Pogo DSTF Construction and Maintenance Plan

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Revision 5: January 2017
# Pogo Drystack Tailings Facility Construction and Maintenance Plan Revisions

<table>
<thead>
<tr>
<th>Revision #</th>
<th>Date</th>
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<th>By</th>
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</thead>
<tbody>
<tr>
<td>3</td>
<td>November 2011</td>
<td>This document replaced DSTF OMS Manual Revision 2 prepared by AMEC in December 2007</td>
<td>Pogo</td>
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<tr>
<td>4</td>
<td>May 2014</td>
<td>Update documents including the information on new diversion ditch construction as-built and DSTF closure study</td>
<td>Pogo</td>
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<tr>
<td>5</td>
<td>January 2017</td>
<td>Update with current 2017-2022 LOM Forecast.</td>
<td>Pogo</td>
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</table>
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1.0 INTRODUCTION

1.1 Objective

Sumitomo Metal Mining Pogo LLC (Pogo) is the operator of the Pogo gold mine, located 38 miles northeast of Delta Junction, Alaska.

The Dry Stack Tailings Facility (DSTF) has been in operation since February 2006. As of end of 2016, about 10.9 million tons (Mt) of material was placed at DSTF, which included 6.9 Mt tons of flotation tailing and 4.0 Mt of waste rock. The capacity of the original facility was estimated to be about 7.5 Mt, and was expanded to 20 Mt by constructing new diversion ditch in September 2013.

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. Subsequently, it was revised and issued as revision two in December 2007 (AMEC, 2007). Pogo updated the OMS Manual and issued as Construction and Maintenance Plan (“Plan”) in November 2011, reflecting the information from DSTF Expansion Preliminary Study (SRK, 2011a) and the field compaction test conducted in March 2011 (SRK, 2011b). Pogo updated the Plan in May 2014 including the as-built design of new diversion ditch (SRK, 2014a), updated stability evaluation (SRK, 2014b), and the DSTF year-by-year plan based on the draft life of mine plan as of end of year 2013. The January 2017 revision updates the Material Placement Schedule in accordance with the current 2016 Life of Mine forecast.

This Plan provides practical steps to construct and maintain the DSTF as designed. It should be noted that the water quality, hydrology, and geochemical monitoring plans were omitted from this Plan and is described in the Pogo Mine Monitoring Plan (Pogo, 2017).
1.2 Document Control and Responsibility

The Environmental Manager is responsible for the preparation and administration of this Plan. Any revisions or updates to the Plan shall be submitted to Alaska Department of Natural Resources (ADNR).

The Maintenance Manager is responsible for the construction of the DSTF. The site specific Standard Operating Procedure (SOP) will be established in accordance with this Plan and will be informed to all relevant personnel.

The Environmental Manager is responsible to implement the monitoring and inspection required by this Plan, and to report to the relevant agencies.
2.0 FACILITY DESCRIPTIONS

2.1 Major Components

Figure 1 shows the plan view of the DSTF as of September 2013. The major components of DSTF include:

- Flow-Through Drains;
- Starter Berm and Toe Berm;
- Shell Area;
- General Placement Area (GPA); and,
- Diversion Ditch.

2.1.1 Flow-Through Drains

All runoff in and around the DSTF 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 of the existing stream as the elevation of GPA rises.

Figure 2 shows 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 a filter material to prevent fines migrating in from the dewatered flotation tailings. The rock fill is placed at about 1H:1V, resulting in a drain base width of 21 ft, crest width of 9 ft and height of 6 ft.

The filter of flow-through drain consists of two layers: Filter 1 and Filter 2. The sand (0.04 inch to 0.2 inch in size) should be used for Filter 1, and the gravel (0.2 inch to 4 inch in size) should be used for Filter 2.

The corresponding flow capacity of the flow-through drains are calculated to be approximately 120 times the daily average flow of 0.47 cfs (200 gpm) measured at the United States Geological Survey gauge on Liese Creek, and this is approximately equivalent to a 1:10,000-year/24-hour storm event with no allowance for freeboard and without the benefits of the diversion ditch (AMEC, 2004a).
Figure 1: General Configuration of DSTF as of September 2013
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 non-mineralized 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.

2.1.3 Shell Area

There are three shells on the DSTF. The first shell (Shell 1) was constructed using non-mineralized rock only to a width of 100 ft on the 3:1 slope. The haul road has been constructed on the Shell 1. The second shell (Shell 2) and third shell (Shell 3), which has been constructed since 2010, is a composite shell which consists of non-mineralized rock and dewatered flotation tailings. Non-mineralized rock is placed at the face slope to a width of 20 feet, and then the dewatered flotation tailings are placed inside of the non-mineralized rock and compacted (see Figure 3). The width of the Shell 2 and Shell 3 is about 180 ft and 150 ft, respectively.
2.1.4 General Placement Area (GPA)

Dewatered flotation tailings and mineralized development rock is co-disposed in the General Placement Area (GPA). The mineralized rock is encapsulated in the tailings to minimize the oxidation of any sulfide minerals present. The mineralized rock may not be placed within 50 ft from the perimeter of DSTF.

The non-mineralized waste rock is placed at the perimeter of DSTF to allow any runoff from precipitation that bypasses the diversion ditch above the site to flow into the flow-through drains. All flows or seepage from the DSTF is collected in the RTP.

2.1.5 Diversion Ditch

The diversion ditch aims to intercept the “non-contact” surface water from areas unaffected by mine development. In order to expand the capacity of DSTF to 20Mt, a new diversion ditch was constructed in 2013.

