**Technical Memorandum**

To: Kathy Arnold  
From: Ronson Chee  
Company: Rosemont Copper Company  
Date: August 31, 2010  
Re: Rosemont Heap Leach Design Responses  
Doc #: 237/10-320877-5.3  
CC: David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum provides information related to the Heap Leach Facility (HLF) that is part of the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is provided 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) in relation to the aquifer protection permit (APP) application submitted to ADEQ in April 2009 (Tetra Tech, 2009a) for the Rosemont Project. The report titled Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009b) was also reviewed as part of the APP application. Specifically, this Technical Memorandum answers the following items as referenced in the following sections:

- **Section 2.0: Geologic Hazards (item no. 1 on page 5 of 18)**

  *It is stated in the facility design report that the geological hazards associated with the Heap Leach Pad, PLS Pond, and Stormwater Pond include areas within the footprint of the facilities where material has been dumped in an uncontrolled manner or where native soils and/or rock had been reworked by human activities. These areas may present a hazard to future construction activities since the placement techniques are undocumented. (missing most of 2nd paragraph of the comment, insert here)… phase of the project. Rosemont must document the mitigation efforts employed to address those hazards, notify ADEQ if requiring action beyond normal construction procedures, and submit a copy to ADEQ as part of the as-built design.*

- **Section 3.0: Subgrade Material (item no. 2 on pages 5/6 of 18/19)**

  *Rosemont plans to construct a heap leach pad that will meet Prescriptive BADCT and proposes the use of geosynthetic clay liner (GCL) of 6 millimeter (mm) thickness and a permeability of $1 \times 10^{-9}$ cm/sec as an engineering equivalent. (Ref. Rosemont Heap Leach Facility Permit Design Report Volume 1 p. 28). As stated in the Arizona Mining BADCT Guidance Manual, Prescriptive BADCT design criteria for a heap leach pad composite liner requires that a geomembrane is underlain by at least 12 inches of native or natural 3/8-inch minus materials compacted in two 6-inch lifts to achieve a saturated hydraulic conductivity no greater than $1 \times 10^{-6}$ cm/sec.*

  *ADEQ will consider Rosemont’s proposed placement of geosynthetic clay liner (GCL) of 6 millimeter (mm) thickness and a permeability of $1 \times 10^{-9}$ cm/sec as an engineering equivalent provided it is demonstrated that:*
a) Strength properties of compacted subgrade under the liner are suitable for bearing load to prevent significant differential settlement.

b) Foundation settlement beneath the proposed pad footprint should not adversely affect the integrity of the Linear Low Density Polyethylene (LLDPE) liner.

- Section 4.0: Leachate Collection Pipes (Piping) Network (item no. 3 on page 6 of 18)

ADEQ will consider the leachate collection and header pipes network layout shown for the Heap Leach Facility Phase 1 and Phase 2 (DWG No. 080-CI-921 and DWG No.080-CI-928) provided the following design criteria are satisfied:

a) The maximum and average hydraulic head over the leach pad liner must not exceed 5 feet and 2 feet, respectively.

b) Pipe loading at the ultimate design height of heap does not threaten the structural integrity of the pipe. The collector and header pipes network can provide sufficient capacity for transporting leachate over the operational life of the facility; withstand the stresses caused by the maximum loading height of the ore heap without significant deformation or buckling and with adequate factor of safety. This demonstration should be based on manufacturer’s technical data on product specification or case studies for the pipes (ADS N-12 Corrugated High Density Polyethylene pipe) used under similar application.

- Section 5.0: Overliner Material (Durability) (item no. 4 on pages 6/7 of 18/19)

Rosemont has proposed the use of 1 1/2-inch minus crushed drainage layer versus 3/4-inch minus material as identified in the BADCT Guidance Manual.

Placement of 3/4-inch, well draining material with a minimum thickness of 18 inches is a design requirement to meet prescriptive BADCT for a heap leach pad overliner protective/drainage layer. ADEQ will consider the proposed use of a 36-inch layer of 1 ½-inch minus crushed material provided the following design criteria are satisfied:

a) As mentioned above, the maximum and average hydraulic head over the leach pad liner must not exceed 5 feet and 2 feet, respectively.

b) Particle size compatibility demonstration that material used will not clog the protective drainage layer and impair overliner drainage capacity.

c) Overliner Material Durability Test to demonstrate that the material used in the protective/drainage layer will not deteriorate when in contact with leachate solution during the service life of the facility.

- Section 6.0: Geomembrane Protection and Liner Puncture Test(ing) (item no. 5 on page 7 of 18)

Rosemont has proposed a minimum of 36 inches of overliner drain fill over the Heap Leach Pad as specified in the design criteria (Tetra Tech 2009) for the geomembrane protection. The material will be screened and or crushed, as needed, to produce a gradation with 100 percent of the material passing the 1.5 inch screen
and less than five percent passing the No. 200 screen. (Ref. Tetra Tech Technical Memorandum - Rosemont Heap Leach Geomembrane Protection, May 4, 2009).

As stated above, placement of 3/4-inch minus, well draining material with a minimum thickness of 18 inches is a design requirement to meet prescriptive BADCT.

ADEQ will consider the use of 1 1/2 -inch minus crushed overliner material if the proposed GCL liner of 6-mm thickness is demonstrated to show no severe indentations when puncture tested under simulated loading conditions by placing the subgrade material, geosynthetic(s), and the overliner material in the test cell. Rosemont has done puncture testing (3 tests) of 60-mil LLDPE with 1.5 inches minus overliner (QMP) drainage layer. However, the test results do not indicate the severity of indentation whether "minor", "moderate", or "severe" dimpling of the geomembrane sample has occurred. There is no indication how these indentations or dimpling affects durability of the geomembrane. ADEQ considers three trials of puncture tests inadequate to verify the liner system behavior under simulated field conditions. Please conduct additional tests. If severe dimpling is noticed in higher frequency which causes noticeable decrease in achievable strain, ADEQ recommends that a cushion or bedding layer should be included between the overliner and the geomembrane as an added protective layer.

- **Section 7.0: Anchor Trench (item no. 6 on page 7 of 18)**

  Please submit stability calculations supporting the design for the anchor trench within the perimeter containment berm. This feature is a critical component with respect to pad stability.

- **Section 8.0: Underdrain/Phase 2 Underdrain System (item no. 7 on page 7 of 18)**

  Current leach pad layout (DWG. 080-Cl-928) shows an underdrain on western perimeter of Phase 2 of the Heap Leach Facility. (Ref. Rosemont Heap Leach Facility, Permit Design Report, Volume 1, May 2009)

  Please indicate the design criteria used for the underdrain indicating estimated amount of surface and subsurface flow designed for discharge through the underdrain system.

- **Section 9.0: Heap Leach Pad Design Modification (item no. 8 on page 8 of 18)**

  In a meeting held on March 17, 2010, between ADEQ project Team and Rosemont personnel and Consultants, Rosemont indicated that the Heap Leach Pad originally designed as two-phase construction is being revised as a single phase construction. Please submit the revised final design, including the final foot print, ultimate design height of the pile, approximate tonnage to be piled on the Heap Leach Pad, stability analysis of the final configuration of the ore heap, revised leachate collection piping network and any other significant changes/modifications made in the final design of the Heap Leach Pad.
Section 10.0: Technical Memorandum, Rosemont Heap Leach Facilities – Liner Leakag... (item 27 on page 12 of 18)

The alert level AL2 (Rapid and Large Leakage) for each of the Raffinate Pond and the PLS Pond is calculated at 15,272 gpd and 46,812 gpd, respectively.

Rosemont's proposed alert level for each of the Raffinate Pond and the PLS Pond appears to be excessively high and shall be revised. Analytical calculations shall be based on system components, taking into account geomembrane defects, transmissivity of the drainage medium, design capacity of the leak collection and removal system (LCRS) rather than discharging capability of the pumping system alone at the LCRS. Please provide revised calculations.

Section 11.0: APP Volume 1, Table 7.16 – Raffinate Pond Volume Requirements (item no. 28 on page 12 of 18)

There is a discrepancy in the Raffinate Pond Volume Requirements and the Total Volume Required. Please reconcile the volumes (Minimum Pool Volume, Design Operating Volume and Freeboard Volume) to reflect the correct volume of raffinate required in the Raffinate Pond.

2.0 Geological Hazards

Figure 03 from the Geologic Hazards Assessment report (Tetra Tech, 2007) is provided in Attachment A of this Technical Memorandum in order to answer item 1 on page 5 of 18. As indicated on the figure, artificial fill material is potentially located within the footprint of the proposed Heap Leach Facility along with other areas of loose soil and rock that may experience erosion during periods of intense rainfall. Artificial fill material located with the footprint of the Heap Leach Facility will either be removed or reworked, i.e., recompacted, to provide a stable foundation. Additionally, loose soil or rock materials will generally be reworked as part of the soil stripping and foundation preparation activities.

The other two (2) hazards listed on the figure, rockfall areas and abandonment mine workings, are not located with the planned facility footprint of the Heap Leach Facility. These facilities are also not present within the planned Phase 2 Pad area which is not shown on Figure 03 in Attachment A. However, in general, the following practices would pertain to these hazards:

- Rockfall Areas: Stability analysis and/or site specific evaluations would be conducted as needed for areas subject to rock fall hazards to determine mitigation measures. As indicated, Potential rock fall areas are not currently present within the footprint of the Heap Leach Facility.

- Abandoned Mine Workings: Workings would be filled as needed depending on the affected facility. The method of filling and degree of compaction/settlement of the fill material would depend on the degree of settlement allowed. This would be a site specific assessment. A site specific closure/remediation plan would be developed for those abandoned mine workings that could adversely affect a facility by causing differential settlement. As indicated, Abandoned Mine Workings are not currently present within the footprint of the Heap Leach Facility. The majority of Abandoned...
Mine Workings are within the Open Pit area which will be mined out. If present, abandoned Mine Workings within the footprint of the Dry Stack Tailings Facility or Waste Rock Storage Area would be filled with loose material (uncompacted).

3.0 Subgrade Material
In response to item 2 on pages 5/6 of 18/19, a load bearing and settlement calculation was performed for the compacted subgrade associated with the Heap Leach Pad. The analysis results show that the subgrade, as currently designed, is suitable to prevent significant differential settlement and any adverse affect on the liner system. A Technical Memorandum titled *Rosemont Heap Leach Pad Settlement Analysis* (Tetra Tech, 2010a) is provided in Attachment B.

4.0 Leachate Collection Piping Network
In response to item 3 on page 6 of 18, calculations were performed which conclude that the hydraulic head over the liner does not exceed maximum and average values of five (5) feet and 2 feet, respectively. Calculations are provided in a Technical Memorandum titled *Rosemont Leachate Collection Pipes Hydraulic Head Calculations* (Tetra Tech, 2010b) in Attachment C.

In response to item 3b on page 6 of 18, pipe crushing calculations were performed which show that the leachate collection pipes are adequate to handle the anticipated loads. Calculations are provided in a Technical Memorandum titled *Rosemont Heap Leach Pad Drain Pipe Deflection* (Tetra Tech, 2010c) in Attachment D.

5.0 Overliner Material Durability
A response to item 4a on page 6 of 18, a Technical Memorandum was prepared as described in Section 4.0 titled *Rosemont Leachate Collection Pipes Hydraulic Head Calculations* (Tetra Tech, 2010b). This Technical Memorandum is provided in Attachment C.

In response to items 4b and 4c on page 7 of 19, a Technical Memorandum was prepared titled *Rosemont Overliner Drainage and Durability* (Tetra Tech, 2010d). This Technical Memorandum is provided in Attachment E. In summary degradation of the overliner material is not anticipated.

6.0 Geomembrane Protection and Liner Puncture Testing
In response to item 5 on page 7 of 18, additional liner puncture tests were conducted on the proposed liner system associated with the Rosemont Heap Leach Pad. Test results indicate that the liner system performed adequately to the anticipated high stacking loads. Details of the liner puncture testing are provided in a Technical Memorandum titled *Liner Puncture Testing for the Proposed Rosemont Heap Leach Pad* (Tetra Tech, 2010e) in Attachment F.

7.0 Anchor Trench
An anchor trench stability analysis was performed in response to item no. 6 on page 7 of 18. A Technical Memorandum was prepared titled *Rosemont Heap Leach Pad Anchor Trench Stability* (Tetra Tech, 2010f) which describes the calculations performed in analyzing the stability of the anchor trench (see Attachment G).
8.0 Phase 2 Pad Underdrain System

In response to item 7 on page 7 of 18, hydrologic and hydraulic calculations were performed to ensure that the underdrain system routing stormwater underneath the Phase 2 Heap Leach Pad is adequate to convey flows. Calculations are provided in Attachment H in a Technical Memorandum titled *Rosemont Phase 2 Heap Leach Pad Underdrain Design* (Tetra Tech, 2010g).

9.0 Heap Leach Pad Design Modification

After analysis of the single heap leach pad option, the Phase 1 and Phase 2 Heap Leach Pad design configuration will be retained as shown in the report titled *Rosemont Heap Leach Facility Permit Design Report* (Tetra Tech, 2009b) submitted to ADEQ in May 2009 as part of the APP application (Tetra Tech, 2009a).

10.0 Heap Leach Facility Liner Leakage Calculations

In response to item no. 27 on page 12 of 18, liner leakage calculations were revised and are presented in the Technical Memorandum titled *Rosemont Heap Leach Facility Permit Design Liner Leakage Calculations* (Tetra Tech, 2010h) in Attachment I.

11.0 Raffinate Pond Volume Requirements

In response to item no. 28 on page 12 of 18, a revision to the method of volume reporting was made in a separate Technical Memorandum titled *Rosemont Raffinate Pond Volume Requirement* (Tetra Tech, 2010i) in Attachment J.
REFERENCES


ATTACHMENT B
TECHNICAL MEMORANDUM
ROSEMONT HEAP LEACH PAD SETTLEMENT ANALYSIS
Technical Memorandum

To: Joel Carrasco
From: Marvin Silva, P.E.
Company: Tetra Tech
Date: August 11, 2010
Re: Rosemont Heap Leach Pad Settlement Analysis
Doc #: 209/10-320877-5.3
CC: David Krizek, P.E.

1.0 Introduction

This Technical Memorandum provides a summary of Tetra Tech's settlement analysis related to the Heap Leach Facility (HLF) at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is 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). Specifically, this Technical Memorandum answers item no. 2 on pages 5 and 6 of 18.

- **Subgrade Material** – Rosemont plans to construct a heap leach pad that will meet Prescriptive BADCT and proposes the use of geosynthetic clay liner (GCL) of 6 millimeter (mm) thickness and a permeability of $1 \times 10^{-9}$ cm/sec as an engineering equivalent. (Ref. Rosemont Heap Leach Facility Permit Design Report Volume 1 p. 28).

As stated in the Arizona Mining BADCT Guidance Manual, Prescriptive BADCT design criteria for a heap leach pad composite liner requires that a geomembrane is underlain by at least 12 inches of native or natural 3/8-inch minus materials compacted in two 6-inch lifts to achieve a saturated hydraulic conductivity no greater than $1 \times 10^{-6}$ cm/sec.

ADEQ will consider Rosemont's proposed placement of geosynthetic clay liner (GCL) of 6 millimeter (mm) thickness and a permeability of $1 \times 10^{-9}$ cm/sec as an engineering equivalent provided it is demonstrated that:

a) Strength properties of compacted subgrade under the liner are suitable for bearing load to prevent significant differential settlement.

b) Foundation settlement beneath the proposed pad footprint should not adversely affect the integrity of the Linear Low Density Polyethylene (LLDPE) liner.

Tetra Tech has proposed to use a geosynthetic clay liner (GCL) underneath the proposed 60 mil double-side textured LLDPE liner in lieu of a 12-inch thick layer of compacted low-permeability material. The purpose of the calculations presented herein was to estimate the maximum settlement in the foundation soils of the Heap Leach Pads, followed by a determination of...
possible differential settlement and its effect on the proposed liner system (against allowable strain). This analysis takes into account loading due to 350 feet of oxide ore material.

The results of our calculations presented in this Technical Memorandum indicate that the liner system will not be damaged by settlement induced by the weight of ore material.

2.0 Settlement Calculation

2.1 Settlement Prediction Method

Foundation settlements were calculated using the Schmertmann strain influence methodology. Originally proposed by Schmertmann (1970) and modified by Schmertmann, Hartmann, and Brown (1978), this method was developed to estimate foundation settlements in cohesionless soils. This procedure provides settlement compatible with field measurements in many different areas. The analysis assumes that the distribution of vertical strain is compatible with a linear elastic half space subjected to a uniform pressure. To utilize this method, the subsurface is broken into layers. Each layer has a constant value of strain and soil modulus. The soil parameters used in the settlement calculation were selected based on the results of the geotechnical investigation performed by Tetra Tech (2007). Settlement is calculated by summing the influence of all layers, as calculated by equation (1):

\[ \Delta H = C_1 C_2 \Delta P \sum_{i=1}^{n} (l_{zi}) \Delta z_i \]  

where:

- \( C_1 \) = embedment correction factor = 1 - 0.5(\( \sigma_{od} / \Delta p \)) ≥ 0.5
- \( C_2 \) = creep correction factor = 1 + 0.2 log (10t)
- \( \sigma_{od} \) = overburden pressure at foundation level or depth d, tsf
- \( \Delta P \) = net foundation pressure increase = q - \( \sigma_{od} \), tsf
- \( t \) = lapsed time in years
- \( l_{zi} \) = influence factor of soil layer i
- \( E_i \) = elastic modulus of soil layer i, 0.2B, tsf
- \( \Delta z_i \) = depth increment i, inches

Schmertmann developed the diagram shown in Illustration 1 to determine the appropriate strain influence factor, \( l_z \), for each layer within the profile. Two distributions are shown: one for square or circular footings (L/B=1, axisymmetric), and a second for strip footings (L/B>10, plane strain). Both are triangular distributions, and the one for square or circular footings begins at a value of 0.1 at the base of the footing, while the one for strip footings begins at a value of 0.2 at the base of the footing. The maximum strain factor, \( l_{zp} \), occurs at a depth equal to B/2 for square footings and B for strip footings, and is calculated using equation (2):

\[ l_{zp} = 0.5 + 0.1 \left( \frac{\Delta P}{\sigma_{od}} \right) \]  

(2)
where:

\[ \sigma'_{izp} = \text{initial effective stress at the depth of maximum strain influence} \]

Axisymmetric: \( \sigma'_{izp} = 0.5 B \gamma' + D \gamma' \)

Plane strain: \( \sigma'_{izp} = B \gamma' + D \gamma' \)

where:

\[ \gamma' = \text{effective unit weight, tcf} \]
\[ B = \text{footing width, ft} \]
\[ D = \text{excavated or embedded depth, ft} \]

\( \text{tons per cubic foot} \)

Values of soil modulus, \( E_s \), were established using the following relationship (NAVFAC, 1986) for coarse sands and sands with little gravel (alluvium):

\[ E_s = 10 N_{spt} \text{ (tsf)} \]  \( \text{(3)} \)

where:

\[ N_{spt} = \text{corrected blow count from standard penetration tests (SPT)} \]

Illustration 1  Strain Influence Factors for Schmertmann's Approximation (From Engineer Manual No. 1110-1-1904, USACOE, 30 Sep 90, page 3-8)
2.2 Settlement Analysis

Based on the results of the geotechnical investigation, the foundation soils below the Heap Leach Pads are expected to consist primarily of alluvium (Gila Conglomerate with some Apache Canyon Formation and some Younger Alluvium) overlying weathered bedrock. The alluvium thickness is variable and it was found to be as thick as 50 feet in the wash area. The alluvium is predominantly a moderately to weakly cemented sand with silt and gravel. Because the bedrock is much stiffer, the majority of the elastic compression is expected to occur in the alluvium deposits. Illustration 2 shows the geometry of the HLF and the profile of the foundation soil. It was assumed that the alluvium deposit has a thickness of 50 feet. Based on this geometry, the maximum foundation loading is expected to occur at point 2, which will have approximately 350 feet of ore material.
The maximum differential settlement is expected to occur between point 1 and point 2. Between these two points the liner system will experience the maximum tensile stresses and the maximum elongation. The liner system will increase in length from the original length \( L_o = 700 \) feet to the final length \( L_f \). The strain on the foundation liner system was calculated using the following steps:

- Use the Schmertmann's method to calculate the settlement \( \Delta H \) at point 2;
- Determine the differential settlement between points 1 and 2 (settlement at point 1 is assumed to be zero);
- Calculate the change in length of the liner system caused by the differential settlement; and
- Calculate the strain on the liner system by dividing the change in the liner length by the initial, pre-settlement liner length between point 1 and point 2, using Equation (4). This calculation assumes that the strain will be uniformly distributed across the liner.

\[
\text{Strain (\%) } = \frac{L_f - L_o}{L_o} \times 100
\]  
(4)

In order to conduct the settlement calculation, it was assumed that the maximum settlement at point 2 will be caused by a strip footing (plane strain) whose strain influence will reach the top of the weathered bedrock (rigid base assumed to be uncompressible). Therefore, a 12.5-feet wide strip footing was assumed in the settlement calculation. This footing width \( B=12.5 \) feet will induce strains up to a depth of 50 feet \( (4B) \), which is the depth of the top of the rigid base. Illustration 3 shows the geometry of the assumed conditions.

Illustration 3  Assumed Geometry for Calculation of Maximum Settlement (not to scale)

2.3 Settlement Results

Table 1 shows the Schmertmann's method to calculate the settlement at point 2. The material parameters used in the settlement calculations are:

\[
\gamma_{om} = \text{unit weight of the ore material} = 125 \text{pcf}
\]
\[
\gamma = \text{unit weight of compacted subgrade and alluvium deposit} = 132.7 \text{pcf}
\]
B = width of slice at point B causing maximum settlement = 12.5 feet

Illustration 3 shows the strain factor distribution calculated using the procedure described in Section 2.1.1. This illustration was used to determine the strain factors in Column 9 of Table 1 using the values of Column 8.

Table 1 Settlement Calculation Table using Schmertmann's Method

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<th>E/N</th>
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</tr>
</tbody>
</table>

\[ \sum_{i=1}^{22} \left( \frac{I_z}{E_i} \right) = 1.577 \]

The settlement at point B is \( \Delta H = C_4 C_2 \Delta p \sum_{i=1}^{n} \left( \frac{I_z}{E_i} \right) \Delta z_i = 41.4 \text{ inches (3.4 feet)} \).
The final length of the liner system due to the differential settlement is

\[ L_f = \sqrt{L_o^2 + \Delta H^2} = \sqrt{700^2 + 3.4^2} = 700.008 \text{ feet}, \text{ and the strain of the liner system is} \]

\[ \text{Strain (\%)} = \frac{L_f - L_o}{L_o} \times 100 = 0.0012\%. \]

---

Illustration 3  Strain Influence Factor Distribution at Point 2

2.4 Criteria for Determining Acceptable Settlements

The maximum allowable strain on the liner system is controlled by the strain tolerance of the LLDPE and the GCL components. The allowable yield strain for the proposed 60 mil double-side textured LLDPE is 12 percent, and the elongation at break is 250 percent. There is additional concern when a geomembrane is exposed to tension perpendicular to seams. In these cases, a general rule-of-thumb is that the allowable strain on the geomembrane is about half the value of the un-seamed sheet material (Giroud et al. 1995). For this reason, horizontal seams are not allowed on side slopes. Tensile stresses applied to a geomembrane parallel to the seams are generally not a large concern, provided that the seams are good quality, and
were installed in accordance with the specifications. For these reasons, strains of up to 12 percent will be considered acceptable for the proposed LLDPE geomembrane.

For GCL materials, the yield strain is not typically included on standard specifications. For these materials, the yield strain is typically controlled by the geotextile layers on the top and bottom of the clay. Geotextiles generally have yield strains in excess of 50 percent. The bentonite component of GCLs also has a high strain tolerance, and can heal cracks (if they occur) over time. If the GCL were to experience such large strains, thinning of the bentonite layer (and a corresponding increase in permeability) would likely be the primary concern. Differential settlement studies performed using GCLs show that they can maintain a hydraulic conductivity below $1 \times 10^{-7}$ cm/sec when subjected to strains between 1 and 10 percent (LaGatta et al. 1997). A second concern would be the GCL panel overlap. To avoid separation of panels caused by strain on the liner system, project specifications include required overlaps twice as large as typical manufacturer recommended overlaps.

In summary, the least strain-tolerant component of the liner system is the GCL. Accordingly, the maximum acceptable strain on the liner system is 10 percent, which is the allowable yield strain of the GCL to maintain the specified hydraulic conductivity.

The settlement calculation shows that the differential settlement on the foundation liner system caused by the ore material will be approximately 3.4 feet over an initial length of 700 feet. This differential settlement will produce an increase in the liner system length of 0.006 feet, which is equivalent to a strain of 0.0012 percent. This strain is below the suggested allowable strain of the GCL of ten percent. Therefore, the liner system will not be damaged by settlement induced by the weight of the ore material and the liner system will maintain its integrity.

3.0 Conclusions

The settlement calculations show that the maximum differential settlement on the foundation induced by the weight of the ore material will not damaged any component of the proposed liner system. The strains imposed on the LLDPE and GCL liners will be within allowable limits.
REFERENCES


ATTACHMENT C
TECHNICAL MEMORANDUM
ROSEMONT LEACHATE COLLECTION PIPES HYDRAULIC HEAD CALCULATIONS
Technical Memorandum

To: Joel Carrasco  
Company: Tetra Tech  
Re: Rosemont Leachate Collection Pipes Hydraulic Head Calculations  
CC: David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum presents supporting design information related to the leachate collection (drain) piping layout for the Heap Leach Pads at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is provided in response to the April 14, 2010 Comprehensive Request for Additional Information provided by the Arizona Department of Environmental Quality (ADEQ) to Rosemont Copper Company (Rosemont). Specifically, this Technical Memorandum answers item no. 3a on page 6 of 18:

- Item 3a – Leachate Collection Pipes Network: ADEQ will consider the leachate collection and header pipes network layout shown for the Heap Leach Facility Phase 1 and 2 (DWG No. 080-C1-921 and DWG No. 080-C1-928) provided the following design criteria are satisfied: The maximum and average hydraulic head over the leach pad liner must not exceed 5 feet and 2 feet, respectively.

2.0 Rosemont Heap Leach Pad Drain Pipe Design – APP Application

Attachment 1 provides the Technical Memorandum titled Rosemont Heap Leach Drain Line Design dated May 4, 2009 (Tetra Tech, 2009a). This Technical Memorandum was originally provided in the May 2009 report titled Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009b) as part of the aquifer protection permit (APP) application (Tetra Tech, 2009c) submitted to ADEQ in February 2009. Attachment 2 provides the following drawings from the May 2009 permit design report. Further details of the piping network are provided in the report.

- DWG No. 080-C1-925: Heap Leach Facility Phase 1 Drain Pipe Layout
- DWG No. 080-C1-929: Heap Leach Facility Phase 2 Drain Pipe Layout

The Heap Leach Facility was designed to meet or exceed the Prescriptive Best Available Demonstrated Control Technology (BADCT) described in the ADEQ Arizona Mining BADCT Manual (ADEQ, 2004). Under the prescriptive BADCT criteria, engineering equivalents to specific elements are deemed acceptable as long as supporting evidence is provided to ADEQ.
The attached Technical Memorandum indicates that the drain pipe spacing was determined by the equation below for estimating the peak hydraulic head on the pad liner system between drain pipes (McWhorter and Sunada, 1977).

\[ H = \frac{L}{2} \left( \frac{W}{K} \right)^{0.5} \]

Where: 
- \( H \) = Maximum Hydraulic Head on liner at the midpoint between pipes;
- \( L \) = Drain Pipe Spacing;
- \( W \) = Application Rate; and
- \( K \) = Hydraulic Conductivity.

Based on the results of the calculations presented in Attachment 1, the following design criteria was maintained:

- Maximum head on liner = 5.0 feet; and
- Average hydraulic head on liner = 2.0 feet.
REFERENCES


1.0 Introduction

This technical memorandum presents the leach pad drain pipe sizing results for the Phase 1 and Phase 2 Heap Leach Pads at the Rosemont Copper Project. The proposed leach pad phases will contain a piping network to accommodate a 100-year, 24-hour storm event in addition to flows from leaching activities. Drain pipe sizing calculations are provided in Attachment A. Drawings of the Heap Leach Facility, including the proposed piping network, are included in the Heap Leach Facility Permit Design Report (Tetra Tech, 2009a). The drain pipes are located within the free draining overliner drain fill material above the pad liner.

