1.0 Introduction

The Technical Memorandum titled Rosemont Flow-Through Drain Design Summary (Tetra Tech, 2010d) was prepared in response to the April 14, 2010 Comprehensive Request for Additional Information from the Arizona Department of Environmental Quality (ADEQ) to Rosemont Copper Company (Rosemont). This request was with regard to the aquifer protection permit (APP) application submitted to ADEQ in 2009 (Tetra Tech, 2009a) for the Rosemont Copper Project (Project). Specifically, Tetra Tech (2010a) answered items nos. 14a and 14b on Page 9 of 18 and no. 15 on page 10 of 18 of the April 14, 2010 ADEQ letter.

- No. 14a: Please provide an estimated amount of surface and subsurface flows designed for discharge through the underdrain system;
- No. 14b: ADEQ recognizes that the underdrain system is designed to discharge the surface and subsurface flows below the tailings pile. However, please demonstrate that the underdrain system will remain functional to effectively discharge surface and subsurface flows without threatening the integrity of the tailings pile. The flow computations should include sediment load for the underdrain system to determine underdrain stability. Underdrain stability in this context implies that there is no net aggregation or degradation of the underdrain bed or clogging of the CPe pipes used in the underdrain system; and
- No. 15: CPe Pipe Deflection – Appendix G-4: Three parallel CPe pipes, each 36-inch diameter, 500 LF (DWG NO. 600-CI-940), are used in the flow-through drain beneath the tailings pile. Please summarize the results of CPe pipe deflection and conclusion as to its effectiveness and suitability in the flow-through drain.

Tetra Tech’s memorandum (2010d) included information previously presented in the Site Water Management Update report (Tetra Tech, 2010f):

- Rosemont Flow-Through Drain Design, April 5, 2010 (Tetra Tech, 2010a)
- Rosemont Flow-Through Drain Sizing, April 5, 2010 (Tetra Tech, 2010c)
Addition information on the flow-through drains was requested by ADEQ in a letter titled *Incomplete Response to Technical Deficiencies* (dated December 3, 2010). This Technical Memorandum responds to a portion of ADEQ's Additional Comment #6 on page 17 of 34 of the December 3, 2010 letter. The entire comment is as follows:

*Review of the Site Water Management Update, Volume 1 of 5, page 35, indicates that the flow-through drain and finger drain design concept is developed after examining the operating Dry Stack Tailings Facility at Pogo, Coeur Alaska Mine Project, near Fairbanks, Alaska. ADEQ is not familiar with the flow-through drain and finger drain system at the Pogo mine. Please describe in detail how the flow-through drain and drain finger design at the Rosemont Dry Stack Tailings Facility compares with the Pogo Project. Please give an account of Pogo’s operating and maintenance experience with the flow-through system and its anticipated long term reliability during operational life of the mine and post-closure period. ADEQ will make final assessment of the Rosemont’s flow-through drain and finger drain system design underneath the Dry Stack Tailings Facility after reviewing and analyzing the requested information.*

Based on a January 5, 2011 meeting with between Rosemont/Rosemont's consultants and ADEQ, and a subsequent meeting between Tetra Tech and ADEQ on January 12, 2011, an assessment of the flow-through drains was made assuming the unlikely event of drain failure, i.e., plugging at the inlets.

### 2.0 Layout of Flow-Through Drains

Attachment 1 provides Figures 37 and 38 from the *Site Water Management Update* report (Tetra Tech, 2010f) that shows the layout of the flow-through drains and their naming convention. Please note that although a single phase heap leach pad is shown, the flow-through drain locations/function will remain consistent for the planned Phase 1 and Phase 2 Heap Leach Pad design presented in *Rosemont Heap Leach Facility Permit Design Report* dated May 2009 (Tetra Tech, 2009b). Rosemont evaluated the option of having a single expanded Phase 1 Pad but has elected to remain with the dual pad concept shown in the May 2009 report.

At closure, the following flow-through drain segments will have inlets on the up-gradient side of the Rosemont Ridge Landform. The Rosemont Ridge Landform (Landform) is the consolidated and contoured earthen structure consisting of waste rock from the Open Pit, a closed Heap Leach Facility encapsulated with waste rock, and a Dry Stack Tailings Facility, also encapsulated with waste rock.