The new north diversion ditch is about 5,850 feet long and runs from Inlet 3 at an elevation of 2,750 feet amsl into the existing north diversion ditch at an elevation of about 2,404 feet amsl. The remaining 2,049 feet of existing north diversion ditch connects to Flume #2 at an elevation of 2,158 feet amsl. Flume #2 is composed of a 750 feet-long, 60-inch diameter open CSP culvert. It discharges into Liese Creek about 700 feet downstream of the RTP Dam.

The new south diversion ditch is about 2,654 feet long and ranges in elevation from about 2,716 to 2,661 feet amsl. The new south diversion ditch connects to the existing ditch at about elevation 2,499 feet amsl via a 342-foot, 24-inch diameter HDPE pipe with intake and outlet structures. The discharge capacity of New South Flume is estimated to be 27 cfs (SRK, 2013a). The 2,329 feet-long existing south diversion ditch connects to Flume #1 at an elevation of 2,195 feet amsl. The water from the south diversion ditch discharges into the spillway via a 427-foot, 20-inch diameter HDPE pipe. The discharge capacity of Flume #1 is estimated to be 20 cfs (AMEC, 2006).

The diversion ditch is designed to intercept a one in 200-year, 24-hour precipitation event (4.6 inches within 24 hours). Minimum one foot of freeboard was incorporated into the design. The estimated design flow (200-year, 24-hour precipitation event) for post-expanded conditions 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), respectively (SRK 2013b).
2.2 Environmental Management

2.2.1 Water Management

The diversion ditch was constructed around the DSTF to divert surface, and near surface, runoff around the DSTF, so that such water becomes “non-contact.” The diverted water is routed to the Liese Creek downstream of the RTP.

Runoff down gradient of the diversion ditch and DSTF seepage are considered “mine-contacted.” These waters are routed to a flow-through drain and into the RTP.

2.2.2 Sedimentation Control

The flotation tailings erosion translates into a sediment load in the RTP, thus specific sedimentation control measures are used to keep erosion to a minimum:

- The slope of each shell is covered with non-mineralized rock, which minimizes the erosion of dewatered flotation tailings;
- The surface of GPA has two percent slopes to the nearest perimeter of GPA to limit erosion on the tailings; and
- The materials dumped on the DSTF are compacted as soon as possible.

2.2.3 Dust Control

Tailings have the potential to create dust, especially when they have been frozen or 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 building silt fences around non-active placement areas.
3.0 CONSTRUCTION DESIGN CRITERIA

3.1 Placement Schedule

The placement schedule was previously updated based on the as-built survey data from September 2013 and the life of mine plan issued in February 2014. Revision 5 of this document incorporates the current Life of Mine forecast from 2016 through 2022. Table 1 shows the placement schedule between 2017 and 2022. Major assumptions used for scheduling are as follows:

- Dry densities of the compacted materials are assumed based on the in-situ measurements and engineering judgments. Historically, the calculated volume using the tonnage record and the assumed dry densities shows good correlation with the surveyed volume. As of September 2013, the surveyed volume of DSTF was about 127.6 million cubic feet ($ft^3$). The calculated volume from the tonnage record was 122.7 million $ft^3$. The discrepancy between these volumes is about 3.8%.

**Assumed material dry densities for scheduling:**

- 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.

- It is assumed that the Shells won’t be constructed by 2022.
- All waste rocks including mineralized rock and non-mineralized rock excavated at the underground mine will be placed at the DSTF.

**Drawings 1 - 7** are the year-by-year drawings for the DSTF between September 2013 and 2019.
Table 1: Material Placement Schedule at the DSTF

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<td>Production</td>
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<td></td>
</tr>
<tr>
<td>Ore Milled</td>
<td>ton</td>
<td>-</td>
<td>904,540</td>
<td>901,533</td>
<td>892,451</td>
<td>802,793</td>
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<td>Tailings Backfilled in Under...</td>
<td>ton</td>
<td>-</td>
<td>369,687</td>
<td>368,459</td>
<td>364,746</td>
<td>328,103</td>
<td>306,986</td>
<td>169,691</td>
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<tr>
<td>Cumulative Tonnage at DSTF</td>
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<tr>
<td>Drystack Tailings</td>
<td>ton</td>
<td>6,899,750</td>
<td>7,434,603</td>
<td>7,967,678</td>
<td>8,495,383</td>
<td>8,970,073</td>
<td>9,414,211</td>
<td>9,659,716</td>
</tr>
<tr>
<td>Waste Rock</td>
<td>ton</td>
<td>3,977,831</td>
<td>4,689,063</td>
<td>5,384,783</td>
<td>6,038,036</td>
<td>6,721,990</td>
<td>7,336,583</td>
<td>7,425,608</td>
</tr>
<tr>
<td>Total</td>
<td>ton</td>
<td>10,877,582</td>
<td>12,123,667</td>
<td>13,332,461</td>
<td>14,533,419</td>
<td>15,692,063</td>
<td>16,750,794</td>
<td>17,085,324</td>
</tr>
<tr>
<td>Shell Area</td>
<td></td>
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</tr>
<tr>
<td>Drystack Tailings</td>
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<td>Waste Rock</td>
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<td>General Placement Area</td>
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</tr>
<tr>
<td>Drystack Tailings</td>
<td>ton</td>
<td>534,853</td>
<td>533,074</td>
<td>527,705</td>
<td>474,690</td>
<td>444,138</td>
<td>245,505</td>
<td>2,759,985</td>
</tr>
<tr>
<td>Waste Rock</td>
<td>ton</td>
<td>711,232</td>
<td>675,720</td>
<td>673,253</td>
<td>683,954</td>
<td>614,593</td>
<td>89,025</td>
<td>3,447,777</td>
</tr>
<tr>
<td>Total</td>
<td>ton</td>
<td>1,246,085</td>
<td>1,208,794</td>
<td>1,200,958</td>
<td>1,158,644</td>
<td>1,058,731</td>
<td>334,530</td>
<td>6,207,742</td>
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</table>