The Heap Leach Facility was designed to meet or exceed the Prescriptive Best Available Demonstrated Control Technology (BADCT) described in the Arizona Department of Environmental Quality (ADEQ) Arizona Mining BADCT Manual (ADEQ, 2004). Under the prescriptive BADCT criteria, engineering equivalents to specific elements are deemed acceptable as long as supporting evidence is provided to ADEQ.

2.0 Pad Drain Pipe Sizing Design

2.1 General

Phase 1 and Phase 2 of the Heap Leach Pad are designed to contain a network of pipes that will be distributed throughout the limits of the Heap Leach Pad and that will collect and convey pregnant leach solution (PLS) in addition to stormwater. The pipe network was designed to accommodate stormwater from a 100-year, 24-hour storm event in addition to 150% of the design capacity of the anticipated PLS solution flow (150% PLS flow + 100 year, 24 hour storm event).
2.2 Assumptions

The following parameters were used in the calculations for the Heap Leach Pad drain pipe design. These parameters were based on design criteria provided in the Rosemont Copper Heap Leach Facility Design Criteria (Tetra Tech, 2009b) and standard engineering practice.

- The leach solution application flow rate was estimated to be 0.0040 gpm/ft², with an active application surface area of 1,000,000 ft².
- Total flow capacity to be accounted for in the pad drain system by taking the design storm event plus 1.5 times the application flow rate (0.0060 gmp/ft²). The 100-year, 24-hour storm precipitation is 4.75 inches (NOAA, 2008).
- Manning’s “n” of 0.014 was selected for the N-12 perforated pipes to account for the manufactured perforations and wear over time.
- Flow conditions assumed pipes were 85% full for maximum conveyance under gravity.
- The drain system design requirement assumed no moisture absorption and retention losses in the ore material.

2.3 Overliner Drain Fill

The design criteria specified an overliner drain fill permeability to be equivalent to $1 \times 10^{-2}$ cm/sec or greater under a 300 ft maximum ore heap load to ensure reasonable spacing of the drain pipes and to ensure fully drained heap conditions. The overliner drain fill thickness used was a minimum thickness of three (3) feet (loose lift) placed above the pad liner surface.

The overliner drain fill will be produced from the crushing of relatively clean ore materials. Some screening may be required to produce a free draining, non-plastic overliner drain fill material with a minus 1.5 inch maximum rock size, less than 20 percent passing the No. 4 ASTM sieve size, and less than 5 percent fines passing the No. 200 ASTM sieve size. The pad liner system requires complete coverage with overliner drain fill as soon as practical to avoid any potential wind or construction related damage.

2.4 Pad Drain Pipe

The pipe network to be constructed consists of a series of four (4) inch drain pipes arranged in a herringbone pattern that will collect and convey the fluid, i.e., PLS and stormwater flows, toward the center of the Heap Leach Pad. The four (4) inch drain pipes will be connected to a series of eight (8) inch primary pipes and 12 inch collector pipes. The 12 inch collector pipes will transmit fluid to 18 inch header pipes located at the toe of the Phase 1 and Phase 2 pads.

The 18 inch header pipes from the Phase 1 Pad will discharge fluids directly into the PLS Pond. The 18 inch header pipes from the Phase 2 Pad will connect to two (2) 18 inch HDPE solution pipes located in the collection ditch adjacent to the north of the Phase 1 pad. Solution from Phase 2 will be contained within the header pipes during normal operation. During storm events, fluid may be conveyed in the lined collection ditch to the PLS Pond and Stormwater Pond.

The heap leach pad drain pipes will consist of 4, 8, 12, and 18 inch diameter corrugated, dual wall, perforated ADS N-12 pipes. Each phase (Phase 1 and Phase 2) will contain two (2) 18 inch header pipes with a minimum grade of 3.0% sloped toward the PLS Pond where the fluid will be collected. Each phase was divided into two (2) areas for design purposes, Phase 1 North and South and Phase 2 North and South areas (see Figure 01).
3.0 Drain Sizing Calculations

1) Phase 1 was divided into two (2) areas using the natural ridgeline, North and South. Each of these two (2) areas were divided into an upper section and a lower section based on slope. Phase 2 was divided using the same procedure. Pad sizes for each area and minimum design pipe slopes are summarized in Table 3.1.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Location</th>
<th>Total Area (ft²)</th>
<th>Minimum Design Slope (ft/ft)</th>
<th>Average Pipe Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 North</td>
<td>Upper</td>
<td>2,051,839</td>
<td>0.03</td>
<td>1800</td>
</tr>
<tr>
<td></td>
<td>Lower</td>
<td></td>
<td></td>
<td>650</td>
</tr>
<tr>
<td>Phase 1 South</td>
<td>Upper</td>
<td>2,318,321</td>
<td>0.03</td>
<td>1400</td>
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<td></td>
<td>Lower</td>
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<tr>
<td></td>
<td>Lower</td>
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<td>1275</td>
</tr>
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2) Primary and collector pipe capacities were calculated using Manning’s equation shown below (Chow, 1959). Pipe design slopes were obtained from design contours within the Heap Leach Pad limits.

\[
Q = \left(1.486/n\right) \times A \times (R)^{2/3} \times (S)^{1/2}
\]

where:
- \(Q\) = discharge (cfs)
- \(n\) = Manning’s roughness coefficient
- \(A\) = cross sectional area of the pipe (ft²)
- \((R)^{2/3}\) = hydraulic radius at 85% capacity
- \((S)^{1/2}\) = pipe slope (ft/ft)

3) The total flow for each area of Phase 1 and Phase 2 was calculated using 150% of the design PLS flow + stormwater generated from a 100-year, 24-hour event. The eight (8) inch primary pipes are located in the upper areas in each of the Phases. The 12 inch
collector pipes are located in the lower areas of the pad where the slopes are less steep. The total flows for each area are summarized in Table 3.2.

Table 3.2 – Summary of Total Flow

<table>
<thead>
<tr>
<th>Phase</th>
<th>Location</th>
<th>Total Flow for 8” Ø primary pipes (gpm)</th>
<th>Total Flow for 12” Ø collector pipes (gpm)</th>
<th>Total Flow for 18” Ø header pipes (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 North</td>
<td>Upper</td>
<td>7,761</td>
<td>8,246</td>
<td>12,843</td>
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<tr>
<td>Phase 1 South</td>
<td>Upper</td>
<td>8,407</td>
<td>9,096</td>
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<tr>
<td>Phase 2 North</td>
<td>Upper</td>
<td>8,515</td>
<td>9,450</td>
<td>12,825</td>
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<td>Phase 2 South</td>
<td>Upper</td>
<td>6,785</td>
<td>7,875</td>
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4) Drain pipe spacing was determined by the equation below for estimating the peak hydraulic head on the pad liner system between drain pipes (McWhorter and Sunada 1977).

\[
H = \left(\frac{L}{2}\right) \times \left(\frac{W}{K}\right)^{0.5}
\]

where:
- \(H\) = maximum mid-point hydraulic head on liner (5 feet for a 2 feet overall head)
- \(L\) = drain pipe spacing (to be determined)
- \(W\) = application rate of 0.006 gpm/ft\(^2\) (150% PLS)
- \(K\) = hydraulic conductivity (permeability) of pad drain material (1 x 10\(^{-2}\) cm/sec)

4.0 Summary

The Heap Leach Pad drain pipe design has been designed to provide engineering equivalence to the prescriptive BADCT standards. The supporting evidence of engineering equivalents are documented herein (Section 3.0). Calculations are provided in Attachment A.

The results summarized in Table 4.1 are the minimum quantities of pipes needed to satisfy the design for each phase of the Heap Leach Pad.
Table 4.1 – Summary of Pipe Quantities

<table>
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<tr>
<th>Phase</th>
<th>Quantity of 8” Ø primary pipes</th>
<th>Quantity of 12” Ø collector pipes</th>
<th>Quantity of 18” Ø collector pipes</th>
</tr>
</thead>
<tbody>
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<td>2</td>
</tr>
<tr>
<td>Phase 1 South</td>
<td>6</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Phase 2 North</td>
<td>7</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>Phase 2 South</td>
<td>5</td>
<td>4</td>
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</tr>
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Prescriptive BADCT criteria for Heap Leach Pads specifies that the hydraulic head over a liner should be less than two (2) feet and no more than five (5) in order to drain the ore and minimize the potential for leakage through the pad liner system. Calculating the hydraulic head using the assumed values in Section 3.0 indicate that for an average overall head of two (2) feet on the liner, the four (4) inch drain pipe minimum spacing of 50 feet is required.

5.0 References


ATTACHMENT A
Slotted ADS Pipe
Mannings n: 0.014
Pipe Slope (m/m): 0.050

<table>
<thead>
<tr>
<th>Pipe Inside Diameter (mm)</th>
<th>Flow (m³/s)</th>
<th>Flow Depth (mm)</th>
<th>Flow Velocity (m/s)</th>
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<tr>
<td>75</td>
<td>0.005</td>
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<td>1.28</td>
</tr>
<tr>
<td>100</td>
<td>0.011</td>
<td>85</td>
<td>1.55</td>
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Slotted ADS Pipe
Mannings n: 0.014
Pipe Slope (m/m): 0.100

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<th>Flow (m³/s)</th>
<th>Flow Depth (mm)</th>
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<tr>
<td>75</td>
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<tr>
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<td>0.016</td>
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Slotted ADS N12 Pipe
Mannings n: 0.014
Pipe Slope (m/m): 0.080

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<th>Flow (m³/s)</th>
<th>Flow Depth (mm)</th>
<th>Flow Velocity (m/s)</th>
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<td>3.12</td>
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<td>0.262</td>
<td>255</td>
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<td>4.74</td>
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</tr>
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<td>760</td>
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Mannings n: 0.014
Pipe Slope (m/m): 0.030

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<th>Flow Velocity (m/s)</th>
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<tr>
<td>760</td>
<td>1.911</td>
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<td>4.65</td>
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Mannings n: 0.014
Pipe Slope (m/m): 0.020

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<th>Flow Depth (mm)</th>
<th>Flow Velocity (m/s)</th>
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</thead>
<tbody>
<tr>
<td>450</td>
<td>0.386</td>
<td>383</td>
<td>2.68</td>
</tr>
</tbody>
</table>
**Client:** Rosemont Copper Company  
**Job No.:** 114-320807  
**Subject:** Heap Leach Facility  
**Details:** Pad Pipe Sizing  
**By:** JAC  
**Date:** April 10, 2009

### Detailed Analysis

**Avg. Solution Outflow Rate (m³/hr):** 681.4 m³/hr  
3,000 gpm  
(From Design Criteria)

**100-Year 24-Hour Precipitation (m):** 0.71 m  
4.75 inches  
(From NOAA Atlas 14)

<table>
<thead>
<tr>
<th>Total Area (m²)</th>
<th>Total Area (ft²)</th>
<th>Primary Pipe Design Slope (m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 - North Upper: 147,328</td>
<td>1,585,824</td>
<td>0.080</td>
</tr>
<tr>
<td>Phase 1 - South Upper: 176,521</td>
<td>1,900,057</td>
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</tr>
<tr>
<td><strong>Total Phase 1:</strong></td>
<td><strong>323,849</strong></td>
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</tr>
<tr>
<td>Phase 2 - North Upper: 181,403</td>
<td>1,952,603</td>
<td>0.080</td>
</tr>
<tr>
<td>Phase 2 - South Upper: 103,232</td>
<td>1,111,177</td>
<td>0.080</td>
</tr>
<tr>
<td><strong>Total Phase 2:</strong></td>
<td><strong>284,634</strong></td>
<td><strong>3,063,780</strong></td>
</tr>
<tr>
<td><strong>Total Pad Upper Area:</strong></td>
<td><strong>608,483</strong></td>
<td><strong>6,549,661</strong></td>
</tr>
</tbody>
</table>

#### Primary Pipes

**150% PLS Solution + 100-Year Storm Flow**

<table>
<thead>
<tr>
<th>Area</th>
<th>PLS Flow (m³/hr)</th>
<th>Storm Flow (m³/hr)</th>
<th>Total Flow (m³/hr)</th>
<th>Total Flow (gpm)</th>
<th>No. Pipes</th>
<th>Pipe Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 - North Upper: 1,022</td>
<td>741</td>
<td>1763</td>
<td>7761</td>
<td>6</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>Phase 1 - South Upper: 1,022</td>
<td>887</td>
<td>1909</td>
<td>8407</td>
<td>6</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>Phase 2 - North Upper: 1,022</td>
<td>912</td>
<td>1934</td>
<td>8515</td>
<td>7</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>Phase 2 - South Upper: 1,022</td>
<td>519</td>
<td>1541</td>
<td>6785</td>
<td>5</td>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>

**Check Pipe Capacity Secondary Pipes**

Calculated Pipe Spacing = 15 m (based on McWhorter equation)

**Controlling Design Pipe Slope** = 0.10 m/m

**150% PLS Solution + 100-Year Storm Flow**

<table>
<thead>
<tr>
<th>Phase</th>
<th>PLS Flow (m³/hr)</th>
<th>Storm Flow (m³/hr)</th>
<th>Total Flow (m³/hr)</th>
<th>Total Flow (gpm)</th>
<th>No. Pipes</th>
<th>Pipe Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 - North Upper: 1,022</td>
<td>741</td>
<td>1763</td>
<td>7761</td>
<td>31</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Phase 1 - South Upper: 1,022</td>
<td>887</td>
<td>1909</td>
<td>8407</td>
<td>33</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Phase 2 - North Upper: 1,022</td>
<td>912</td>
<td>1934</td>
<td>8515</td>
<td>34</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Phase 2 - South Upper: 1,022</td>
<td>519</td>
<td>1541</td>
<td>6785</td>
<td>27</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

15m spacing ok for capacity
### Heap Leach Facility By: JAC

Details: Pad Pipe Sizing  
Date: April 10, 2009

**Avg. Solution Outflow Rate (m³/hr):** 681.4  
100-Year 24-Hour Precipitation (m): 0.121

- From Design Criteria
- From NOAA Atlas 14

<table>
<thead>
<tr>
<th>Total Area (m²)</th>
<th>Total Area (ft²)</th>
<th>Collector Pipe Design Slope (m/m)</th>
<th>Header Pipe Design Slope (m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Phase 1 - North:</strong> 169,254</td>
<td>1,821,836</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Phase 1 - South:</strong> 207,666</td>
<td>2,235,299</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Total Phase 1:</strong> 376,920</td>
<td>4,057,135</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Phase 2 - North:</strong> 223,628</td>
<td>2,407,107</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Phase 2 - South:</strong> 152,506</td>
<td>1,641,557</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Total Phase 2:</strong> 376,133</td>
<td>4,048,664</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td><strong>Total Pad Area:</strong> 753,053</td>
<td>8,105,799</td>
<td>0.030</td>
<td>0.030</td>
</tr>
</tbody>
</table>

#### Header Pipes

<table>
<thead>
<tr>
<th>Area</th>
<th>PLS Flow (m³/hr)</th>
<th>Storm Flow (m³/hr)</th>
<th>Total Flow (m³/hr)</th>
<th>Total Flow (gpm)</th>
<th>No. Pipes</th>
<th>Pipe Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1</td>
<td>1.022</td>
<td>1,895</td>
<td>2,917</td>
<td>12,843</td>
<td>2</td>
<td>450</td>
</tr>
<tr>
<td>Phase 2</td>
<td>1.022</td>
<td>1,891</td>
<td>2,913</td>
<td>12,825</td>
<td>2</td>
<td>450</td>
</tr>
</tbody>
</table>

#### Collector Pipes

<table>
<thead>
<tr>
<th>Area</th>
<th>PLS Flow (m³/hr)</th>
<th>Storm Flow (m³/hr)</th>
<th>Total Flow (m³/hr)</th>
<th>Total Flow (gpm)</th>
<th>No. Pipes</th>
<th>Pipe Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 - North</td>
<td>1.022</td>
<td>851</td>
<td>1,873</td>
<td>8,246</td>
<td>4</td>
<td>300</td>
</tr>
<tr>
<td>Phase 1 - South</td>
<td>1.022</td>
<td>1,044</td>
<td>2,066</td>
<td>9,096</td>
<td>4</td>
<td>300</td>
</tr>
<tr>
<td>Phase 2 - North</td>
<td>1.022</td>
<td>1,124</td>
<td>2,146</td>
<td>9,450</td>
<td>4</td>
<td>300</td>
</tr>
<tr>
<td>Phase 2 - South</td>
<td>1.022</td>
<td>767</td>
<td>1,789</td>
<td>7,875</td>
<td>4</td>
<td>300</td>
</tr>
</tbody>
</table>
Pipe Spacing

<table>
<thead>
<tr>
<th>Units</th>
<th>English</th>
</tr>
</thead>
<tbody>
<tr>
<td>Application Rate:</td>
<td>12.43 ft³/day/ft²</td>
</tr>
<tr>
<td>Maximum Desired Head on the Liner:</td>
<td>5.0 ft</td>
</tr>
<tr>
<td>Hydraulic Conductivity:</td>
<td>28.35 ft/day</td>
</tr>
<tr>
<td>Pipe Spacing:</td>
<td>49.5 ft</td>
</tr>
<tr>
<td>Average Hydraulic Head on Liner:</td>
<td>2.0 ft</td>
</tr>
</tbody>
</table>

Application Rate Converter

<table>
<thead>
<tr>
<th>Application Rate</th>
<th>gpm/ft²</th>
<th>l/hr/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0060</td>
<td>14.6690</td>
<td>0.0147</td>
</tr>
<tr>
<td>0.35</td>
<td>12.43</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Hydraulic Conductivity Converter

<table>
<thead>
<tr>
<th>Hydraulic Conductivity</th>
<th>cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0100</td>
<td></td>
</tr>
</tbody>
</table>

\[
H = \frac{L}{2} \left( \frac{W}{K} \right)^{0.5}
\]

Where:

- \( H \) = Maximum Hydraulic Head on Liner at Midpoint between Pipes
- \( L \) = Drain Pipe Spacing
- \( W \) = Application Rate
- \( K \) = Hydraulic Conductivity

1) Permeability in drain layer of 1 x 10⁻² cm/s for a fully loaded scenario with lift height at 300 feet (Design Criteria)
ATTACHMENT D
TECHNICAL MEMORANDUM
ROSEMONT HEAP LEACH PAD DRAIN PIPE DEFLECTION
Technical Memorandum

To: Joel Carrasco  
From: Mike Thornbrue  
Company: Tetra Tech  
Date: August 13, 2010  
Re: Rosemont Heap Leach Pad Drain Pipe Deflection  
Doc #: 214/10-320877-5.3  
CC: David R. Krizek, P.E. (Tetra Tech); and Troy Meyer, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum documents the estimated deflection of the solution collection pipes related to the Heap Leach Facility (HLF) at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is 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). Specifically, this Technical Memorandum answers item no. 3b on page 6 of 18.

- Item 3b – Leachate Collection Pipes Network: Pipe loading at the ultimate design height of the heap does not threaten the structural integrity of the pipe. The collector and header pipes network can provide sufficient capacity for transporting leachate over the operation life of the facility; withstand the stresses caused by the maximum loading height of the ore heap without significant deformation or buckling and with adequate factor of safety. This demonstration should be based on manufacturer’s technical data on product specifications or case studies for the pipes (ADS-N-12 Corrugated High Density Polyethylene pipe) used under similar application.

The results of our calculations indicate that the collection pipes will provide sufficient capacity without significant deformation caused by the maximum loading of the heap.

2.0 Drain Pipe Design

As documented in the technical memorandum titled, “Rosemont Heap Leach Drain Pipe Design” (Tetra Tech, 2009), the Heap Leach Facility has been designed to meet or exceed the recommendations for Best Available Demonstrated Control Technology (BADCT) as established by the Arizona Department of Environmental Quality (ADEQ) (2004).

The drain pipe network will consist of corrugated, dual wall, perforated N-12 high-density, polyethylene (HDPE) pipes buried in three (3) feet of permeable (about 1x10⁻² cm/sec) Overliner Drain Fill (ODF) material. The drain pipes will be arranged in a herringbone pattern that will convey Pregnant Leach Solution (PLS) and stormwater to the PLS Pond located at the base of
the Heap Leach Pad. The four (4) inch diameter pipes will connect to a series of ten (10) inch diameter primary pipes, 15-inch diameter collector pipes, and 18-inch diameter header pipes. The pipe network has been designed to convey 150% of the anticipated PLS flows plus the stormwater from the 100-year, 24-hour storm event to the PLS Pond under gravity flow conditions.

3.0 Plastic Piping Design Guidance

Prescriptive BADCT design guidance recommends that the drainage gravel and solution collection piping provide for the removal of solution from the base of the leach ore pile such that the average and maximum hydraulic head over the liner are less than two (2) feet and five (5) feet, respectively.

The Association of State Highway and Transportation Officials (AASHTO) recommends a maximum allowable long-term strain (deflection) of five (5) percent for buried, corrugated polyethylene pipes (AASHTO, 2007). However, this standard is intended for roadway design and is impractical as guidance for heap leach pad design.

In a paper published in 2005, John F. Lupo, P.E. noted that non-pressurized HDPE pipes may have severe deformation and buckling if the vertical deflections in the pipe exceed 20%. Therefore, Mr. Lupo recommends allowing a maximum vertical deflection of 15% if the pipes are sized to accommodate a reduction in flow capacity due to deflection (Lupo, 2005).

In 2007, Knight Piésold conducted a large-scale laboratory vertical load test on segment of 24-inch (600-mm) diameter Corrugated Plastic Tubing (CPT) pipe manufactured by Advanced Drainage Systems Incorporated (ADS). The testing was conducted at the U.S. Bureau of Reclamation (USBR) in Denver, CO using a full scale cross-section of a heap leach pad liner system including soil liner, geomembrane, protective soil, a drain pipe, and a drainage layer. The testing was conducted to evaluate the performance of the pipe under the loading conditions expected on a heap leach pad stacked to a height of 170 meters (558 feet). The testing indicated a deformation of 5.1 inches, approximately 21%, at 140 meters (459 feet) and a vertical deformation of 7.02 inches, approximately 29%, at 170 meters (558 feet). The deformations were considered acceptable because the design of the leach pad piping system allowed for a 50% reduction in flow capacity due to pipe deformations.

4.0 Pipe Deformation Calculations

Tetra Tech performed deflection calculations for each pipe size that will be used on the proposed Heap Leach Pad. The calculations provided in Attachment 1 are based on the work of Burns and Richard (1964).

The calculations are based on the following information for each pipe size (ADS, 2009):

- Outside Diameter of the Pipe;
- Wall Thickness at the Valley of the Corrugation;
- Unit Area of the Wall;
- Unit Moment of Inertia; and
- The Distance from the Inner Wall to the Neutral Axis of the Pipe.
The calculations are also based on the following information (AASHTO, 2007):

- The Flexural Modulus of 110,000 pounds per square inch (psi).

The following values were estimated based on engineering judgment and experience:

- The Modulus of Soil Reaction at 5’ of cover for the ODF was estimated at 1,000 psi. The Modulus of Soil Reaction can range from 470 psi to over 1,000 psi based on the geostatic load of the material above the ODF;
- Poisson’s Ration was estimated to be 0.25. Typical values for this application range from 0.25 to 0.30. A value of 0.25 provides the most conservative calculation (Coduto, 2001);
- The Unit Weight of the soil (leach pad) was estimated to be 125 pounds per cubic foot (pcf); and
- The maximum possible height of the stacked ore of 450 feet was selected for the height of fill above the crown of the pipe.

5.0 Conclusions

In general, if the leach ore is stacked to a height of 450 feet, each pipe is likely to experience an 18% vertical deformation as shown in Attachment 1. The pipe deflection results in a loss of 10% of the cross sectional area of the pipes. As previously stated, the drain pipe system has been designed to accommodate 150% of the PLS design flows plus the flows from a 100-year, 24-hour storm event. Therefore, the deflection is acceptable.
REFERENCES


### Pipe Crushing Calculations

#### 4" ADS N-12

**Heap Leach Facility Design**

**Pipe Crushing Calculations**

**Rosemont Copper Company**

---

#### PIPE PARAMETERS - AASHTO M294, Type S

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Radius (in), R</td>
<td>2.50</td>
</tr>
<tr>
<td>Outside Diameter (in), D</td>
<td>4.8</td>
</tr>
<tr>
<td>Thickness (in), t</td>
<td>0.03</td>
</tr>
<tr>
<td>Unit Area of Wall (in^2/in), A</td>
<td>0.063</td>
</tr>
</tbody>
</table>

#### Calcuations of Ring Shortening

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Diameter (in), D</td>
<td>4.8</td>
<td></td>
</tr>
<tr>
<td>Unit Moment of Inertia (in^4/in), I</td>
<td>0.0014</td>
<td></td>
</tr>
<tr>
<td>Flexural Modulus (psi), E_f</td>
<td>110,000</td>
<td></td>
</tr>
<tr>
<td>Ring Compression Modulus (psi), E_c</td>
<td>110,000</td>
<td></td>
</tr>
<tr>
<td>Flexural Stiffness (psi), K_f</td>
<td>6E_fI/R^3</td>
<td></td>
</tr>
<tr>
<td>Ring Compression Stiffness (psi), K_c</td>
<td>E_cA/R</td>
<td></td>
</tr>
<tr>
<td>Distance From Inner Wall to N.A. (in), c</td>
<td>0.12, 0.209, 0.302</td>
<td></td>
</tr>
</tbody>
</table>

#### Soil Parameters - Good Granular Soil

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Soil Reaction at 5' of Cover (psi), E'_5</td>
<td>1000</td>
</tr>
<tr>
<td>Modulus of Soil Reaction (psi), E'_0</td>
<td>4161</td>
</tr>
<tr>
<td>Poisson's Ratio, ( \mu )</td>
<td>0.25</td>
</tr>
<tr>
<td>Constr Mod (psi), M' = E'(1-\mu)/(1+\mu)(1-2\mu)</td>
<td>4992.8</td>
</tr>
<tr>
<td>Lateral Stress Ratio = K</td>
<td>0.333</td>
</tr>
<tr>
<td>Sym Lateral Stress Ratio = B</td>
<td>0.667</td>
</tr>
<tr>
<td>Antisym Lat Stress Ratio = C</td>
<td>0.333</td>
</tr>
</tbody>
</table>

#### Soil/Structure Parameters (full slippage)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ring Flexibility Ratio, UF = (1+K)M'_c/K_c</td>
<td>2.40</td>
</tr>
<tr>
<td>Bending Flexibility Ratio, VF = (1-K)M'_c/K_f</td>
<td>55.9</td>
</tr>
</tbody>
</table>

#### Load Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Soil (lb/ft^3)</td>
<td>125</td>
</tr>
<tr>
<td>Height of Fill Above Crown (ft)</td>
<td>450.0</td>
</tr>
<tr>
<td>Surcharge Pressure (psi), P</td>
<td>390.6</td>
</tr>
</tbody>
</table>

---

### COMMENTS

1. This is 4" diameter ADS N-12
2. Flexural and compressive modulus are taken as 110,000 psi.
3. Typical \( E'_0 \) values (in psi) for various soils are listed in the table below:

#### STRESS FUNCTION COEFFICIENTS

| \( \cos(2\theta) \), \( a_{2**} \) | 0.974 |
| \( \sin(2\theta) \), \( b_{2**} \) | 0.961 |

#### LOAD PARAMETERS

<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Standard AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine-grained soils with less than 25% sand (CL, ML, DL-ML)</td>
<td>500</td>
</tr>
<tr>
<td>Coarse-grained soils with fines (SM, SC)</td>
<td>600</td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines (SP, SW, GP, GW)</td>
<td>700</td>
</tr>
</tbody>
</table>