- South 2 Drain (adjacent to Open Pit)
- South 1 Drain (adjacent to Open Pit)
- South 1 Drain - South 1 Collector Drain (at former PWTS Pond location)
There will also be additional connections to the flow-through drain system and up-gradient ponding areas, such as the Pit Stormwater Pond and the Crusher Stormwater Pond. These areas are shown on Figures 19 and 20 of the Site Water Management Update report (Tetra Tech, 2010f). These figures are provided in Attachment 2.

The Pit Stormwater Pond will be eliminated due to expansion of the Open Pit. However, the Crusher Stormwater Pond will remain at closure. Culverts are designed to pass stormwater from these ponds into the South 1 “flow-through” Drain.

A possible closure modification of the former Crusher Stormwater Pond to would be to extend a “collector” drain off the South 1 Drain and into the pond area. Regardless, the culverts as shown on Figure 20 in Attachment 1 are designed to handle the General and Local Probable Maximum Precipitation (PMP) storm events at maximum watershed conditions. As designed, these culverts discharge into the South 1 Drain. As shown on Figure 19, the contributing watershed to the Crusher Stormwater Pond will also become reduced over time.

As a note, the South 3 “flow-through” Drain will become buried as the Waste Rock Storage Facility is developed and will not have an inlet area at closure (Figure 38 in Attachment 1).

The sections below describe the potential consequences of failure under closure conditions, i.e., loss of containment at the drain inlets and plugging of the inlet, for each one of the flow-through drains listed above, including the former Crusher Stormwater Pond. Except for storm and inlet area containment volumes where needed for emphasis, the design information contained in the technical memoranda listed in Section 1.0 is not repeated herein.

3.0 South Flow-Through Drains

The following flow-through drain inlets/ponding areas are discussed in this section:

- Inlet to South 2 Drain (adjacent to Open Pit)
- Inlet to South 1 Drain (adjacent to Open Pit)
- Inlet to South “South 1 Collector Drain” (at former PWTS Pond location)
- Outlet from former Crusher Stormwater Pond Area

3.1 Inlet to South 2 “Flow-Through” Drain (adjacent to Open Pit)

In the event of loss of containment volume at the inlet of the South 2 Drain, and plugging of the drain, storm flows would eventually report to the Open Pit.
3.2 **Inlet to South 1 “Flow-Through” Drain (adjacent to Open Pit)**

In the event of loss of containment volume at the inlet of the South 1 Drain, and plugging of the drain, storm flows would eventually report to the Open Pit.

3.3 **Inlet to “South 1 Collector Drain” (at former PWTS Pond location)**

An extension of the South 1 “flow-through” Drain leads to the former PWTS Pond location as shown on Figure 38 (Attachment t1), herein termed the South 1 Collector Drain. In the event the inlet to the “South 1 Collector Drain” becomes plugged, storm runoff would pool in the former PWTS Pond Area and evaporate or infiltrate into the ground and/or seep into the flow-through drain. Based on the Technical Memorandum titled *Rosemont Flow-Through Drain Sizing* (Tetra Tech, 2010c), the maximum anticipated runoff from the General PMP event reporting directly to the former PWTS Pond Area is estimated to be 348.8 acre-feet, corresponding to an elevation of about 4,950 feet above mean sea level (amsl).

The entire basin encompassing the former PWTS Pond location, up to an elevation of 5,010 feet amsl, is about 5,720 acre-feet. This water surface elevation (WSE) corresponds to a spill point into the former Crusher Stormwater Pond. This maximum containment is about 16 times the runoff volume from a General PMP event. However, at this WSE, the former Settling Basin area would be affected as well as the inlet to the North 1 Drain (see Section 4.1).

Section 5.0 provides the results of stability analyses performed in relation to stormwater ponding in the former PWTS Pond Area, as well as potential contingencies related to draining this area to the Open Pit.

3.4 **Outlet from former Crusher Stormwater Pond Area**

In the event of loss of containment volume and plugging of the culverts/inlet of the former Crusher Stormwater Pond, storm flows would eventually report to the former PWTS Pond Area (at a WSE of 5,010 feet amsl). WSE’s in excess of about 5,025 feet amsl would flow back into the Open Pit.