End of Year Crest Elevation of GPA

<table>
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</thead>
<tbody>
<tr>
<td></td>
<td>ft</td>
<td>2,571</td>
<td>2,585</td>
<td>2,598</td>
<td>2,611</td>
<td>2,623</td>
<td>2,635</td>
<td>2,639</td>
</tr>
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</table>
3.2 Tailings Characterization

Laboratory tests of the flotation tailings were carried out in 2009 by Golder Associates. In addition, a compaction test was carried out in March 2011 to evaluate the influence of the frozen flotation tailings on compaction. SRK conducted additional geotechnical tests using the Shelby Tube samples collected from piezometer drill holes. **Table 2** summarizes the geotechnical properties of flotation tailings obtained by these tests.

3.3 Development Rock Characterization

It is assumed that development rock placed and compacted will have a dry in-place density of approximately 125 lb/ft\(^3\) (2.00 t/m\(^3\)). 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.

3.4 Structural Stability Evaluation

The stability of the 20 Mt DSTF was previously studied by AMEC using the conceptual design (AMEC, 2004a). SRK updated the construction design for the 20 Mt DSTF (see **Drawing 8**), and evaluated its structural stability considering the variability of pseudo-static loadings, phreatic surfaces, and strength parameter (friction angle) of materials (SRK, 2011a). SRK updated slope stability evaluation as a part of DSTF Closure Study, considering the additional geotechnical tests and monitoring information on the phreatic surface obtained from piezometer holes (SRK, 2014b). This section summarizes the results of stability evaluation.
### Table 2: Geotechnical Properties of Flotation Tailings

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Properties</th>
<th>Testing Method</th>
<th>Information Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>2.56</td>
<td>ASTM D854-06</td>
<td>2011 Compaction Test</td>
</tr>
<tr>
<td>Optimum Moisture Content</td>
<td>15% - 16%</td>
<td>Standard Proctor (ASTM D-698)</td>
<td>2011 Compaction Test</td>
</tr>
<tr>
<td>Maximum Dry Density</td>
<td>109 lb/ft³ (1.74 t/m³)</td>
<td>Standard Proctor (ASTM D-698)</td>
<td>2011 Compaction Test</td>
</tr>
<tr>
<td>Shear Strength (Saturated)</td>
<td>Effective Friction Angle 34.4 degree&lt;sup&gt;(1)&lt;/sup&gt; Cohesion - 63 psf</td>
<td>Triaxial Compression Test (CU- Test) (ASTM D-4767)</td>
<td>Golder Associates (2009)</td>
</tr>
<tr>
<td>Shear Strength (Saturated)</td>
<td>Effective Friction Angle 34.4 - 35 degree&lt;sup&gt;(2)&lt;/sup&gt; Cohesion - 0.7 psf</td>
<td>Triaxial Compression Test (CU- Test) (ASTM D-4767)</td>
<td>SRK (2014)</td>
</tr>
<tr>
<td>Direct Shear Strength (90% Compaction)</td>
<td>Friction Angle - 37 degree Cohesion – 140 psf</td>
<td>Direct Shear Test (ASTM D-3080)</td>
<td>2011 Compaction Test</td>
</tr>
<tr>
<td>Direct Shear Strength (95% Compaction)</td>
<td>Friction Angle - 39 degree Cohesion – 90 psf</td>
<td>Direct Shear Test (ASTM D-3080)</td>
<td>2011 Compaction Test</td>
</tr>
<tr>
<td>Direct Shear Strength (100% Compaction)</td>
<td>Friction Angle - 41 degree Cohesion – 60 psf</td>
<td>Direct Shear Test (ASTM D-3080)</td>
<td>2011 Compaction Test</td>
</tr>
</tbody>
</table>

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):
   1. 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
   2. 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.
3.4.1 Design Criteria

The design criteria used for the stability analysis were specified in the original design report (AMEC, 2004a). Stability analysis of embankment slopes requires assessment of the structure’s ability to withstand the effects of self-weight (static) and earthquake induced (pseudo-static) loading conditions under both operating and closure conditions. In the original design report, it was considered the minimum allowable factor of safety (FoS) under static loading conditions during operations and closure conditions to be 1.5. During pseudo-static conditions, the minimum allowable FoS was selected as 1.1.

3.4.2 Seismic and Excess Pore Pressure 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). In summary, the peak ground acceleration (PGA) of 0.2 g (i.e., 20% of acceleration due to gravity) has a recurrence interval of 2,475 years at the site, and represents the Maximum Design Earthquake (MDE) for the project (AMEC, 2004b). The PGA was reduced by half to 0.1 g for input to the slope stability model as a horizontal acceleration. The one-half reduction in PGA for slope stability analysis accounts for the duration of ground acceleration necessary to damage earth and rock structures (the PGA is an instantaneous acceleration) as well as the attenuation provided by earth and rock structures (AMEC, 2004b; SRK, 2014b).

Vertical acceleration can be a considerable component of earthquake ground motion, especially in close proximity to a seismic source. The ratio of peak vertical to peak horizontal ground acceleration generally decreases with increasing distance from the seismic source. Based on engineering judgment and literature review, a vertical ground acceleration 0.7 times horizontal ground acceleration was selected for the sensitivity analysis (AMEC, 2014b).