---

### Tetra Tech

February 2010

---

* N.A. = Neutral Axis
### Pipe Crushing Calculations

10" ADS N-12

<table>
<thead>
<tr>
<th>PIPE PARAMETERS - AASHTO M294, Type S</th>
<th>RESPONSE OF PIPE WALL</th>
<th>CALCULATION OF RING SHORTENING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Radius (in), R = 5.94</td>
<td>deg radial radial radial</td>
<td>c.c.w.</td>
</tr>
<tr>
<td>Outside Diameter (in), D = 11.36</td>
<td>from press defl defl wall</td>
<td>radial tang circum wall bend</td>
</tr>
<tr>
<td>Thickness (in), t = 0.04</td>
<td>10.64 (-0.292) 0.000 1012 61</td>
<td>-7387 -2294 -1964 -9681 -9351</td>
</tr>
<tr>
<td>Unit Area of Wall (in²/in), A = 0.137</td>
<td>164.9 (-0.251) 0.118 1011 58</td>
<td>-7383 -2178 -1865 -9561 -9248</td>
</tr>
<tr>
<td>Unit Moment of Inertia (in⁴/in), I = 0.008</td>
<td>165.7 (-0.131) 0.222 1010 49</td>
<td>-7372 -1844 -1579 -9216 -8950</td>
</tr>
<tr>
<td>Flexural Modulus (psi), Eₚ = 110,000</td>
<td>166.8 0.051 0.299 1008 36</td>
<td>-7355 -1331 -1140 -8687 -8495</td>
</tr>
<tr>
<td>Ring Compression Modulus (psi), Eᵢ = 110,000</td>
<td>168.3 0.276 0.340 1005 19</td>
<td>-7334 -703 -602 -8038 -7936</td>
</tr>
<tr>
<td>Flexural Stiffness (psi), Kₚ = 110,000</td>
<td>171.2 0.739 0.299 999 -16</td>
<td>-7292 594 508 -6698 -6783</td>
</tr>
<tr>
<td>Ring Compression Stiffness (psi), Kᵢ = 2,539</td>
<td>173.4 1.083 0.000 995 -42</td>
<td>-7260 1556 1332 -5704 -5928</td>
</tr>
<tr>
<td>Distance From Inner Wall to N.A. (in), c = 0.30</td>
<td>173.4 1.083 0.000 995 -42</td>
<td>-7260 1556 1332 -5704 -5928</td>
</tr>
<tr>
<td>10.64 (-0.292) 0.000 1012 61</td>
<td>-7387 -2294 -1964 -9681 -9351</td>
<td>0</td>
</tr>
</tbody>
</table>

**SOIL PARAMETERS - Good Granular Soil**

| Modulus of Soil Reaction at 5' of Cover (psi), E₆₅ = 1000 | 172.4 0.922 0.222 997 -30 | -7275 1106 947 -6169 -6328 | 80 |-7275 -0.066135 -0.0685 |
| Modulus of Soil Reaction (psi), E₆ = 4,161 | 173.4 1.083 0.000 995 -42 | -7260 1556 1332 -5704 -5928 | 90 |-7260 -0.066001 -0.0684 |
| Poisson's Ratio, u = 0.25 | 173.4 1.083 0.000 995 -42 | -7260 1556 1332 -5704 -5928 | 100 |-7260 -0.066001 -0.0684 |
| Constr Mod (psi), Mₚ = Eₚ(1-ul)/(1+(1+u)(1-2ul)) = 4992.8 | 172.4 0.922 0.222 997 -30 | -7275 1106 947 -6169 -6328 | 110 |-7275 -0.066135 -0.0685 |
| Lateral Stress Ratio = K = u/(1-ul) = 0.667 | 171.2 0.739 0.299 999 -16 | -7292 594 508 -6698 -6783 | 120 |-7292 -0.066289 -0.0687 |
| Sym Lateral Stress Ratio = B = (1/2)(1+K) =0.667 | 168.3 0.515 0.340 1002 1 | -7312 -35 -30 -7347 -7342 | 130 |-7312 -0.066477 -0.0689 |
| Antisym Lat Stress Ratio = C = (1/2)(1-K) =0.333 | 168.3 0.515 0.340 1002 1 | -7312 -35 -30 -7347 -7342 | 140 |-7312 -0.066677 -0.0691 |
| 171.2 0.739 0.299 999 -16 | -7292 594 508 -6698 -6783 | 120 |-7292 -0.066289 -0.0687 |

**SOIL/STRUCTURE PARAMETERS (full slippage)**

| Ring Flexibility Ratio, UF = (1+K)Mₚ/Kᵢ = 2.62 | 165.7 -0.131 -0.222 1010 49 | -7372 -1844 -1579 -9216 -8950 | 160 |-7372 -0.067018 -0.0694 |
| Bending Flexibility Ratio, VF = (1-K)Mₚ/Kf = 131.9 | 164.9 -0.251 -0.118 1011 58 | -7383 -2178 -1865 -9561 -9248 | 170 |-7383 -0.0671 -0.0695 |

**STRESS FUNCTION COEFFICIENTS**

1. This is 10" diameter ADS N-12
2. Flexural and compressive modulus are taken as 110,000 psi.
3. Typical E'₅ values (in psi) for various soils are listed in the table below:

<table>
<thead>
<tr>
<th>LOAD PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Soil (lb/ft³) = 125</td>
</tr>
<tr>
<td>Height of Fill Above Crown (ft) = 450.0</td>
</tr>
<tr>
<td>Surcharge Pressure (psi), P = 390.6</td>
</tr>
</tbody>
</table>

**COMMENTS**

SUM (I/2 circle) = -1.3111

**Vertical deflection (%) = 18.24**

Critical Buckling Pressure (psi), Pcr = 122.2

Radial Soil Pressure at Crown (psi), P = 390.6

Arc length of each sector (in) = 1.0360

**CIRCUMFERENCE SHORTENING**

-2.62
### PIPE PARAMETERS - AASHTO M294, Type S

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Radius (in)</td>
<td>R = 9.18</td>
</tr>
<tr>
<td>Outside Diameter (in)</td>
<td>D = 17.57</td>
</tr>
<tr>
<td>Thickness (in), t = 0.150</td>
<td></td>
</tr>
<tr>
<td>Unit Area of Wall (in^2), A</td>
<td>0.260</td>
</tr>
</tbody>
</table>

### RESPONSE OF PIPE WALL

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Diameter (in)</td>
<td>17.52</td>
</tr>
<tr>
<td>Unit Moment of Inertia (in^2/in)</td>
<td>0.054</td>
</tr>
</tbody>
</table>

### CALCULATION OF RING SHORTENING

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sum (I/2 circle)</td>
<td>-1.8458</td>
</tr>
</tbody>
</table>

### STRESS FUNCTION COEFFICIENTS

1. This is 15" diameter ADS N-12
2. Flexural and compressive modulus are taken as 110,000 psi.
3. Typical E'5 values (in psi) for various soils are listed in the table below:

### LOAD PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Soil (lb/ft^3)</td>
<td>125</td>
</tr>
<tr>
<td>Height of Fill Above Crown (ft)</td>
<td>450.0</td>
</tr>
<tr>
<td>Surcharge Pressure (psi), P</td>
<td>390.6</td>
</tr>
</tbody>
</table>

### COMMENTS

1. This is 15" diameter ADS N-12
2. Flexural and compressive modulus are taken as 110,000 psi.
3. Typical E'5 values (in psi) for various soils are listed in the table below:

### Type of soil

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Standard AASHTO Relative Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine-grained soils with less than 25% sand (CL, ML, DL-ML)</td>
<td>85% 90% 95%</td>
</tr>
<tr>
<td>Coarse-grained soils with fines (SM, SC)</td>
<td>165.2</td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines (SP, SW, GP, GW)</td>
<td>196.8</td>
</tr>
</tbody>
</table>

### COMPARISON

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical deflection (%)</td>
<td>17.52</td>
</tr>
<tr>
<td>Horizontal deflection (%)</td>
<td>-10.79</td>
</tr>
<tr>
<td>Critical Buckling Pressure (psi), Pcr</td>
<td>165.2</td>
</tr>
<tr>
<td>Radial Soil Pressure at Crown (psi), Prc</td>
<td>196.8</td>
</tr>
<tr>
<td>Arc length of each sector (in)</td>
<td>1.6017</td>
</tr>
</tbody>
</table>

### CIRCUMFERENCE SHORTENS

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine-grained soils with less than 25% sand (CL, ML, DL-ML)</td>
<td>500 700 1000</td>
</tr>
<tr>
<td>Coarse-grained soils with fines (SM, SC)</td>
<td>600 1000 1200</td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines (SP, SW, GP, GW)</td>
<td>700 1000 1600</td>
</tr>
<tr>
<td>Parameter</td>
<td>Value</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>Effective Radius (in)</td>
<td>11.014</td>
</tr>
<tr>
<td>Outside Diameter (in)</td>
<td>21.2</td>
</tr>
<tr>
<td>Thickness (in), I = 0.161</td>
<td></td>
</tr>
<tr>
<td>Unit Area of Wall (in²/in)</td>
<td>0.28</td>
</tr>
</tbody>
</table>

**Pipe Parameters - AASHTO M294, Type S**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Diameter (in), D = 110,000</td>
<td></td>
</tr>
<tr>
<td>Flexural Modulus (psi), E_f</td>
<td></td>
</tr>
<tr>
<td>Ring Compression Modulus (psi), E_c</td>
<td></td>
</tr>
<tr>
<td>Flexural Stiffness (psi), K_f</td>
<td></td>
</tr>
<tr>
<td>Ring Compression Stiffness (psi), K_c</td>
<td></td>
</tr>
<tr>
<td>Distance From Inner Wall to N.A. (in), c</td>
<td>0.58</td>
</tr>
</tbody>
</table>

**Soil Parameters - Good Granular Soil**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Soil Reaction at 5' of Cover (psi), E'</td>
<td>1000</td>
</tr>
<tr>
<td>Modulus of Soil Reaction (psi), E'</td>
<td>4,161</td>
</tr>
<tr>
<td>Poisson's Ratio, u</td>
<td>0.25</td>
</tr>
<tr>
<td>Constr Mod (psi), M* = E*(1+u)/(1-u) = 4993</td>
<td></td>
</tr>
<tr>
<td>Lateral Stress Ratio = K = E*(1-u)/E = 0.333</td>
<td></td>
</tr>
<tr>
<td>Sym Lateral Stress Ratio = B = (1/2)((1+K) = 0.667</td>
<td></td>
</tr>
<tr>
<td>Antsym Lat Stress Ratio = C = (1/2)((1-K) = 0.333</td>
<td></td>
</tr>
</tbody>
</table>

**Soil/Structure Parameters (full slippage)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ring Flexibility Ratio, UF = (1+K)M*/K_c = 2.42</td>
<td></td>
</tr>
<tr>
<td>Bending Flexibility Ratio, VF = (1-K)M*/K = 88.7</td>
<td></td>
</tr>
</tbody>
</table>

**Comments**

1. This is 18" diameter ADS N-12
2. Flexural and compressive modulus are taken as 110,000 psi.
3. Typical E' values (in psi) for various soils are listed in the table below:

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Standard AASHTO</th>
<th>Relative Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine-grained soils with less than 25% sand (CL, ML, DL-ML)</td>
<td>500 700 1000</td>
<td>85% 90% 95%</td>
</tr>
<tr>
<td>Coarse-grained soils with fines (SM, SC)</td>
<td>600 1000 1200</td>
<td>85% 90% 95%</td>
</tr>
<tr>
<td>Coarse-grained soils with little or no fines (SP, SW, GP, GW)</td>
<td>700 1000 1600</td>
<td>85% 90% 95%</td>
</tr>
</tbody>
</table>

**Load Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Soil (lb/ft³)</td>
<td>125</td>
</tr>
<tr>
<td>Height of Fill Above Crown (ft)</td>
<td>450.0</td>
</tr>
<tr>
<td>Surcharge Pressure (psi), P</td>
<td>390.6</td>
</tr>
</tbody>
</table>

**Stress Function Coefficients**

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant Term, a₀ = 0.321</td>
<td></td>
</tr>
<tr>
<td>cos(2*theta), a₂ = 0.983</td>
<td></td>
</tr>
<tr>
<td>sin(2*theta), b₂ = 0.975</td>
<td></td>
</tr>
</tbody>
</table>

**Vertical deflection (%) = 17.93**

Horizontal deflection (%) = -10.18

Critical Buckling Pressure (psi), P_cr = 149.0

Radial Soil Pressure at Crown (psi), P_a = 183.4

Arc length of each sector (in) = 1.9223
Technical Memorandum

To: Kathy Arnold  
From: Joel Carrasco  
Company: Rosemont Copper Company  
Date: August 20, 2010  
Re: Rosemont Overliner Drainage and Durability  
Doc #: 223/10-320877-5.3  
CC: David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum documents the durability of the overliner drain fill (ODF) planned for use as part of the Heap Leach Pad liner system at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is 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). Specifically, this Technical Memorandum answers item no. 4b and 4c on pages 6 and 7 of 18. Item no. 4a was answered in a separate Technical Memorandum in response to item no. 3 on page 6 of 18.

- **Overliner Material** – Rosemont has proposed the use of 1 ½ -inch minus crushed drainage layer versus ¾ -inch minus material as identified in the BADCT Guidance Manual.

  Placement of ¾ -inch, well draining material with a minimum thickness of 18 inches is a design requirement to meet prescriptive BADCT for a heap leach pad overliner protective/drainage layer. ADEQ will consider the proposed use of 36-inch layer of 1 ½ -inch minus crushed material provided the following design criteria are satisfied:

  a) As mentioned above, the maximum and average hydraulic head over the leach pad liner must not exceed 5 feet and 2 feet, respectively. (this was answered in a separate Technical Memorandum)

  b) Particle size compatibility demonstration that material used will not clog the protective drainage layer and impair overliner drainage capacity.

  c) Overliner Material Durability test to demonstrate that the material used in the protective/drainage layer will not deteriorate when in contact with the leachate solution during the service life of the facility.

The drainage and durability analysis results indicate that the ODF drainage capacity and durability will not be adversely affected during the service life of the Heap Leach Facility.
2.0 Overliner Material Compatibility

The heap leach pad ODF provides liner protection from exposure to the climate, vehicle tracks, and ore placement via haul trucks. The ODF also reduces the hydraulic head on the pad liner when constructed in combination with supplemental drain pipes placed at a spacing determined by the leaching solution application rate and the permeability characteristics of the drain rock.

The ODF material will be screened and or crushed, as needed, to produce a gradation with 100 percent of the material passing the 1.5 inch screen and less than five (5) percent passing the No. 200 screen. The permeability of the ODF will be approximately $1 \times 10^{-2}$ cm/sec. A low content of fines (minus 200 screen) will maintain the required permeability and not clog.

In a paper published in 2005, John F. Lupo, P.E., a range of gradations was recommended for overliner drain fill in order to maintain proper drainage and compatibility with the drainage pipe network. Illustration 1 presents the typical gradation envelope for typical overliner material based on this analysis. As indicated by Illustration 1, the envelope ranges from 100 percent passing the 1.5 inch screen and a maximum of 10 percent passing the minus 200 mesh. Therefore, the selected gradation of the ODF material at Rosemont is well within the gradation envelope to prevent plugging of the overliner drain fill and associated perforated piping network.

![Illustration 1 Typical Gradation Envelope](image-url)
3.0 Overliner Material Durability

Quartz Monzonite Porphyry (QMP) is one of the rock materials available early in the mine production schedule. A sample of QMP was collected from within the proposed Open Pit limits and tested for suitability as ODF for the Heap Leach Pad. The material was subjected to slake durability (ASTM D 4644) testing with sulfuric acid soaking to simulate acid leaching and to verify chemical resistance to the leachate solution. The material was also subjected to testing for resistance to degradation by abrasion and impact (ASTM C 535) to measure the degradation as percent loss.

The slake durability test results indicated no loss of durability upon soaking in 0.5 percent sulfuric acid solution and minimal loss upon soaking in 5.0 percent solution strength. The test results indicated that the material will exhibit negligible degradation under the expected operational conditions and that it is suitable for use as overliner drain fill for the Heap Leach Pad.

The resistance to degradation test results show a uniform hardness ratio of 0.25 which indicates that the material is fairly uniform in hardness. The material is therefore expected to show little degradation during the handling of the material as well as under high loads. Testing results of the QMP overliner drain fill material are provided in Attachment 1.
REFERENCES


**Project Name:** Advanced Terra Testing  
**Quarry Source:** Rosemont  
**Project Number:** 292004

Grading Used: Grading "3"

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Required Grading (gm)</th>
<th>Actual Grading Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passing</td>
<td>Retained</td>
<td>QMP30 - Drain Layer</td>
</tr>
<tr>
<td>2.5 in. (63.0 mm)</td>
<td>2.0 in. (50.0 mm)</td>
<td>---</td>
</tr>
<tr>
<td>2.0 in. (50.0 mm)</td>
<td>1.5 in. (37.5 mm)</td>
<td>---</td>
</tr>
<tr>
<td>1.5 in. (37.5 mm)</td>
<td>1.0 in. (25.0 mm)</td>
<td>5000 ± 50</td>
</tr>
<tr>
<td>1.0 in. (25.0 mm)</td>
<td>3/4 in. (19.0 mm)</td>
<td>5000 ± 50</td>
</tr>
<tr>
<td>Total Aggregate Accumulated</td>
<td></td>
<td>10000 ± 100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9959.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Weight (gm)</th>
<th>9959.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unwashed Weight at 200 Rev. (gm)</td>
<td>9475.9</td>
</tr>
<tr>
<td>Washed Weight at 1000 Rev. (gm)</td>
<td>8002.2</td>
</tr>
</tbody>
</table>

| Percent Loss at 200 Revolutions | 4.9 |
| Percent Loss at 1000 Revolutions | 19.6 |
| Uniform Hardness Ratio | 0.250 |

*No. 12 (1.70 mm) Sieve was used to determine "Percent Loss"*

MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D 6913

CLIENT       Tetra Tech M&M
BORING NO.    QMP 30 Jan. 2009
DEPTH
SAMPLE NO.    Drainage Layer Material
SOIL DESCR.   Rosemont
LOCATION      Rosemont
JOB NO.       2688-20
SAMPLED      Yes
DATE +#4 WASHED 02/21/09 WAR
DATE -#4 WASHED 02/21/09 WAR
WASH SIEVE    No
DRY SIEVE     No

MOISTURE DATA

HYGROSCOPIC    Yes
NATURAL        No

| Wt. Wet Soil & Pan (g) | 631.02 |
| Wt. Dry Soil & Pan (g) | 623.90 |
| Wt. Lost Moisture (g)  | 7.12  |
| Wt. of Pan Only (g)    | 14.03 |
| Wt. of Dry Soil (g)    | 609.87 |
| Moisture Content %     | 1.2   |
|
Wt. Partial -#4 Sample Wet (g) 534.68
Wt. Partial Sample Dry (g)    528.51

Wt. Total Sample Wet (g)  36104.40
Weight of + #4 Before Washing (g)  33908.30
Weight of + #4 After Washing (g)  33908.30
Weight of - #4 Wet (g) 2196.10
Weight of - #4 Dry (g) 2170.76
Wt. Total Sample Dry (g)  36079.06
Calc. Wt. "W" (g) 8784.09
Calc. Mass + #4 8255.58

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td>1 1/2&quot;</td>
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<td>0.00</td>
<td>0.00</td>
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<td>0.0</td>
<td>100.0</td>
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<td>3/4&quot;</td>
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<td>25316.00</td>
<td>25316.00</td>
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<td>29.8</td>
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<tr>
<td>3/8&quot;</td>
<td>0.00</td>
<td>6623.00</td>
<td>6623.00</td>
<td>31939.00</td>
<td>88.5</td>
<td>11.5</td>
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<td>#4</td>
<td>0.00</td>
<td>1969.30</td>
<td>1969.30</td>
<td>33908.30</td>
<td>94.0</td>
<td>6.0</td>
</tr>
<tr>
<td>#10</td>
<td>7.00</td>
<td>258.53</td>
<td>251.53</td>
<td>251.53</td>
<td>96.8</td>
<td>3.2</td>
</tr>
<tr>
<td>#20</td>
<td>3.10</td>
<td>108.12</td>
<td>105.02</td>
<td>356.55</td>
<td>98.0</td>
<td>2.0</td>
</tr>
<tr>
<td>#40</td>
<td>3.23</td>
<td>55.63</td>
<td>52.40</td>
<td>408.95</td>
<td>98.6</td>
<td>1.4</td>
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<td>#60</td>
<td>3.27</td>
<td>32.16</td>
<td>28.89</td>
<td>437.84</td>
<td>99.0</td>
<td>1.0</td>
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<tr>
<td>#100</td>
<td>3.79</td>
<td>24.01</td>
<td>20.22</td>
<td>458.06</td>
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<td>0.8</td>
</tr>
<tr>
<td>#140</td>
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<td>13.70</td>
<td>10.62</td>
<td>468.68</td>
<td>99.3</td>
<td>0.7</td>
</tr>
<tr>
<td>#200</td>
<td>3.10</td>
<td>12.41</td>
<td>9.31</td>
<td>477.99</td>
<td>99.4</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Data entered by: MLM
Data checked by: MLM
Date: 02/23/2009
FileName: TMM0QMP3
### SLAKE DURABILITY
#### ASTM D 4644

**CLIENT:** Tetra Tech M&M  
**LOCATION:** Rosemont Liner and Aggregate Project Site  
**JOB NO.:** 2688-20  
**DATE TESTED:** 2/19-26/09 BKL

<table>
<thead>
<tr>
<th>Submerged 5 days at</th>
<th>0.5% H2SO4 solution</th>
<th>5.0% H2SO4 solution</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sample No.</strong></td>
<td>* QMP 30Jan2009</td>
<td>* QMP 30Jan2009</td>
</tr>
<tr>
<td><strong>Depth</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Rock Type</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Wt. wet Sample &amp; Drum (g)</td>
<td>1669.8</td>
<td>1718.1</td>
</tr>
<tr>
<td>Initial Wt. dry Sample &amp; Drum (g)</td>
<td>1665.0</td>
<td>1714.5</td>
</tr>
<tr>
<td>1st Cycle Wt. Sample &amp; Drum (g)</td>
<td>1661.7</td>
<td>1711.2</td>
</tr>
<tr>
<td>2nd Cycle Wt. Sample &amp; Drum (g)</td>
<td>1660.1</td>
<td>1709.8</td>
</tr>
<tr>
<td>Wt. of Drum (g)</td>
<td>1126.6</td>
<td>1185.9</td>
</tr>
<tr>
<td>Initial Wt Dry Sample (g)</td>
<td>539.0</td>
<td>528.5</td>
</tr>
<tr>
<td>1st Cycle Wt. Sample (g)</td>
<td>535.1</td>
<td>525.3</td>
</tr>
<tr>
<td>2nd Cycle Wt. Sample (g)</td>
<td>533.5</td>
<td>523.9</td>
</tr>
<tr>
<td>1st Cycle Durability (%)</td>
<td>99.28</td>
<td>99.38</td>
</tr>
<tr>
<td>2nd Cycle Durability (%)</td>
<td>98.98</td>
<td>99.13</td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>0.79</td>
<td>0.69</td>
</tr>
<tr>
<td>Type (I, II, III)</td>
<td>I</td>
<td>I</td>
</tr>
</tbody>
</table>

**Comments:** Based upon client request for simulation of copper leaching and effect on the drainage, samples were submerged for 5 days at 0.5% and 5.0% H2SO4 prior to Slake test to be performed.

**Data Entered By:** BKL  
**Date:** 03/01/2009  
**Data Checked By:** HN  
**Date:** 3/1/09  
**Filename:** TMSDQMP3
Tetra Tech M&AM
2688-20
Rosemont Liner and Aggregate Test Program
Sample: QMP 30 Jan 2009

After 5 days soaking
0.5% H₂SO₄
Tetra Tech MDN
2688-20
Rosemont Liner and Aggregate Test Program
Sample: QMP 30 Jan 2009
Begin Soaking 2/14/09 10:00am 0.5% H₂SO₄
Tetra Tech M4-M
2688-20
Rosemont Liner and Aggregate Test Program
Sample: QMP 30 Jan 2009
Begin Soaking 5% H2SO4
2/19/09 10:00am
Tetra Tech MWW
2688-20
Rosemont Liner and Aggregate Test Program
Sample: QMP 30 Jan 2009
before test 0.5% H2SO4
ATTACHMENT F
TECHNICAL MEMORANDUM
LINER PUNCTURE TESTING FOR THE PROPOSED ROSEMONT HEAP LEACH PAD
Technical Memorandum

To: Kathy Arnold
From: Mike Thornbrue

Company: Rosemont Copper Company
Date: August 26, 2010

Re: Liner Puncture Testing for the Proposed Rosemont Heap Leach Pad
Doc #: 232/10-320877-5.3

CC: Joel Carrasco (Tetra Tech); and David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum summarizes the protocol and test results for additional liner puncture tests performed in support of the Rosemont Heap Leach Pad liner system at the Rosemont Copper Project (Project) in Pima County, Arizona. This information is 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). Specifically, this Technical Memorandum answers item no. 5 on page 7 of 18.

- **Geomembrane Protection and Liner Puncture Test** – Rosemont has proposed a minimum of 36 inches of overliner drain fill over the Heap Leach Pad as specified in the design criteria (Tetra Tech 2009) for the geomembrane protection. The material will be screened and or crushed, as needed, to produce a gradation with 100 percent of the material passing the 1.5 inch screen and less than five percent passing the No. 200 screen. (Ref. Tetra Tech Technical Memorandum – Rosemont Heap Leach Geomembrane Protection, May 4, 2009).

As stated above, placement of ¾ -inch minus, well draining material with a minimum thickness of 18 inches is a design requirement to meet prescriptive BADCT.

ADEQ will consider the use of the 1 ½ -inch crushed overliner material if the proposed GCL liner of 6-mm thickness is demonstrated to show no severe indentations when puncture tested under simulated loading conditions by placing the subgrade material, geosynthetic(s), and the overliner material in the test cell. Rosemont has done puncture testing (3 tests) of 60-mil LLDPE with 1.5 inches minus overliner (QMP) drainage layer. However, the test results do not indicate the severity of indentation whether “minor”, “moderate”, or “severe”. Dimpling of the geomembrane sample has occurred. There is no indication how these indentations or dimpling affects durability of the geomembrane. ADEQ considers three trials of puncture tests inadequate to verify the liner system behavior under simulated field conditions. Please conduct additional tests. If severe dimpling is noticed in higher frequency which causes noticeable decrease in achievable strain, ADEQ
recommends that a cushion or bedding layer should be included between the overliner and the geomembrane as an added protective layer.

The current design of the Project Heap Leach Pad liner system calls for 1.5-inch minus Overliner Drain Fill (ODF) material over a 60-mil, double-sided textured, linear low-density, polyethylene (LLDPE) geomembrane. The additional liner puncture testing performed evaluated 1.5-inch minus ODF material and its potential effect on the proposed 60-mil LLDPE liner. The Heap Leach Pad is planned for a maximum depth of material over the pad liner of about 330 feet.

2.0 Recommended Design Section

The following is the current proposed liner and drainage system design section (bottom to top) to be employed at Rosemont:

- Six (6) inches (minimum) thickness of bedding soil (Liner Bedding Fill) derived from local grading or borrow sources with a maximum particle size of 1.5 inch minus material compacted to 95% of the maximum dry density and within two (2) percent of the optimum moisture content as determined by ASTM D-698. The surface of the Liner Bedding Fill will be prepared to produce a relatively smooth surface suitable for installation of geosynthetics. The sample (TTTP-09-01, BU-01) (Gila Conglomerate) was used for this bedding soil in the liner puncture tests;

- Reinforced geosynthetic clay liner (GCL) consisting of a layer of sodium bentonite between two (2) non-woven geotextiles, which are needle-punched together (Cetco Bentomat DN, or equivalent);

- 60-mil, double-sided textured, LLDPE geomembrane liner; and

- A drainage layer (ODF) placed over the 60-mil LLDPE liner, crushed to minus 1.5-inch and screened over 0.5 inch, with no more than five (5) percent fines (passing 200 mesh). The QMP 30Jan2009 sample (QMP) was used for this drainage layer in the liner puncture tests.

3.0 Test Program

3.1 Initial Liner Puncture Testing (2009)

In support of the Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009), liner puncture testing was previously completed for the Heap Leach Pad. The testing was conducted to 390 psi which is equivalent to a geostatic load of 450 feet of stacked heap leach material. The previous testing also used 1.5-inch minus QMP ODF, a 60-mil, double-sided textured, LLDPE liner, a GCL, and site specific subgrade materials.

The initial testing included three (3) puncture tests that resulted in minor to moderate indentations with no punctures. These 2009 test results are provided in Attachment 1.
### 3.2 Additional Liner Puncture Testing (2010)

The additional high-stress compression testing for liner puncture evaluated the following combinations:

- A Heap Leach Pad liner section using 60-mil, double-sided textured, LLDPE liner coupled with 1.5-inch minus ODF; and
- A Heap Leach Pad liner section using 60-mil, double-sided textured, LLDPE liner coupled with 1.5-inch ODF with a 1/2-inch angular rock under the GCL and above the Liner Bedding Fill (planned defect).

Four (4) additional tests were completed using the standard (no defect) puncture testing scenario. Additionally, two (2) tests were completed with the planned defect. Table 1 provides a matrix of the liner puncture testing scenarios.

<table>
<thead>
<tr>
<th>ODF Max Grain Size</th>
<th>60-mil LLDPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Defect</td>
<td>1.5-inch</td>
</tr>
<tr>
<td>Planned Defect*</td>
<td>1.5-inch</td>
</tr>
<tr>
<td>Total Tests</td>
<td></td>
</tr>
</tbody>
</table>

* Planned Defect is an angular 1/2-inch rock placed under the GCL and above the Liner Bedding Fill.

