This technical memorandum supersedes any previous liner leakage estimates or facility descriptions for the PWTS Pond presented in the Site Water Management Plan (Tetra Tech, 2007b), the Mine Plan of Operations (WestLand Resources, Inc., 2007), and the Aquifer Protection Permit (APP) application (Tetra Tech, 2009a).

The proposed PWTS Ponds (previously named the PWTS Pond) will be located southeast of the Plant Site as shown on Figure 01. The facility will be divided into two (2) cells (ponds) identified as the Process Water (PW) Pond and the Temporary Storage (TS) Pond.

The PW Pond will receive:
- Recovered water from Tailings Thickeners;
- Recovered water from the Tailings Filter Plant;
- Overflow from the Settling Basin;
- Fresh water make-up;
- Accumulated groundwater and stormwater from the Open Pit; and
- Stormwater run-off from the Plant Site.

The TS Pond will collect stormwater run-off from a small drainage basin and overflows from the PW Pond. Water collected in the PW Pond will be pumped to the Reclaimed Water Tanks to be distributed by gravity pipeline for use in the process circuit. Water collected in the TS Pond will be pumped to the PW Pond and recycled into the process circuit.

The PWTS Ponds have been designed to meet or exceed the Best Available Demonstrated Control Technology (BADCT). The PW Pond was designed to meet prescriptive BADCT standards for a process solution pond as described in the Arizona Department of Environmental Quality (ADEQ) Arizona Mining BADCT Guidance Manual (ADEQ, 2004). The design of the TS Pond meets or exceeds the criteria established for a non-stormwater pond. Both these surface impoundments are categorical APP facilities pursuant to A.R.S. §49-241(B)(1).

During the first four (4) years of operations, the PWTS embankment, which forms the southern side of the PWTS Ponds Facility, will be a free standing structure. In Year 4, construction of the
Dry Stack Tailings Facilities will reach a height equal to the top of the embankment, thereby buttressing the structure. After Year 4, the elevation of the Dry Stack Tailings Facility will continue to rise, thus preventing the possibility of water overtopping from the PWTS Ponds Facility. The PW Pond will be managed to optimize the containment of process water, and the TS Pond will be kept at low fill levels or dry. Temporary or permanent pumping systems in each pond will pump process water and stormwater to the Reclaimed Water Tanks for distribution into the process circuit as needed. The pumping systems will allow each pond to be emptied for inspection.
1.0 PWTS Ponds Leak Detection System

The PWTS Ponds are divided into two (2) cells, the PW Pond and the TS Pond. The design and function of each pond is different; thus the leak detection systems for each pond were evaluated separately. Per its function, the PW Pond is a process solution pond and the TS Pond is a non-stormwater pond for the evaluation of prescriptive BADCT criteria. The Arizona Mining BADCT Guidance Manual (ADEQ, 2004) provides the following definition:

“Non-Storm Water Ponds include lined ponds that receive seepage from tailing impoundment(s), waste dump(s), and/or process areas where potential pollutant constituents in the seepage have concentrations that are relatively low (e.g., compared to process solutions) but exceed Arizona Surface Water Quality Standards. Non-Storm Water Ponds also include secondary containment structures and overflow ponds that contain process solution for short periods of time due to process upsets or rainfall events.”

1.1 PW Pond

The PW Pond was designed to meet prescriptive BADCT standards for a process solution pond.

PW Pond – Liner Design

The PW Pond liner system will consist of the following (from bottom to top):

- A minimum six (6) inch thick layer of properly compacted bedding soil (prepared subgrade);
- A sodium bentonite Geosynthetic Clay Liner (GCL);
- A 60 mil High Density Polyethylene (HDPE) secondary (bottom) geomembrane liner;
- A geonet or equivalent Leak Collection and Removal System (LCRS); and
- A 60 mil HDPE primary (top) geomembrane liner.

A cross section of the PW Pond liner system is presented in Illustration 1.01.
A LCRS will consist of a geonet or equivalent layer between the primary and secondary liners to allow any leaks through the primary liner to flow to the monitoring sump. The sump will be equipped with an automatic, fluid-level activated pump. The pump will be sized to remove fluids such that the head on the secondary liner is minimized.

The bottom of the PW Pond will be graded to the sump at a minimum 3.0% slope to facilitate gravity collection.

**PW Pond – Alert Level Calculations**

The purpose of the following analysis was to evaluate the Potential Leakage Rate (PLR) through the primary liner of the proposed PW Pond and to determine Alert Levels (ALs).

The Alert Level 1 (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 Alert Level 2 (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 top liner of the liner system.

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 is used to calculate the ALs for double-lined process solution ponds.

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

Where:
- \( Q \) = rate of liquid migration or PLR through a geomembrane hole (cubic meters per second \([m^3/s]\));
- \( C_B \) = Dimensionless coefficient related to the shape of the edges of the hole (for sharp edges \( C_B = 0.6 \));
- \( a \) = Hole area \([square meters (m^2)]\);
- \( g \) = Acceleration due to gravity \([meters per second per second (m/s^2)]\); and
- \( h_w \) = Liquid depth on top of the geomembrane \([meters (m)]\).