SRK (SRK, 2014b) evaluated the sensitivity of the pseudostatic stability model to excess pore pressure with the B-bar coefficient of the computer program SLIDE (Version 5.026), which can be varied from 0 (no excess pore pressure from vertical stress change) to 1 (excess pore pressure equals vertical stress change). B-bar coefficients of 1 were assumed for the compacted tailings, GPA, and interface materials. B-bar coefficients of 0 were assumed for rock shell and flow-through drain, starter berm and toe berm, overburden, and bedrock materials.
3.4.3 Material Strength Parameters

AMEC (AMEC, 2004a) modeled the shells with moderate shear strength and GPA with no shear strength, whereas SRK (SRK, 2011a; SRK, 2014b) modeled 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, Pogo collected geotechnical parameters and samples from the sonic boreholes drilled in the DSTF for laboratory index and shear strength test. Table 3 summarizes the material parameters used in the stability analysis conducted by SRK (SRK, 2014b).

<table>
<thead>
<tr>
<th>Material</th>
<th>Bulk Unit Weight (pcf)</th>
<th>Saturated Unit Weight (pcf)</th>
<th>Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compacted Tailings¹</td>
<td>118</td>
<td>128</td>
<td>34</td>
</tr>
<tr>
<td>General Placement Area¹,²</td>
<td>118/125</td>
<td>128/135</td>
<td>34</td>
</tr>
<tr>
<td>Rock Shell</td>
<td>125</td>
<td>135</td>
<td>38</td>
</tr>
<tr>
<td>Flow-through Drain</td>
<td>125</td>
<td>135</td>
<td>38</td>
</tr>
<tr>
<td>Starter Berm and Toe Berm</td>
<td>125</td>
<td>130</td>
<td>32</td>
</tr>
<tr>
<td>Overburden</td>
<td>125</td>
<td>130</td>
<td>32</td>
</tr>
<tr>
<td>Bedrock</td>
<td>156</td>
<td>156</td>
<td>40</td>
</tr>
<tr>
<td>Interface ³</td>
<td>118</td>
<td>128</td>
<td>varies</td>
</tr>
</tbody>
</table>

Notes:
1. Nonlinear shear strength envelope was also analyzed based on triaxial shear test result for tailings Sample 10838/10839 Comp (SRK, 2014b).
2. Unit weights were varied between tailings and waste rock values for general placement area (GPA) materials.
3. "Interface" material type created to facilitate analysis of non-circular failure surfaces at boundaries between material types (see Section 2.4). The bulk and saturated unit weights are minimum values for materials present at the interface. The shear strengths are assumed to be the same as the weaker material at the interface.
4. Pounds per cubic foot
3.4.4 Phreatic Surface

One significant difference among three stability analyses conducted by AMEC and SRK 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 the phreatic surface presented in Figure 4 based on the following observations:

   • 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.

   • 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.

   • 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.

   • 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.
Figure 4: Phreatic Surface Used for Stability Analysis (SRK, 2014b)
3.4.5 Stability Analysis

The slope stability analysis was performed using the computer program SLIDE (Version 5.026). SLIDE is a two-dimensional, slope stability analysis program for evaluating the FoS of circular or non-circular failure surfaces in a defined slope. SLIDE analyzes the stability of slip surfaces using vertical slice, limit equilibrium methods (e.g., Bishop, Janbu, Spencer, etc.). Individual slip surfaces can be analyzed, or random search methods can be applied to locate the critical slip surface for a given slope. Spencer’s method was used as it considers both force and moment equilibrium.

The potential failure surfaces through the shells, GPA, and native soil layers were evaluated in SLIDE using the circular failure mode, while failure surfaces along the rock shell/flotation tailings interfaces and flow-through drain/DSTF interface were evaluated using the non-circular (block) failure mode. Both circular failure and block failure modes were evaluated for static and pseudostatic conditions.

There were four potential block failure planes evaluated in this study and they are described as follows:

- Block failure plane 1: the failure surface is between the non-mineralized waste rock in Shell 1 and the underlying tailings of the general placement area;
- Block failure plane 2: the failure surface is between the non-mineralized waste rock in Shell 2 and the underlying compacted tailings;
- Block failure plane 3: the failure surface is between the non-mineralized waste rock in Shell 3 and the underlying compacted tailings; and
- Block failure plane 4: the failure surface is between the DSTF and the flow-through drain.

The results of the slope stability analysis are summarized in Table 4 and show that the predicted stability of the critical cross-section satisfies the minimum allowable FoS for both static (1.5) and pseudostatic (1.1) conditions. Table 4 shows the lowest FoS resulting from the different material parameters listed in Table 3, analyzed phreatic surfaces in Figure 4, and seismic/excess pore pressure parameters. Results of the analysis show minimal sensitivity of the pseudostatic 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 4: Results of DSTF Slope Stability Evaluations

<table>
<thead>
<tr>
<th>Section A-A’</th>
<th>Circular Failure Surface</th>
<th>Noncircular Failure Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FoS - Static</td>
<td>FoS - Seismic</td>
</tr>
<tr>
<td>Circular Failure</td>
<td>1.77</td>
<td>1.22</td>
</tr>
<tr>
<td>Block Failure Plane 1</td>
<td>--</td>
<td>2.40</td>
</tr>
<tr>
<td>Block Failure Plane 2</td>
<td>--</td>
<td>2.14</td>
</tr>
<tr>
<td>Block Failure Plane 3</td>
<td>--</td>
<td>2.02</td>
</tr>
<tr>
<td>Block Failure Plane 4</td>
<td>--</td>
<td>2.21</td>
</tr>
</tbody>
</table>

3.4.6 Liquefaction Analysis

SRK (SRK, 2014b) conducted the liquefaction analysis using a 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 procedure using SPT data was 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 and subjected to vibration, e.g., earthquake ground motions. Among the soil samples collected from the three boreholes drilled in the 2012, only one sample, which was approximately 97 ft below ground surface (bgs) at SB-1, was below the established water table and was therefore used for liquefaction analysis. The result of the liquefaction analysis indicates the sampled soil from SB-1 has a FoS of 2.3 against liquefaction. Given the scope of observations in this study and the results of this analysis, liquefaction of the DSTF materials during the MDE is considered unlikely.
4.0 COMPACTION TEST IN 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.