The testing apparatus was prepared to resemble field loading conditions. The bedding layer (Liner Bedding Fill) was compacted to a minimum of 95% of the maximum dry density at the optimum moisture content as determined by ASTM D-698. The surface of the Liner Bedding Fill was prepared to produce a relatively smooth surface suitable for installation of the geosynthetics. The GCL was placed directly on the bedding layer.

When evaluating the planned defect, a 1/2-inch QMP rock was placed directly on the Liner Bedding Fill in the approximate center of the apparatus, followed by the GCL and the LLDPE liner. Prior to placing the ODF, the location of the planned defect was indicated both on the LLDPE and GCL using a marker, such as a white-out pen. The QMP ODF was preferentially selected for its angularity and was placed on the liner using standard methods.

The maximum planned depth of material on the Heap Leach Pad will be about 330 feet. The test load used was 508 psi, which is equivalent to about 585 feet of material. Assuming that the material has a density of 125 pounds per cubic foot (pcf), the depth of the material can be converted to a pressure as follows:

\[
585 \text{ feet} \times \frac{125 \text{ lbs}}{\text{ft}^3} \times \frac{1 \text{ ft}^2}{144 \text{ in}^2} = 508 \text{ psi}
\]

Based on these criteria, the following loading and testing procedures were followed:
An estimated test load of 508 psi was applied to the liner system to simulate the equivalent static load of 585 feet of material on the Heap Leach Pad. Tracking numbers were provided on the liners (both GCL and LLDPE) prior to loading;

- The static load was held for 48 hours;
- Following unloading of the apparatus, the liners (both GCL and LLDPE) were photographed and examined for indentations. The indentations were classified as minor, moderate, major, or severe. Major or severe indentations were noted (numbered) and the liners re-photographed. The location of the planned defect, as appropriate, was also noted;
- The LLDPE liner was vacuum tested to five (5) psi and results noted. If leaks were detected, they were correlated to the numbered indentations;
- The major or severe indentations (indentation depths) on the LLDPE liner were counted and measured immediately after unloading, including the planned defect location; and
- The indentation on the GCL was noted and measured in the location of the planned defect.

The gradation of the Liner Bedding Fill and Overliner Drain Fill were provided along with photographs of the material. A gradation was run for each test to account for possible breakdown of the materials if reused.

The results of the additional testing indicated minor to moderate indentations on the liner with no punctures. The results are provided in Attachment 2.

4.0 Conclusion

To date, seven (7) standard liner puncture tests have been performed using the proposed liner cross section consisting of the following from bottom to top:

- A prepared subgrade;
- A GCL;
- A 60-mil, double sided textured, LLDPE geomembrane, and
- 1.5-inch minus QMP ODF.

The results of all of the tests have indicated minor to moderate indentations with no puncture failures. Therefore, the 1.5-inch minus ODF has been demonstrated to show no severe indentations when tested under simulated loading conditions.

Additionally, there have been two (2) liner puncture tests conducted using a planned defect between the subgrade and the GCL. The puncture testing with the planned defect caused significant damage to the GCL. However, the LLDPE experienced moderate to major indentation with no puncture failure.
REFERENCES

ATTACHMENT 1
LINER PUNCTURE TESTING RESULTS
MARCH 2009
March 23, 2009

Mr. Troy Meyer  
Tetra Tech M&M  
3031 West Ina Road  
Tucson, Arizona 85741

Report for Rosemont Project  
Results of Geosynthetic Liner Puncture Test  
ATT Job No. 2688-20

Dear Mr. Troy Meyer,

The following report presents the results of the liner puncture test performed using the TTTP-09-01 Bedding Layer, QMP 30Jan2009 Drainage Layer, Bentomat DN (Lot #200839LO, Roll # 10699) GCL, and 60 mil double sided textured LLDPE (No Lot # or Roll # provided) samples, in accordance with your request. The purpose of the liner puncture test was to determine if the 60 mil double sided textured LLDPE liner, when placed between compacted layers of TTTP-09-01 Bedding Layer and QMP 30Jan2009 Drainage Layer material with the Bentomat DN GCL under the liner, would develop punctures or pinholes when a simulated 450 foot heap height load (390 psi) was placed on the sample.

The scope of work was authorized and defined in e-mail messages and conversations between Troy Meyer of Tetra Tech M&M (TTMM), with Mr. Kerry Repola of Advanced Terra Testing (ATT). The work described in this document itemizes the testing requirements, which were followed and are described in subsequent sections of this report.
SAMPLE RECEIPT AND CONDITION

The TTTP-09-01 Bedding Layer sample (6 5-gallon buckets) and the QMP 30Jan2009 Drainage Layer (6 5-gallon buckets) was shipped to ATT via UPS. The samples arrived at the ATT laboratory in Lakewood, Colorado on February 04, 2009. Following receipt inspection, Tetra Tech M&M forwarded instructions regarding the preparation of the material for the subsequent test program.

SAMPLE PREPARATION

The TTTP-09-01 sample was prepared over the 1" sieve prior to testing in accordance with the instructions provided by Tetra Tech M&M. The QMP 30Jan2009 sample was manually crushed over the 1.5" sieve prior to testing. A representative sized sample was then split from the total bulk sample to perform other requested testing. A portion of the -1" TTTP-09-01 material was also used in performing a modified proctor test (ASTM D1557). The remaining material was moisture conditioned to an 8.1% moisture content as determined by the proctor. The QMP 30Jan 2009 material was assumed to be at a dry condition and a dpclp density was determined by pouring the material from approximately 3’ to simulate field conditions. A total of 3 trials were run and the results were averaged. A maximum of 2% difference in density was allowed between each trial.

TEST APPARATUS

The test cell designed for this application is 12.0 inches in inside diameter (ID) and is 19 inches in inside height. The actual size of the test specimen was 12.0 inches in diameter with a height of 6 inches for the bottom TTTP-09-01 Bedding Layer, and 12.0 inches of height for the QMP 30Jan2009 Drainage Layer. The base plate for the cylinder is 1.0 inches thick. The base plate is machined to fit inside the cylinder and is sealed with an "O" ring to prevent leakage. A removable bail affixed to the
base plate, facilitates transportation by crane, as the assembled cell with specimen and loading plates has an approximate mass of 350 pounds.

Load was applied by means of a wide-bay loading frame. The loading frame has a capacity of 220,000 pounds, and is hydraulically controlled such that a constant load can be applied to the sample to imitate a constant heap height throughout the test.

PROCEDURE - LINER PUNCTURE TEST
A six inch layer of TTTP-09-01 Bedding Layer was compacted in the base of the test cell, to a dry density of 122 pc/ at 8.1% moisture content. A twelve inch diameter sample of GCL was placed, non-woven side up, on top of the compacted Bedding Layer. A twelve inch diameter sample of Double Sided Textured 60 mil LLDPE liner material was placed on top of the GCL. A twelve inch layer of QMP 30Jan2009 Drainage Material was then placed at 78.84 pc/ dry density on top of the LLDPE material. The one inch loading plate was then placed on top of the QMP 30Jan2009 drainage material, and the test cell was placed into the load frame. Hydraulic pressure was then applied to the sample via the loading plate, until a load equivalent to 450 feet of heap height (390 psi) was achieved. See figure 1 for the diagram. The load was maintained on the sample for a minimum of 24 hours. The load was then released from the sample and the top layer of QMP 30Jan2009 material was carefully removed to expose the geosynthetic liner material.

SUMMARY AND OBSERVATIONS - LINER PUNCTURE TEST
Visual observations of the Double Sided Textured 60 mil LLDPE liner material revealed that although several indentations were present, no punctures were detected when the sample was inspected over a bright light source. Additionally, vacuum testing performed on the liner sample indicated that no punctures were present. Pictures of the LLDPE are included in this report.
This concludes the presentation of data and observations pertinent to this project. During the testing process we observed no anomalies or circumstances that deviate
from our standard approach in conducting these tests. The data presented are applicable only to this sample and described test conditions. The test results do not apply to other materials or test conditions. It has been a pleasure to provide these testing services for you. If you have any questions or require further information, please feel free to contact us.

Yours very truly,
ADVANCED TERRA TESTING, Inc.

[Signature]
William Rausch
Senior Geosynthetics Technician

[Signature]
Christopher Wienecke
Laboratory Director/Owner
Figure 1. Cell Cross Section for Tetra Tech M&M.
Tetra Tech M&M
2G88-20
Rosemont
60mil Double Sided Textured LLDPE
(After Puncture Test)

TM2688/TMDP60M2
03/09/09
Mike Thornbrue  
Tetra Tech  
3030 West Ina Road  
Tucson, AZ 85741

August 24, 2010

RE: Geostatic Point Load on  
   TTP-09-01(BU-01), Gila Conglomerate  
   Bentomat DN, Lot 201005LO, Roll 1308  
   Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09  
   QMP 30Jan2009, Drainage Layer

Mike Thornbrue,

In accordance with your request, we have completed the six Geostatic Point Load Tests consisting of four tests configured in a typical configuration, and two utilizing a planned defect. A Final report is enclosed. If you have any questions, please feel free to give us a call.

Sincerely,

Advanced Terra Testing, Inc.

Mary McFadden  
Geosynthetic Technician  
Signing For Kerry M. Repola  
Laboratory Manager

William Rausch  
Geosynthetic Senior Technician
FINAL REPORT
Geostatic Point Load Testing

Prepared for:

Mike Thornbrue
Tetra Tech
3030 West Ina Road
Tucson, AZ 85741

Prepared by:

Advanced Terra Testing, Inc.
833 Parfet Street, Unit A
Lakewood, Colorado 80215

Project No. 2688-29

August 20, 2010
This laboratory test program report is for the exclusive use of Tetra Tech and consists of six Geostatic Point Load Tests. The series tests consisted of four tests configured with six inches of bedding layer, a clay liner, a LLDPE liner and twelve inches of drainage aggregate, while the other 2 tests were the same configuration with an angular half inch rock placed in between the bedding layer and the clay liner. All materials were supplied to Advanced Terra Testing, Inc. (ATT) by Tetra Tech. The test series was performed at a normal stress of 508 pounds per square inch, as required by Tetra Tech.

GEOSYNTHETIC AND SOIL MATERIAL

- TTTP-09-01(BU-01), Gila Conglomerate
- Bentomat DN, Lot 201005LO, Roll 1308
- Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09
- -1.5” QMP 30Jan2009, Drainage Layer

GEOSTATIC POINT LOAD TEST

Sample Preparation
The soil samples, TTTP-09-01(BU-01) and QMP 30Jan2009, used were collected from a previous job for the Rosemont Copper Project, ATT job number 2688-20. A proctor was run in accordance with ASTM D698 on the TTTP-09-01(BU-01) sample in order to determine the maximum dry density and optimum moisture content of the sample. A Grain Size Analysis was conducted on the QMP 30Jan2009 sample in order to determine whether the grain size distribution was within the requirements. Due to limited sample size, material for both of the soil samples were reused as needed. Each time it was necessary to reuse the QMP 30Jan2009 sample a Grain Size Analysis was ran to ensure the material was still within the requirements. The actual test data for the Proctor and the Grain Size Analysis are presented in Appendix A.

Geostatic Point Load Configurations

The Geostatic Point Load Tests as requested by Tetra Tech were configured as follows:

Geostatic Point Load Test Numbers 1-4:

For this set of tests, the TTTP-09-01(BU-01) sample was compacted in a six inch layer to 95% of its maximum dry density at optimum moisture content (125.4 pounds per cubic foot and 9.9% moisture content) in a twelve inch diameter mold. A twelve inch diameter piece of Bentomat DN was then placed on top of the compacted TTTP-09-01(BU-01), a twelve inch diameter piece of the 60 mil textured liner was then placed on top of the Bentomat DN. Finally a twelve inch layer of -1.5” QMP 30Jan2009 was placed at a
density of 83.6 pounds per cubic foot into the top of the twelve inch diameter mold. A one inch thick steel plate was placed on top of the configuration to distribute the load evenly across the surface. Pressure was applied through a fifty five ton hydraulic press. Pressure was held at a continuous pressure of five hundred and eight pounds per square inch for a minimum of forty-eight hours. The actual test data as reported is presented in Appendix A.

For each test in this Geostatic Point Load Test, fresh Geosynthetic materials were prepared for each Puncture Test. Due to limited sample the TTTP-09-01(BU-01) and QMP 30Jan2009 material was reused as needed, a Grain Size Analysis was used to confirm that the drainage layer had not broken down and still met the requirements.

**Geostatic Point Load Test Numbers 5 & 6:**

For this set of tests, the TTTP-09-01(BU-01) sample was compacted in a six inch layer to 95% of its maximum dry density at optimum moisture content (125.4 pounds per cubic foot and 9.9% moisture content) in a twelve inch diameter mold. A half inch angular rock was placed on top of the compacted soil layer. A twelve inch diameter piece of Bentomat DN was then placed on top of the compacted TTTP-09-01(BU-01) and angular rock, then a twelve inch diameter piece of the 60 mil textured liner was then placed on top of the Bentomat DN. Finally a twelve inch layer of -1.5” QMP 30Jan2009 was placed at a density of 83.6 pounds per cubic foot into the top of the twelve inch diameter mold. A one inch thick steel plate was placed on top of the configuration to distribute the load evenly across the surface. Pressure was applied through a fifty five ton hydraulic press. Pressure was held at a continuous pressure of five hundred and eight pounds per square inch for a minimum of forty eight hours. The actual test data as reported is presented in Appendix A.

For each test in this Geostatic Point Load Test, fresh Geosynthetic materials were prepared for each Puncture Test. Due to limited sample the TTTP-09-01(BU-01) and QMP 30Jan2009 material was reused as needed, a Grain Size Analysis was used to confirm that the drainage layer had not broken down and still met the requirements.

**Observations**

After each test was performed a vacuum test was performed on the 60 mil textured liner to determine if any punctures were present in the liner, for all six samples it was determined that no punctures were in the liner. Depth measurements were taken on six of the most apparent indentations in the liner, moderate indentation were observed in all six tests, the actual depth measurements as reported are presented in Appendix A. For tests five and six major indentations were present at the position of half inch angular rock. Although a major indentation was present no punctures were detected during the vacuum test of the liner. Pictures of the two liners, the half inch angular rocks used, the top surface of the QMP 30Jan2009 layer and bottom inch of the QMP Jan2009 layer are presented in Appendix B.
For each Geostatic Point Load Test conducted the actual data and a schematic of the configuration are presented in Appendix A. The consolidation of each configuration was measured and a final density of the QMP 30Jan2009 was calculated. Consolidation was assumed to have occurred in the QMP 30Jan2009 layer.

This concludes our report for the Geostatic Point Load Testing performed as requested by Tetra Tech. The results reported apply only to the materials supplied and do not apply to other materials or test conditions.
APPENDIX A

GEOSTATIC POINT LOAD TEST DATA,
SCHEMATIC of the CONFIGURATION,
PROCTOR
&
GRAIN SIZE ANALYSIS
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Job No.: 2688-29
Location: Rosemont Copper Project
Project No.: --
Test Series: Puncture 1
Test Date: 7/19/10 WAR

Load Applied: 508 psi
Duration of Test: 48 Hours
Initial Height (in): -0.23
Final Height (in): 2.51
Total Height Change (in): 2.74

Configuration (Bottom to Top)

Bottom Layer
Boring Number: TTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
Dry Density: 125.4 pcf
Moisture Content: 9.9%
Height: 6.0 in
Diameter: 12.0 in

Puncture Test Results
Vacuum Test (pass/fail): Pass
Indentation Depth 1 (in): 0.1296
Indentation Depth 2 (in): 0.1221
Indentation Depth 3 (in): 0.1049
Indentation Depth 4 (in): 0.1398
Indentation Depth 5 (in): 0.1185
Indentation Depth 6 (in): 0.1324

Geosynthetic Configuration
Bottom Layer: Bentomat DN, Lot 201005LO, Roll1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30Jan2009
Sample Number: Drainage Layer
Sample Depth: --
Initial Dry Density: 83.6 pcf
Moisture Content: 0.00%
Height: 12.0 in
Diameter: 12.0 in

Data Entered By: WAR
Date: 7/30/2010
File Name: TMGSP1

Data Checked By:
Date: 8/10
Figure 1. Cell Cross Section for Tetra Tech M&M.
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Load Applied: 508 psi
Job No.: 2688-29
Duration of Test: 48 Hours
Location: Rosemont Copper Project
Initial Height (in): -0.15
Project No.: --
Final Height (in): 2.20
Test Series: Puncture 2
Total Height Change (in): 2.35
Test Date: 7/22/10 WAR

Configuration (Bottom to Top)

Bottom Layer
Boring Number: TTTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
Dry Density: 125.4pcf
Moisture Content: 9.9%
Height: 6.0 in
Diameter: 12.0 in

Puncture Test Results

Vacuum Test (pass/fail): Pass
Indentation Depth 1 (in): 0.0826
Indentation Depth 2 (in): 0.1348
Indentation Depth 3 (in): 0.106
Indentation Depth 4 (in): 0.1076
Indentation Depth 5 (in): 0.0924
Indentation Depth 6 (in): 0.1243

Geosynthetic Configuration

Bottom Layer: Bentomat DN, Lot 201005LO, Roll 1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30Jan2009
Sample Number: Drainage Layer
Sample Depth: --
Initial Dry Density: 83.6 pcf
Moisture Content: 0.00%
Height: 12.0 in
Diameter: 12.0 in

Data Entered By: WAR
Date: 7/30/2010
File Name: TMGSP2

Data Checked By: [Signature]
Date: 8/2/10
Tetra Tech M&M  2688-29
Rosemont Copper project
Puncture 2

LINER PUNCTURE TEST
CELL CROSS SECTION
508 psi
Normal Load
For 48 Hours

LOADING PLATE

QMP 30Jan2009
Drainage Layer
83.6 lbs/ft^3
0% Moisture

60 mil. Double Sided
Textured LLDPE

Bentomat DN GCL
Gila Conglomerate
TTTP-09-01(BU-01)
125.4 lbs/ft^3
9.9% Moisture

BASE

Initial Height
- .15"

Final Height
2.20"
Height Change
2.35"

Density of Drainage Layer
Initial
83.6 lbs/ft^3
Final
104.0 lbs/ft^3

Note: consolidation assumed to occur in the Drainage layer section.

Figure 1. Cell Cross Section for Tetra Tech M&M.
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Job No.: 2688-29
Location: Rosemont Copper Project
Project No.: --
Test Series: Puncture 3
Test Date: 7/26/10 WAR

Load Applied: 508 psi
Duration of Test: 48 Hours
Initial Height (in): 0.05
Final Height (in): 2.35
Total Height Change (in): 2.30

Configuration (Bottom to Top)

Bottom Layer
Boring Number: TTTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
Dry Density: 125.4 pcf
Moisture Content: 9.9%
Height: 6.0 in
Diameter: 12.0 in

Puncture Test Results
Vacuum Test (pass/fail): Pass
Indentation Depth 1 (in): 0.1188
Indentation Depth 2 (in): 0.1111
Indentation Depth 3 (in): 0.1271
Indentation Depth 4 (in): 0.1082
Indentation Depth 5 (in): 0.1245
Indentation Depth 6 (in): 0.1506

Geosynthetic Configuration
Bottom Layer: Bentomat DN, Lot 201005LO, Roll 1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30Jan2009
Sample Number: Drainage Layer
Sample Depth: --
Initial Dry Density: 83.6 pcf
Moisture Content: 0.00%
Height: 12.0 in
Diameter: 12.0 in

Data Entered By: WAR
Date: 7/30/2010
File Name: TMGSP3

Data Checked By:
Date: 8/10/2010
Figure 1. Cell Cross Section for Tetra Tech M&M.
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Job No.: 2688-29
Location: Rosemont Copper Project
Project No.: --
Test Series: Puncture 4
Test Date: 08/02/10 WAR

Load Applied: 508 psi
Duration of Test: 48 Hours
Initial Height (in): -0.11
Final Height (in): 2.19
Total Height Change (in): 2.30

Configuration (Bottom to Top)
Bottom Layer
Boring Number: TTTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
Dry Density: 125.4 pcf
Moisture Content: 9.9%
Height: 6.0 in
Diameter: 12.0 in

Puncture Test Results
Vacuum Test (pass/fail): Pass
Indentation Depth 1 (in): 0.1061
Indentation Depth 2 (in): 0.0924
Indentation Depth 3 (in): 0.1063
Indentation Depth 4 (in): 0.1243
Indentation Depth 5 (in): 0.1188
Indentation Depth 6 (in): 0.0805

Geosynthetic Configuration
Bottom Layer: Bentomat DN, Lot 201005LO, Roll1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30Jan2009 -1.5"
Sample Number: Drainage Layer
Sample Depth: --
Initial Dry Density: 83.6 pcf
Moisture Content: 0.00%
Height: 12.0 in
Diameter: 12.0 in

Data Entered By: MLM
Date: 8/16/2010
File Name: TMGSP4

Data Checked By: [Signature]
Date: 8/16/10
Figure 1. Cell Cross Section for Tetra Tech M&M.
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Job No.: 2688-29
Location: Rosemont Copper Project
Project No.: --
Test Series: Puncture 5 (Planned Defect)
Test Date: 08/05/10 WAR

Load Applied: 508 psi
Duration of Test: 48 Hours
Initial Height (in): -0.18
Final Height (in): 2.07
Total Height Change (in): 2.25

Configuration (Bottom to Top)

Bottom Layer
Boring Number: TTTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
   Dry Density: 125.4 pcf
   Moisture Content: 9.9%
   Height: 6.0 in
   Diameter: 12.0 in

Vacuum Test (pass/fail): Pass
Indentation Depth 1 (in): 0.2763
Indentation Depth 2 (in): 0.1120
Indentation Depth 3 (in): 0.0928
Indentation Depth 4 (in): 0.0884
Indentation Depth 5 (in): 0.1001
Indentation Depth 6 (in): 0.0969

Geosynthetic Configuration
Bottom Layer: Bentomat DN, Lot 201005LO, Roll 1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30 Jan 2009 -1.5"
Sample Number: Drainage Layer
Sample Depth: --
   Initial Dry Density: 83.6 pcf
   Moisture Content: 0.00%
   Height: 12.0 in
   Diameter: 12.0 in

Data Entered By: MLM
Date: 8/16/2010
File Name: TMGSP5

Data Checked By: WMP
Date: 8/16/10
LINER PUNCTURE TEST
CELL CROSS SECTION

508 psi
Normal Load
For 48 Hours

LOADING PLATE

QMP 30Jan2009
Drainage Layer
83.6 lbs/ft^3
0% Moisture

60 mil. Double Sided
Textured LLDPE

Bentomat DN GCL

Gila Conglomerate
TTTP-09-01(BU-01)
125.4 lbs/ft^3
9.9% Moisture

BASE

12 in

Initial Height
-.18"

Final Height
2.07"

Height Change
2.25"

Density of
Drainage Layer
Initial
83.6 lbs/ft^3
Final
102.9 lbs/ft^3

12 in

6 in

Note: consolidation
assumed to occur in the
Drainage layer section.

Figure 1. Cell Cross Section for Tetra Tech M&M.
Geostatic Point Load Test
ATT Method

Client: Tetra Tech M&M
Job No.: 2688-29
Location: Rosemont Copper Project
Project No.: --
Test Series: Puncture 6 (Planned Defect)
Test Date: 08/07/10 WAR

Load Applied: 508 psi
Duration of Test: 48 Hours
Initial Height (in): 0.04
Final Height (in): 2.04
Total Height Change (in): 2.00

Configuration (Bottom to Top)
Bottom Layer
Boring Number: TTTP-09-01(BU-01)
Sample Number: Gila Conglomerate
Sample Depth: --
  Dry Density: 125.4pcf
  Moisture Content: 9.9%
  Height: 6.0 in
  Diameter: 12.0 in

Puncture Test Results
Vacuum Test (pass/fail): Pass
  Indentation Depth 1 (in): 0.2107
  Indentation Depth 2 (in): 0.0863
  Indentation Depth 3 (in): 0.0746
  Indentation Depth 4 (in): 0.1176
  Indentation Depth 5 (in): 0.0785
  Indentation Depth 6 (in): 0.1103

Geosynthetic Configuration
Bottom Layer: Bentomat DN, Lot 201005LO, Roll1308
Top Layer: Agru, 60 mil LLDPE Double Sided Textured, Roll 330443-09

Top Layer
Boring Number: QMP 30Jan2009 -1.5"
Sample Number: Drainage Layer
Sample Depth: --
  Initial Dry Density: 83.6pcf
  Moisture Content: 0.00%
  Height: 12.0 in
  Diameter: 12.0 in

Data Entered By: MLM
Date: 8/16/2010
File Name: TMGSP6

Data Checked By: [Signature]
Date: 8/16/10
LINER PUNCTURE TEST
CELL CROSS SECTION

508 psi
Normal Load
For 48 Hours

LOADING PLATE

QMP 30Jan2009
Drainage Layer
83.6 lbs/ft^3
0% Moisture

60 mil. Double Sided
Textured LLDPE

Bentomat DN GCL

Gila Conglomerate
TTTP-09-01(BU-01)
125.4 lbs/ft^3
9.9% Moisture

BASE

Initial Height
-.04"

Final Height
2.04"

Height Change
2.00"

Density of Drainage Layer
Initial
83.6 lbs/ft^3
Final
100.3 lbs/ft^3

Note: consolidation assumed to occur in the Drainage layer section.

Figure 1. Cell Cross Section for Tetra Tech M&M.
Moisture Determination

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<th>3</th>
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<tr>
<td>Wt. of Moisture added (ml)</td>
<td>240.00</td>
<td>200.00</td>
<td>160.00</td>
<td>120.00</td>
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<td>Wt. of soil &amp; dish (g)</td>
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<td>541.56</td>
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<td>Corrected Moisture Content</td>
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<td>12.37</td>
<td>10.90</td>
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Density determination

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<td>Net wt. of wet soil (lb)</td>
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<td>Net wt of dry soil (lb)</td>
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<td>Dry Density, (pcf)</td>
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<td>30</td>
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Note: If a rock correction is used the dry density has been calculated using the corrected moisture content.
Proctor Compaction Test
TTTP-09-01(BU-01), Gila Conglomerate

Zero Air Voids Curve @ SG reported below

- Best Fit Curve
- Actual Data
- Zero Air Voids Curve @ SG = 2.75

Optimum moisture content = 9.9
Maximum dry density = 132.0
ASTM D 698 B, Rock correction applied? Y
MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D 6913

CLIENT: Tetra Tech M&M
JOB NO.: 2688-29

BORING NO.: QMP 30Jan2009
DEPTH:
SAMPLE NO.: Drainage Layer
SOIL DESCR.: -1.5" post-test
LOCATION: Rosemont Copper Project

SAMPLED
DATE +#4 DRY SIEVED: 07/28/10 QRS
DATE -#4 WASHED: 07/29/10 PW
WASH SIEVE: Yes
DRY SIEVE: No

MOISTURE DATA

HYGROSCOPIC: Yes
NATURAL: No

Wt. of Pan Only (g) 8.17
Wt. of Dry Soil (g) 182.76
Moisture Content % 0.7

Wt. Total Sample
Wet (g) 15409.40
Weight of + #4
Before Washing (g) 13799.00
Weight of + #4
After Washing (g) 13799.00
Weight of - #4
Wet (g) 1610.40
Weight of - #4
Dry (g) 1599.37
Wt. Total Sample
Dry (g) 15398.37

Wt. Partial -#4 Sample Wet (g) 209.98
Wt. Partial Sample Dry (g) 208.54
Calc. Wt. "W" (g) 2007.79
Calc. Mass + #4 1799.25

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<td>1 1/2&quot;</td>
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<td>106.67</td>
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Data entered by: MLM  Date: 08/12/2010
Data checked by: [Signature] Date: 8/12/10
FileName: TMM03009
APPENDIX B

PHOTOGRAPHS
TetraTech M&M 2688-29
Rose mont Copper Project

Puncture Z
Rosemont Copper Project

Puncture 2
Puncture 3
Tetra Tech M+M 2688-29
Rosemont Copper Project
PUNCTURE: 5
1
Tetra Tech M+M 2688-29
Rose mont Copper Project

Puncture
Tetra Tech M&I 2688-29
Rosemont Copper Project

Puncture 5
Tetra Tech M+M  ZC88-29
Rosemont Copper Project

PUNCTURE 6
Tetra Tech M+M 2688-29
Rosemont Copper Project

PUNCTURE 6
TetraTech M&H 2688-29
Rosmont Copper Project

Puncture 6
ATTACHMENT G
TECHNICAL MEMORANDUM
ROSEMONT HEAP LEACH PAD ANCHOR TRENCH STABILITY
1.0 Introduction

This Technical Memorandum provides a summary of Tetra Tech's anchor trench stability analysis related to the Heap Leach Facility (HLF) at the proposed Rosemont Copper Project (Project) in Pima County, Arizona. This information is 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). Specifically, this Technical Memorandum answers item no. 6 on page 7 of 18.