The PLR calculation results are expressed as cubic meters per second per defect \((m^3/s/defect)\). This value is then converted to gallons per day \((gpd)\) per defect, multiplied by the number of defects per acre, and multiplied by the lined surface area \((LSA)\) of the pond in acres.

**Calculation of AL1 for the PW Pond**

The maximum hydraulic head \((h_w)\) on the liner will be 56 feet \((17.1 \text{ meters})\), and the PW Pond’s LSA will be 535,729 square feet \((sf)\) or 12.3 acres. Table 1.01 presents the parameters and
calculation results for the PLR through a single hole in the primary liner of the PW Pond that is two (2) millimeters (mm) in diameter \( [a = 3.14 \text{ square millimeters (mm}^2 \text{)]}.\)

| Table 1.01  PLR for the PW Pond (AL1) |
|----------------|---------------------------------|
| \( C_B \) =   | 0.6 (dimensionless)             |
| \( a \) =     | 3.14 (mm\(^2\))                |
| \( g \) =     | 9.81 (m/s\(^2\))               |
| \( h_w \) =   | 17.1 (m)                        |
| \( Q \) =     | 3.45E-5 PLR (m\(^3\)/s/defect) |

The calculations yielded a PLR of \( Q = 3.45E-5 \text{ m}^3/\text{s/defect} \). This can be converted to gallons per day (gpd) per defect as follows:

\[
\frac{3.45E-5 \text{ m}^3/\text{s}}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ s}}{\text{min}} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{24 \text{ hr}}{\text{day}} = 787.4 \text{ gpd/defect}
\]

To establish AL1, the PLR is multiplied by the defect rate and then multiplied by the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. This defect rate allows for seam defects resulting from fabrication or installation factors that still may exist after intensive quality assurance (Giroud and Bonaparte, 1989). The PW Pond has a LSA of 535,729 sf or 12.3 acres.

\[
AL1 = \frac{787.4 \text{ gpd/defect}}{\text{defect}} \times \frac{1 \text{ defect}}{\text{acre}} \times 12.3 \text{ acres} = 9,685.5 \text{ gpd}
\]

Based on the conditions presented, AL1 for the PW Pond was calculated to be 9,686 gpd.

**Calculation of AL2 for the PW 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 1.02 presents the parameters and calculation results for the PLR.

| Table 1.02  PLR for the PW Pond (AL2) |
|----------------|---------------------------------|
| \( C_B \) =   | 0.6 (dimensionless)             |
| \( a \) =     | 100 (mm\(^2\))                 |
| \( g \) =     | 9.81 (m/s\(^2\))               |
| \( h_w \) =   | 17.1 (m)                        |
| \( Q \) =     | 1.10E-3 PLR (m\(^3\)/s/defect) |

The calculations yielded a PLR of \( Q = 1.10E-3 \text{ m}^3/\text{s/defect} \). This can be converted to gpd per defect as follows:
To establish AL2, the PLR is multiplied by the defect rate and then multiplied by the LSA of the pond in acres. A defect rate of one (1) hole per acre was selected. The PW Pond has a LSA of 535,729 sf or 12.3 acres.

\[
AL2 = \frac{25,106.7 \text{ gpd}}{\text{defect} \times \text{acre}} \times 12.3 \text{ acres} = 308,812.6 \text{ gpd}
\]

Based on the conditions presented, AL2 for the PW Pond was calculated to be 308,813 gpd.

**Conclusions**

The ALs for the PW Pond, as measured by the amount of fluid pumped out of a LCRS for a typical process solution pond, were calculated to be:

- AL1 = 9,686 gpd; and
- AL2 = 308,813 gpd.

If it is determined during normal operations that the amount of fluid pumped back to the PW Pond from the LCRS exceeds the AL1, Rosemont will take action to determine the cause. This action may include physical inspection, mechanical leak detection, electric leak location, or other methods to determine what is causing the AL1 exceedance in order to maintain the liner integrity such that the AL2 is not exceeded.

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

### 1.2 TS Pond

The TS Pond is designed for temporary and emergency storage only and will be dry during normal operations. Additionally, a temporary or permanent pumping system will be utilized in the TS Pond to remove impounded water within 90 days and recycle the water into the reclaim water system.

The TS Pond is considered a non-stormwater pond for the evaluation of BADCT. Prescriptive design for this type of facility includes the following liner system (from bottom to top):

- A prepared subgrade consisting of a minimum of six (6) inches of minus 3/8 inch native or natural materials compacted to 95% maximum dry density (ASTM D-698); and
- A single geomembrane of at least 30 mil thickness (60 mil if HDPE).

The TS Pond liner system will consist of the following (from bottom to top):

- A minimum six (6) inch thick layer of properly compacted bedding soil (prepared subgrade);
A sodium bentonite GCL; and
A 60 mil HDPE geomembrane liner.