4.1 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).
4.2 Results

4.2.1 Soil Temperatures and Frost Penetration

Table 5 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 5: Summary of Soil Temperature of Dumped Tailings Piles

<table>
<thead>
<tr>
<th>Trial</th>
<th>Surface Temp (°F)</th>
<th>3’ Depth Temp (°F)</th>
<th>5’ Depth Temp (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Day Trial</td>
<td>31</td>
<td>72</td>
<td>n/a</td>
</tr>
<tr>
<td>2 Day Trial</td>
<td>15</td>
<td>36</td>
<td>n/a</td>
</tr>
<tr>
<td>3 Day Trial</td>
<td>10</td>
<td>35</td>
<td>42</td>
</tr>
<tr>
<td>7 Day Trial</td>
<td>7</td>
<td>30</td>
<td>n/a(1)</td>
</tr>
</tbody>
</table>

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

4.2.2 Material Properties and Field Density Measurements

Table 6 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 7 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 7 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 6: Laboratory Tests Results – Material Properties

<table>
<thead>
<tr>
<th>Trial</th>
<th>Moisture Content</th>
<th>Specific Gravity</th>
<th>Standard Proctor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Surface</td>
<td>6” below surface</td>
<td>3' below surface</td>
</tr>
<tr>
<td>1 Day</td>
<td>17.9</td>
<td>n/a</td>
<td>17.9</td>
</tr>
<tr>
<td>2 Days</td>
<td>20.2</td>
<td>n/a</td>
<td>17.7</td>
</tr>
<tr>
<td>3 Days</td>
<td>13.9</td>
<td>16.5</td>
<td>17.2</td>
</tr>
<tr>
<td>7 Days</td>
<td>10.5</td>
<td>19.7</td>
<td>16.8</td>
</tr>
</tbody>
</table>

Table 7: Field Density Measurements

<table>
<thead>
<tr>
<th>Duration of Pile Exposure</th>
<th>Compaction Effort Trial</th>
<th>Nuclear Densometer</th>
<th>% to Maximum Dry Density</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Density (pcf)</td>
<td>Moisture (%)</td>
</tr>
<tr>
<td>1 Day</td>
<td>4 Passes</td>
<td>102.0</td>
<td>16.2</td>
</tr>
<tr>
<td></td>
<td>6 Passes</td>
<td>105.4</td>
<td>15.4</td>
</tr>
<tr>
<td></td>
<td>8 Passes</td>
<td>105.1</td>
<td>16.7</td>
</tr>
<tr>
<td>2 Days</td>
<td>4 Passes</td>
<td>102.3</td>
<td>16.8</td>
</tr>
<tr>
<td></td>
<td>6 Passes</td>
<td>103.7</td>
<td>16.1</td>
</tr>
<tr>
<td></td>
<td>8 Passes</td>
<td>106.4</td>
<td>16.7</td>
</tr>
<tr>
<td>3 Days</td>
<td>4 Passes</td>
<td>98.4</td>
<td>16.8</td>
</tr>
<tr>
<td></td>
<td>6 Passes</td>
<td>100.6</td>
<td>16.9</td>
</tr>
<tr>
<td></td>
<td>8 Passes</td>
<td>102.7</td>
<td>17.1</td>
</tr>
<tr>
<td>7 Days</td>
<td>4 Passes</td>
<td>90.0</td>
<td>15.5</td>
</tr>
<tr>
<td></td>
<td>6 Passes</td>
<td>87.8</td>
<td>15.3</td>
</tr>
<tr>
<td></td>
<td>8 Passes</td>
<td>86.4</td>
<td>15.6</td>
</tr>
</tbody>
</table>

4.2.3 Shear Strength

Table 8 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 8: Summary of Direct Shear Results

<table>
<thead>
<tr>
<th>Sample Compaction Effort</th>
<th>Average Dry Density of Specimen (pcf)</th>
<th>Average Cohesion (psf)</th>
<th>Average Friction Angle (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90%</td>
<td>99.0</td>
<td>140</td>
<td>37</td>
</tr>
<tr>
<td>95%</td>
<td>105.1</td>
<td>90</td>
<td>39</td>
</tr>
<tr>
<td>100%</td>
<td>109.9</td>
<td>60</td>
<td>41</td>
</tr>
</tbody>
</table>

4.2.4 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.
5.0 CONSTRUCTION PROCEDURES

This section describes the construction procedures of the DSTF.

5.1 General Placement Area

Materials are placed on the GPA year-round. This section describes the construction procedures for the GPA including Shell 1 and associated structures.

5.1.1 Shell 1 Construction

The first shell (Shell 1) has been constructed using non-mineralized rock since the commencement of operation. Shell 1 has a width of 100 ft on the 3:1 slope. Non-mineralized rock is dampened and spread into 3-ft loose lift. Then the lift is compacted with three passes of a D7 Dozer.

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

5.1.2 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.

The trees, shrubs, and topsoil along the perimeter of DSTF are removed and non-mineralized rock is placed on the slope surface at a thickness of approx. 1 ft. This layer works as water drainage to route the run-off water on the GPA into the flow-through drain.