- Anchor Trench – Please submit stability calculations supporting the design for the anchor trench within the perimeter containment berm. This feature is a critical component with respect to pad stability.

The results of our calculations indicate that the anchor trench will be stable and will provide sufficient resistance to the forces developed in the geomembrane due to the differential settlement.

2.0 Anchor Trench Stability

2.1 Tensile Strength Capacity of Anchor Trench

The tensile strength capacity of the anchor trench was evaluated using the methodology presented by Koerner (1999). The methodology is based on a static equilibrium analysis of the problem. Illustration 1 shows the free body diagram for the geomembrane considered to develop the analytical equations.

The proposed analytical equation for determination of the allowable geomembrane tension from the anchor trench is:

\[ \sum F_x = 0 \]  \hspace{1cm} (1)

\[ T_{allow} \cos \beta = F_{Uo} + F_{Lo} + F_{LT} - P_a + P_p \]  \hspace{1cm} (2)
where:

\[ T_{\text{allow}} = \text{allowable force in geomembrane stress} = \sigma_{\text{allow}} t; \]
\[ \sigma_{\text{allow}} = \text{allowable stress in geomembrane}; \]
\[ t = \text{thickness of geomembrane}; \]
\[ \beta = \text{side slope angle}; \]
\[ F_{U0} = \text{shear force above geomembrane due to cover soil (negligible for thin cover soils)}; \]
\[ F_{L0} = \text{shear force below geomembrane due to cover soil}; \]
\[ F_{LT} = \text{shear force below geomembrane due to vertical component of } T_{\text{allow}}; \]
\[ P_a = \text{active earth pressure against the backfill side of the anchor trench; and} \]
\[ P_p = \text{passive earth pressure against the in-situ side of the anchor trench}. \]

**Illustration 1**  Cross Section of Geomembrane with Anchor Trench and Related Stresses and Forces Involved (modified from Koerner, 1999).
The shear forces above and below the geomembrane are defined as:

\[ F_{U0} = \sigma_n \tan \delta_U (L_{RO}) \]  
\[ F_{L0} = \sigma_n \tan \delta_L (L_{RO}) \]  
\[ F_{LT} = 0.5 \left( \frac{2T_{allow} \sin \beta}{L_{RO}} \right) (L_{RO}) \tan \delta_L \]

where:
- \( \sigma_n \) = applied normal stress from cover soil;
- \( \delta_U \) = angle of shearing resistance between geomembrane (LLDPE) and cover soil (upper material);
- \( \delta_L \) = angle of shearing resistance between geomembrane (LLDPE) and GCL (lower material); and
- \( L_{RO} \) = length of geomembrane runout.

The active and passive earth pressures are defined as:

\[ P_a = \frac{1}{2} (\gamma_{AT} d_{AT}) K_a d_{AT} + (\sigma_n) K_a d_{AT} \]  
\[ P_a = (0.5 \gamma_{AT} d_{AT} + \sigma_n) K_a d_{AT} \]  
\[ P_p = (0.5 \gamma_{AT} d_{AT} + \sigma_n) K_p d_{AT} \]

Where:
- \( \gamma_{AT} \) = unit weight of soil in anchor trench;
- \( d_{AT} \) = depth of anchor trench;
- \( K_a \) = coefficient of active earth pressure = \( \tan^2(45 - \varphi/2) \);
- \( K_p \) = coefficient of passive earth pressure = \( \tan^2(45 + \varphi/2) \); and
- \( \varphi \) = angle of shearing resistance of respective soil.

Combining equations (2) and (5), the tensile strength capacity of the anchor trench (\( T_{allow} \)) is:

\[ T_{allow} = \frac{(F_{U0} + F_{L0} - P_a + P_p)}{\cos \beta \sin \beta \tan \delta_L} \]
2.2 Calculations and Results

Illustration 2 shows the geometry of the anchor trench and soil parameters used to calculate the capacity of the proposed anchor trench.

Illustration 2  Geometry of Anchor Trench and Related Soil Parameters

The following conservative assumptions were considered in the calculation of the anchor trench capacity:

- The soil cover above the geomembrane was neglected, therefore $\sigma_n=0$, $F_{Uo}=0$, and $F_{Lo}=0$;
- The weakest interface friction was used in the calculation. This is the friction between the LLDPE liner and the GCL ($\delta_L=20.3^\circ$);
- The geomembrane runout length within the anchor trench was neglected.

Based on the above assumptions, the calculated anchor trench capacity is

$$T_{allow} = \frac{F_{Uo} + F_{Lo} - P_a + P_p}{\cos \beta - \sin \beta \tan \delta_L} = \frac{-P_a + P_p}{\cos \beta - \sin \beta \tan \delta_L}$$

and:

$$P_a = 0.5K_a \gamma_{AT} d_{AT}^2 = 6.19 \text{ lb ft}^2$$

$$P_p = 0.5K_p \gamma_{AT} d_{AT}^2 = 55.69 \text{ lb ft}^2$$
$T_{allow} = 59.49 \frac{lb}{in}$

According to the settlement calculations presented in a separate Technical Memorandum titled *Rosemont Heap Leach Pad Settlement Analysis* dated August 11, 2010 (Tetra Tech, 2010), the maximum strain in the liner system will be 0.0012 percent. Using the tensile elongation at yield of 12 percent and the tensile strength at yield of 100 lb/in for the proposed LLDPE liner, the maximum tensile stress in the geomembrane due to differential settlement will be:

$T_{max} = \frac{100 \frac{lb}{in}}{12\%} \cdot 0.0012\% = 0.01 \frac{lb}{in}$

According to these analyses the anchor trench will provide sufficient resistance to the forces developed in the geomembrane due to the differential settlement. The maximum force to be experienced by the geomembrane was calculated to be 0.01 lb/in, while the anchor trench provides an allowable resistance force equal to 59.49 lb/in.

3.0 Conclusions

The calculations show that the anchor trench will provide sufficient resistance to the forces developed in the geomembrane due to the differential settlement.
REFERENCES


ATTACHMENT H
TECHNICAL MEMORANDUM
ROSEMONT PHASE 2 HEAP LEACH PAD UNDERDRAIN DESIGN
1.0 Introduction

This Technical Memorandum summarizes the methods and computations employed to estimate the capacity of the Phase 2 Heap Leach Pad Underdrain System as shown on DWG 080-CI-928 (Attachment 1) for the proposed Rosemont Copper Project (Project) Heap Leach Facility (HLF) in Pima County, Arizona. DWG 080-CI-928 was previously submitted to the Arizona Department of Environmental Quality (ADEQ) in May 2009 as part of the Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009a). The May 2009 HLF permit report is part of the Aquifer Protection Permit (APP) application submitted to the ADEQ in April 2009 (Tetra Tech, 2009b). This computation summary is in response to the April 14, 2010 Comprehensive Request for Additional Information from ADEQ to Rosemont Copper Company (Rosemont). Specifically, this Technical Memorandum answers item no. 7 on page 7 of 18.

- **Underdrain**

  Current leach pad layout (DWG. 080-CI-928) shows an underdrain on western perimeter of Phase 2 of the Heap Leach Facility. (Ref. Rosemont Heap Leach Facility Permit Design Report, Volume 1, May 2009)

  Please indicate the design criteria used for the underdrain design indicating estimated amount of surface and subsurface flow designed for discharge through the underdrain system.

2.0 Underdrain System

The Phase 2 Pad Underdrain system is comprised of a 3’ x 3’ trench filled with Overliner Drain Fill (minus 1.5-inch rock) wrapped in an 8-ounce non-woven geotextile. An 8-inch diameter perforated ADS N-12 pipe is located within the drain rock. This underdrain system is designed to convey up-gradient stormwater underneath the Phase 2 Pad which follows a natural wash.

The 8-inch perforated drain pipe is sized to convey the peak flow from a 100-year, 24-hour storm event. The drain rock provides additional flow capacity above the 100-year, 24-hour event. The Phase 2 Pad Underdrain system includes a sediment trap at the upstream end of the underdrain.
Only storm flow is assumed to pass through the underdrain, i.e., no subsurface flow contributions.

3.0 Methodology and Precipitation

The Natural Resource Conservation Service (NRCS) method was selected to perform the hydrologic calculations. The NRCS method described herein is based on two (2) components, the Curve Number procedure (to determine initial losses and excess precipitation) and the unit hydrograph method (to derive the hydrograph resulting from excess rainfall).

HEC-HMS, developed by the U.S. Army Corps of Engineers, incorporates the NRCS method and was selected to determine the discharge resulting from the sub-basin that contributes to the Phase 2 Underdrain system located along the west side of the proposed Phase 2 Heap Leach Pad.

The NRCS has developed a widely used curve number procedure for estimating runoff from storm events. Rainfall initial losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors (curve numbers). Curve numbers represent the runoff potential of a soil type (i.e., the higher the curve number, the higher the runoff potential).

A detailed discussion of the NRCS method can be found in the Technical Memorandum titled *Rosemont Hydrology Method Justification* (Tetра Tech, 2010).

Precipitation data was acquired from the National Oceanic and Atmospheric Association (NOAA) Atlas 14 Point Precipitation website (NOAA, 2009).

4.0 Hydrology

The NRCS method was selected as the most appropriate for this level of analysis based on the information available and on the analysis expectations.

The underdrain was modeled using an 8-inch diameter N-12 perforated pipe to drain the 18.31 acre area as shown on Figure 1 provided in Attachment 2.

4.1 Rainfall Excess

The NRCS method determines rainfall runoff volume using the following relationship:

\[ Q = \frac{(P - 0.2S)^2}{P + 0.8S} \]

Where,

- Q is the accumulated runoff volume in inches;
- P is the accumulated precipitation in inches, (100-year, 24-hour rainfall depth = 4.75”);
- S is the maximum soil water retention parameter, \((S = \frac{1000}{CN} - 10)\), in inches; and
- CN is the curve number.
4.2 Curve Number

The Curve Number selected for the contributing basin is 75 based on previous studies. The initial abstraction used was $0.2S = 0.667^\prime$.

4.3 Lag Time

The SCS lag time is defined as:

$$T_{LAG} = \frac{L^{0.8}(S+1)^{0.7}}{1900\sqrt{Y}}$$

Where,

- $T_{LAG}$ = lag time, in hours;
- $S$ = the maximum soil water retention parameter;
- $L$ = hydraulic length, in feet; and
- $Y$ = watershed slope in %.

5.0 Results

The model results show that the storage capacity at the entry of the underdrain pipe reaches a maximum pool elevation of 5134.47 feet above mean sea level (ft amsl) with a total inflow of 3.43 acre-feet (ac-ft) based on the 100-year, 24-hour storm event. The total storage capacity upstream of the pipe is approximately 1.5 ac-ft. The time that the pipe will take to drain is about 19.5 hour with a minimum peak storage of 1.2 ac-ft. A summary of the results are shown in Table 1.

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<th>Basin ID</th>
<th>Area (ac)</th>
<th>Tlag (hr)</th>
<th>Qvolume (ac-ft)</th>
<th>Qpeak (cfs)$^1$</th>
<th>Maximum Head (ft)</th>
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$^1$cfs = cubic feet per second

Detail results from the HEC-HMS model output is provided in Attachment 3. Calculations assume that flows only pass through the 8-inch pipe for the 100-year, 24-hour event and not through the drain rock.
REFERENCES


ATTACHMENT 1
PHASE 2 UNDERDRAIN
ATTACHMENT 2

PHASE 2 UNDERDRAIN PIPE
ATTACHMENT 3
UNDERDRAIN CALCULATIONS
### Hydrologic Element Results

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<th>Peak Discharge (CFS)</th>
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**Computed Results**

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<tr>
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<td>End of Run:</td>
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<td>Met1</td>
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<td>Control Specifications:</td>
<td>Control1</td>
</tr>
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</table>
Technical Memorandum

To: Kathy Arnold

From: Mike Thornbrue

Company: Rosemont Copper Company

Date: August 20, 2010

Re: Rosemont Heap Leach Facility Permit Design Liner Leakage Calculations

Doc #: 221/10-320877-5.3

CC: Joel Carrasco (Tetra Tech)
    David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction

This Technical Memorandum supersedes any previous liner leakage estimates submitted in the Aquifer Protection Permit (APP) Application (Tetra Tech, 2009a), the Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009b), and the feasibility level Heap Leach Facilities Design (Tetra Tech, 2007) for the proposed Heap Leach Facility (HLF) at the proposed Rosemont Copper Project (Project) in Pima County, Arizona.

This information is 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). Specifically, this Technical Memorandum answers item no. 27 on page 12 of 18.

- Item 27 - Technical Memorandum, Rosemont Heap Leach Facilities – Liner Leakage Calculations April 27, 2009

  The alert level AL2 (Rapid and Large Leakage) for each of the Raffinate Pond and the PLS Pond is calculated at 15,272 gpd and 46,812 gpd, respectively.

  Rosemont's proposed alert level for each of the Raffinate Pond and the PLS Pond appears to be excessively high and shall be revised. Analytical calculations shall be based on system components, taking into account geomembrane defects, transmissivity of the drainage medium, design capacity of the leak collection and removal system (LCRS) rather than discharging capability of the pumping system alone at the LCRS. Please provide revised calculations.

The purpose of this Technical Memorandum is to document the calculations used to evaluate the level of engineering control achieved for various liner systems as part of the Best Available Demonstrated Control Technology (BADCT) analysis for the Heap Leach Facility at the proposed Rosemont Copper Project (Project). The calculations were used to estimate potential leakage rates (PLRs) through geomembrane liner systems for the proposed facilities. Additionally, calculations were performed to determine Alert Level (AL) liner leakage rates for
potential flow to the Leak Collection and Removal System (LCRS) in the Raffinate and PLS Ponds. This memorandum is organized as follows:

- Section 2.0 presents the equations used for the liner leakage calculations;
- Section 3.0 presents the BADCT analysis of alternative liner systems for the Stormwater Pond;
- Section 4.0 presents the BADCT analysis of alternative liner systems for the Heap Leach Pad;
- Section 5.0 presents the BADCT analysis of alternative liner systems Raffinate and PLS Ponds;
- Section 6.0 presents the calculations used to determine the proposed Alert Level 1 (AL1) and Alert Level 2 (AL2) for the Raffinate Pond;
- Section 7.0 presents the calculations used to determine the proposed AL1 and AL2 for the PLS Pond; and
- Section 8.0 presents a summary of the previous sections.

The configuration and location of the Raffinate Pond has been updated from the design shown in the Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009b) dated May 2009. The BADCT and liner leakage calculations are based on this new design. A stability analysis (Tetra Tech, 2009c) was also performed on the new pond location (see Attachment 1).

2.0 Equations

The calculations used in this memorandum are based on either Giroud’s Equation or Bernoulli’s Equation for free flow through an opening.

2.1 Giroud’s Equation

The leakage through a circular defect in a liner system that includes a low permeability component (soil or geosynthetic clay liner) along with a geomembrane liner was estimated using Giroud's Equation (Giroud, 1997):

\[ Q = 0.976 \ C_q \left[ 1 + 0.1 \left( \frac{h}{t_g} \right)^{0.95} \right] \ d^{0.2} \ h^{0.9} \ k_s^{0.74} \]

Where:

- \( Q \) = Rate of liquid migration or PLR [cubic meters per second (m³/s)];

The PLR is represented as the rate of liquid migration through a composite liner system. This is an accurate representation of the degree of engineering control achieved by a BADCT liner system. By maximizing the degree of engineering control, the rate of liquid migration or PLR is minimized.
C_{Qq} = Contact Quality Factor (CQF) that represents the contact interface between the low permeability component and the geomembrane liner (dimensionless);

This factor is dimensionless and ranges from 0.21 for good contact and 1.15 for poor contact. Typically, a GCL/geomembrane interface has a better CQF than a soil/geomembrane interface. However, a good CQF was used for all systems to provide a uniform comparison.

h = Height of liquid on top of geomembrane (m);

Giroud's Equation assumes that the hydraulic head on the liner to be less than or equal to three (3) meters (m). The empirical investigations published by Giroud and Bonaparte (1989) showed that permeation, leakage through a geomembrane liner without holes, may not be negligible in scenarios with more than three (3) meters of hydraulic head. Giroud’s Equation does not take permeation into account.

t_s = Thickness of the low permeability component (m);

The thickness of the low permeability component directly affects the amount of time necessary for a fluid to flow through the material.

d = Diameter of circular defect (m); and

Giroud’s Equation assumes a circular defect in the geomembrane liner having a diameter between 0.0005 m and 0.025 m. A single, two (2) millimeter (mm) diameter [area (a) = 3.14 mm²] hole per acre allows for seam defects that still may exist after intensive quality assurance resulting from fabrication or installation factors (Giroud and Bonaparte, 1989).

k_s = Hydraulic conductivity of the low permeability component (m/s).

The prescriptive BADCT permeability standard of 1x10^{-6} cm/s was used for the low permeable soil (LPS) calculations. A standard geosynthetic clay liner (GCL) permeability of 5x10^{-9} cm/s was selected for the GCL calculations (Cetco, 2009).

2.2 Bernoulli’s Equation for Free Flow Through an Opening

The rate of liquid migration or PLR through a geomembrane liner that is not placed directly on a low permeability component can be calculated using Bernoulli’s Equation for free flow through an opening. This equation was used to calculate the Alert Levels for the double-lined solution ponds. This equation can also be used to calculate the rate of liquid migration or PLR through liner systems that do not utilize a LPS or GCL beneath a geomembrane liner.

\[ Q = C_g a \sqrt{2gh_w} \]

Where:

\[ Q = \text{rate of liquid migration or PLR through a geomembrane hole (m}^3/\text{s}); \]
3.0 BADCT Analysis for the Stormwater Pond

As stated in Section 2.0, Giroud's Equation assumes the hydraulic head on the liner to be less than or equal to three (3) meters and therefore does not account for permeation of liquids through the liner. The Stormwater Pond is designed with a maximum capacity in excess of three (3) meters of hydraulic head. Because of this, the calculations used in this section to estimate the potential leakage rates do not account for permeation of liquids through the liner.

The maximum hydraulic head on the liner in the Stormwater Pond was determined using the total depth of the pond (20 feet) and subtracting the required freeboard (3 feet). This results in an overall maximum hydraulic head of 17 feet (5.18 meters). The total lined surface area (LSA) of the pond was estimated to be approximately 246,325 square feet (sf) or 5.65 acres. This area does not include the potential expansion of the pond as shown on the design drawings.

3.1 Stormwater Pond with a Low Permeability Soil Layer

The PLR for the Stormwater Pond having a composite liner system consisting of LPS and a single geomembrane liner was calculated using Giroud's Equation as presented in Section 2.0. The following values were established to represent the variables of the equation.

- Height of liquid on top of geomembrane: The maximum head allowed by the design (5.18 meters) was selected;
- Diameter of circular defect: A defect rate of one (1) hole per acre that is two (2) millimeters (mm) in diameter was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989);
- Thickness of LPS: The calculation used a LPS layer having a thickness of six (6) inches (0.1524 meters) underneath the geomembrane liner; and
- Hydraulic conductivity of low permeability component: The prescriptive BADCT permeability standard of 1x10-6 cm/s was used for the LPS layer (ADEQ, 2004).

Table 3.1 presents the PLR through a composite liner system comprised of a six (6) inch thick layer of LPS and a geomembrane liner.
### Table 3.1 PLR for the Stormwater Pond (LPS and Geomembrane)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(C_{qp})</td>
<td>0.21</td>
<td>(dimensionless)</td>
</tr>
<tr>
<td>(h)</td>
<td>5.18</td>
<td>(m)</td>
</tr>
<tr>
<td>(d)</td>
<td>0.002</td>
<td>(m)</td>
</tr>
<tr>
<td>(t_s)</td>
<td>0.1524</td>
<td>(m)</td>
</tr>
<tr>
<td>(k_s)</td>
<td>1.0E-8</td>
<td>(m/s)</td>
</tr>
<tr>
<td>(Q)</td>
<td>1.20E-06</td>
<td>(m³/s/defect)</td>
</tr>
</tbody>
</table>

The calculation yielded a PLR of \(Q = 1.20E-6 \text{ m}^3/\text{s/defect}\). This can be converted to gallons per day (gpd) per defect as follows:

\[
\frac{1.20E - 6 \text{ m}^3 / \text{s}}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ seconds}}{\text{minute}} \times \frac{60 \text{ minutes}}{\text{hour}} \times \frac{24 \text{ hours}}{\text{day}} = 27.39 \text{ gpd/d}\]

To establish the total potential leakage (TPL), the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre that is two (2) millimeters (mm) in diameter was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Stormwater Pond has an LSA of 246,325 square feet (sf) or 5.65 acres.

\[
TPL = \frac{27.39 \text{ gpd}}{\text{defect}} \times \frac{1 \text{ defect}}{\text{acre}} \times 5.65 \text{ acres} = 154.7 \text{ gpd}
\]

Therefore, the TPL through the liner system of the Stormwater Pond using LPS as the low permeability component is approximately 155 gpd.

### 3.2 Stormwater Pond with a Geosynthetic Clay Liner

The PLR for the Stormwater Pond having a composite liner system consisting of a GCL and a single geomembrane liner was calculated using Giroud’s Equation as presented in Section 2.0. The following values were established to represent the variables of the equation.

- Height of liquid on top of geomembrane: The maximum head allowed by the design (5.18 meters) was selected;
- Diameter of circular defect: A defect rate of one (1) hole per acre that is two (2) millimeters (mm) in diameter was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989);
- Thickness of GCL: The calculation used a GCL having a thickness of six (6) mm underneath the geomembrane liner; and
- Hydraulic conductivity of low permeability component: A GCL permeability of 5x10-9 cm/s was selected for the GCL layer (Cetco, 2009).

Table 3.2 presents the PLR through a composite liner system comprised of a GCL and a geomembrane liner.
Table 3.2 PLR for the Stormwater Pond (GCL and Geomembrane)

| $C_{qp}$ | 0.21 | (dimensionless) |
| $h$ | 6.4 | (m) |
| $d$ | 0.002 | (m) |
| $t_s$ | 0.0060 | (m) |
| $k_s$ | 5.0E-11 | (m/s) |
| $Q$ | 3.88E-07 | PLR (m$^3$/s/defect) |

The calculation yielded a PLR of $Q = 3.88 \times 10^{-7}$ m$^3$/s/defect. This can be converted to gpd per defect as follows:

$$\frac{3.88E - 7m^3}{s} \times \frac{264.17 gallons}{m^3} \times \frac{60s}{min} \times \frac{60min}{hr} \times \frac{24hr}{day} = 8.96 \text{ gpd/defect}$$

To establish the TPL, the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Stormwater Pond has an LSA of 246,325 square feet (sf) or 5.65 acres.

$$TPL = \frac{8.96 \text{ gpd}}{\text{defect}} \times \frac{1 \text{ defect}}{\text{acre}} \times 5.65 \text{ acres} = 50.62 \text{ gpd}$$

Therefore, the TPL through the liner system of the Stormwater Pond using GCL as the low permeability component is approximately 51 gpd.

### 3.3 Prescriptive BADCT Liner System for the Stormwater Pond

Because a low permeability component (GCL or LPS) is not required for prescriptive BADCT design of a non-stormwater pond, i.e., for an overflow pond that will contain process solution for short periods of time due to process upsets of rainfall events, the PLR for the Stormwater Pond could be estimated using Bernoulli’s equation for free flow through an opening as presented in Section 2.0. This assumes a prescriptive BADCT design is applied to the Stormwater Pond.

The following values were established to represent the variables of the equation.

- Dimensionless coefficient (CB): related to the shape of the edges of the hole (for sharp edges CB = 0.6);
- The hole area (a): A single two (2) mm diameter (a = 3.14 mm2) hole per acre allows for seam defects resulting from fabrication or installation factors that still may exist after intensive quality assurance (Giroud and Bonaparte, 1989); and
- Liquid depth on top of the geomembrane (hw): The maximum hydraulic head allowed by the design (5.18 meters) was used to estimate the PLR.

Table 3.3 presents the PLR for a prescriptive BADCT lined Stormwater Pond.
Table 3.3 PLR for a Prescriptive BADCT Stormwater Pond

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<th>$C_B$</th>
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<td>(m/s$^2$)</td>
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<td>$h_w$</td>
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<td>(m)</td>
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<tr>
<td>$Q$</td>
<td>1.90E-05</td>
<td>PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 1.90E-5$ m$^3$/s/defect. This can be converted to gpd per defect as follows:

$$Q = 1.90E-5 \text{ m}^3/\text{s/defect}$$

$$= \frac{1.90E-5 \text{ m}^3/\text{s/defect}}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{60 \text{ min}}{\text{day}} \times \frac{24 \text{ hr}}{\text{day}} = 433.66 \text{ gpd/defect}$$

To establish the TPL, the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Stormwater Pond has an LSA of 246,325 square feet (sf) or 5.65 acres.

$$TPL = \frac{433.66 \text{ gpd/defect}}{\text{defect}} \times 1\text{defect/acre} \times 5.65\text{acres} = 2,450.18 \text{ gpd}$$

Therefore, the TPL through a prescriptive BADCT lined Stormwater Pond is approximately 2,450 gpd.

3.4 Conclusions

As indicated in Sections 3.1 through 3.3, the minimum TPL through the Stormwater Pond would be:

- 155 gpd for a geomembrane/LPS composite liner system;
- 51 gpd for a geomembrane/GCL composite liner system; and
- 2,450 gpd for a prescriptive BADCT liner system for a non-stormwater pond.

As stated by Giroud and Bonaparte (1989), “It also appears that unitized leakage rates due to permeation through the geomembrane may not be negligible in the case of liquid impoundments; however, additional research is needed in this area before firm conclusions are drawn.” The permeation of fluid through the geomembrane is therefore not included in the calculated rates shown above.

Calculations were performed for the purpose of evaluating BADCT for different liner designs and to establish the degree of engineering control for each design. The proposed liner system for the Stormwater Pond is a geomembrane/GCL composite liner system. The BADCT comparison indicated that this liner system achieves a greater degree of engineering control when compared to that achieved by the prescriptive BADCT design for a non-stormwater pond.
4.0 BADCT Analysis for the Heap Leach Pad

This section presents calculations for the estimated TPL through the Heap Leach Pad Liner system. TPLs were calculated for two (2) liner systems.

- A composite liner system consisting of one (1) foot of LPS ($10^{-6}$ cm/sec material) and a geomembrane liner; and
- A composite liner system consisting of a GCL and a geomembrane liner.

Both systems were evaluated with a liner defect rate of one (1) hole per acre that is 11.3 mm in diameter. According to Giroud and Bonaparte (1989), a failure of the geomembrane due to accidental punctures may be represented by a single 11.3 mm diameter ($a = 100$ mm²) hole per acre.

The PLR through a Heap Leach Pad liner can be estimated using the Giroud's Equation (Giroud, 1997) as described in Section 2.0. The following values were established to represent the variables of the equation for the Heap Leach Pad liner leakage calculation.

- CQF (Cqo): Typically, a GCL/geomembrane interface has a better CQF than a soil/geomembrane interface. For these calculations, a CQF of 0.21 (the appropriate CQF associated with GCL) was selected for both liner systems. Typically, a GCL/geomembrane interface has a better CQF than a soil/geomembrane interface. However, a good CQF was used for both systems to provide a uniform comparison;

- Height of liquid on top of geomembrane (h): An average hydraulic head of two (2.0) feet (0.6096 meters) allowed on a Heap Leach Pad Liner by prescriptive BADCT was selected (ADEQ, 2004);

- Diameter of circular defect (d): A single 11.3 mm diameter hole per acre of liner was selected. This defect rate allows for damage incurred during placement of overliner materials or accidental punctures;

- Thickness of LPS or GCL (ts): The prescriptive BADCT standard of one (1) foot (0.3048 meters) of LPS was selected for the scenario presented in Table 4.1 (ADEQ, 2004). A standard GCL thickness of six (6) mm was selected for the GCL scenario presented in Table 3.02 (Cetco, 2009);

- Hydraulic conductivity of low permeability component (ks): The prescriptive BADCT permeability standard of 1x10^{-6} cm/s was selected for the LPS scenario presented in Table 4.1 (ADEQ, 2004). A standard GCL permeability of 5x10^{-9} cm/s was selected for the GCL scenario presented in Table 4.2 (Cetco, 2009); and

- LSA: The LSA for phases 1 and 2, including the area of the expanded phase 1 pad, is estimated to be 9,438,077 sf or 217 acres.

