed slopes and shell, and the Drytoe. If any displacement is apparent, the extent of the deformation will be reviewed for consistency with the design parameters. Any deviations from design parameters will be reviewed with a qualified engineer, and a specific mitigation plan will be developed.

8. REFERENCES

AMEC, 2004a, Drystack Tailings Facility Geotechnical Design Report.

AMEC, 2004b, RTP Dam Design Report.

Teck-Pogo, 2004, Pogo Project Final Feasibility Study

AMEC, 2006, RTP Dam 2004-2005 As-built Report

AMEC, 2007, Pogo Mine Drystack Tailings Facility OMS Manual – Revision Two.

SRK, 2011a, Pogo Drystack Tailings Facility Expansion Preliminary Study.

SRK, 2011b, Pogo Mine – Findings of Winter Field Program and Preliminary Recommendations for Dry Stack Storage Facility Construction and QA/QC Procedures.

SRK, 2012, Dry Stack Tailings Facility (DSTF) Expansion Detailed Design.

Pogo, 2018, Pogo Mine Monitoring Plan.

SRK, 2013a, South Diversion Flume, As-Built Hydraulic Capacity.

SRK, 2013b, DSTF Diversion Ditches Design Calculations

SRK, 2014a, Dry Stack Tailings Facility Expansion – Final As-Built Report for Diversion Channels.

SRK, 2014b, DSTF Closure Study – Slope Stability Analysis.

SRK, 2014c, DSTF Closure Study – DSTF Vibrating Wire Piezometers.

AECOM, 2019, 2019 Geotechnical Investigation Temporary Ore Storage and Future Expansion Dry Stack Tailings Facility, Pogo Mine, Alaska

AECOM, 2020, 2020 Geotechnical Investigation, Dry Stack Tailings Facility, Pogo Mine, Alaska

AECOM, 2021, 2021 Geotechnical Review Report, Dry Stack Tailings Facility, Pogo Mine, Alaska

International Commission on Large Dams (ICOLD), 2022, Tailings Dam Safety, Bulleting No. 194, Version 1.0.

Youd, T.L. and Idriss, I.M. ed. (1997). Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. National Center for Earthquake Engineering Research Technical Report NCEER-97-0022.

Youd T.L., Idriss I.M., Andrus R.D., Arango I., Castro G., Christian J.T., Dobry R., Finn W.D.L., Harder Jr L.F., Hynes M.E., Ishihara K., Koester J.P., Liao S.S.C., Marcuson III W.F., Martin G.R., Mitchell J.K., Moriwaki Y., Power M.S., Robertson P.K., Seed R.B., Stokoe II K.H.. Liquefaction resistance of soils: summary report from the 1996 NCEER and1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J Geotechn Geoenviron Engng, ASCE 2001;127(10):817–33

9. RELATED DOCUMENTS

Document Name	Document Number
DSTF Density Testing	PGO-ENV-003-SWP
DSTF Tailing and Rock Placement	PGO-ENV-025-SWP
DSTF Piezometer Data Downloading and Compiling Manual	PGO-ENV-002-SWP
Pogo Mine Monitoring Plan	PGO-ENV-011-PLA
Recycled Tailings Pond Operating and Maintenance Manual	PGO-ENV-008-MAN

10. APPENDICES

APPENDIX I – Weekly Inspection Form
APPENDIX II – Compaction Test March 2011

10.1 APPENDIX I – Weekly and Monthly Inspection Forms (INX)

Reference No: 294151

Inspections - Pogo Checklist

PGO - ENV - RTP Dam & Dry Stack Weekly Inspection - PGO - ENV - RTP Dam & Dry Stack Weekly Inspection

Prompt	Yes	No	N/A	Explanation	Comments
Date of inspection:	☐	☐	☐		
Seepage Collection Wells	☐	☐	☐	Are all pumps running in Auto Mode? Do the well motor speeds and water levels indicate that the wells are working properly? Are lights (emergency and standard) functional?	
RTP Dam	☐	☐	☐	Are dam faces free of vegetation, erosion, collapse, subsidence? Is downstream dam free of seepage? Is dam crest free of subsidence and damage to facilities? Are reservoir walls free of erosion and collapse? Are dam abutments (north and south) free of erosion and seepage?	
Spillway Inlet (Concrete) and Outfall (Flume)	☐	☐	☐	Is spillway inlet (concrete) free of new cracks and properly connected to flume (culvert)? Are existing cracks stable? Have any new cracks formed? Is spillway outfall (flume) free of damage, obstacles and erosion on the ground? Are spillway abutments (north and south) free of erosion and seepage?	
Drystack	☐	☐	☐	is the dry stack free of unusual cracks and signs of settlement? is the dry stack free of bulging and seepage? Is the dry stack free of erosion, rills, and gullies? Are 2% slopes being maintained?	
Describe and document any maintenance activities completed in response to deficiencies noted in previous inspections.	☐	☐	☐	Notes:	
Any unusual events? Describe and document dam performance. (Seismic, weather, etc.)	☐	☐	☐	Notes:	