5.1.3 Dewatered flotation Tailings Placement

The dewatered flotation tailings is dumped 15-ft apart, and then spread into maximum 12-inch loose lift. Compaction then proceeds with a minimum of four passes of a smooth drum roller having a minimum 12-ton equivalent weight.

Operation During Winter Conditions

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

- Windrows of dewatered flotation tailings have to be dozed down and spread 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 flotation tailings may become difficult to compact if water is allowed to infiltrate. In order to minimize 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; and
- Once dewatered flotation 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.

5.1.4 Mineralized Rock Placement

The mineralized rock needs to be encapsulated in the dewatered flotation tailings and the following procedures applied:

- The mineralized rock won’t be placed within 50 ft from the perimeter of DSTF;
- The mineralized rock is dumped and then spread into 3-feet loose lift. Compaction then proceeds with minimum three passes of a D7 dozer; and
- Once three lifts are placed, the mineralized rock will be covered with two one-foot dewatered flotation tailings layers before placing another lift of mineralized rock.

5.2 Shell Area

This section describes the construction procedures for Shell 2 and Shell 3 which consist of non-mineralized rock and dewatered flotation tailings.

5.2.1 Construction Period

The previous DSTF OMS Manual (AMEC, 2007) prescribed that the Shell would be constructed during a typical four month summer construction period. However,
compaction test conducted in March 2011 confirmed that the dewatered flotation tailings can be compacted appropriately during winter/freezing conditions once the appropriate construction procedures are consistently followed. Therefore, it is now planned to construct the Shells year-round.

5.2.2 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 sufficiently advanced. 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 shells.

5.2.3 Shell Construction Procedures

Shell 2 and Shell 3 are composite shells which consist of compacted dewatered flotation tailings and non-mineralized rock placed on the slope surface of the shells. The construction procedures for these shells are as follows:

- Non-mineralized rock is used to form a crest of the shells. 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. The crest of non-mineralized rock will have a width of 20 ft on the 3:1 slope; and

- The dewatered flotation tailings are dumped 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.
Operation during Winter Condition

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

- Between November and February, the windrows of dewatered flotation tailings have to be dozed down and spread within one day;
- In October and March to May, the windrows of dewatered flotation tailings have to be dozed down and spread within three days; and
- The placement area needs to be regularly clear to prevent build-up of snow and ice.

Operation in Wet Conditions

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

- Prior to placement of dewatered flotation tailings, the saturated and softened surface will be scraped off;
- Windrows of dewatered flotation tailings 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 will be suspended.
6.0 MONITORING

6.1 Geotechnical Monitoring

The compaction of dewatered flotation tailings at the Shells is important for overall stability of the DSTF and to ensure volume capacity. It is necessary to achieve a nominal 90% Standard Proctor of the dry density to secure the designed shear strength. The construction procedures for GPA and Shells aim to compact the dewatered flotation tailings to achieve a minimum of 90% Standard Proctor of the dry density. The geotechnical monitoring will verify compaction of the dewatered flotation tailings during the construction of Shell 2 and Shell 3 for adherence to design standards.

There is no specific monitoring requirement for the dewatered flotation tailings placement at GPA, because it can be deduced from the monitoring results at the Shell, and cumulative compaction effort by piling up the lifts can be expected at GPA.

6.1.1 Geotechnical Monitoring for Shell Construction

During construction of Shell 2 and Shell 3, the QA/QC program shown in Table 9 will be implemented.

The location of densometer readings and grab samples will be documented using handheld GPS and indicated on a site plan, and included with the data collected for the QC program. If QC testing is completed by an independent third party technician and soils testing laboratory, only the sand cone testing indicated in the proposed QA plan will be completed at a frequency of every 80,000 tons of tailings placed and compacted within each shell. If QC testing is completed by Pogo personnel, QA testing will be carried out by an independent certified technician and soils testing laboratory.

The results of geotechnical monitoring will be recorded using the data sheet shown in Appendix A.

In case the average of in-situ dry densities is less than the target (90% of Standard Proctor), that layer of dewatered flotation tailings will be re-compacted until the target dry density will be achieved.

6.2 Annual Survey

A detailed survey of DSTF will be conducted annually in September. The survey data will be compared with the year-by-year plan. If a significant discrepancy is identified, the plan may be updated.
Table 9: Geotechnical Monitoring Items during Shell Construction

<table>
<thead>
<tr>
<th>QA/QC</th>
<th>Test Description</th>
<th>ASTM Method</th>
<th>Test Frequency</th>
<th>Test Procedures</th>
<th>Target</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>In-situ Nuclear Densometer</td>
<td>D6938-10</td>
<td>Every 20,000 tons of tailings placed in each shell</td>
<td>Performed on material placed and compacted in all areas within 24 hours prior to test day. Maximum testing spacing of 30 ft to a target depth of 12 inches. Test density results should be reported in pcf and moisture content in %. Compare results to laboratory Standard Proctor test results.</td>
<td>Avg. Density of 98.1 pcf or 90% Standard Proctor</td>
</tr>
<tr>
<td></td>
<td>Standard Proctor</td>
<td>D698-07</td>
<td></td>
<td>Completed for three equally spaced grab samples from each test area.</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Moisture Content</td>
<td>D2216</td>
<td></td>
<td>Completed for three equally spaced grab samples from each test area.</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Grain Size Distribution</td>
<td>D422</td>
<td></td>
<td>Completed for three equally spaced grab samples from each test area.</td>
<td>Verify tailings consistency</td>
</tr>
<tr>
<td></td>
<td>In-situ Nuclear Densometer</td>
<td>D6938-10</td>
<td>Every 80,000 tons of tailings placed in each shell</td>
<td>Performed on material placed and compacted in all areas within 24 hours prior to test day. Maximum testing spacing of 30 ft to a target depth of 12 inches. Test density results should be reported in pcf and moisture content in %. Compare results to laboratory Standard Proctor test results.</td>
<td>As above</td>
</tr>
<tr>
<td></td>
<td>Sand Cone Test(1)</td>
<td>D1556-07</td>
<td></td>
<td>One test for every ten densometer tests completed.</td>
<td>Consistency with ASTM D6938-10 results</td>
</tr>
<tr>
<td></td>
<td>Standard Proctor</td>
<td>D698-07</td>
<td></td>
<td>Completed for three equally spaced grab samples from each test area.</td>
<td>As above</td>
</tr>
<tr>
<td></td>
<td>Moisture Content</td>
<td>D2216</td>
<td></td>
<td>Completed for the three samples collected for the Proctor test.</td>
<td>As above</td>
</tr>
<tr>
<td></td>
<td>Grain Size Distribution</td>
<td>D422</td>
<td></td>
<td>Completed for the three samples collected for the Proctor test.</td>
<td>As above</td>
</tr>
</tbody>
</table>