4.1 Potential Leakage through a Geomembrane/LPS lined Heap Leach Pad

Table 4.1 presents the PLR through a heap leach pad liner system consisting of a one (1) foot thick LPS layer and a geomembrane liner.
Table 4.1 PLR for the Heap Leach Pad (LPS and Geomembrane)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>LPS Values</th>
<th>Geomembrane Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{q_0}$</td>
<td>0.21</td>
<td>(dimensionless)</td>
</tr>
<tr>
<td>$h$</td>
<td>0.6096</td>
<td>(m)</td>
</tr>
<tr>
<td>$d$</td>
<td>0.0113</td>
<td>(m)</td>
</tr>
<tr>
<td>$t_s$</td>
<td>0.0304</td>
<td>(m)</td>
</tr>
<tr>
<td>$k_s$</td>
<td>1.0E-8</td>
<td>(m/s)</td>
</tr>
<tr>
<td>$Q$</td>
<td>7.68E-08</td>
<td>PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 7.68E-8$ m$^3$/s/defect. This can be converted to gpd per defect as follows:

$$\frac{7.68E - 8m^3}{s} \times \frac{264.17\text{gallons}}{m^3} \times \frac{60s}{\text{min}} \times \frac{60\text{min}}{\text{hr}} \times \frac{24\text{hr}}{\text{day}} = 1.75\text{gpd/defect}$$

To establish the TPL, the PLR is multiplied by the defect rate and the LSA of the leach pad in acres. A defect rate of one (1) hole per acre that is 11.3 mm in diameter was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Heap Leach Pad has an LSA of 9,438,077 sf or 217 acres.

$$TPL = \frac{1.75\text{gpd}}{\text{defect}} \times \frac{1\text{defect}}{\text{acre}} \times 217\text{acres} = 379.75\text{gpd}$$

Therefore, the TPL through the liner system of the Heap Leach Pad using LPS as the low permeability component is approximately 380 gpd.

4.2 Potential Leakage through a Geomembrane/GCL lined Heap Leach Pad

Table 4.2 presents the leakage through a heap leach pad liner system consisting of a GCL and a geomembrane liner.

Table 4.2 PLR for the Heap Leach Pad (GCL and Geomembrane)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>GCL Values</th>
<th>Geomembrane Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{q_0}$</td>
<td>0.21</td>
<td>(dimensionless)</td>
</tr>
<tr>
<td>$h$</td>
<td>0.6096</td>
<td>(m)</td>
</tr>
<tr>
<td>$d$</td>
<td>0.0113</td>
<td>(m)</td>
</tr>
<tr>
<td>$t_s$</td>
<td>0.006</td>
<td>(m)</td>
</tr>
<tr>
<td>$k_s$</td>
<td>5.0E-11</td>
<td>(m/s)</td>
</tr>
<tr>
<td>$Q$</td>
<td>1.16E-08</td>
<td>PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 1.16E-8$ m$^3$/s/defect. This can be converted to gpd per defect as follows:

$$\frac{1.16E - 8m^3}{s} \times \frac{264.17\text{gallons}}{m^3} \times \frac{60s}{\text{min}} \times \frac{60\text{min}}{\text{hr}} \times \frac{24\text{hr}}{\text{day}} = 0.26\text{gpd/defect}$$
To establish the TPL, the PLR is multiplied by the defect rate and the LSA of the leach pad in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Heap Leach Pad has an LSA of 9,438,077 sf or 217 acres.

\[
TPL = \frac{0.26\text{gpd}}{\text{defect per acre}} \times \frac{1\text{defect}}{\text{acre}} \times 217\text{acres} = 56.42\text{gpd}
\]

Therefore, the TPL through the liner system of the Heap Leach Pad using GCL as the low permeability component is approximately 56 gpd.

4.3 Conclusions
As indicated in Sections 4.1 through 4.2, the TPL through the Heap Leach Pad Liner system would be:

- 380 gpd for a geomembrane/LPS liner system; and
- 56 gpd for a geomembrane/GCL composite liner system.

These calculations were performed for the purpose of evaluating BADCT for different liner designs and to establish the degree of engineering control for each design. The proposed liner system for the Rosemont Heap Leach Pad is a composite liner consisting of a GCL and a geomembrane liner. Without having a GCL or LPS liner underneath the geomembrane, the TPL would be approximately 1,030,197 gpd.

5.0 Liner Leakage Analysis for the Raffinate and PLS Ponds
The following calculations were used to determine the TPL through the bottom liner of the Raffinate and PLS Ponds. The PLR was estimated using Giroud's Equation (Giroud, 1997) as presented in Section 2.0. The PLR is based on the assumption that the fluid on the bottom liner will be contained within the LCRS sump, and that the sump will contain one (1) defect that is two (2) mm in diameter. The Raffinate Pond and PLS Ponds have the same sump dimensions and a depth of 1.5 feet (0.4572 meters).

5.1 Potential Leakage through the Bottom Liner of the Raffinate and PLS Ponds
Table 5.1 presents the PLR through the bottom liner of the Raffinate or PLS Pond. The calculations used the same method to quantify the PLR through a geomembrane/GCL composite liner system as presented in Section 3.2 for a single-lined pond.
Table 5.1 PLR for the Bottom Liner of the Raffinate/PLS Pond

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{qo}$</td>
<td>0.21 (dimensionless)</td>
</tr>
<tr>
<td>$h$</td>
<td>0.4572 (m)</td>
</tr>
<tr>
<td>$d$</td>
<td>0.002 (m)</td>
</tr>
<tr>
<td>$t_o$</td>
<td>0.0060 (m)</td>
</tr>
<tr>
<td>$k_s$</td>
<td>5.0E-11 (m/sec)</td>
</tr>
<tr>
<td>$Q$</td>
<td>4.97E-09 PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 4.97E-9$ m$^3$/s/defect. This can be converted to gpd per defect as follows:

$$\frac{4.97E - 9 m^3}{s/\text{defect}} \times \frac{264.17 \text{gallons}}{m^3} \times \frac{60 \text{s}}{\text{min}} \times \frac{60 \text{min}}{\text{hr}} \times \frac{24 \text{hr}}{\text{day}} = \frac{0.113 \text{gpd}}{\text{defect}} \times 1 \text{defect} = 0.113 \text{gpd}$$

Therefore, the estimated TPL through the bottom liner of either the Raffinate or PLS Pond is approximately 0.113 gpd or 14.5 fluid ounces per day.

As indicated in Appendix P of the APP application (Tetra Tech, 2009), if the Raffinate Pond design utilized six (6) inches of LPS, the TPL would be approximately one (1) gallon per day. In order to quantify the total degree of engineering control achieved by the prescriptive BADCT design of the Raffinate and PLS Ponds, a TPL of 129 gpd was calculated for a liner system that did not include a GCL or LPS layer.

5.2 Leak Collection and Removal System

The Arizona Mining BADCT Guidance Manual (ADEQ, 2004) Section 2.3.2.5 recommends the following for a Process Solution Pond Leak Collection and Removal System (LCRS):

"The LCRS shall be designed to result in minimal hydraulic head on the lower liner and provide for the collection and removal of liquids from between the upper and lower liner. The LCRS shall consist of a layer of sand, gravel, geonet or other permeable material located between the two (2) geomembranes. Materials used as drainage media must achieve a flow capacity equivalent to a one (1) foot thick layer with a saturated hydraulic conductivity of $10^{-2}$ cm/sec or greater and three (3) percent slope, and must be chemically compatible with the solution stored in the pond. The LCRS must drain to a sump design to facilitate liquid extraction and leak monitoring. A three (3) percent minimum slope is required to promote drainage to a collection sump. ..."

In order to show that the minimum slope of the pond bottom meets prescriptive BADCT guidance, Tetra Tech calculated the flow capacity of an inner liner system consisting of the prescriptive BADCT criteria of a one (1) foot thick layer of $10^{-2}$ cm/sec material at a 3% slope and the selected geonet with a transmissivity of 9.66 gal/minute/foot at a 1% minimum slope.

The following calculations use Darcy’s velocity estimate for a confined aquifer with one (1) dimensional flow in a homogenous media (Schwartz / Zhang, 2003):

$$q = \frac{k * h_o - h_i}{\Delta L}$$

$$Q = H * B * q$$
Where:

\[ q = \text{Darcy flow velocity (cm/s)}; \]
\[ Q = \text{Darcy Flow (cm}^3/\text{s)}; \]
\[ k = \text{hydraulic conductivity (cm/s)}; \]
\[ h_o = \text{hydraulic head at the origin (cm)}; \]
\[ h_L = \text{hydraulic head at the length L (cm)}; \]
\[ L = \text{Length of the segment (cm)}; \]
\[ H = \text{Thickness of the material}; \text{ and} \]
\[ B = \text{Unit Width of one (1) cm}. \]

5.2.1 Prescriptive BADCT LCRS

The prescriptive BADCT guidance for an LCRS recommends a one (1) foot thick layer of material with a saturated hydraulic conductivity of $10^{-2}$ cm/sec or greater and three (3) percent slope. The calculations below represent a hypothetical pond bottom that is 100 feet long.

\[ k = 1 \times 10^{-2} \text{ cm/s}; \]
\[ h_o = \text{Three (3) feet} = 91.44 \text{ cm}; \]
\[ h_L = \text{Zero (0)}; \]
\[ L = \text{100 feet} = 3,048 \text{ cm}; \text{ and} \]
\[ H = \text{One (1) foot} = 30.48 \text{ cm}; \text{ and} \]
\[ B = \text{One (1) cm}. \]

Therefore:

\[ q = \frac{10^{-2} \times (91.44 - 0)}{3,048} = 3 \times 10^{-4} \text{ cm/s} \]
\[ Q = 3 \times 10^{-4} \times 30.48 \times 1 = 9.1 \times 10^{-3} \text{ cm}^3/\text{s} \]

The flow capacity of the prescriptive BADCT LCRS is $9.1 \times 10^{-3}$ cm$^3$/sec.

5.2.2 Proposed LCRS

The Technical Specifications associated with the final design of the Rosemont Heap Leach Facilities specify a geonet that is 60 mil thick with a transmissivity of 9.66 gallons per minute per foot. The proposed minimum slope for the pond bottom is 0.5%. The calculations below represent a hypothetical pond bottom that is 100 feet long.

\[ k = \frac{T}{H} \text{ where } T = \text{Transmissivity} = 9.66 \text{ gallons per minute per foot}; \]
\[ h_o = \text{Six (6) inches} = 15.24 \text{ cm}; \]
\[ h_L = \text{Zero (0)}; \]
L = 100 feet = 3,048 cm; and
H = 60 mil = 0.06 inches = 0.1524 cm.

In order to use Darcy’s law, the Transmissivity of the geonet must be converted to a hydraulic conductivity as follows:

\[ T = \frac{9.66 \text{gallon}}{\text{min} \times \text{foot}} \times \frac{3.785 \text{cm}^3}{\text{gallon}} \times \frac{1 \text{min}}{60 \text{s}} = \frac{609 \text{cm}^3}{\text{ft} \times \text{s}} \]

\[ k = \frac{T}{H} = \frac{609 \text{cm}^3}{\text{ft} \times \text{s}} \times \frac{1 \text{ft}}{30.48 \text{cm}} = \frac{19.98 \text{cm}^2}{\text{s}} / 0.1524 \text{cm} = \frac{131.1 \text{cm}}{\text{s}} \]

Therefore:

\[ q = \frac{131.1 \times (15.24 - 0)}{3,048} = \frac{3,995.9}{3,048} = 0.6555 \text{cm/s} \]

\[ Q = \frac{0.6555 \text{cm/s}}{s} \times 0.1524 \text{cm}(\text{height}) \times 1 \text{cm}(\text{width}) = 0.1 \text{cm}^3/\text{s} \]

The flow capacity of the proposed LCRS is 0.1 cm³/second at a 0.5% slope.

The flow capacity of the proposed LCRS is greater than that indicated in the prescriptive BADCT guidance. Therefore, the proposed LCRRS for the Rosemont PLS Pond and Raffinate Pond provides a greater degree of engineering control than the prescriptive BADCT LCRS system.

6.0 Raffinate Pond – Alert Level Leakage Rate Calculations

The purpose of the following analysis was to determine the PLR through the upper liner of the double-lined Raffinate Pond in order to propose Alert Level 1 (AL1) and Alert Level 2 (AL2).

The AL1 leakage rate is used to evaluate the liner performance in a process solution pond under typical operating conditions. The AL1, as measured by the amount of fluid pumped by the pond’s LCRS, is a low-level trigger that may indicate the presence of a small hole or defect in the top geomembrane of a double-lined, process solution pond.

The AL2 leakage rate, as measured by the amount of fluid pumped by the pond’s LCRS, is a high-level trigger that indicates a serious malfunction of the liner system.

The leakage rates were calculated using Bernoulli’s equation for free flow through an opening as previously described in Section 2.0. The calculations are dependent on the area of the hole (a), the maximum hydraulic head on the liner (hw), and the LSA of the pond.

For the Raffinate Pond, the maximum hydraulic head on the liner was determined using the total depth of the pond (26 feet) and subtracting the required three (3) feet of freeboard for a maximum hydraulic head of 23 feet (7.0 meters). The LSA of the pond was estimated to be 41,733 sf or 0.96 acres.
6.1 Calculation of the AL1 for the Raffinate Pond

Table 6.1 presents the parameters and calculation results for the PLR through the top liner of the Raffinate Pond.

Table 6.1 PLR Calculation for the Raffinate Pond (AL1)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_B )</td>
<td>0.6</td>
</tr>
<tr>
<td>( a )</td>
<td>3.14</td>
</tr>
<tr>
<td>( g )</td>
<td>9.81</td>
</tr>
<tr>
<td>( h_w )</td>
<td>7.0</td>
</tr>
<tr>
<td>( Q )</td>
<td>2.21E-05</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of \( Q = 2.21 \times 10^{-5} \text{ m}^3/\text{s/defect} \). This can be converted to gpd per defect as follows:

\[
\frac{2.21 \times 10^{-5} \text{ m}^3}{\text{s/defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ s}}{\text{min}} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{24 \text{ hr}}{\text{day}} = \frac{504.31 \text{ gpd}}{\text{defect}}
\]

To establish AL1, the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Raffinate Pond has an LSA of 41,733 sf or 0.96 acres.

\[
\text{AL1} = \frac{504.31 \text{ gpd}}{\text{defect}} \times \frac{1 \text{ defect}}{\text{acre}} \times 0.96 \text{ acre} = 484.1 \text{ gpd}
\]

Based on the assumptions presented herein, AL1 for the Raffinate Pond was calculated to be 484 gpd.

6.2 Calculation of AL2 for the Raffinate Pond

According to Giroud and Bonaparte (1989), a failure of the geomembrane due to poor design, or accidental punctures, may be represented by a single 11.3 mm diameter (\( a = 100 \text{ mm}^2 \)) hole per acre. Table 6.2 presents the parameters and calculation results for the PLR through the top liner of the Raffinate Pond.

Table 6.2 PLR Calculation for the Raffinate Pond (AL2)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_B )</td>
<td>0.6</td>
</tr>
<tr>
<td>( a )</td>
<td>100</td>
</tr>
<tr>
<td>( g )</td>
<td>9.81</td>
</tr>
<tr>
<td>( h_w )</td>
<td>7.0</td>
</tr>
<tr>
<td>( Q )</td>
<td>7.04E-04</td>
</tr>
</tbody>
</table>
The calculations yielded a PLR of \( Q = 7.04 \times 10^{-4} \text{ m}^3/\text{s/defect} \). This can be converted to gpd per defect as follows:

\[
\frac{7.04 \times 10^{-4} \text{ m}^3}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ s}}{\text{min}} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{24 \text{ hr}}{\text{day}} = \frac{16,060.9 \text{ gpd}}{\text{defect}}
\]

To establish \( AL_2 \), the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The Raffinate Pond has an LSA of 41,733 sf or 0.96 acres.

\[
AL_2 = \frac{16,060.9 \text{ gpd}}{\text{defect}} \times \frac{1}{\text{acre}} \times 0.96 \text{ acre} = 15,418.5 \text{ gpd}
\]

Based on the assumptions presented herein, \( AL_2 \) for the Raffinate Pond was calculated to be 15,418.5 gpd.

The system will be design with the pumping capacity to accommodate the \( AL_2 \) leakage rate of 15,418.5 gpd or 10.7 gpm. However, the Alert Level will be lowered to provide a factor of safety of 1.5 as follows:

\[
15,418.5 \div 1.5 = 10,279 \text{ gpd}
\]

Additionally, because the \( AL_2 \) level for the Raffinate Pond is very large, it is necessary to verify that the geonet drainage layer has sufficient capacity to transmit the flow to the sump.

As presented in Section 5.2.2, the specified geonet drainage layer for the Raffinate Pond has a Transmissivity of 9.66 gallons per minute per foot. This can be converted to gallons per day per foot as follows:

\[
\frac{9.66 \text{ gallons}}{\text{min/foot}} \times \frac{60 \text{ min}}{\text{hour}} \times \frac{24 \text{ hour}}{\text{day}} = 13,910.4 \text{ gpd/ft}
\]

The minimum bottom width of the Raffinate Pond is about 30 feet. Therefore, the geonet drainage layer will be able to transmit approximately 417,312 gpd and will accommodate the design flow of 15,418.5 gpd.

### 6.3 Conclusions

The ALs for the Raffinate Pond, as measured by the amount of fluid potentially pumped by the LCRS, were calculated to be:

- \( AL_1 = 484 \text{ gpd} \);
- \( AL_2 = 10,279 \text{ gpd} \) or 7.1 gpm; and
- Pumping Capacity = 15,418.5 gpd or 10.7 gpm.

If it is determined during normal operations that the amount of fluid pumped by the LCRS exceeds \( AL_1 \), Rosemont will take action to determine the cause. This action may include
physical inspection, mechanical leak detection, electric leak location, or other methods as appropriate.

If it is determined during normal operations that the amount of fluid pumped by the LCRS exceeds AL2, the contingency plan should be followed as described in Section 8.0 of the Rosemont Copper Project APP application (Tetra Tech, 2009a).

**7.0 PLS Pond – Alert Level Leakage Rate Calculations**

The purpose of the following analysis was to determine the PLR through the upper liner of the double-lined PLS Pond in order to propose AL1 and AL2 values.

PLRs are calculated using Bernoulli’s equation for free flow through an opening (as previously described in Section 2.0). As indicated in Section 5.0, the calculations are dependent on the area of the hole (a), the maximum hydraulic head on the liner (h_w), and the pond’s LSA.

For the PLS Pond, the maximum hydraulic head on the liner is defined as the vertical distance from the invert of the spillway to the bottom of the pond which is 17 feet (5.18 meters), and the pond’s LSA is 147,607 sf or 3.39 acres.

**7.1 Calculation of the AL1 for the PLS Pond**

Table 7.1 presents the parameters and calculation results for the PLR through the top liner of the PLS Pond.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_b</td>
<td>0.6</td>
</tr>
<tr>
<td>a</td>
<td>3.14</td>
</tr>
<tr>
<td>g</td>
<td>9.81</td>
</tr>
<tr>
<td>h_w</td>
<td>5.18</td>
</tr>
<tr>
<td>Q</td>
<td>1.90E-05 PLR (m³/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 2.06E-5$ m³/s/defect. This can be converted to gpd per defect as follows:

$$1.90E^{-5} \text{ m}^3/\text{s} \times 264.17 \text{ gallons/m}^3 \times 60 \text{s/min} \times 60 \text{ min/hr} \times 24 \text{ hr/day} = 433.7 \text{ gpd/defect}$$

To establish AL1, the PLR is multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte (1989). The PLS Pond has an LSA of 147,607 sf or 3.39 acres.

$$AL_1 = \frac{433.7 \text{ gpd/defect}}{\text{defect}} \times \frac{1 \text{ defect}}{\text{acre}} \times 3.39 \text{ acres} = 1,470.2 \text{ gpd}$$
Based on the assumptions presented herein, AL1 for the PLS Pond was calculated to be 1,470 gpd.

### 7.2 Calculation of AL2 for the PLS Pond

According to Giroud and Bonaparte (1989), a failure of the geomembrane due to poor design, or accidental punctures, may be represented by a single 11.3 mm diameter ($a = 100 \text{ mm}^2$) hole per acre. Table 7.2 presents the parameters and calculation results for the PLR through the top liner of the PLS Pond.

#### Table 7.2 PLR Calculation for the PLS Pond (AL2)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_B$</td>
<td>0.6</td>
</tr>
<tr>
<td>$a$</td>
<td>100</td>
</tr>
<tr>
<td>$g$</td>
<td>9.81</td>
</tr>
<tr>
<td>$h_w$</td>
<td>5.18</td>
</tr>
<tr>
<td>$Q$</td>
<td>6.05E-04 PLR ($\text{m}^3/\text{s}/\text{defect}$)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 6.05E-4 \text{ m}^3/\text{s}/\text{defect}$. This can be converted to gpd per defect as follows:

$$\frac{6.05E-4 \text{ m}^3/\text{s}}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ s}}{60 \text{ min}} \times \frac{60 \text{ min}}{1 \text{ hr}} \times \frac{24 \text{ hr}}{1 \text{ day}} = 13,808.7 \text{ gpd/defect}$$

To establish AL2, the PLR would be multiplied by the defect rate and the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate is based on empirical investigations published by J.P. Giroud and Bonaparte, (1989). The PLS Pond has a LSA of 147,607 sf or 3.39 acres.

$$AL2 = \frac{13,808.7 \text{ gpd/defect} \times 1 \text{ defect}}{\text{acre}} \times 3.39 \text{ acres} = 46,811.5 \text{ gpd}$$

Based on the assumptions presented herein, AL2 for the Raffinate Pond was calculated to be 46,812 gpd.

The system will be designed with the pumping capacity to accommodate the AL2 leakage rate of 46,812 gpd or 32.5 gpm. However, the Alert Level will be lowered to provide a factor of safety of 1.5 as follows:

$$46,812.5 \div 1.5 = 31,208.3 \text{ gpd}$$

Additionally, because the AL2 level for the Raffinate Pond is very large, it is necessary to verify that the geonet drainage layer has sufficient capacity to transmit the flow to the sump.

As presented in Section 5.2.2, the specified geonet drainage layer for the Raffinate Pond has a Transmissivity of 9.66 gallons per minute per foot. This can be converted to gallons per day per foot as follows:
The bottom width of the Raffinate Pond is about 198 feet. Therefore, the geonet drainage layer will be able to transmit approximately 2,754,259.2 gpd and will accommodate the AL2 flow of 46,812 gpd.

### 7.3 Conclusions

The ALs for the PLS Pond, as measured by the amount of fluid potentially pumped by the LCRS, were calculated to be:

- AL1 = 1,470 gpd;
- AL2 = 31,208 gpd or 21.7 gpm; and
- Pumping Capacity = 31,208 gpd or 32.5 gpm.

If it is determined during normal operations that the amount of fluid pumped by the LCRS exceeds AL1, Rosemont will take action to determine the cause. This action may include physical inspection, mechanical leak detection, electric leak location, or other methods as appropriate.

If it is determined during normal operations that the amount of fluid pumped by the LCRS exceeds AL2, the contingency plan should be followed as described in Section 8.0 of the Rosemont Copper Project APP application (Tetra Tech, 2009a).

### 8.0 Summary and Conclusions

Table 8.1 summarizes the calculated TPLs through the lined facilities associated with the Heap Leach Facility at the proposed Rosemont Copper Project.

<table>
<thead>
<tr>
<th>Facility</th>
<th>Total Potential Leakage (gallons per day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No LPS or GCL</td>
</tr>
<tr>
<td>Raffinate Pond</td>
<td>129</td>
</tr>
<tr>
<td>Heap Leach Pad</td>
<td>1,030,197</td>
</tr>
<tr>
<td>PLS Pond</td>
<td>129</td>
</tr>
<tr>
<td>Stormwater Pond</td>
<td>2,450</td>
</tr>
</tbody>
</table>

As shown in Table 8.1, GCL achieves a better degree of engineering control when compared to LPS. Also, LPS achieves a superior degree of engineering control when compared to liner systems without a low permeability layer. These calculations were performed for the purpose of
evaluating BADCT for different liner designs and to establish the degree of engineering control for each design.

To allow for added safety the pumping capacity will be sized to handle the calculated AL2 level flow but the alert level will be adjusted with a factor of safety of 1.5. This will result in the alert levels for the Raffinate and PLS Ponds as shown below:

The Alert Levels for the Raffinate Pond were calculated to be:

- AL1 = 480 gpd;
- AL2 = 10,182 gpd or 7.1 gpm; and
- Pumping Capacity = 15,272 gpd or 10.6 gpm.

The Alert Levels for the PLS Pond were calculated to be:

- AL1 = 1,470 gpd;
- AL2 = 31,208 gpd or 21.7 gpm; and
- Pumping Capacity = 31,208 gpd or 32.5 gpm.
REFERENCES


ATTACHMENT 1

TECHNICAL MEMORANDA

ROSEMONT RAFFINATE POND STABILITY ANALYSIS
Technical Memorandum

To: File
Cc: Joel Carrasco and Troy Meyer (Tetra Tech)
From: Jiny Carrera and Alyssa Kohlman
Project #: 114-320807-5.3
Subject: Rosemont Raffinate Pond Stability Analysis
Date: September 24, 2009

1.0 Introduction

This technical memorandum summarizes the results of a slope stability analysis performed on the embankment of the Raffinate Pond for the proposed Rosemont Copper Project (Project) in Pima County, Arizona. Information provided in this technical memorandum supersedes any previous stability or facility descriptions for the Raffinate Pond presented in the Leaching Facilities Design (Tetra Tech, 2007b), the Aquifer Protection Permit (APP) Application (Tetra Tech, 2009a), and the Rosemont Heap Leach Facility Permit Design Report (Tetra Tech, 2009c). The Raffinate Pond is located near the northeast corner of the Plant Site Area. The Raffinate Pond is proposed to be double-lined with a leak collection and removal system (LCRS) over a Geosynthetic Clay Liner (GCL). The Raffinate Pond will store raffinate from the SX-EW Plant before it is pumped to the Heap Leach Pad.

Only the most critical section (steepest, maximum height) of the Raffinate Pond embankment was analyzed. This section is shown on Figure 01 (Section A). Both block and global (rotational) failures were evaluated for a shallow, full height failure condition, and a loss of containment failure condition for the outside embankment slope ratio of 2H:1V. Attachment 1 contains the model outputs from the stability analysis.

The stability analysis was performed assuming that the channel (natural drainage) near the toe of the pond embankment (Figure 01) will be filled with compacted Willow Canyon material (Site Grading Fill). The stability figures in Attachment 1 show the channel-fill and pond embankment.

2.0 Construction of Model Cross Section

As shown on Figure 01, the Raffinate Pond is underlain by Arkose material from the Willow Canyon Formation on the northern and eastern edges and is underlain by older alluvium on the southern and western edges. The embankment is proposed to consist of locally excavated weathered bedrock from the Willow Canyon Formation. The critical section of the Raffinate Pond Embankment occurs at the southeast corner where the older alluvium is present (Figure 01). Due to the variable thickness of the older alluvium, the quantity to be removed for cut to fill construction is unknown. Based on testing results of the Willow Canyon in the Plant Site Area, the Willow Canyon is anticipated to have a lower strength value than the older alluvium. Therefore, the older alluvium was neglected in the stability analysis.
3.0 Design Criteria

Design of the Raffinate Pond embankment is governed by requirements of the Arizona Department of Environmental Quality (ADEQ) as detailed in the Arizona Mining BADCT Guidance Manual (ADEQ, 2004). Based on these requirements, the minimum stability criteria adopted for the Raffinate Pond embankment is presented in Table 3.01. Per BADCT requirements (ADEQ, 2004), site specific testing of material shear strength was performed. Additionally, a quality control testing program will be conducted during construction to determine grain size, plasticity index, moisture, density, etc., of the materials used to construct the Raffinate Pond embankment.

Table 3.01 Minimum Stability Requirements (with testing)

<table>
<thead>
<tr>
<th>Analysis Condition</th>
<th>Required Minimum Factor of Safety</th>
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<tr>
<td>Static</td>
<td>1.30</td>
</tr>
<tr>
<td>Pseudostatic</td>
<td>1.00</td>
</tr>
</tbody>
</table>

As documented in the Geologic Hazards Assessment (Tetra Tech, 2007a), the site seismicity was analyzed for two (2) levels of ground motion: the Maximum Probable Earthquake (MPE) and the Maximum Credible Earthquake (MCE). These values are 0.045g for the MPE and 0.326g for the MCE. In order to determine the appropriate design earthquake, the applicable rules pertaining to the Raffinate Pond embankment were reviewed.