4.0 **North Flow-Through drains**

The following flow-through drain inlets/ponding areas are discussed in this section:

- North 1 Drain (immediately north of former Settling Basin location)
- North 1 Collector Drain
- North 2 Collector Drain
- North 2 Drain (inlet at location of Detention Basin No. 2)
- North 3 Drain (two locations) (inlets at location of Detention Basin No. 3)

4.1 **North 1 “Flow-Through” Drain (immediately north of former Settling Basin location)**

In the event of loss of containment volume at the inlet of the North 1 Drain, and plugging of the drain, storm flows would report to the former Settling Basin area and eventually to the former PWTS Pond Area.
As indicated in Section 3.3, the former Settling Basin, in combination with the former PTWS Pond Area, has an estimated maximum storage capacity of 5,720 acre-feet up to elevation 5,010 feet amsl. WSE’s in excess of 5,010 feet amsl would spill into the former Crusher Stormwater Pond area. WSEs in excess of about 5,025 feet amsl would report to the Open Pit.

Based on the Technical Memorandum titled Rosemont Flow-Through Drain Sizing (Tetra Tech, 2010c), the maximum anticipated runoff from the General PMP event reporting to the North 1 Drain inlet is estimated to be 901.9 acre-feet.

Under worst case conditions, the combined volumes from Section 3.3 (348.8 acre-feet) and 901.9 acre-feet would report to the former PWTS Pond location. This equals about 1,250.7 acre-feet of run-off, or 21 percent of the total available storage volume prior to spilling into the former Crusher Stormwater Pond area. This equates to a storage capacity of about 4.5 times the combined runoff volume from a General PMP event from these two (2) watershed areas.

4.2 North 1 Collector “Flow-through” Drain

In the event of loss of containment volume at the inlet of the North 1 Collector Drain, and plugging of the drain, storm flows would eventually report to the former Settling Basin/PWTS Pond Area. A minor watershed area reports to this drain inlet.

4.3 North 2 Collector “Flow-Through” Drain

In the event of loss of containment volume at the inlet of the North 2 Collector Drain, and plugging of the drain, storm flows would eventually report to the North 1 Collector Drain inlet area and eventually into the former Settling Basin/PWTS Pond Area. A minor watershed area reports to this drain inlet.

4.4 North 2 “Flow-Through” Drain (inlet at location of Detention Basin No. 2)

In the event of loss of containment volume at the inlet of the North 2 Drain, and plugging of the drain, storm flows would report to Permanent Diversion Channel No. 2 and eventually to Detention Basin No. 3.

4.5 North 3 “Flow-Through” Drain (2 locations) (inlets at location of Detention Basin No. 3)

In the event of loss of containment volume at the inlet of the North 3 Drain, and plugging of the drain, storm flows would report to Permanent Diversion Channel No. 2 and eventually to the Compliance Point Dam, downstream of the Rosemont Ridge Landform.

5.0 Contingency Summary

In the unlikely event of containment loss at the flow-through drain inlet areas associated with the Rosemont Ridge Landform, or total plugging of the drains, protection of the facilities at the Rosemont Copper Project under closure conditions is still maintained. Excess runoff may either routed to the Open Pit, routed downstream of the Landform via stormwater channels designed to handle the Local PMP event, or report to a large basin in the former PWTS Pond area. This basin is able to contain multiple PMP event volumes from up-gradient watershed areas. In the event of stormwater ponding at the former PWTS Pond area, Stability analyses were performed
at various ponding depths. Stability results are summarized in Section 6.0 of this Technical Memorandum.

6.0 Former PWTS Pond Area Evaluation

Since the former PWTS Pond Area has the potential to pool stormwater in the event of plugging at the “South 1 Collector Drain” inlet, additional stability analyses were performed in this area assuming hypothetical pooling depths of 25 feet, 50 feet, 75 feet, and 125 feet. This corresponds to pooling depths of 4,925 feet amsl, 4,950 feet amsl, 4,975 feet amsl, and 5,025 feet amsl. As previously, the maximum anticipated runoff from the General PMP event reporting directly to the former PWTS Pond Area is estimated to be about 348.8 acre-feet, corresponding to an elevation of about 4,950 feet amsl. Although unrealistic, a pooling depth of 125 feet was also analyzed. This is the maximum possible pooling depth based on a topographic high between the former PWTS Pond area and the Open Pit.

In order to assess the effect of the embankment above the former PWTS Pond area, stability analyses were performed along the cross section shown in Attachment 3. Attachment 4 provides the analysis results. The stability evaluation was performed by AMEC Earth & Environmental (AMEC, 2011).