22/04/2023 6:31:22AM   InControl - Event Checklist   Page 1 of 1

Reference No: 273312

Inspections - Pogo Checklist

PGO - ENV - Liese Creek Flume Monthly Inspection - PGO - ENV - Liese Creek Flume Monthly Inspection

Prompt	Yes	No	N/A	Explanation	Comments
Date of inspection:	☐	☐	☐		
Diversion Ditch Headwall / Sump - North & South	☐	☐	☐	Does the sump need sediment removed? Are there any operational issues present (excessive vegetation, erosion, overflow, blockage, channel migration away from the headwall / sump)? Any maintenance required?	
Diversion Ditches - North, South (Upper), South (Lower)	☐	☐	☐	Are Diversion ditches free of obstacles and damage? Are diversion ditches free of erosion, sediment accumulation, aufeis, obstacles, and damage?	
Flume #1 Dry stack Toe	☐	☐	☐	Has debris / sediment been cleared from the flume? Is the flume free of erosion and settling? Has the stilling well been flushed? Download / calibrate data logger. Inspect desiccant for data logger, replace if pink. Note the manual weir reading in the comments.	
Flume #2 South Diversion Ditch Return (below Seepage Collection Wells)	☐	☐	☐	Has debris / sediment been cleared from the flume? Is the flume free of erosion and settling? Has the stilling well been flushed? Download / calibrate data logger. Inspect desiccant for data logger, replace if pink. Note the manual weir reading in the comments.	
Flume #3 North Diversion Ditch Return	☐	☐	☐	Has debris / sediment been cleared from the flume? Is the flume free of erosion and settling? Has the stilling well been flushed? Download / calibrate data logger. Inspect desiccant for data logger, replace if pink. Note the manual weir reading in the comments.	
Flume #4 Liese Creek (Rd. 7)	☐	☐	☐	Has debris / sediment been cleared from the flume? Is the flume free of erosion and settling? Has the stilling well been flushed? Download / calibrate data logger. Inspect desiccant for data logger, replace if pink. Note the manual weir reading in the comments.	

Prompt	Yes	No	N/A	Explanation	Comments
Describe and document any maintenance activities completed in response to deficiencies noted in previous inspections.				Notes:	
Any unusual events? Describe and document flumes performance during unusual events. (Seismic, weather, etc.)				Notes:	

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10.2 APPENDIX II – Compaction Test March 2011

The previous DSTF OMS Manual describes that "windrows of tailings have to be dozed down and spread within 1 hour" during winter conditions. However, it is not practical to implement this rule.

In order to evaluate the influence of frozen dewatered flotation tailings on the compaction and to establish appropriate compaction procedures during winter season, a compaction test was conducted in March 2011. A technical memorandum was provided by SRK (SRK, 2011b). This section summarizes the results of this test.

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Methodology

Four different scenarios were tested on site to assess the potential impact of time lags between the dumping of tailings material into heaps on the surface of the DSTF and subsequent spreading of that material under freezing conditions. The four-time lags tested were 1, 2, 3, and 7 days between the time tailings were dumped on the surface of the DSTF and when material was spread into one-foot thick lifts and then compacted with a vibratory roller. Air temperature measured during the test period was between -9- and 27-degrees F.

At each site when the specified time had elapsed dumped materials were spread using a CAT D7 track type dozer to create a one-foot thick lift that was approximately 30 ft by 60 ft. Each pad was then subjected to three different of compaction passes (four, six and eight passes) with a CAT CS 563 vibratory compactor (approximately 12 tons operating weight).

The following field measurements and laboratory tests were conducted:

Soil temperature measurements using a handheld infrared gauge;

In-situ density and water content measurements using nuclear densometer (ASTM D6938-10),

Sand cone test (ASTM D1556-07),

Standard Proctor (ASTM D698-07),

Moisture content (ASTM D2216); and

Direct shear test (ASTM D3080).