In the unlikely event of a failure of the Raffinate Pond embankment, solutions would flow into the Plant Water/Temporary Storage (PWTS) Ponds. Additionally, the operational life of the facility is relatively short (less than 10 years). Any damage caused by the failure of the Raffinate Pond embankment would be limited to the Project site (i.e., property owned or controlled by the dam owner) and is unlikely to result in loss of human life. Therefore, the MPE was utilized for pseudostatic analyses. To allow for damping and attenuation of the bedrock acceleration within a slope or embankment, and to account for the rigid body pseudostatic model, the pseudo-static coefficient used in the model was a conservative estimate of horizontal ground motion equivalent to 2/3 of the MPE, or 0.03g.

4.0 Modeling Methods

The slope stability analysis was conducted using the Slope/W component of the GeoStudio 2007 software package produced by Geo-Slope International, Ltd. Slope/W was used to perform limiting equilibrium analyses using the general limit equilibrium (GLE) method, which satisfies both force and moment equilibrium. The Slope/W program incorporates a search routine to locate those failure surfaces with the least factor of safety within user defined search limits. Trial failure surfaces were defined with “entry and exit” and “block specified” slip surfaces, resulting in a range of possible locations to search for the most critical (lowest factor of safety) potential failure surface. Full height failure surfaces intersecting the crest of the embankment were considered.

Since the facility will be lined, the analysis was conducted assuming that steady-state seepage will not occur through the embankment. The phreatic surface used in the model was applied to
the foundation materials of the pond and was set just below the ground surface to account for the occasional seepage that is observed in the area. Pseudostatic analyses were conducted to evaluate the performance of the embankment under seismic conditions, using 2/3 of the design peak ground acceleration of the MPE, or 0.03g. The pseudostatic analyses subjects the two-dimensional sliding mass to a horizontal acceleration equal to an earthquake coefficient multiplied by the acceleration of gravity.

5.0 Material Properties

The material properties presented in Table 5.01 for the embankment and foundation materials were determined from field and laboratory testing (Tetra Tech, 2009b), experience with similar materials, and professional judgment. Direct shear testing was completed on young alluvium (which is expected to have equal or greater strength properties than the older alluvium) and the Willow Canyon Formation collected during drilling and test pit sampling. As explained in Section 2.0, the older alluvium was conservatively neglected in the stability analysis. Material properties for the in-situ Willow Canyon material were based on the direct shear testing results of the remolded Willow Canyon material. This is appropriate since the in-situ material is expected to have similar strength properties to the remolded material.

<table>
<thead>
<tr>
<th>Material</th>
<th>Strength Model</th>
<th>Phi (degrees)</th>
<th>Cohesion (psf)</th>
<th>Unit Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Willow Canyon Formation</td>
<td>Mohr-Coulomb</td>
<td>34</td>
<td>0</td>
<td>118</td>
</tr>
<tr>
<td>(Arkose) (in situ and remolded)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.0 Results

Both static and pseudostatic factors of safety against a full-height failure of the Raffinate Pond embankment were found to be adequate. Shallow failures were analyzed as were deep seated failures with block and circular failure modes. The shallow failures represent the minimum factors of safety; however, failures of this type can be easily repaired and are unlikely to cause significant damage or loss of fluid from the Raffinate Pond. Deep seated failures have higher factors of safety and represent a failure that is likely to release fluid from the Raffinate Pond. The results of the analyses are provided in Attachment 1 to this memo and summarized in Table 6.01.

<table>
<thead>
<tr>
<th>Case</th>
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<tr>
<td></td>
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<tr>
<td>Shallow, Full-Height Failure – Circular</td>
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<tr>
<td>Shallow, Full-Height Failure – Block</td>
<td>1.69</td>
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<tr>
<td>Loss of Containment – Circular</td>
<td>1.67</td>
</tr>
<tr>
<td>Loss of Containment – Block</td>
<td>2.39</td>
</tr>
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</table>
7.0 References


ATTACHMENT 1
ROSEMONT FINAL DESIGN
RAFFINATE POND
Static - Circular
Horz Seismic Value: 0

Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1
Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1

ROSEMONT FINAL DESIGN
RAFFINATE POND
Static - Block
Horz Seismic Value: 0

Distance (ft) x 1000

Elevation (ft) x 1000
ROSEMONT FINAL DESIGN
RAFFINATE POND
Pseudostatic - Block
Horz Seismic Value: 0.03

Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1

Distance (ft)

Elevation (ft) (x 1000)

Willow Canyon Formation
Remolded Willow Canyon
Remolded Willow Canyon (Site Grading Fill)
Existing Ground
ROSEMONT FINAL DESIGN
RAFFINATE POND
Static - Circular, Loss of containment
Horz Seismic Value: 0

Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1
ROSEMONT FINAL DESIGN
RAFFINATE POND
Static, Loss of Containment
Horz Seismic Value: 0

Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1

Willow Canyon Formation
Remolded Willow Canyon
(Site Grading Fill)
Existing Ground
Name: Willow Canyon Fm in-situ
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Remolded Willow Canyon Fm
Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °
Phi-B: 0 °
Piezometric Line: 1

ROSEMONT FINAL DESIGN
RAFFINATE POND
Pseudostatic - Block
Horz Seismic Value: 0.03

Elevation (ft) (x 1000)
Distance (ft)
ATTACHMENT J
TECHNICAL MEMORANDUM
ROSEMONT RAFFINATE POND VOLUME REQUIREMENT
Technical Memorandum

To: Joel Carrasco
From: Ronson Chee
Company: Tetra Tech
Date: August 3, 2010
Re: Rosemont Raffinate Pond Volume Requirement
Doc #: 206/10-320877-5.3
CC: David R. Krizek, P.E. (Tetra Tech)

1.0 Introduction
This Technical Memorandum provides an updated volume requirement for the planned Raffinate Pond at the Rosemont Copper Project (Project) in Pima County, Arizona. This updated pond volume requirement is compared to the pond volume requirement submitted in the Aquifer Protection Permit (APP) application provided to the Arizona Department of Environmental Quality (ADEQ) in February 2009 (Tetra Tech, 2009a). This information is in response to the April 14, 2010 Comprehensive Request for Additional Information from ADEQ to Rosemont Copper Company (Rosemont). Specifically, this Technical Memorandum answers item no. 28 on page 12 of 18.

- **APP Volume 1, Table 7.16 – Raffinate Pond Volume Requirements:** There is a discrepancy in the Raffinate Pond Volume Requirements and the Total Volume Required. Please reconcile the volumes (Minimum Pool Volume, Design Operating Volume and Freeboard Volume) to reflect the correct volume of raffinate required in the Raffinate Pond.

2.0 Raffinate Pond Volume – APP Application
The capacity and storage design of the Raffinate Pond is described in Section 7.5.8 of the February 2009 APP application. Pond volume requirements are summarized in Table 7.16 on the APP application. A direct duplication of Section 7.5.8 from the February 2009 APP application is provided below:

**7.5.8 Capacity and Storage Design (Original)**
The Raffinate Pond will be sized to provide lined storage for the equivalent of four (4) hours of operational flows assuming maximum values for precipitation, evaporation, and ore moisture. In addition, the pond will be sized to contain a minimum pool depth of ten (10) feet with three (3) feet of freeboard. The dimensions and volume requirements for the Raffinate Pond are presented in Tables 7.15 and 7.16, respectively.
Table 7.15  Raffinate Pond Dimensions

<table>
<thead>
<tr>
<th>Pond Base Width (feet)</th>
<th>Pond Base Length (feet)</th>
<th>Pond Top Width (feet)</th>
<th>Pond Top Length (feet)</th>
<th>Total Pond Depth (feet)</th>
<th>Pond Side Slopes (H:V)</th>
<th>Total Pond Volume (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.0</td>
<td>175.0</td>
<td>180.5</td>
<td>255.5</td>
<td>16.1</td>
<td>2.5:1</td>
<td>495,000</td>
</tr>
</tbody>
</table>

Table 7.16  Raffinate Pond Volume Requirements

<table>
<thead>
<tr>
<th>Minimum Pool Depth (feet)</th>
<th>Minimum Pool Volume (ft³)</th>
<th>Design Operating Volume (ft³)</th>
<th>3 Feet Freeboard Volume (ft³)</th>
<th>Total Volume Required (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0</td>
<td>251,843</td>
<td>99,594</td>
<td>128,803</td>
<td>495,000</td>
</tr>
</tbody>
</table>

Table 7.15 provided “theoretical dimensions” based on water balance calculations. These dimensions are the minimum dimensions and pond volume necessary to satisfy the water balance calculations. Due to the complex nature of satisfying water balance requirements, along with actual design considerations, a range of suitable Raffinate Pond dimensions was created as a guide to designing the actual pond configuration. As long as the actual design dimensions fell within the suitable range of “theoretical dimensions”, the pond geometry was deemed adequate.

The “Total Volume Required” entry in Table 7.16 does not equate to the summation of the volume requirements. The “Total Volume Required” entry in Table 7.16 should have been labeled “Sum of Required Volumes” and be equal 479,240 cubic feet (ft³) (251,843 ft³ + 98,594 ft³ + 128,803 ft³). The “Total Volume Required” value of 495,000 ft³ listed in Table 7.15 is the volume of the actual Raffinate Pond design assumed in the APP application.

The summation of the volume requirements in Table 7.16 is 479,240 ft³, which is less than the total pond volume of 495,000 ft³ provided by the assumed design. This indicates an adequate pond design.

3.0  Raffinate Pond Volume – Updated

Since the submittal of the APP application, the Raffinate Pond has been through a series of changes. The changes included: an increase to eight (8) hours of operational flows versus four (4) hours and a change in location and geometry. These changes were assumed in the Rosemont Heap Leach Facility Permit Design Report dated May 2009 and provided to ADEQ (Tetra Tech, 2009b). In addition to the updated pond dimensions, the table entries were modified herein to clarify the volume requirements. Information for the updated tables was taken from the Technical Memorandum titled Rosemont - Water Balance and Pond Sizing dated April 17, 2009. This Technical Memorandum, which is included herein as Attachment 1, was
previously provided in the *Rosemont Heap Leach Facility Permit Design Report* dated May 2009. Based on this April 17, 2009 memo, the following should replace Section 7.5.8 in the APP application.

### 7.5.8 Capacity and Storage Design (Updated)

The Raffinate Pond will be sized to provide lined storage for the equivalent of eight (8) hours of operational flows assuming maximum values for precipitation, evaporation, and ore moisture. In addition, the pond will be sized to contain a minimum pool depth of ten (10) feet with three (3) feet of freeboard. The dimensions and volume requirements for the Raffinate Pond are presented in Tables 7.15 and 7.16, respectively.

#### Table 7.15  Raffinate Pond Dimensions

<table>
<thead>
<tr>
<th></th>
<th>Pond Base Width (feet)</th>
<th>Pond Base Length (feet)</th>
<th>Pond Top Width (feet)</th>
<th>Pond Top Length (feet)</th>
<th>Total Pond Depth (feet)</th>
<th>Pond Side Slopes (H:V)</th>
<th>Total Pond Volume (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Requirements (Theoretical)</td>
<td>55</td>
<td>96.3</td>
<td>166.9</td>
<td>208.1</td>
<td>22.4</td>
<td>2.5:1</td>
<td>399,730</td>
</tr>
<tr>
<td>Actual</td>
<td>50</td>
<td>115</td>
<td>165</td>
<td>230</td>
<td>23</td>
<td>2.5:1</td>
<td>452,305</td>
</tr>
</tbody>
</table>

#### Table 7.16  Raffinate Pond Volume Requirements

<table>
<thead>
<tr>
<th>Minimum Pool Volume</th>
<th>Design Operating Volume</th>
<th>3 Feet Freeboard Volume</th>
<th>Sum of Required Volumes</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft³)</td>
<td>(ft³)</td>
<td>(ft³)</td>
<td>(ft³)</td>
</tr>
<tr>
<td>98,887</td>
<td>194,435</td>
<td>95,992</td>
<td>389,314</td>
</tr>
</tbody>
</table>

In Table 7.15, the “Design Requirements (Theoretical)” entries are from the water balance calculations provided in Attachment 1. The “Actual” entries are the final design layout of the Raffinate Pond as shown in the *Rosemont Heap Leach Facility Permit Design Report* dated May 2009. Table 7.16 shows a change in the column entry from “Total Volume Required” to “Sum of Required Volumes” in comparison to the original Table 7.16 headings. Additionally, the first two (2) columns in the original Table 7.16 were combined, i.e., the column “Minimum Pool Depth” was deleted and the column heading “Minimum Pool Volume” was also changed to “10 Feet Minimum Pool Volume”. The “Sum of Required Volumes” entry in Table 7.16 (389,314 ft³) differs from the “Total Pond Volume” (399,730 ft³) in Table 7.15 based on the theoretical design.
requirements due to the iterative nature of the water balance calculation and iteration tolerance - and therefore the difficulty in matching the two (2) numbers exactly.
REFERENCES


1.0 Introduction

This Technical Memorandum presents methodologies and procedures for sizing of the various ponds proposed by Rosemont Copper for construction as part of the Rosemont Copper Project heap leach facility, which entails:

- Pregnant Leach Solution (PLS) Pond;
- Raffinate Solution Pond; and
- Stormwater Pond (for runoff from the Heap Leach Pad).

Once the ponds are sized, estimate the monthly fresh water make-up requirements of the Rosemont heap leach facility.

2.0 Method

Sizing of ponds involved in a heap leach water cycle follows a four-step process:

1. Create a schematic diagram of the heap leach water cycle focusing on inflows to the system, outflows from the system, and portions of the system where water storage exists.

2. Identify the subset of the water cycle on the schematic diagram that must be computed (using the principle of conservation of mass) in order to determine all normal inflows to the pond that is being sized.

3. Estimate maximum inflow rates to the pond that is being sized (in an average year) then compute design volumes by multiplying the aforementioned inflow rates by prescribed design durations.

4. Assume a base width and then compute the dimensions of the pond by stacking the design volumes and computing the height associated with each “slice” subject to the geometric constraints of the pond (side slopes, length to width ratio, etc.)
Water balance calculations to validate the pond sizing and estimate the monthly fresh water make-up requirements of the heap leach water cycle are based on the principle of conservation of mass, which may be expressed in its most basic form as:

$$\Delta S = (\Sigma I - \Sigma O) \times \Delta t$$

Where:

- $\Delta S$ = Change in system volume;
- $\Sigma I$ = Sum of inflows to system;
- $\Sigma O$ = Sum of outflows from system; and
- $\Delta t$ = Elapsed time.

For storage structures within the Rosemont heap leach water balance model, normal operations may entail changes in the volume of water stored at the end of a simulated time increment (months). The estimated pond sizes are validated if their respective capacities are not exceeded during simulated operations in an average year.

At the system-wide level, the Rosemont heap leach water balance must operate under the criteria that $\Delta S = 0$. At the Rosemont site, local climatic conditions dictate that additional fresh make-up water has to be added to the system in given months, i.e., the Rosemont heap leach water balance cycle is a net evaporative system.

3.0 Assumptions

Leach ore properties
- Maximum Leach ore production rate = 38,000 tons per day (tpd) (Actual varies by year);
- Leach ore “as-mined” moisture content = 2.0%
- Leach ore field capacity = 7.0%
- Leach ore field capacity for make-up water = 10.0%

Raffinate solution application
- Raffinate application method = drip emitters
- Raffinate application evaporative loss rate = 3.0% (of the raffinate application rate)
- Target PLS flow rate to PLS Pond = 2,500 gallons per minute (gpm)

Heap Leach Pad design
- Active leach surface = 1,000,000 square feet (ft²)
- Phase 1 lined area = 5,476,769 ft²
- Phase 2 lined area = 4,511,933 ft²
- Ultimate lined area = 9,988,702 ft²
Pond design

- PLS Pond is sized for 8 hours of operational flow plus 24 hours of heap drain-down flow with an additional 3 feet (ft) of dry freeboard
- PLS drain-down flow rates are assumed equivalent to the estimated PLS Pond operational flow rates
- Raffinate Solution Pond is sized for 8 hours of operational flow with an additional 3 ft of dry freeboard
- Stormwater Pond is sized for the 100 year (yr), 24 hour precipitation event, which equals 4.75 inches (in), over the lined heap leach pad area with an additional 3 ft of dry freeboard. In addition, the Stormwater Pond has been sized to allow for a one hour shutdown in the leaching operation during the peak of the 100 year, 24 hour storm event. Six possible construction configurations of the heap leach were investigated for pond sizing, all having differing amounts of liner and/or ore. They are as follows:
  - Case 1: Assuming Phase 1 of the Heap Leach Pad is lined, but ore placement has not begun.
  - Case 2: Assuming both Phases of the Heap Leach Pad are lined and the Phase 1 pad contains 8.6MT of ore (end of year 1 estimated total).
  - Case 3: Assuming both Phases of the Heap Leach Pad are lined, but ore placement has not begun.
  - Case 4: Assuming both Phases of the Heap Leach Pad are lined and the Phase 1 pad contains 29.3 MT of ore (end of year 2 estimated total).
  - Case 5: Assuming both Phases of the Heap Leach Pad are lined, the Phase 1 pad contains 29.3 MT of ore (end of year 2 estimated total), and the Phase 2 pad contains 28.6 MT of ore (end of year 2 estimated total).
  - Case 6: Assuming both Phases of the Heap Leach Pad are lined, the Phase 1 pad contains 40 MT of ore (end of year estimated total), and the Phase 2 pad contains 35 MT of ore (end of year estimated total).
- Process pond (PLS and Raffinate) minimum operating depth = 10 ft
- Pond side slopes (interior) = 2.5 horizontal to 1 vertical (2.5:1)
- Pond shapes are frustums of inverted rectangular pyramids

Climate

- Pond surface evaporation coefficient = 0.70 (of Pan) as given in *Handbook of Applied Hydrology* (Chow, 1964)
- Wetted leach surface evaporation coefficient = 0.85 (of Pan) to account for the additional area exposed on the surface of the heap leach pad
- Average year precipitation values were taken from Santa Rita Experimental Range, Arizona: Average Total Precipitation (inches) (WRCC, 2008)
Wet and dry year precipitation values were established by averaging the rainfall during the twenty wettest years and the twenty driest years, respectively. The raw precipitation data was obtained from *Nogales 6N, Arizona: Monthly Total Precipitation (inches)* (WRCC, 2008).

Pan evaporation values were taken from Technical Memo, Rosemont Copper Project Design Storm and Precipitation Data/Design Criteria (Tetra Tech, April 2009).

4.0 Calculations

A schematic diagram of the Rosemont heap leach balance cycles is shown on Figure 1.

PLS Pond sizing focused on the subset of the Rosemont heap leach water balance cycle centered on the PLS Pond. As shown in Figure 1, normal inflow rates to the PLS Pond include the pregnant leach solution flow from the heap leach pad \( G_{hl} \) and direct precipitation on the PLS Pond \( P_{pp} \). Details of the PLS Pond sizing are given in Attachment 1, with associated water balance formulas taken from Figure 1. (Note that since the normal inflow rate \( G_{hl} \) is specified, average precipitation and evaporation data are not required for sizing the PLS Pond.)

Sizing of the Stormwater Pond was performed using three parameters: the lined area of the heap leach pad, design storm runoff from the heap leach pad, and direct precipitation on the Stormwater Pond during the design storm event. Details of the Stormwater Pond sizing are given in Attachment 2.

Raffinate Pond sizing focused on the subset of the Rosemont heap leach water balance cycle centered on inflows and outflows from the Raffinate Pond. Referring to Figure 1, the barren raffinate solution outflow rate to the heap leach pad \( B_{rp} \) can’t exceed the sum of the barren raffinate solution inflow rate \( B_{sx} \), direct precipitation on the Raffinate Pond \( P_{rp} \), and the fresh water make-up rate \( F_{fs} \). Accordingly, the normal inflow rate to the heap leach pad was taken as \( B_{rp} \). Details of the Raffinate Pond sizing are given in Attachment 3, with associated water balance formulas taken from Figure 1. (Note that for this calculation, the greatest required solution application rate is in the month of June at 3,253 gallons per minute from the dry year calculations.)

One year of Rosemont heap leach water cycle operations was simulated under average, dry, and wet year precipitation conditions using a water balance model based on the schematic and water balance equations shown on Figure 1. This model, detailed in Attachment 4a-d, was used to verify pond sizing and to determine monthly fresh water make-up required at the site. Similarly, Case 2 of the aforementioned heap leach configurations was used in the ultimate Stormwater Pond sizing because it produced the highest probable volume and it provided the closest approximation to the one year heap arrangement.

5.0 Conclusions/Results

For the sizing of each pond, various geometric alternatives were computed in order to provide a range of possible dimensions (see Table 2 for pond configurations and results). These results are detailed in Table 2 and are summarized below:
PLS Pond – depending on the selected geometric configuration (in Table 2), the PLS Pond should have an ultimate capacity between 44 and 60 acre-feet.

Stormwater Pond – depending on the selected geometric configuration, the Stormwater Pond should have an ultimate capacity between 84 and 97 acre-feet.

Raffinate Pond – depending on the selected geometric configuration, the Raffinate Pond should have an ultimate capacity between 8 and 14 acre-feet.

The Rosemont heap leach water balance cycle was simulated with average, wet, and dry year climate values (see Climate Data Table 1) and pond configurations as indicated in Table 2 to determine the fresh water make-up requirements. The heap leach water balance results are detailed in Table 3. Key results are summarized below:

- As was mentioned in the Section 2.0, the Rosemont heap leach water balance cycle is a net evaporative system. The water balance results agree with this assertion as fresh water make-up is predicted year-round for all scenarios investigated.

- The results show that the consumptive maximum fresh water requirements for the Rosemont Copper Project are 753 gpm during the driest 20 years.

6.0 References


TABLES
Table 1 - Average Climate Data

<table>
<thead>
<tr>
<th>Month No.</th>
<th>Operating Month</th>
<th>Avg. Year Precipitation¹ (in/mo)</th>
<th>Dry Year Precipitation² (in/mo)</th>
<th>Wet Year Precipitation² (in/mo)</th>
<th>Pan Evaporation³ (in/mo)</th>
<th>Pond Evaporation⁴ (in/mo)</th>
<th>Active W.S. Evaporation⁵ (in/mo)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>2</td>
<td>Feb</td>
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<td>0.14</td>
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<td>3</td>
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<td>10</td>
<td>Oct</td>
<td>1.62</td>
<td>0.08</td>
<td>3.16</td>
<td>6.15</td>
<td>4.31</td>
<td>5.23</td>
</tr>
<tr>
<td>11</td>
<td>Nov</td>
<td>1.15</td>
<td>0.18</td>
<td>1.30</td>
<td>4.11</td>
<td>2.88</td>
<td>3.49</td>
</tr>
<tr>
<td>12</td>
<td>Dec</td>
<td>1.96</td>
<td>0.13</td>
<td>3.21</td>
<td>3.89</td>
<td>2.72</td>
<td>3.31</td>
</tr>
</tbody>
</table>

| Yearly Total | 22.17 | 5.45 | 32.09 | 71.52 | 50.06 | 60.79 |

¹ Precipitation values were taken from Santa Rita Experimental Range, Arizona: Average Total Precipitation (inches) (WRCC, 2008).
² Precipitation values calculated from Nogales 6N, Arizona: Monthly Total Precipitation (inches) (WRCC, 2008).
³ Pan evaporation values were taken from Technical Memo, Rosemont Copper Project Design Storm and Precipitation Data/Design Criteria (Tetra Tech, Apr 2009).
⁴ Pond evaporation values were computed from pan evaporation values using an assumed pan evaporation coefficient of 0.70 as given in Handbook of Applied Hydrology (Chow, 1964).
⁵ Wetted surface evaporation values were computed from pan evaporation values using an assumed pan evaporation coefficient of 0.85 to account for the additional area exposed on the wetted surface of the heap leach pad.
Table 2 - Pond Sizing Results

<table>
<thead>
<tr>
<th>Selected Pond</th>
<th>Base Width (ft)</th>
<th>Base Length (ft)</th>
<th>Top Width (ft)</th>
<th>Top Length (ft)</th>
<th>Total Depth (ft)</th>
<th>Ultimate Capacity (acre-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PLS Pond</strong></td>
<td>200.0</td>
<td>350.0</td>
<td>296.2</td>
<td>446.2</td>
<td>19.2</td>
<td>43.9</td>
</tr>
<tr>
<td></td>
<td>215.0</td>
<td>376.3</td>
<td>308.2</td>
<td>469.5</td>
<td>18.6</td>
<td>47.6</td>
</tr>
<tr>
<td></td>
<td>230.0</td>
<td>402.5</td>
<td>320.7</td>
<td>493.2</td>
<td>18.1</td>
<td>51.6</td>
</tr>
<tr>
<td></td>
<td>245.0</td>
<td>428.8</td>
<td>333.4</td>
<td>517.2</td>
<td>17.7</td>
<td>55.8</td>
</tr>
<tr>
<td></td>
<td>260.0</td>
<td>455.0</td>
<td>346.5</td>
<td>541.5</td>
<td>17.3</td>
<td>60.2</td>
</tr>
<tr>
<td><strong>Raffinate Solution Pond</strong></td>
<td>40.0</td>
<td>70.0</td>
<td>162.2</td>
<td>192.2</td>
<td>24.4</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>55.0</td>
<td>96.3</td>
<td>166.9</td>
<td>208.1</td>
<td>22.4</td>
<td>9.2</td>
</tr>
<tr>
<td></td>
<td>70.0</td>
<td>122.5</td>
<td>173.7</td>
<td>226.2</td>
<td>20.7</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td>85.0</td>
<td>148.8</td>
<td>182.2</td>
<td>246.0</td>
<td>19.4</td>
<td>12.1</td>
</tr>
<tr>
<td></td>
<td>100.0</td>
<td>175.0</td>
<td>192.1</td>
<td>267.1</td>
<td>18.4</td>
<td>13.9</td>
</tr>
<tr>
<td><strong>Stormwater Pond</strong></td>
<td>360.0</td>
<td>450.0</td>
<td>459.0</td>
<td>499.0</td>
<td>19.8</td>
<td>84.0</td>
</tr>
<tr>
<td></td>
<td>400.0</td>
<td>450.0</td>
<td>485.9</td>
<td>535.9</td>
<td>17.2</td>
<td>86.4</td>
</tr>
<tr>
<td></td>
<td>450.0</td>
<td>500.0</td>
<td>524.4</td>
<td>574.4</td>
<td>14.9</td>
<td>89.5</td>
</tr>
<tr>
<td></td>
<td>500.0</td>
<td>550.0</td>
<td>565.3</td>
<td>615.3</td>
<td>13.1</td>
<td>93.1</td>
</tr>
<tr>
<td></td>
<td>550.0</td>
<td>600.0</td>
<td>608.0</td>
<td>658.0</td>
<td>11.6</td>
<td>97.1</td>
</tr>
</tbody>
</table>
Table 3 - Summary Year Water Balance Results

<table>
<thead>
<tr>
<th>Make-Up Water Value Description</th>
<th>Avg. Year Make-Up Water (gpm)</th>
<th>Dry Year Make-Up Water (gpm)</th>
<th>Wet Year Make-Up Water (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>566</td>
<td>597</td>
<td>538</td>
</tr>
<tr>
<td>Maximum</td>
<td>744</td>
<td>753</td>
<td>737</td>
</tr>
</tbody>
</table>
CLIENT: Rosemont Copper Co.
PROJECT: Heap Leach Facility
JOB NO: 320807
SUBJECT: Water Balance & Pond Sizing
BY: EMS
DETAILS: Heap Leach Process Schematic 
DATE: 4/17/2009

Note: overflows from PLS Pond go to Storm Water Pond (not shown) and are pumped back within 72 hours.

Design Parameters:
Ore production = 38,000 tpd
Phase 1 lined pad area = 4,370,161 ft²
Phase 2 lined pad area = 4,511,933 ft²
Ultimate lined pad area = 8,882,094 ft²
PLS flow rate (Ghl) = 2,500 gpm

Legend:
- Pumped Flow
- Environmental Flow
- Barren Solution Flow from SX/EW Plant to Raffinate Pond
- Evaporation from Heap Leach
- Evaporation from PLS Pond
- Fixed water flow from Fresh Water Source to Raffinate Pond
- Pregnant Solution Flow from Heap Leach to PLS Pond
- Pumped Solution Flow from PLS Pond to SX/EW Plant

Figure 1 - Water Balance Schematic
ATTACHMENT 1
PLS POND SIZING
In order to size the PLS Pond, the normal operations volume and the drain-down volume (in the event of a spill) must be determined.