A cross section of the liner system for the TS Pond is presented in Illustration 1.02.

Illustration 1.02 TS Pond Liner System

As noted, prescriptive BADCT design does not include a low permeability component. Therefore, the proposed design will exceed prescriptive BADCT. Additionally, the system will not have a leak detection system, or LCRS, since the facility is not double-lined.
2.0 Stability Analysis

A stability analysis for the PWTS Ponds was performed using the Slope/W component of the GeoStudio 2007 software package produced by Geo-Slope International, Ltd. The cross sections selected for stability analysis were based on the maximum or critical sections as taken at the east (TS Pond) and west (PW Pond) cells of the PWTS Ponds embankment. One (1) section of the TS Pond and two (2) sections of the PW Pond were analyzed. Figure 02 shows the location of the sections analyzed. Sections 2.1 through 2.5 presents the design criteria, methods and results of slope stability analyses performed.

2.1 Construction of Model Cross Section

Both the upstream and downstream embankment slopes of the PWTS Ponds are proposed to be 2.5H:1V. During construction of the PW Pond, a 35 foot thick layer of 12 inch run-of-mine (ROM) drain rock will be placed in the drainage prior to the construction of the embankment. This rock will be used for post-closure drainage of the upstream area via a flow-through drain (AMEC, 2009). The embankment of both ponds is proposed to consist of locally excavated weathered bedrock from the Willow Canyon Formation. The liner system will be placed over the embankment, including the ROM drain layer, thus preventing solutions in the PW Pond from flowing through the embankment. The ROM drain rock, however, will not be part of the TS side of the embankment.

As shown in Figure 01, the PWTS Embankment is mostly underlain by the Willow Canyon Formation. However, a small deposit of older alluvium is present under the southeastern edge of the embankment. The critical sections of the TS Pond, Section A on Figure 02, does not encounter alluvium, therefore, an alluvium layer was not included in the model. Additionally, testing indicates that the Willow Canyon Formation in the Plant Site area may have a lower strength value than older alluvium.

The PW Pond maximum embankment height does not occur at perpendicular angles to the upstream and downstream side slopes. This maximum embankment height is indicated as Section B on Figure 02 and does encounter alluvium. Since the critical section of the PW Pond does not occur perpendicular to the embankment, a constructed worst case scenario section was developed. This constructed section was created by combining the two (2) maximum slopes for the upstream (Section C on Figure 02) and downstream (Section D on Figure 02) sides of the embankment.

As previously mentioned, a flow-through drain (35 foot high ROM layer) will intersect the PW side of the embankment. Therefore, the constructed maximum section was analyzed for two (2) configurations of the ROM drain layer. This accounts for the variation of ROM thickness depending on the location of the section (Section C or D on Figure 02). The first configuration, shown as Illustration 2.05, is identified as PW1. In this configuration, the ROM drain layer ranges from 0 to 35 feet. The second configuration, shown as Illustration 2.09, is identified as PW2. In this configuration, the ROM drain layer is 35 feet high.

The PWTS Ponds were modeled using two (2) feet of freeboard below the embankment’s crest, resulting in a water elevation of 4,948 feet above mean sea level (amsl). This is conservative
since the site hydrology studies estimated that the PW and TS Ponds will have adequate capacity to contain the 6-hour Probable Maximum Precipitation (PMP) event while maintaining over 25 acre-feet of capacity. The phreatic surface was added to the model to account for the weight of the water against the embankment and not to evaluate seepage since the embankment will be HDPE lined. However, the HPDE liner was not included in the stability model since it does not create a critical slip surface.

Local groundwater elevations have been recorded in well HC-3B, located approximately 1,000 feet from the PWTS Ponds. Groundwater level measurements in this well ranged from approximately 4,785 to 4,815 feet amsl (Errol L. Montgomery & Associates, 2009). The bottom elevation of the PW Pond is estimated to be 4,892 feet amsl resulting in a depth to groundwater of 77 to 107 feet below the pond bottom. The bottom elevation of the TS Pond is estimated to be 4,895 feet amsl, resulting in a depth to groundwater of 80 to 110 feet below the pond bottom. The groundwater elevation is below the modeling boundaries such that the stability of the facility is not affected. Therefore, groundwater was not included in the model cross section.

2.2 Stability Requirements

Design of the PWTS Ponds is governed by requirements of ADEQ as detailed in the Arizona Mining BADCT Guidance Manual (ADEQ, 2004) and by the Arizona Department of Water Resources (ADWR) as described in Arizona Administrative Code (A.A.C.) Title 12, Chapter 15. Based on these requirements, the minimum stability criteria adopted for the PWTS embankment are presented in Table 2.01. 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 PWTS embankment.

Table 2.01 Minimum Stability Requirements (with testing)

<table>
<thead>
<tr>
<th>Analysis Condition</th>
<th>Required Minimum Factor of Safety</th>
</tr>
</thead>
<tbody>
<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 PWTS embankment were reviewed.