Under all conditions simulated, the computed safety factors met or exceeded the prescriptive factors of safety for both static and seismic loading conditions per ADEQ’s BADCT Guidance Manual (ADEQ, 2004). Ponded stormwater would eventually evaporate or infiltrate into the ground and/or seep into the flow-through drain.

Although not warranted based on the stability results, preventing stormwater pooling in the former PWTS Pond area would required significant channel cuts to route stormwater to the Open Pit. Excavation cuts of over 200 feet would be anticipated.
References


Attachment 1

Figures 37 & 38 – Site Water Management Plan
Attachment 2

Figures 19 & 20 – Site Water Management Plan
Attachment 3
Stability Analysis Cross Section Location
AMEC Earth and Environmental (AMEC) has prepared this memorandum at the request of Rosemont Copper to provide responses to concerns from the Arizona Department of Environmental Quality (ADEQ) regarding the stability of the Dry Stack Tailings Storage Facility (TSF) adjacent to the Process Water Temporary Storage (PWTS) Pond. The subject addressed in this memorandum pertains to evaluation of the Dry Stack TSF stability, assuming closure geometry, with ponding water up to 125-feet deep against the facility. This memorandum also aims to mitigate concerns over potential liquefaction of the tailings due to the potential for temporary storage of water within the former PWTS Pond Area.

1.0 Slope Stability

Slope stability analyses were completed in support of the closure conditions of the Rosemont Copper Project Dry Stack TSF. Using the proposed closure configuration of the PWTS Pond area received from Tetra Tech, static and pseudostatic slope stability analyses were conducted under effective stress conditions using the computer program SLIDE 5.0, a commercially available computer program (Rockscience, 2007) which enables the user to conduct limit equilibrium slope stability calculations by a variety of methods.

1.1 Methodology

For the failure mechanisms considered in the analyses, slope stability was evaluated using limit equilibrium methods based on Spencer’s method of analysis (Spencer, 1967). Spencer’s method is a method of slices (consideration of potential failure masses as rigid bodies divided into adjacent regions or "slices," separated by vertical boundary planes). It is based on the principle of limiting equilibrium (i.e., the method calculates the shear strengths that would be required to just maintain equilibrium along the selected failure plane, and then determines a safety factor by dividing the available shear strength by the driving shear stress). Consequently, safety factors calculated by Spencer’s, or by any other limiting equilibrium method, indicate the percentage by which the available shear strength exceeds, or falls short of, that required to maintain equilibrium. Therefore, safety factors in excess of 1.0 indicate stability and those less than 1.0 indicate instability, while the greater the mathematical difference between a safety factor and 1.0, the larger the margin of safety (for safety factors in excess of 1.0), or the more extreme the likelihood of failure (for safety factors less than 1.0). The minimum required safety factors used in accordance with the BADCT Guidance Manual guidelines are 1.3 and 1.0 for static and seismic analyses, respectively, where appropriate laboratory and field strength testing has been conducted.

Stability analyses were conducted under both static and seismic loading conditions. Pseudostatic based analyses are commonly used to apply equivalent seismic loading on earthfill structures. In an actual seismic event, the peak acceleration would be sustained for only a fraction of a second. Actual seismic time histories are characterized by multiple frequency attenuating motions. The accelerations produced by seismic events rapidly reverse motion and generally tend to build to a peak acceleration that quickly decays to lesser accelerations. Consequently, the duration that a mass is actually subjected to a
unidirectional, peak seismic acceleration is finite, rather than infinite. The pseudo-static analyses conservatively model seismic events as constant acceleration and direction, i.e., an infinitely long pulse. Therefore, it is customary for geotechnical engineers to take only a fraction of the predicted peak maximum acceleration when modeling seismic events using pseudo-static analyses. The stability of the Dry Stack TSF under earthquake loading was evaluated based on the Maximum Credible Earthquake (MCE) of magnitude 7.1 and a pseudostatic coefficient equal to two-thirds the Peak Ground Acceleration (PGA), i.e., 0.24 g. This represents a conservative approach as Hynes-Griffin and Franklin (1984) suggest using ½ the peak horizontal ground acceleration.

1.2 Model Development

The cross section under consideration was selected as representative of the most critical configuration through the South Dry Stack TSF and the former PWTS Pond Area as shown on Figure 1. This cross section was developed from the closure geometry provided by Tetra Tech.