In order to determine these values the pregnant solution flow from the Heap Leach Pad must be estimated as shown below:

<table>
<thead>
<tr>
<th>Flow Rate</th>
<th>Evaporation</th>
<th>Drain-Down</th>
<th>Total</th>
<th>Drain-Down</th>
</tr>
</thead>
<tbody>
<tr>
<td>(gpm)</td>
<td>(in/hr)</td>
<td>(ft³)</td>
<td>(ft³)</td>
<td>(ft³)</td>
</tr>
<tr>
<td>2300</td>
<td>0.70</td>
<td>160,417</td>
<td>641,987</td>
<td></td>
</tr>
</tbody>
</table>

Since the PLS flow rate is known, the normal operations and the drain-down volumes can be calculated directly without having to consider environmental effects.

Now, estimate the PLS Pond size based on given design assumptions and Arizona BADC guidelines:

An iterative solution process was used, with assumptions in yellow cells and calculated comparison results in green cells.

Based on the results of this calculation, the PLS Pond can range in size from roughly 44 to 60 acre-ft, depending on the desired depth and plan area used.
ATTACHMENT 2
STORMWATER POND SIZING
### Model Assumptions

<table>
<thead>
<tr>
<th>Field Capacity of ore</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture of ore from Mine</td>
<td>2%</td>
</tr>
<tr>
<td>Tons Placed</td>
<td>38,000 t/d</td>
</tr>
<tr>
<td>Process Flow</td>
<td>2,500 gpm</td>
</tr>
<tr>
<td>Permibility</td>
<td>0.00328 ft/sec</td>
</tr>
<tr>
<td>Total Heap Slope Area</td>
<td>300 ft by 0 sq ft</td>
</tr>
<tr>
<td>Runoff from slopes</td>
<td>25%</td>
</tr>
<tr>
<td>Plant Flow</td>
<td>2,500 gpm</td>
</tr>
</tbody>
</table>

### Rainfall Details

- **CLIENT:** Rosemont Copper Co
- **PROJECT:** Heap Leach Facility
- **JOB NO:** 320807

### Change in Pond Volume

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Pond Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>20</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>30</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>40</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>50</td>
<td>400,000 cu ft</td>
</tr>
</tbody>
</table>

### Runoff from slopes

- **Percent:** 25%
- **Distance Above:** 200 ft
- **Velocity:** 0.00328 ft/sec

### Volume

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0 cu ft</td>
</tr>
<tr>
<td>10</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>20</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>30</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>40</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>50</td>
<td>400,000 cu ft</td>
</tr>
</tbody>
</table>

### Pump to Heap

- **Number of Days:** 5
- **Average Distance Above:** 200 ft
- **Average Velocity:** 0.00328 ft/sec

### Cumulative Total Volume

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Cumulative Total Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0 cu ft</td>
</tr>
<tr>
<td>10</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>20</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>30</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>40</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>50</td>
<td>400,000 cu ft</td>
</tr>
</tbody>
</table>

### Total Inflow

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Total Inflow</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0 cu ft</td>
</tr>
<tr>
<td>10</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>20</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>30</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>40</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>50</td>
<td>400,000 cu ft</td>
</tr>
</tbody>
</table>

*Note: All volumes in cubic feet.*
Model Assumptions

- Value
  - Evaporation: 3%
  - Field Capacity of ore: 7%
  - Moisture of ore from Mine: 2%
  - Tons Placed: 38,000 t/d
  - Permeability: 0.00328 ft/sec
  - Total Lined Area: 8,590,509 sq ft
  - Runoff from slopes: 25%
  - Plant Flow: 2,500 gpm

<table>
<thead>
<tr>
<th>Hour</th>
<th>Ore Depth</th>
<th>% of Rainfall</th>
<th>Time (hrs)</th>
<th>Inflow Rate (cfs)</th>
<th>Pond Volume (cu ft/hr)</th>
<th>Total Inflow (cu ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0.00000</td>
<td></td>
<td>-20,052</td>
<td>-3,138</td>
<td>-3,138</td>
</tr>
<tr>
<td>1</td>
<td>100.0%</td>
<td>0.00000</td>
<td>1.00</td>
<td>2,910,886</td>
<td>2,763</td>
<td>2,763</td>
</tr>
<tr>
<td>2</td>
<td>97.8%</td>
<td>0.00085</td>
<td>2.00</td>
<td>2,950,544</td>
<td>2,637</td>
<td>2,637</td>
</tr>
<tr>
<td>3</td>
<td>95.1%</td>
<td>0.00296</td>
<td>3.00</td>
<td>2,929,291</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>4</td>
<td>94.0%</td>
<td>0.00381</td>
<td>4.00</td>
<td>2,940,708</td>
<td>2,505</td>
<td>2,505</td>
</tr>
<tr>
<td>5</td>
<td>93.2%</td>
<td>0.00494</td>
<td>5.00</td>
<td>2,975,089</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>6</td>
<td>92.5%</td>
<td>0.00587</td>
<td>6.00</td>
<td>3,004,906</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>7</td>
<td>91.8%</td>
<td>0.00680</td>
<td>7.00</td>
<td>3,035,623</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>8</td>
<td>91.9%</td>
<td>0.00773</td>
<td>8.00</td>
<td>3,066,339</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>9</td>
<td>91.9%</td>
<td>0.00866</td>
<td>9.00</td>
<td>3,097,056</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>10</td>
<td>91.9%</td>
<td>0.00959</td>
<td>10.00</td>
<td>3,127,772</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>11</td>
<td>91.9%</td>
<td>0.01052</td>
<td>11.00</td>
<td>3,158,489</td>
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<td>2,536</td>
</tr>
<tr>
<td>12</td>
<td>91.9%</td>
<td>0.01145</td>
<td>12.00</td>
<td>3,189,205</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
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<td>91.9%</td>
<td>0.01238</td>
<td>13.00</td>
<td>3,219,922</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>14</td>
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<td>0.01331</td>
<td>14.00</td>
<td>3,250,639</td>
<td>2,536</td>
<td>2,536</td>
</tr>
<tr>
<td>15</td>
<td>91.9%</td>
<td>0.01424</td>
<td>15.00</td>
<td>3,281,356</td>
<td>2,536</td>
<td>2,536</td>
</tr>
</tbody>
</table>

*Note: Days to consume water by ore wetting alone.*

17 days
### Model Assumptions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture of ore from Mine</td>
<td>2%</td>
</tr>
<tr>
<td>Permibility</td>
<td>0.00328 ft/sec</td>
</tr>
<tr>
<td>Total Heap Slope Area</td>
<td>3,468,618 sq ft</td>
</tr>
<tr>
<td>Shutdown Duration</td>
<td>1 hr</td>
</tr>
</tbody>
</table>

### Hour Ore Depth

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Ore Depth</th>
<th>Wetting</th>
<th>Evaporation</th>
<th>Rainfall</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>99.0%</td>
<td>0.00211 ft</td>
<td>48,363 cu ft</td>
</tr>
<tr>
<td>50</td>
<td>20,052 cu ft/hr</td>
<td>97.8%</td>
<td>0.00296 ft</td>
<td>72,579 cu ft</td>
</tr>
<tr>
<td>30</td>
<td>20,052 cu ft/hr</td>
<td>97.0%</td>
<td>0.00381 ft</td>
<td>94,562 cu ft</td>
</tr>
<tr>
<td>25</td>
<td>20,052 cu ft/hr</td>
<td>96.0%</td>
<td>0.00381 ft</td>
<td>69,269 cu ft</td>
</tr>
<tr>
<td>20</td>
<td>20,052 cu ft/hr</td>
<td>93.8%</td>
<td>0.00507 ft</td>
<td>88,811 cu ft</td>
</tr>
<tr>
<td>15</td>
<td>20,052 cu ft/hr</td>
<td>92.5%</td>
<td>0.00677 ft</td>
<td>116,873 cu ft</td>
</tr>
<tr>
<td>10</td>
<td>20,052 cu ft/hr</td>
<td>90.0%</td>
<td>0.00904 ft</td>
<td>20,052 cu ft</td>
</tr>
<tr>
<td>5</td>
<td>20,052 cu ft/hr</td>
<td>87.5%</td>
<td>0.01245 ft</td>
<td>10,340 cu ft</td>
</tr>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>85.0%</td>
<td>0.01945 ft</td>
<td>164,105 cu ft</td>
</tr>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>83.1%</td>
<td>0.01945 ft</td>
<td>164,105 cu ft</td>
</tr>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>82.0%</td>
<td>0.02191 ft</td>
<td>164,105 cu ft</td>
</tr>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>81.0%</td>
<td>0.02436 ft</td>
<td>164,105 cu ft</td>
</tr>
<tr>
<td>0</td>
<td>20,052 cu ft/hr</td>
<td>80.0%</td>
<td>0.02681 ft</td>
<td>164,105 cu ft</td>
</tr>
</tbody>
</table>

### Volume Details

<table>
<thead>
<tr>
<th>Time (h)</th>
<th>Total Inflow</th>
<th>Pond Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>200,000 cu ft</td>
<td>200,000 cu ft</td>
</tr>
<tr>
<td>2</td>
<td>400,000 cu ft</td>
<td>400,000 cu ft</td>
</tr>
<tr>
<td>3</td>
<td>600,000 cu ft</td>
<td>600,000 cu ft</td>
</tr>
<tr>
<td>4</td>
<td>800,000 cu ft</td>
<td>800,000 cu ft</td>
</tr>
<tr>
<td>5</td>
<td>1,000,000 cu ft</td>
<td>1,000,000 cu ft</td>
</tr>
<tr>
<td>6</td>
<td>1,200,000 cu ft</td>
<td>1,200,000 cu ft</td>
</tr>
<tr>
<td>7</td>
<td>1,400,000 cu ft</td>
<td>1,400,000 cu ft</td>
</tr>
<tr>
<td>8</td>
<td>1,600,000 cu ft</td>
<td>1,600,000 cu ft</td>
</tr>
<tr>
<td>9</td>
<td>1,800,000 cu ft</td>
<td>1,800,000 cu ft</td>
</tr>
<tr>
<td>10</td>
<td>2,000,000 cu ft</td>
<td>2,000,000 cu ft</td>
</tr>
<tr>
<td>11</td>
<td>2,200,000 cu ft</td>
<td>2,200,000 cu ft</td>
</tr>
<tr>
<td>12</td>
<td>2,400,000 cu ft</td>
<td>2,400,000 cu ft</td>
</tr>
<tr>
<td>13</td>
<td>2,600,000 cu ft</td>
<td>2,600,000 cu ft</td>
</tr>
<tr>
<td>14</td>
<td>2,800,000 cu ft</td>
<td>2,800,000 cu ft</td>
</tr>
<tr>
<td>15</td>
<td>3,000,000 cu ft</td>
<td>3,000,000 cu ft</td>
</tr>
<tr>
<td>16</td>
<td>3,200,000 cu ft</td>
<td>3,200,000 cu ft</td>
</tr>
<tr>
<td>17</td>
<td>3,400,000 cu ft</td>
<td>3,400,000 cu ft</td>
</tr>
</tbody>
</table>

### Diagrams

- **Total Inflow**
  - Graph showing inflow over time from 0 to 60 hours.
- **Pond Volume**
  - Graph showing pond volume over time from 0 to 60 hours.

---

*Note: Percentages indicate the wetting and evaporation percentage of the rainfall amount.*
<table>
<thead>
<tr>
<th>Event Size</th>
<th>Total Heap Slope Area</th>
<th>Total Lined Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.75 in</td>
<td>6,440,739 sq ft</td>
<td>8,590,509 sq ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Model Assumptions</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evaporation</td>
<td>3%</td>
</tr>
<tr>
<td>Moisture of ore from Mine</td>
<td>2%</td>
</tr>
</tbody>
</table>

### Water Balance & Pond Sizing

**BY:** EMS

**CLIENT:** Rosemont Copper Co

<table>
<thead>
<tr>
<th>Leach</th>
<th>LinerArea</th>
<th>Time to Travel</th>
<th>Volume</th>
<th>Ore Wetting</th>
<th>Evaporation</th>
<th>Pond Volume</th>
<th>Process Flow</th>
<th>Cumulative Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2,149,770 sq ft</td>
<td>27 days</td>
<td>Number of days to consume water by ore wetting alone</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Graphs

1. **Total Inflow**
   - Graph shows inflow over time.

2. **Pond Volume**
   - Graph shows volume increase over time.
### Process Flow
- **2,500 gpm**

### Permibility
- **0.00328 ft/sec**
- **300 ft**
- **445,225 sq ft**
- **25.00 hr**

### Total Heap Slope Area
- **7,048,617 sq ft**
- **240 ft**
- **1,819,421 sq ft**
- **20.00 hr**

### Total Lined Area
- **8,590,509 sq ft**
- **180 ft**
- **1,893,254 sq ft**
- **15.00 hr**

### Shutdown Duration
- **1 hr**
- **0 ft**
- **1,541,892 sq ft**
- **0.00 hr**

---

### Hourly Ore Depth

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Ore Depth</th>
<th>Volume (cu ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>-20,052</td>
<td>-3,138</td>
</tr>
<tr>
<td>59</td>
<td>-20,052</td>
<td>2,008,801</td>
</tr>
<tr>
<td>58</td>
<td>-20,052</td>
<td>2,031,991</td>
</tr>
<tr>
<td>57</td>
<td>-20,052</td>
<td>2,055,180</td>
</tr>
<tr>
<td>52</td>
<td>-20,052</td>
<td>2,171,130</td>
</tr>
<tr>
<td>50</td>
<td>-20,052</td>
<td>2,217,510</td>
</tr>
<tr>
<td>46</td>
<td>-20,052</td>
<td>2,309,987</td>
</tr>
<tr>
<td>44</td>
<td>-20,052</td>
<td>2,355,378</td>
</tr>
<tr>
<td>38</td>
<td>2,596</td>
<td>2,479,519</td>
</tr>
<tr>
<td>37</td>
<td>4,362</td>
<td>2,496,975</td>
</tr>
<tr>
<td>35</td>
<td>9,012</td>
<td>2,527,201</td>
</tr>
<tr>
<td>31</td>
<td>25,202</td>
<td>2,555,463</td>
</tr>
<tr>
<td>30</td>
<td>33,963</td>
<td>2,550,313</td>
</tr>
<tr>
<td>29</td>
<td>36,867</td>
<td>2,536,403</td>
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<tr>
<td>28</td>
<td>48,163</td>
<td>2,519,588</td>
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<tr>
<td>26</td>
<td>84,804</td>
<td>2,453,365</td>
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<tr>
<td>23</td>
<td>100.0%</td>
<td>0.00000</td>
</tr>
<tr>
<td>20</td>
<td>99.6%</td>
<td>0.00211</td>
</tr>
<tr>
<td>19</td>
<td>99.0%</td>
<td>0.00211</td>
</tr>
<tr>
<td>18</td>
<td>98.5%</td>
<td>0.00296</td>
</tr>
<tr>
<td>17</td>
<td>97.8%</td>
<td>0.00296</td>
</tr>
<tr>
<td>12</td>
<td>92.5%</td>
<td>0.00677</td>
</tr>
<tr>
<td>11</td>
<td>90.8%</td>
<td>0.00677</td>
</tr>
<tr>
<td>9</td>
<td>83.1%</td>
<td>0.01945</td>
</tr>
<tr>
<td>7</td>
<td>78.2%</td>
<td>0.01945</td>
</tr>
<tr>
<td>2</td>
<td>31.8%</td>
<td>0.06301</td>
</tr>
<tr>
<td>8</td>
<td>86.1%</td>
<td>0.01184</td>
</tr>
<tr>
<td>9</td>
<td>86.1%</td>
<td>0.01184</td>
</tr>
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</table>

---

### Diagrams
- **Total Inflow**
- **Pond Volume**
Determine the design storm runoff volume to the Phase 1+2 Stormwater Pond and estimate the Phase 1+2 Stormwater Pond size based on given design assumptions and Arizona BADCt guidelines:

<table>
<thead>
<tr>
<th>Side Slopes</th>
<th>Minimum Depth</th>
<th>Design Storm Depth</th>
<th>Design Storm Duration</th>
<th>Dry Freeboard</th>
<th>Lined Area</th>
<th>Ro. Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>z (H:1V)</td>
<td>(ft)</td>
<td>(in)</td>
<td>(hr)</td>
<td>(ft)</td>
<td>(ft²)</td>
<td>(ft³)</td>
</tr>
<tr>
<td>2.5</td>
<td>0.0</td>
<td>4.75</td>
<td>24.0</td>
<td>3.0</td>
<td>8,590,509</td>
<td>2,908,134</td>
</tr>
</tbody>
</table>

Case 2 was used to calculate the amount of lined area and runoff volume

\[
A_0 = W_o \times L_o \\
A_z = (W_o + 2zd) \times (L_o + 2zd) \\
V_o = \frac{1}{3} d \times (A_o + (A_o A_z)^{1/2} + A_z)
\]

An iterative solution process was used, with assumptions in yellow cells and calculated comparison results in green cells. (Tools → Goal Seek)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>360.0</td>
<td>400.0</td>
<td>144,000</td>
</tr>
<tr>
<td>400.0</td>
<td>450.0</td>
<td>180,000</td>
</tr>
<tr>
<td>450.0</td>
<td>500.0</td>
<td>225,000</td>
</tr>
<tr>
<td>500.0</td>
<td>550.0</td>
<td>275,000</td>
</tr>
<tr>
<td>550.0</td>
<td>600.0</td>
<td>330,000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design (Direct) Precipitation Event - Calculation</th>
<th>Dry Freeboard - Calc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Width Wo (ft)</td>
<td>Assumed Direct Depth d Ace (in)</td>
</tr>
<tr>
<td>360.0</td>
<td>4.77</td>
</tr>
<tr>
<td>400.0</td>
<td>4.77</td>
</tr>
<tr>
<td>450.0</td>
<td>4.77</td>
</tr>
<tr>
<td>500.0</td>
<td>4.77</td>
</tr>
<tr>
<td>550.0</td>
<td>4.77</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stormwater Pond Top - Calculation Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Width Wo (ft)</td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>360.0</td>
</tr>
<tr>
<td>400.0</td>
</tr>
<tr>
<td>450.0</td>
</tr>
<tr>
<td>500.0</td>
</tr>
<tr>
<td>550.0</td>
</tr>
</tbody>
</table>

Based on the results of this calculation, the Stormwater Pond can range in size from roughly 84 to 97 acre-ft, depending on the desired depth and area used.
ATTACHMENT 3
RAFFINATE POND SIZING
In order to size the Raffinate Pond, the normal operations volume must be determined. This estimation was made using the total operational outflows from the Raffinate Pond, as shown below:

### Raffinate Pond Operating Flow Estimate (Part 1 - Design Criteria)

<table>
<thead>
<tr>
<th>PLS Flow Rate</th>
<th>Application Method</th>
<th>Soln. App. Rate (%)</th>
<th>Active Plan Area (ft²)</th>
<th>W.S. Evaporation Coefficient</th>
<th>Ore App. Rate (ton/day)</th>
<th>Ore Moisture Content (%)</th>
<th>Ore Field Capacity (%)</th>
<th>Operations Duration (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,500 gpm</td>
<td>Dripper</td>
<td>70.0</td>
<td>17,500</td>
<td>251,843</td>
<td>5.0</td>
<td>43,810</td>
<td>194,435</td>
<td>25.5</td>
</tr>
</tbody>
</table>

Based on the results of this calculation, the Raffinate Pond can range in size from roughly 5 to 11 acre-ft, depending on the desired depth and plan area used.

### Raffinate Pond Operating Flow Estimate (Part 2 - Average Year Climate Input)

#### Operating Months
<table>
<thead>
<tr>
<th>Month</th>
<th>Days in Month</th>
<th>Avg. Precipitation (in/mo)</th>
<th>Pan Evaporation (in/mo)</th>
<th>W.S. Evaporation (in/mo)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>31</td>
<td>1.63</td>
<td>4.13</td>
<td>3.98</td>
</tr>
<tr>
<td>Feb</td>
<td>28</td>
<td>1.68</td>
<td>4.28</td>
<td>3.98</td>
</tr>
<tr>
<td>Mar</td>
<td>31</td>
<td>0.89</td>
<td>6.10</td>
<td>3.98</td>
</tr>
<tr>
<td>Apr</td>
<td>31</td>
<td>0.62</td>
<td>10.75</td>
<td>3.98</td>
</tr>
<tr>
<td>May</td>
<td>31</td>
<td>5.37</td>
<td>7.83</td>
<td>3.98</td>
</tr>
<tr>
<td>Jun</td>
<td>31</td>
<td>4.32</td>
<td>6.68</td>
<td>3.98</td>
</tr>
<tr>
<td>Jul</td>
<td>30</td>
<td>2.15</td>
<td>4.40</td>
<td>3.98</td>
</tr>
<tr>
<td>Aug</td>
<td>30</td>
<td>1.15</td>
<td>4.11</td>
<td>3.98</td>
</tr>
<tr>
<td>Sep</td>
<td>31</td>
<td>1.76</td>
<td>3.89</td>
<td>3.98</td>
</tr>
</tbody>
</table>

Now, estimate the Raffinate Pond size based on given design assumptions and Arizona BADCT guidelines for June, which requires the greatest volume:

### Raffinate Pond Operating Flow Estimate (Part 3 - Calculation)

#### Raffinate Pond Base - Calculation

- **Base Area Width** $W_b$ = $W_o \times L_o$
- **Dry Freeboard** $d_b$ = $(W_o + 2zd) \times (L_o + 2zd)$
- **Design Storm Depth** $d_v$ = $V_o / (\Delta s \times \Delta b)$

#### Design Precipitation Event - Calculation

- **Runoff Vol.** $V_o$ = $A_o \times \Delta b$
- **Storm Vol.** $V_s$ = $A_o \times \Delta s$
- **Error** $V_s - V_o$

#### Raffinate Pond Base - Calculation

<table>
<thead>
<tr>
<th>Base Width ($W_b$ ft)</th>
<th>Assumed Runoff Area ($A_o$ ft²)</th>
<th>Calculated Runoff Area ($A_o$ ft²)</th>
<th>Calculated Runoff Vol. ($V_o$ ft³)</th>
<th>Calculated Storm Vol. ($V_s$ ft³)</th>
<th>Error ($V_s - V_o$) ft³</th>
<th>Dry Area Volume ($V_d$ ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40.0</td>
<td>48.1</td>
<td>26,096</td>
<td>29,282</td>
<td>0</td>
<td>3,186</td>
<td>104,461</td>
</tr>
<tr>
<td>70.0</td>
<td>48.1</td>
<td>26,096</td>
<td>29,282</td>
<td>0</td>
<td>3,186</td>
<td>104,461</td>
</tr>
<tr>
<td>100.0</td>
<td>48.1</td>
<td>26,096</td>
<td>29,282</td>
<td>0</td>
<td>3,186</td>
<td>104,461</td>
</tr>
</tbody>
</table>

### Raffinate Pond Top - Calculation Results

<table>
<thead>
<tr>
<th>Base Width ($W_t$ ft)</th>
<th>Top Width ($W_t$ ft)</th>
<th>Total Depth ($L_t$ ft)</th>
<th>Top Length ($L_a$ ft)</th>
<th>Area ($A_t$ ft²)</th>
<th>Total Volume ($V_t$ acre-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40.0</td>
<td>24.4</td>
<td>162.7</td>
<td>192.2</td>
<td>31,182</td>
<td>0.7</td>
</tr>
<tr>
<td>70.0</td>
<td>24.4</td>
<td>162.7</td>
<td>192.2</td>
<td>31,182</td>
<td>0.7</td>
</tr>
<tr>
<td>100.0</td>
<td>24.4</td>
<td>162.7</td>
<td>192.2</td>
<td>31,182</td>
<td>0.7</td>
</tr>
</tbody>
</table>

*Used for fresh water make-up est.*

Based on the results of this calculation, the Raffinate Pond can range in size from roughly 5 to 11 acre-ft, depending on the desired depth and plan area used.
ATTACHMENT 4
HEAP LEACH WATER BALANCE CALCULATIONS
### Raffinate Parameter Input

<table>
<thead>
<tr>
<th>PLS Flow Rate (gpm)</th>
<th>Application Method</th>
<th>Soln. App. Loss Rate (%)</th>
<th>Active Area (ft²)</th>
<th>Ore App. Rate (ton/d)</th>
<th>Ore Moisture Content (%)</th>
<th>Ore Field Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,500</td>
<td>Dripper</td>
<td>3.0%</td>
<td>1,000,000</td>
<td>38,000</td>
<td>2.0%</td>
<td>10.0%</td>
</tr>
</tbody>
</table>

### Ore Production Input

<table>
<thead>
<tr>
<th>PLS Pond Input</th>
<th>Raffinate Pond Input</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Width (ft)</td>
<td>215.0</td>
</tr>
<tr>
<td>Pond Area (ft²)</td>
<td>144,711</td>
</tr>
<tr>
<td>Initial Pond Volume (ft³)</td>
<td>0</td>
</tr>
<tr>
<td>Ultimate Capacity (ft³)</td>
<td>70.0</td>
</tr>
<tr>
<td>Initial Pond Volume (ft³)</td>
<td>0</td>
</tr>
</tbody>
</table>
## Water Balance Calculations - Average Year Precipitation

<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>Days/Month</th>
<th>Precipitation on Heap Area (ft³/mo)</th>
<th>Incoming Ore Moisture (ft³/mo)</th>
<th>Ore Moisture Retention (ft³/mo)</th>
<th>PLS Flow to PLS Pond (ft³/mo)</th>
<th>Raffinate Flow (ft³/mo)</th>
<th>Evaporation from Heap Area (ft³/mo)</th>
<th>Evaporation from PLS Pond (ft³/mo)</th>
<th>Evaporation from PLS Pond (ft³/mo)</th>
<th>Pregnant PLS Pond Solution Flow (ft³/mo)</th>
<th>PLS Pond Water Volume (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Jan</td>
<td>31</td>
<td>135,833</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
<td>14,918,749</td>
<td>19,657</td>
<td>34,863</td>
<td>14,903,542</td>
<td>14,903,542</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Feb</td>
<td>28</td>
<td>121,667</td>
<td>682,051</td>
<td>3,410,256</td>
<td>13,474,999</td>
<td>13,474,999</td>
<td>17,606</td>
<td>36,129</td>
<td>13,456,476</td>
<td>13,456,476</td>
<td>0</td>
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<tr>
<td>1</td>
<td>Mar</td>
<td>31</td>
<td>121,667</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
<td>14,918,749</td>
<td>19,657</td>
<td>34,863</td>
<td>14,903,542</td>
<td>14,903,542</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Apr</td>
<td>30</td>
<td>57,600</td>
<td>730,769</td>
<td>3,653,846</td>
<td>14,437,499</td>
<td>14,437,499</td>
<td>17,606</td>
<td>60,019</td>
<td>14,876,337</td>
<td>14,876,337</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>May</td>
<td>31</td>
<td>20,000</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
<td>14,918,749</td>
<td>19,657</td>
<td>34,863</td>
<td>14,903,542</td>
<td>14,903,542</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Jun</td>
<td>30</td>
<td>51,667</td>
<td>730,769</td>
<td>3,653,846</td>
<td>14,437,499</td>
<td>14,437,499</td>
<td>17,606</td>
<td>67,222</td>
<td>14,834,021</td>
<td>14,834,021</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Jul</td>
<td>31</td>
<td>405,833</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
<td>14,918,749</td>
<td>19,657</td>
<td>74,778</td>
<td>14,354,230</td>
<td>14,354,230</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Aug</td>
<td>31</td>
<td>360,000</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
<td>14,918,749</td>
<td>19,657</td>
<td>87,222</td>
<td>14,946,449</td>
<td>14,946,449</td>
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<tr>
<td>1</td>
<td>Sep</td>
<td>30</td>
<td>179,187</td>
<td>730,769</td>
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<td>14,437,499</td>
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<td>135,000</td>
<td>755,128</td>
<td>3,775,641</td>
<td>14,918,749</td>
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<td>95,833</td>
<td>730,769</td>
<td>3,653,846</td>
<td>14,437,499</td>
<td>14,437,499</td>
<td>17,606</td>
<td>74,778</td>
<td>14,416,673</td>
<td>14,416,673</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>Dec</td>
<td>31</td>
<td>163,333</td>
<td>755,128</td>
<td>3,775,641</td>
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## SX/EW Plant

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## Water Balance & Pond Sizing

**Client:** Rosemont Copper Co.  
**Project:** Heap Leach Facility  
**Subject:** Water Balance & Pond Sizing  
**Details:** Attachment 4c: Water Balance Calculations - Dry Year Precipitation

### Heap Leach

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### SX/EW Plant

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### Water Balance & Pond Sizing

#### Details: Attachment 4d: Water Balance Calculations - Wet Year Precipitation

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### Raffinate Pond: Ult. Capacity = 2,074,753 ft³

### SX/EW Plant: Ult. Capacity = 457,639 ft³