Based on A.A.C. Title 12, Chapter 15 Section 1206(A) Table 2, the PWTS Ponds were defined as an intermediate size dam since the embankment has a maximum height of 80 feet and the total storage capacity of both ponds is 360 acre-feet.

A.A.C. R12-15-1206(B)(2)(a) defines a Very Low Hazard Dam Potential as:

\[ \text{“Failure or improper operation of a dam would be unlikely to result in loss of human life and would produce no lifeline losses and very low economic and intangible losses.”} \]
Losses would be limited to the 100 year floodplain or property owned or controlled by the dam owner under long-term lease. The Department (ADWR) considers loss of life unlikely because there are no residences or overnight camp sites.”

As previously indicated, after Year 4 the Dry Stack Tailings Facilities will be above the elevation of the PWTS embankment. This will effectively buttress the embankment and significantly reduce the likelihood of a failure. However, in the unlikely event of a failure of the PWTS embankment prior to Year 4, water would flow into the Dry Stack Tailings Facilities. Therefore, any damage caused by the failure of the PWTS 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. As per ADWR dam safety regulations, the PWTS embankment qualifies as an intermediate size, very low hazard dam.

Since the PWTS embankment qualifies as an intermediate size, very low hazard dam, 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.

2.3 Modeling Methods

As previously mentioned, stability analysis was performed using the Slope/W component of GeoStudio 2007 software package. It should be noted that seepage analyses of the PWTS embankment was not conducted because the facility will be lined with a HDPE geomembrane and GCL. Therefore, steady-state seepage conditions are not expected to develop in the embankment.

Slope/W was used to conduct limiting equilibrium analyses using the general limit equilibrium (GLE) method, which satisfies both force and moment equilibrium. This program incorporates search routines to determine the critical, lowest factor of safety failure surface. Slope/W was used to conduct analyses of slope stability considering global (rotational) stability of the embankment and full-height failure surfaces affecting the crest of the embankment on the upstream and downstream sides of the embankments.

To evaluate the performance of the embankment under seismic loading, pseudo-static stability analyses were performed. This pseudo-static analysis subjects a two-dimensional sliding mass to a horizontal acceleration equal to an earthquake coefficient multiplied by the acceleration of gravity.

2.4 Material Properties

The material properties presented in Table 2.02 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 and remolded Willow Canyon materials collected during drilling and test pit sampling. Although the testing was completed on the young alluvium, it is appropriate to use those test results for the older alluvium since the older alluvium is expected to have equal or greater strength properties. Material properties for the in-situ Willow Canyon Formation were based on the direct
shear testing of the remolded Willow Canyon material. This is considered an appropriate approach since the in-situ material is expected to have similar strength properties to the remolded material. The ROM drain rock material properties were developed using a bilinear strength function based on Leps (1970) medium-strength rock fill.

Table 2.02 Material Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Phi (degrees)</th>
<th>Cohesion (psf)</th>
<th>Unit Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remolded Willow Canyon (Embankment Fill)</td>
<td>34</td>
<td>0</td>
<td>118</td>
</tr>
<tr>
<td>Older Alluvium</td>
<td>39</td>
<td>0</td>
<td>127</td>
</tr>
<tr>
<td>Willow Canyon (in-situ Bedrock)</td>
<td>34</td>
<td>0</td>
<td>118</td>
</tr>
<tr>
<td>ROM Drain Rock *</td>
<td>0-60°: 38</td>
<td>60° +: 34</td>
<td>0</td>
</tr>
</tbody>
</table>

* Only used in the PW Pond embankment model.

2.5 Results

Illustrations 2.01 through 2.12 show the results of the stability analyses performed. The minimum factors of safety determined by the analyses are summarized in Table 2.03. Based on these results, the static and pseudo-static factors of safety against failure of the PWTS Ponds embankment are adequate under normal operating conditions. It should be noted that block sliding failure of the embankment over the natural ground slope was evaluated and found not to be a critical failure mode for the facility; that is, the safety factor for this mode of failure was significantly higher than for global failure.

Table 2.03 Results of Slope Stability Analyses for the PWTS Embankment (Global Failure)

<table>
<thead>
<tr>
<th>Cell</th>
<th>Scenario</th>
<th>Safety Factor</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Static</td>
<td>Pseudostatic (MPE)</td>
</tr>
<tr>
<td>TS</td>
<td>Downstream</td>
<td>1.80</td>
<td>1.65</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td>1.82</td>
<td>1.68</td>
<td></td>
</tr>
<tr>
<td>PW1 (ROM below Embankment)</td>
<td>Downstream</td>
<td>1.75</td>
<td>1.58</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td>1.71</td>
<td>1.57</td>
<td></td>
</tr>
<tr>
<td>PW2 (ROM throughout Embankment)</td>
<td>Downstream</td>
<td>1.73</td>
<td>1.58</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td>1.87</td>
<td>1.72</td>
<td></td>
</tr>
</tbody>
</table>
Illustration 2.01    PWTS Embankment – TS Pond – Downstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - TS Pond