At final closure, the toe and face of the South Dry Stack TSF forms a containment berm for the former PWTS Pond Area with a floor elevation of approximately 4,900-feet above mean sea level (AMSL). A flow-through drain consisting of select rockfill routes water from this area beneath the Dry Stack TSF to beyond the limits of the structure. Under normal conditions, the high hydraulic conductivity of the flow-through drain will minimize the surface water ponding depth in the former PWTS Pond Area. In the event of reduced effectiveness due to localized sedimentation or blinding at the flow-through drain interface, ponding may temporarily increase the surface water elevation in the former PWTS Pond Area. Slope stability analyses were conducted to examine the effect of hypothetical 25, 50, 75 and 125-foot ponding depths with surface water elevations of 4,925, 4,950, 4,975, and 5,025 feet AMSL, respectively. Seepage analyses were performed at each surface water depth interval, described above, to evaluate the limits of the resulting phreatic level developed within the Dry Stack TSF. As a note, the 125-foot ponding depth was selected since this represents the maximum depth prior to spillover to the open pit.

1.3 Material Properties

The cross section under consideration is composed of the following common material types: (1) alluvial foundation soil, (2) tailings, (3) compacted tailings, (4) rockfill, and (5) flow-through drain material. The material properties for each were estimated based upon laboratory testing including triaxial and direct shear tests; the geotechnical field investigation including standard penetration tests; and experience with similar materials. Material properties required for the stability analyses include soil unit weight, shear strength, and hydraulic parameters. For a full discussion of the laboratory and field programs, refer to the AMEC report entitled “Rosemont Copper Company Dry Stack Tailings Storage Facility Final Design Report,” dated April 15, 2009. Material properties used in the analyses are summarized below.

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<th>Material Type</th>
<th>Moist Unit Weight (pcf)</th>
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<th>Cohesion (psf)</th>
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<td>38</td>
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<td>$1 \times 10^{-1}$</td>
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1.4 Results

Results of the aforementioned seepage and slope stability analyses for the cross section under consideration are shown on Figures 1, 2, 3, and 4, and summarized below. For tailing impoundment facilities, the minimum factors of safety, as required by the BADCT Guidance Manual, are 1.3 and 1.0 for static and seismic analyses, respectively, where appropriate strength laboratory and field testing has been completed. As summarized in the table below, the proposed Dry Stack TSF facility is stable under the conditions noted above as the computed values meet or exceed the prescriptive factors of safety for both static and seismic loading conditions.

<table>
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<th>PWTS Pond Depth (ft)</th>
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In summary, the proposed closure configuration of the Dry Stack TSF in the vicinity of the former PWTS Pond Area is stable for both static and seismic loading conditions. The above stability analysis is considered conservative because the flow-through drains do not indicate a propensity for blinding (Tetra Tech 2010a, b, c, d) and are anticipated to function as designed, but regardless, are globally stable with surface water ponding depths in the former PWTS Pond Area of 25, 50, 75 and 125 feet. Deformation associated with seismic loading is considered to be negligible as the factor of safety under seismic loading is equal to or greater than one and materials under consideration do not express strain-softening characteristics. Some minor maintenance and repair may be required due to localized slope sloughing or raveling under seismic loading.

1.5 Liquefaction

Liquefaction can be generally defined as the loss of shear strength in loose, saturated, and cohesionless soils due to the generation of excess pore pressures as a result of large shear strains induced by undrained cyclic loading. This phenomenon is generally caused by seismic loading in which loose, saturated soils tend to contract and displace pore water. If the soil is unable to dissipate the increasing pore pressure generated from the pore water displacement, undrained loading results and a loss of effective shear strength occurs. Liquefaction is common in loose, saturated, and cohesionless sand but has also been noted to occur in material such as low plasticity clay and silt or cohesionless gravels.

The tailings within the Dry Stack TSF will not likely be saturated as evidenced from the seepage analysis and are anticipated to have a moisture content of 18 percent (by dry weight) or less. Furthermore, the dry stack tailings will be under large confining pressures producing a uniformly dense fill, hence the propensity for liquefaction will be very low and is not anticipated to occur, especially at depths exceeding 30-40 feet below the surface. The dry stack tailings will be further contained around the entire perimeter by a zone of compacted dry stack tailings, overlain by a Rock Buttress, both with average lift widths of approximately 250 feet.