Note: 1. QA tests, apart from the Sand Cone Test, are not required if the QC program is conducted by a certified, independent lab.
6.3 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) (see Figure 5 and Table 10) 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).

RST vibrating wire piezometers (VWP) were installed in each of the three boreholes to determine the presence and extent of saturated zones within the DSTF and to monitor changes in pore pressure. DSTF temperatures were also measured using thermistors located within each VWP sensor. The installation depth of each sensor is presented in Table 11.

The data should be downloaded at a minimum of twice per year; once following the freshet, and once just prior to the onset of winter. The second downloading event is also an opportunity to check the condition of the datalogger and battery.

6.4 Reporting

The results of the monitoring described in this section will be reported in the quarterly monitoring reports and annual monitoring report.
### Table 10: Summary of DSTF Piezometer Drill holes

<table>
<thead>
<tr>
<th>Drillhole</th>
<th>Description</th>
<th>Easting</th>
<th>Northing</th>
<th>Collar Elevation (ft asl*)</th>
<th>Drilled Depth (ft)</th>
<th>Dip (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB1</td>
<td>Starter berm</td>
<td>1817073</td>
<td>3819217</td>
<td>2,408</td>
<td>106.5</td>
<td>90</td>
</tr>
<tr>
<td>GP1</td>
<td>General placement</td>
<td>1817382</td>
<td>3819010</td>
<td>2,486</td>
<td>147</td>
<td>90</td>
</tr>
<tr>
<td>RR1</td>
<td>‘Red Rock’ area</td>
<td>1817696</td>
<td>3818669</td>
<td>2,509</td>
<td>97</td>
<td>90</td>
</tr>
</tbody>
</table>

Note: 1 *ft asl – feet above sea level.

### Table 11: Summary of Vibrating Wire Piezometer Installation

<table>
<thead>
<tr>
<th>Drillhole</th>
<th>Drilled Depth (ft)</th>
<th>Details of Vibe Wire Sensor</th>
</tr>
</thead>
</table>
| SB-1      | 106.5              | Depth: 25 fbgl *  
Elev. 2383 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22850  
Datalogger SN: 2639  
Logger channel: VW2, Therm2  
Depth: 104.5 fbgl  
Elev. 2303.5 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22851  
Datalogger SN: 2639  
Logger channel: VW1, Therm1 |
| GP-1      | 147                | Depth: 63 fbgl  
Elev. 2423 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22852  
Datalogger SN: 2640  
Logger channel: VW1, Therm 1  
Depth: 137 fbgl  
Elev. 2349 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22853  
Datalogger SN: 2640  
Logger channel: VW2, Therm 2 |
| RR-1      | 97                 | Depth: 2 fbgl  
Elev. 2507 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW23152  
Datalogger SN: 2640  
Logger channel: VW5, Therm 5  
Depth: 61 fbgl  
Elev. 2448 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22854  
Datalogger SN: 2640  
Logger channel: VW3, Therm 3  
Depth: 94 fbgl  
Elev. 2415 ft  
Range: 0.7 Mpa  
Vibe wire SN: VW22855  
Datalogger SN: 2640  
Logger channel: VW4, Therm 4 |

Note: 1 *fbgl – feet below ground level.
Figure 5: As-Built Sonic Borehole and Vibrating Wire Piezometer Locations (SRK, 2014b)
7.0 INSPECTION

7.1 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;
- 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 B). If any unusual situation is found, it will be reported to the Maintenance Manager and Environmental Manager.

7.2 Occasional Inspection
The DSTF will be inspected by Environmental personnel after extreme rainfall (two inches within 24 hours) or an appreciable earthquake (felt by site personnel).
8.0 REFERENCES

AMEC, 2004b, RTP Dam Design Report.
Teck-Pogo, 2004, Pogo Project Final Feasibility Study
AMEC, 2006, RTP Dam 2004-2005 As-built Report
SRK, 2011a, Pogo Drystack Tailings Facility Expansion Preliminary Study.
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, DSTFClosure Study – Slope Stability Analysis.
Appendix A

DSTF Shell Geotechnical Monitoring Data Sheet
Pogo Mine DSTF Shell Geotechnical Monitoring Data Sheet

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Reported by</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Shell No.</td>
<td>Elevation (ft)</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Date Compacted</td>
<td></td>
</tr>
</tbody>
</table>