Static, Global Failure

Name: Willow Canyon Fir (in-situ and remolded)
Unit Weight: 118 psf
Cohesion: 0 psf
Phi: 34°
Illustration 2.02  PWTS Embankment – TS Pond – Upstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - TS Pond
Static, Upstream Failure

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 psf
Cohesion: 0 psf
Phs: 34°
Illustration 2.03  PWTS Embankment – TS Pond – Downstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - TS Pond

Pseudostatic, 2/3 MPE Ground Acceleration
Horiz seismic Value: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 psf
Cohesion: 0 psf
Phi: 34°
Illustration 2.04  PWTS Embankment – TS Pond – Upstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - TS Pond
Pseudostatic, 2/3 MPE Ground Acceleration
Horz seismic Value: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 psf
Cohesion: 0 psf
Phi: 34°

Existing Ground
Phreatic Surface
Remolded Willow Canyon
Willow Canyon Formation
Illustration 2.05  PWTS Embankment – PW Pond – PW1 – Downstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW1 Section
Static, Global Failure

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34°

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39°

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38°
Phi 2: 34°
Bilinear Normal: 7200 psf
Illustration 2.06  PWTS Embankment – PW Pond – PW1 – Upstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW1 Section
Static, Upstream Failure

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39 °

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38 °
Phi 2: 34 °
Bilinear Normal: 7200 psf

Phreatic Surface
2.5H:1V
12" ROM Drain Rock
35°
Illustration 2.07  PWTS Embankment – PW Pond – PW1 – Downstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW1 Section

Pseudostatic, 2/3 MPE Ground Acceleration
Horz Seismic Load: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34°

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39°

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38°
Phi 2: 34°
Bilinear Normal: 7200 psf
Illustration 2.08  PWTS Embankment – PW Pond – PW1 – Upstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW1 Section

Pseudostatic, Upstream Failure, 2/3 MPE Ground Acceleration
Horz Seismic Load: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39 °

Name: 12” ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38 °
Phi 2: 34 °
Bilinear Normal: 7200 psf
Illustration 2.09        PWTS Embankment – PW Pond – PW2 – Downstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW2 Section

Static, Global Failure

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34°

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39°

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38°
Phi 2: 34°
Bilinear Normal: 7200 psf

Elevation (x 1000)

Distance
Illustration 2.10   PWTS Embankment – PW Pond – PW2 – Upstream Side Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW2 Section

Static, Upstream Failure

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34°

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39°

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38°
Phi 2: 34°
Bilinear Normal: 7200 psf
Illustration 2.11  PWTS Embankment – PW Pond – PW2 – Downstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW2 Section

Pseudostatic, 2/3 MPE Ground Acceleration
Horz Seismic Load: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34°

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39°

Name: 12" ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38°
Phi 2: 34°
Bilinear Normal: 7200 psf
Illustration 2.12  PWTS Embankment – PW Pond – PW2 – Upstream Side Pseudo-Static Stability Analysis

ROSEMONT COPPER COMPANY
PWTS PONDS - PW Pond - PW2 Section

Pseudostatic, 2/3 MPE Ground Acceleration
Horz Seismic Load: 0.03

Name: Willow Canyon Fm (in-situ and remolded)
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 34 *

Name: Older Alluvium
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 39 *

Name: 12” ROM Drain Rock
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi 1: 38 *
Phi 2: 34 *
Bilinear Normal: 7200 psf

Phreatic Surface
1.72
25°
Remolded Willow Canyon
2.5H:1V
Willow Canyon Formation
12” ROM Drain Rock
35°
Older Alluvium
3.0 Evaluation of BADCT Alternatives

Since the PWTS Ponds are divided into two (2) cells, the PW Pond and the TS Pond, and the design and function of each pond is different, BADCT alternatives for each pond were evaluated separately.

3.1 PW Pond

The PW Pond is being treated as a process solution pond for the evaluation of prescriptive BADCT standards. Prescriptive BADCT for a process solution pond includes the following liner system (from bottom to top):

- A prepared subgrade consisting of a minimum of six (6) inches of native or natural materials compacted to 95% maximum dry density (ASTM D-698);
- A low permeability soil (LPS) layer consisting of a minimum of six (6) inches of 3/8 inch minus native or natural materials compacted to 95% maximum dry density (ASTM D-698) with a maximum hydraulic conductivity of $1 \times 10^{-6}$ cm/s;
- A secondary geomembrane liner of at least 30 mil thickness (60 mil if HDPE);
- An LCRS layer; and
- A primary geomembrane liner of at least 30 mil thickness (60 mil if HDPE).

A GCL was selected as an alternative to the LPS layer of the PW Pond liner system due to the absence of onsite clay borrow materials. The GCL surface also provides greater rock puncture protection to the overlying geomembrane liner. A smoothed and compacted subgrade surface, graded to drain, is required for the GCL.