Results

Soil Temperatures and Frost Penetration

Table 11, Summary of Soil Temperatures of Dumped Tailings Piles summarizes the soil temperature recorded on site. Measured soil temperatures indicate increased frost penetration depth with increased exposure time to freezing conditions. Frost penetration depth ranged from approximately 3 inches from the surface of dumped tailings piles after one day exposure to depths in excess of 3 ft in material heaped for the seven-day test. After seven days it is estimated that up to two-thirds (by volume) of tailings material dumped is frozen.

Table 11: Summary of Soil Temperature of Dumped Tailings Piles

Trial	Surface Temp (°F)	3' Depth Temp (°F)	5' Depth Temp (°F)
1 Day Trial	31	72	n/a
2 Day Trial	15	36	n/a
3 Day Trial	10	35	42
7 Day Trial	7	30	n/a(1)

Note: *(1) Completely frozen at depth and unable to excavate for temperature measurement.*

Material Properties and Field Density Measurements

Table 12, Laboratory Test Results-Material Properties, summarizes the material properties of tailings material placed during the test program. The results show the specific gravity and Standard Proctor values are very consistent and indicative of a well-controlled process in which the filtered tailings are produced. Moisture content results near the surface of dumped tailings steadily decreased with increased exposure time.

Table 13, Field Density Measurements, summarizes field density testing results from the nuclear densometer. It indicates a general trend of increasing in situ density as the number of compaction passes increased. Nuclear densometer results also show that compacted density achieved tended to decrease with increasing exposure time. Table 8 shows that the heaps exposed three days or less meet 90% Standard Proctor with a minimum four compaction passes, and one day and two days duration heaps meet 95% Standard Proctor with a minimum six compaction passes.

Table 12: Laboratory Tests Results – Material Properties

Trial	Moisture Content			Specific Gravity	Standard Proctor	
	Surface	6" below surface	3' below surface		Maximum Dry Density (pcf)	Optimum Moisture Content (%)
1 Day	17.9	n/a	17.9	2.56	109.3	15.0
2 Days	20.2	n/a	17.7	2.56	109.3	15.3
3 Days	13.9	16.5	17.2	2.54	109.3	15.7
7 Days	10.5	19.7	16.8	2.55	107.9	16.3

Table 13: Field Density Measurements

Duration of Pile Exposure	Compaction Effort Trial	Nuclear Densometer		% to Maximum Dry Density
		Density (pcf)	Moisture (%)	
1 Day	4 Passes	102.0	16.2	93.3
	6 Passes	105.4	15.4	96.4
	8 Passes	105.1	16.7	96.2
2 Days	4 Passes	102.3	16.8	93.6
	6 Passes	103.7	16.1	94.9
	8 Passes	106.4	16.7	97.3
3 Days	4 Passes	98.4	16.8	90.0
	6 Passes	100.6	16.9	92.0
	8 Passes	102.7	17.1	94.0
7 Days	4 Passes	90.0	15.5	83.4
	6 Passes	87.8	15.3	81.4
	8 Passes	86.4	15.6	80.1

Shear Strength

Table 14, Summary of Direct Shear Results, shows the results of direct shear tests. The tests were completed on remoulded samples compacted to 90, 95, and 100% Standard Proctor compaction effort. The laboratory results showed a general increase in material friction angle along with compaction effort, and adequate shear strength can be developed in the dewatered flotation tailings at 90% Standard Proctor compaction in comparison with the design criteria of 32 degree in friction angle of dewatered flotation tailings.

Table 14: Summary of Direct Shear Results

Sample Compaction Effort	Average Dry Density of Specimen (pcf)	Average Cohesion (psf)	Average Friction Angle (degree)
90%	99.0	140	37
95%	105.1	90	39
100%	109.9	60	41

Major Findings from Compaction Test in March 2011

This section summarizes the major findings obtained from the compaction test conducted in March 2011.

Dewatered flotation tailings can be placed in the DSTF within the limits of both GPA and Shell during winter conditions once the appropriate construction procedures are consistently followed.

Adequate shear strength which exceeds the design criteria can be developed in the dewatered flotation tailings at 90% Standard Proctor compaction.

To achieve 90% Standard Proctor compaction effort during winter/freezing conditions, dewatered flotation tailings should be spread within three days of placement and compacted with a minimum of four passes using a 12-ton compactor.