**GPS Coordinates (degree)**

<table>
<thead>
<tr>
<th>Nuclear Densometer Grid</th>
<th>Map</th>
</tr>
</thead>
<tbody>
<tr>
<td>A N: W:</td>
<td>Upstream</td>
</tr>
<tr>
<td>B N: W:</td>
<td>Densometer Grid</td>
</tr>
<tr>
<td>C N: W:</td>
<td>Sampling Location</td>
</tr>
<tr>
<td>D N: W:</td>
<td>1 2 3</td>
</tr>
</tbody>
</table>

**Sampling Location**

<table>
<thead>
<tr>
<th>1 N: W:</th>
<th>Sampling Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 N: W:</td>
<td></td>
</tr>
<tr>
<td>3 N: W:</td>
<td></td>
</tr>
</tbody>
</table>

**Moisture Content / Standard Proctor Test (Three samples per monitoring)**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Dry Density (pcf)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Nuclear Densometer (30 ft grid, Target Depth: 12 inch)**

<table>
<thead>
<tr>
<th>Number of measurements</th>
<th>Items</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moisture Content (%)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dry Density (pcf)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>% of Standard Proctor</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Sand Cone Test (One test for every ten densometer measurements) (QA Program)**

<table>
<thead>
<tr>
<th>Test Hole No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry Density (pcf)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: All lab test reports should be attached to this data sheet.
Appendix B

Weekly Inspection Form
### Inspection Information

<table>
<thead>
<tr>
<th>Location: Drystack</th>
<th>Inspection Date:</th>
<th>Time:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inspector:</td>
<td>Status:</td>
<td></td>
</tr>
</tbody>
</table>

#### Comments:

<table>
<thead>
<tr>
<th>Inspection Criteria</th>
<th>Y</th>
<th>N</th>
<th>Inspection Findings</th>
<th>Corrective Actions</th>
<th>Date Actions Completed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free of Unusual Cracks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Free of Bulging</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Free of Signs of Settlement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Free of Seepage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Free of Erosion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Inspection Instructions

#### Inspection Certification

Authorized Signature: __________________________  Date: __________________________
Appendix C

Drawings
Existing South Diversion Ditch

CROSS SECTION A - A'

PLAN VIEW

DRY STACK STORAGE AREA

NOTES
1. Contour interval is 5 ft.
2. Base contour data received from POGO mine site dated May 20, 2010.

Pogo Dry Stack Tailings Facility
Construction & Maintenance Plan Revision 4

Dry Stack Tailings Facility
Plan and Section
September 2013 As-built

Sumitomo Metal Mining Pogo LLC
LEGEND
- Existing Diversion Ditch
- New Diversion Ditch
- Extent of End of 2014 Material Placement
- Flow-through Drain Extension
- Green Rock Outer Shell
- GPA Materials
- Dry Stack Tailings

NOTES
1. Contour interval is 5 ft.
2. Base contour data received from POGO mine site dated May 20, 2010.

PLAN VIEW
DRY STACK STORAGE AREA

New North Diversion Ditch
Existing North Diversion Ditch
New South Diversion Ditch
Existing South Diversion Ditch

Dry Stack Tailings Facility
Construction & Maintenance Plan Revision 4
Sumitomo Metal Mining Pogo LLC
Existing South Diversion Ditch

CROSS SECTION A - A'

PLAN VIEW

DRY STACK STORAGE AREA

NOTES
1. Contour interval is 5ft.
2. Base contour data received from POGO mine site dated May 20, 2010.

SUMITOMO METAL MINING POGO LLC

POGO Dry Stack Tailings Facility
Construction & Maintenance Plan Revision 4

Dry Stack Tailings Facility
Plan and Section
End of 2015
Existing South Diversion Ditch

CROSS SECTION A - A'

PLAN VIEW

DRY STACK STORAGE AREA

NOTES
1. Contour interval is 5ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
LEGEND
- Existing Diversion Ditch
- New Diversion Ditch
- Extent of End of 2017 Material Placement
- Flow-through Drain Extension
- Green Rock Outer Shell
- GPA Materials
- Dry Stack Tailings

NOTES
1. Contour interval is 5 ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
LEGEND
- Existing Diversion Ditch
- New Diversion Ditch
- Extent of End of 2018 Material Placement
- Flow-through Drain Extension
- Green Rock Outer Shell
- GPA Materials
- Dry Stack Tailings

NOTES
1. Contour interval is 5 ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
NOTES
1. Contour interval is 5 ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
LEGEND
- Existing Diversion Ditch
- Proposed Diversion Ditch
- Extent of Year 2030 Material Placement
- Rock Drain Extension Required for 20Mt
- Green Rock Outer Shell
- GPA Materials
- Tailings

NOTES
1. Contour interval is 10ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
3. 20 M ton configuration developed for assessment of proposed ultimate configuration.

PLAN VIEW
DRY STACK STORAGE AREA

CROSS SECTION A - A'

20M; FL 2640R

General Placement Material

Existent South Diverion Ditch

Proposed North Diversion Ditch Alignment

Proposed South Diversion Ditch Alignment

Current Limit of Disturbed Area
Proposed Limit of Disturbed Area
(additional 1,767,145 ft²)

Existing North Diversion Ditch

Existing South Diversion Ditch

Current Limit of Disturbed Area

Proposed Limit of Disturbed Area
(additional 1,767,145 ft²)

NOTES
- Contour interval is 10ft.
- Base contour data received from POGO mine site dated May 20, 2010.
- 20 M ton configuration developed for assessment of proposed ultimate configuration.

1. Contour interval is 10ft.
2. Base contour data received from POGO mine site dated May 20, 2010.
3. 20 M ton configuration developed for assessment of proposed ultimate configuration.