In order to compare the degree of engineering control achieved by using an LPS or GCL as the low permeability component of the liner system, the Total Potential Leakage (TPL) was calculated for two (2) liner systems. The first uses a six (6) inches (0.1524 meters) thick layer of LPS. The second system uses a GCL.

The PLR through the bottom liner of the PW Pond was estimated using Giroud's Equation (Giroud, 1997) to calculate the leakage through a circular defect in a liner system that includes a low permeability component (LPS or GCL), along with a geomembrane liner. The PLR was based on the assumption that the fluid on the bottom liner will be contained within the LCRS sump, the sump will contain one (1) defect that is two (2) mm in diameter, and the LCRS sump will have a depth of 1.5 feet (0.4572 meters).

Giroud's Equation (Giroud, 1997):

$$Q = 0.976 C_{qo} [1 + 0.1(h/t_s)^{0.95}] d^{0.2} h^{0.9} k_s^{0.74}$$
Where:

\[ Q = \text{Rate of liquid migration or PLR (m}^3/\text{s}); \]

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.

\[ C_{q0} = \text{Contact Quality Factor represents the contact interface between the low permeability component and the geomembrane liner (dimensionless);} \]

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;

\[ h = \text{Height of liquid on top of geomembrane (m);} \]

Giroud's Equation assumes that the hydraulic head on the liner to be less than or equal to three (3) meters. 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. The design depth of the PW Pond sump, 1.5 feet (0.4572 meters), was selected;

\[ t_s = \text{Thickness of the low permeability component (m);} \]

The thickness of the low permeability component directly effects the amount of time necessary for a fluid to flow through the material. These calculations use six (6) inches (0.1524 meters) of LPS and a GCL thickness of six (6) mm underneath the geomembrane liner;

\[ d = \text{Diameter of circular defect (m);} \]

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 \([\text{area } (a) = 3.14 \text{ mm}^2]\) 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). Therefore, a defect rate of one (1) hole per sump that is two (2) millimeters (mm) in diameter was selected;

\[ k_s = \text{Hydraulic conductivity of the low permeability component (m/s).} \]

The prescriptive BADCT permeability standard of \(1 \times 10^{-6} \text{ cm/s}\) was used for the LPS calculation. A standard geosynthetic clay liner (GCL) permeability of \(5 \times 10^{-9} \text{ cm/s}\) was selected for the GCL calculation (Cetco, 2009).
Table 3.01 presents the calculations used to determine the PLR through the bottom liner of the PW Pond liner system using LPS.

Table 3.01  PLR Through the Bottom Liner of the PW Pond using LPS

<table>
<thead>
<tr>
<th>$C_{qo}$</th>
<th>0.21</th>
<th>(dimensionless)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$</td>
<td>0.4572</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/sec)</td>
</tr>
<tr>
<td>$Q$</td>
<td>4.51E-08</td>
<td>PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 4.51E-8$ m$^3$/s/defect. This can be converted to gallons per day (gpd) per defect as follows:

$$\frac{4.51E-8 \text{ m}^3/\text{s}}{\text{defect}} \times \frac{264.17 \text{ gallons}}{\text{m}^3} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{1 \text{ day}}{24 \text{ hr}} = 1.03 \text{ gpd}$$

To establish the TPL, the PLR is multiplied by the defect rate. Because the sump is less than one acre in area, a single defect was selected.

$$TPL = \frac{1.03 \text{ gpd}}{\text{defect}} \times 1 \text{ defect} = 1.03 \text{ gpd}$$

The calculations indicate that a prescriptive BADCT lined process solution pond could potentially discharge approximately 1.0 gpd.

Table 3.02 presents the calculations used to determine the PLR through the bottom liner of the PW Pond liner system using GCL.

Table 3.02  PLR Through the Bottom Liner of the PW Pond using GCL

<table>
<thead>
<tr>
<th>$C_{qo}$</th>
<th>0.21</th>
<th>(dimensionless)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$</td>
<td>0.4572</td>
<td>(m)</td>
</tr>
<tr>
<td>$d$</td>
<td>0.002</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>4.97E-09</td>
<td>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 gallons per day (gpd) per defect as follows:
To establish the TPL, the PLR is multiplied by the defect rate. Because the sump is less than one acre in area, a single defect was selected.

\[
TPL = \frac{0.113 \text{ gpd}}{\text{defect}} \times \text{defect} = 0.113 \text{ gpd}
\]

The calculations indicate that a PW Pond liner system using GCL could potentially discharge approximately 0.113 gpd or 14.5 fluid ounces per day.

**Cost Comparison**

The PW Pond has a LSA of approximately 535,729 sf. In order to account variables such as seam overlaps, material placed in anchor trenches, and installation waste, approximately 589,302 sf of GCL (110% of the LSA to account for waste) or 9,921 cubic yards (CY) of LPS would be required to construct the pond. Table 3.03 summarizes the estimated costs for the delivery and installation of GCL versus LPS for the Raffinate Pond.