The filtered tailings within the Dry Stack TSF are anticipated to have saturation levels between 60 and 90 percent based upon previous seepage analyses. Despite the saturation levels however, for liquefaction to occur, the unsaturated soils must also be in a contractive (loose) state, thereby allowing pore pressures to build during the seismic loading. The tailings to be used to form the structural zone of the containment structure will be compacted to 90 percent of the maximum dry density. The tailings behind this compacted zone (the majority of the tailings in the TSF) will not be compacted; however our testing has shown that after a depth of 50 feet, the tailings achieve a density of approximately 90 percent of the maximum dry density due to self-weight compaction. Extensive triaxial shear testing laboratory testing of
the tailings was completed on tailings samples remolded to between 80 and 90 percent of the maximum dry density (standard proctor) and at saturation levels between approximately 70 and 80 percent. All 8 different lithologies tested exhibited strain-hardening, or dilatant behavior. Dense soils that exhibit dilatancy are unlikely to liquefy as the generated pore pressures are relieved as the soil dilates.

As part of the design of the Dry Stack TSF, we considered the increased saturation (moisture content) due to self-weight consolidation of the tailings. This was evaluated by conducting compaction tests on actual samples and measuring the saturation with applied load. The results of these tests indicated that some of the tailings could potentially become saturated under an equivalent tailing column of 200 feet while others do not become saturated under 800 feet of tailings. However, the tailings density at these depths exceeds 90 percent of the maximum dry density. These materials are quite dense and dilate under shear, indicating liquefaction will not be an issue. However, after an seismic event such as the MCE occurs, localized deformations or sloughing could occur and may require minor maintenance or repair.

The alluvial soils underlying the Dry Stack TSF have an approximate average depth of 40 feet. Thirty-eight geotechnical borings were advanced within or near the footprint of the Dry Stack TSF and were drilled to depths exceeding 100 feet. As evidenced by blow counts obtained from SPT, the majority of encountered native soils were very dense or hard for granular and fine grained material, respectively, and consisted mainly of sand with varying amounts of gravel, silt, and clay. Furthermore, groundwater was not encountered in geotechnical borings with the exception of borings very close to a small surface water impoundment. Therefore, the alluvial soils underlying the Dry Stack TSF are not susceptible to liquefaction.

If you have any questions or comments regarding these responses or would like to discuss the stability evaluation in further detail, please contact us.

Sincerely,

AMEC Earth & Environmental, Inc.

Justin Hall, P.E.      Dave Weidinger, E.I.T.
Senior Engineer      Staff Engineer

JWH:jwh
References


Hynes, M.E., F.G. Franklin. 1984. Rationalizing the Seismic Coefficient Method. U.S. Army Corps of Engineers (USACE), Waterways Experiment Station, Miscellaneous Paper GL-84-13, Vicksburg, MS.


STABILITY RESULTS

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MATERIAL PROPERTIES

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NOTES:

1. TAILINGS REPORTING TO THE DRY STACK FACILITY EXCEEDING THE MAXIMUM ALLOWABLE MOISTURE CONTENT OF 18% MUST BE PLACED AT LEAST 1,100 FT AWAY FROM THE UPSTREAM CREST AT ANY ELEVATION.
2. CLOSURE GEOMETRY PROVIDED BY TETRA TECH.
### Stability Results

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### Notes:

1. Tailing reporting to the dry stack facility exceeding the maximum allowable moisture content of 18% must be placed at least 1,100 ft away from the upstream crest at any elevation.
2. Closure geometry provided by Tetra Tech.
STABILITY RESULTS

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<td>FRICTION ANGLE (DEGREES)</td>
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<tr>
<td>5</td>
<td>FLOW-THROUGH DRAIN</td>
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NOTES:

1. TAILINGS REPORTING TO THE DRY STACK FACILITY EXCEEDING THE MAXIMUM ALLOWABLE MOISTURE CONTENT OF 18% MUST BE PLACED AT LEAST 1,100 FT AWAY FROM THE UPSTREAM CREST AT ANY ELEVATION.

2. CLOSURE GEOMETRY PROVIDED BY TETRA TECH.
**STABILITY RESULTS**

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**MATERIAL PROPERTIES**

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<th>UNIT WEIGHT MOIST (pcf)</th>
<th>FRICTION ANGLE (DEGREES)</th>
<th>COHESION (psf)</th>
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TITLE: SOUTH DRY STACK TAILINGS FACILITY & 125-FOOT PONDING DEPTH STABILITY CROSS SECTION