**Table 3.03  GCL vs. LPS Costs – PW Pond**

<table>
<thead>
<tr>
<th>Material (Units)</th>
<th>Quantity</th>
<th>Delivered</th>
<th>Placement</th>
<th>Total Cost</th>
<th>TPL (gpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GCL (sf)</td>
<td>589,302</td>
<td>$0.40</td>
<td>$235,721</td>
<td>$176,791</td>
<td>0.113</td>
</tr>
<tr>
<td>LPS (CY)*</td>
<td>9,921</td>
<td>$24.00</td>
<td>$238,104</td>
<td>$69,447</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Costs were developed based on assumptions regarding an offsite borrow source located 15 miles from the Project Site.

The GCL cost comparison was based on the Rosemont Copper Heap Leach Facility Cost Estimate – Final (Tetra Tech, 2008). The LPS cost estimate is based on estimates performed for the Freeport McMoRan Copper & Gold Safford Leach Project. The delivered cost for LPS includes:

- The costs for excavating and hauling the material from an assumed suitable borrow source approximately 15 miles from the Project site. This cost is estimated to be $17.00 per CY.
- Because a borrow source for LPS is not available onsite, the purchase of LPS from offsite would be necessary to install LPS for this project. This base cost was estimated to be $5.00 per CY.
- Costs for characterizing, developing, closing, and reclaiming a borrow source were estimated to be $2.00 per CY.

As indicated in Table 3.03, GCL would be approximately $104,961 more expensive to install when compared with the LPS estimate. At most sites, GCL costs would far exceed the LPS costs due to the availability of onsite sources. However, the Project site does not have a source
of LPS, thus requiring import from an offsite borrow source. Additionally, the GCL layer provides a greater degree of engineering control.

3.2 TS Pond

The TS Pond is designed for temporary and emergency storage only and will be dry during normal operations. The TS Pond provides temporary storage of stormwater for a 54.6 acre drainage area partially associated with the dry stack tailings area and any potential overflow from the PW Pond. Overflow from PW Pond would result from a failure of the reclaim water pumping system coincident with a storm exceeding the 100-year, 24-hour design storm event.

Two (2) BADCT alternatives were considered for the liner system of the TS Pond. The alternatives included:

- A prescriptive BADCT liner system for a non-stormwater pond; and
- A modified BADCT liner system for a non-stormwater pond that incorporates a low permeability GCL.

Non-Stormwater Pond Liner System

For a non-stormwater pond, prescriptive BADCT requires a liner system consisting of the following (from bottom to top):

- A minimum six (6) inches of minus 3/8-inch native or natural materials compacted to a minimum of 95% of the maximum dry density (ASTM D-698); and
- A geomembrane liner of at least 30 mil thickness (60 mil if HDPE) (ADEQ, 2004).

Because a low permeability component is not required for a prescriptive BADCT design, the PLR can be estimated using Bernoulli’s equation for free flow through an opening as presented in Section 1.1.

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

- Dimensionless coefficient (C_b): This coefficient is related to the shape of the edges of the hole (for sharp edges C_b = 0.6);
- The hole area (a): A single, two (2) mm diameter (a = 3.14 mm^2) 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);
- Liquid depth on top of the geomembrane (h_w): The maximum hydraulic head of 55 feet (16.8 meters) was used to estimate the PLR; and
- Lined Surface Area (LSA): The TS Pond has a lined surface area of 311,610 sf or 7.15 acres.

Table 3.04 presents the PLR for a prescriptive BADCT lined non-stormwater pond.
Table 3.04  PLR for a Prescriptive BADCT Lined Non-Stormwater Pond

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_B$</td>
<td>0.6 (dimensionless)</td>
</tr>
<tr>
<td>$a$</td>
<td>3.14 (mm$^2$)</td>
</tr>
<tr>
<td>$g$</td>
<td>9.81 (m/s$^2$)</td>
</tr>
<tr>
<td>$h_w$</td>
<td>16.8 (m)</td>
</tr>
<tr>
<td>$Q$</td>
<td>$3.42E-05$ PLR (m$^3$/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of $Q = 3.42E-5$ m$^3$/s/defect. This can be converted to gallons per day (gpd) per defect as follows:

$$\frac{3.42E - 5m^3}{s\text{ defect}} \times \frac{264.17\text{Gallons}}{m^3} \times \frac{60s}{\text{min}} \times \frac{60\text{min}}{\text{hr}} \times \frac{24\text{hr}}{\text{day}} = \frac{780.6\text{gpd}}{\text{defect}}$$

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. A single, small 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).

$$\text{TPL} = \frac{780.6\text{gpd}}{\text{defect}} \times \frac{1\text{defect}}{\text{acre}} \times 7.15\text{acres} = 5,581\text{gpd}$$

The calculations indicate that a prescriptive BADCT lined non-stormwater pond could potentially discharge 5,581 gpd through the liner system.

**Modified Non-Stormwater Pond Liner System**

This section presents the TPL for a modified BADCT lined non-stormwater pond using a composite liner system comprised of a GCL and geomembrane liner.

The leakage through a circular defect in a liner system that includes a low permeability component (soil or GCL), along with a geomembrane liner, was estimated using Giroud's Equation (Giroud, 1997) as previously presented.

$$Q = 0.976 \ C_{q_0} \ [1 + 0.1(h/t_s)^{0.95}] \ d^{0.2} \ h^{0.9} \ k_s^{0.74}$$

Where:
- $Q$ = Rate of liquid migration or PLR (m$^3$/s);
- $C_{q_0}$ = Contact Quality Factor represents the contact interface between the low permeability component and the geomembrane liner (dimensionless);
- $h$ = Height of liquid on top of geomembrane (m);
- $t_s$ = Thickness of the low permeability component (m);
This calculation uses a GCL having a thickness of six (6) mm;

\[ d = \text{Diameter of circular defect (m)}; \] and

A defect rate of one (1) hole per acre that is two (2) millimeters (mm) in diameter was selected.

\[ k_s = \text{Hydraulic conductivity of the low permeability component (m/s)}. \]

A standard GCL permeability of \(5 \times 10^{-9}\) cm/s was selected (Cetco, 2009).

Table 3.05 presents the PLR for a modified BADCT lined non-stormwater pond using a composite liner system comprised of a GCL and geomembrane liner.

### Table 3.05  PLR for a Modified BADCT Lined Non-Stormwater Pond

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(C_{qo})</td>
<td>0.21</td>
<td>(dimensionless)</td>
</tr>
<tr>
<td>(h)</td>
<td>16.8</td>
<td>(m)</td>
</tr>
<tr>
<td>(d)</td>
<td>0.002</td>
<td>(m)</td>
</tr>
<tr>
<td>(t_s)</td>
<td>0.0060</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>3.37E-06</td>
<td>PLR (m(^3)/s/defect)</td>
</tr>
</tbody>
</table>

The calculations yielded a PLR of \(Q = 3.37E-6\) m\(^3\)/s/defect. This can be converted to gallons per day (gpd) per defect as follows:

\[
\frac{3.37E - 6\ m^3/s}{\text{defect}} \times \frac{264.17\ gallons}{m^3} \times \frac{60\ min}{hr} \times \frac{60\ hr}{day} = \frac{76.92\ gpd}{\text{defect}}
\]

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. A single, small 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).

\[
TPL = \frac{76.92\ gpd}{\text{defect}} \times \frac{1\text{defect}}{\text{acre}} \times 7.15\text{ acres} = 550.0\text{ gpd}
\]

The calculations indicate that a modified BADCT lined non-stormwater pond could potentially discharge 550 gpd through the liner system.

### Cost Comparison

The TS Pond has a LSA of approximately 311,610 sf. Approximately 342,771 sf of material (110% of the LSA) will be required to construct each layer of the proposed TS Pond in order to account for variables such as seam overlaps, material placed in anchor trenches, and installation waste.
Table 3.06 presents the estimated costs for constructing the different liner systems for the TS Pond. Additional costs such as mobilization, clearing and grubbing, earthwork, subgrade preparation, etc., have been assumed to be equal for the two (2) alternatives.

**Table 3.06  TS Pond BADCT Cost Comparison**

<table>
<thead>
<tr>
<th>Item</th>
<th>Units</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
<th>PLR (gpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prescriptive BADCT Non-Stormwater Pond</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liner</td>
<td>sf</td>
<td>342,711</td>
<td>$ 0.72</td>
<td>$ 246,795</td>
<td></td>
</tr>
<tr>
<td>Total Cost</td>
<td></td>
<td></td>
<td></td>
<td>$ 246,795</td>
<td>5,581</td>
</tr>
<tr>
<td>Modified BADCT Non-Stormwater Pond</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GCL</td>
<td>sf</td>
<td>342,711</td>
<td>$ 0.70</td>
<td>$ 239,940</td>
<td></td>
</tr>
<tr>
<td>Liner</td>
<td>sf</td>
<td>342,711</td>
<td>$ 0.72</td>
<td>$ 246,795</td>
<td></td>
</tr>
<tr>
<td>Total Cost</td>
<td></td>
<td></td>
<td></td>
<td>$ 486,735</td>
<td>550</td>
</tr>
</tbody>
</table>

Note: These estimates are not intended to be used for bidding or budgeting purposes.

As indicated in Table 3.07, there will be an additional cost of $239,940 associated with using a modified BADCT liner system over the prescriptive BADCT liner system for a non-stormwater pond. However, the use of a GCL results in a greater engineering control. The cost estimate is based on the Rosemont Copper Heap Leach Facility Cost Estimate – Final (Tetra Tech, 2008).

Despite the higher cost of including a GCL in the construction of the TS Pond, Rosemont will use this design to achieve a higher degree of engineering control.
4.0 Conclusions

Overall, it is expected that the PWTS Ponds Facility will function without any structural failures that may cause a discharge to the underlying aquifer.

This technical memorandum was prepared in accordance with generally accepted engineering practices and applicable BADCT requirements.
5.0 References


