REPORT

PERIODIC SAFETY INSPECTION REPORT NO. 2 – LOWER SLATE LAKE TAILINGS DAM

Alaska Jurisdictional Dam No. AK00308
Kensington Mine, Alaska

Submitted To: Coeur Alaska Inc.
3031 Clinton Drive, Suite 202
Juneau, AK 99801

Submitted By: Golder Associates Inc.
2121 Abbott Road, Suite 100
Anchorage, AK 99507 USA

Distribution:

3 Copies – Coeur Alaska Inc.
1 Copy – Charles Cobb, Alaska Dam Safety Engineer
2 Copies – Golder Associates Inc.

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EXECUTIVE SUMMARY

This report presents the results of the second periodic safety inspection (PSI) of the Lower Slate Lake Dam at Kensington Mine, Alaska, which was conducted by Golder Associates Inc. (Golder) on June 24, 2014. This inspection was performed as a follow-on to one prior periodic safety inspection performed by Knight Piésold Limited (KPL) on June 28, 2011. The findings presented in this report are intended to satisfy regulations covered under Title 11, Chapter 93, Article 3 of Alaska Administrative Code (11 AAC 93), which addresses the safety of all non-federally regulated dams in Alaska.

Based on our inspection, the Lower Slate Lake Dam and ancillary structures generally appear to be in satisfactory condition and have been maintained well. The significant conclusions and recommendations resulting from our inspection of the dam and review of design, construction, and operations documents are summarized below. Refer to the report text for additional details.

- The lake level is rising at a faster rate than planned, which is mainly attributed to low Tailings Treatment Facility (TTF) water treatment plant (WTP) rates. The updated 2014 water balance modeling suggests that if the TTF WTP rates can be increased and maintained at 1,000 gallons per minute (gpm) or 1,500 gpm through the end of 2014 or longer, the lake level will remain within current dam capacity until construction of the planned Stage 3 of the dam is completed as scheduled in 2017. This modeling shows the mean and minimum predicted lake levels, which are very similar, can be maintained below the maximum 200-year storm containment elevation of 697.3 feet. The mean predicted lake level is based on the average of 100 iterations or realizations used for the model; therefore, the majority of the realization results for each time step are very similar to the minimum predicted value. The modeling suggests that there is a risk the maximum predicted lake level could rise to the maximum 200-year storm containment elevation during the fourth quarter of 2014 or later in 2015 if the increase in WTP rates described above does not occur, but we understand that this increase is within the WTP permitted capacity and achievable. We understand that CAK plans to design the Stage 3 Dam in 2015 with construction in 2016.
- The Operations and Maintenance (O&M) Manual appears to be thorough and appropriate for this facility. However, the manual does not describe that the maximum elevation of the eastern upstream grout trench (710.5 feet) is 4.5 feet lower than the dam crest elevation, and there appears to be a discrepancy between the maximum operating elevation of 695 feet in the O&M Manual and the 697.3 feet elevation used in the water balance. The O&M Manual also does not include the requirement for a PSI by a qualified professional engineer every three years in accordance to the Alaska Administrative Code 11 AAC 93.159(a) and the Class II hazard potential classification for the dam. In addition, the stream gauge at East Fork Slate Creek, immediately downstream of the dam, is no longer monitored as the stream flow is essentially equivalent to the discharges from the Parshall Flume and the TTF WTP discharge. A gauging station and transducer are maintained at the East Fork Slate Creek as a backup. We recommend that the O&M Manual be revised to account for these discrepancies and noted changes.
- The Emergency Action Plan (EAP) appears to be thorough and appropriate for this facility, but it could be improved by highlighting the limitations of the upstream east grout trench (in the project description section of the EAP) as noted in the previous bullet. Preventive measures for excessive seepage through the embankment are included in the EAP and these measures could be required if the lake level rose above the 710.5 feet

elevation. The requirement for a functional exercise in the EAP can also be removed as is it not necessary for a Class II dam.

- The hydrology design parameters for the spillway and water balance were based on a review of site and regional data as well as synthetic analyses through modeling to validate peak flows. Based on our review of the design documents, the approach and analyses to develop the hydrology design parameters appears reasonable. The methods used to develop the design storms and analyses used to design the spillway also appear to be reasonable and conservative.
- The permeability, density, and strength values for the materials used in the static (pre-earthquake and post-liquefaction) and seismic (pseudo-static) slope stability analyses appear reasonable as well as the analyses methods for the various evaluation scenarios. Based on our review of the design earthquakes using the United States Geological Survey (USGS) seismic hazard maps, the Operating Basis Earthquake (OBE) peak ground acceleration (PGA) has increased to 0.171g compared to 0.13g used in the seismic stability design. This higher OBE PGA of 0.171g is still lower than the yield accelerations, which ranged from 0.30g to 0.45g for the operations case; therefore, the factor of safety is still above one with minor deformations. While the details of the deterministic analyses were not included in the reference documents reviewed, the MCE PGA appears to be reasonable. We recommend a detailed review of the seismic hazard deterministic and probabilistic analyses during the Stage 3 Dam design including updated seismic data from USGS.

LOWER SLATE LAKE TAILINGS DAM
AK00306

Kensington Mine, near Juneau, Alaska
Latitude 58.8073 degrees and Longitude -135.0386 degrees

Owned by: Coeur Alaska Inc.

Size Classification: N/A

Hazard Classification: Class II

Inspector:
Steven L. Anderson, PE
Golder Associates Inc.
Associate and Senior Geotechnical Engineering Consultant

STATE OF ALASKA
49TH
12/16/14
STEVEN L. ANDERSON
CE-9120
REGISTERED PROFESSIONAL ENGINEER

Approved By:
Thomas G. Krzewinski, PE, D.GE, F. ASCE
Golder Associates Inc.
Principal and Senior Geotechnical Engineering Consultant

Reviewed By: Charles F. Cobb, PE
State Dam Safety Engineer

Approved By: Jan 10, 2015
Charles F. Cobb, PE
State Dam Safety Engineer
Alaska Department of Natural Resources

Golder Associates

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1.0 INTRODUCTION

This report documents our periodic safety inspection (PSI) of the Lower Slate Lake Tailings Dam at the Kensington Mine, Alaska that was written in general compliance with Section 10.4 of the "Guidelines for Cooperation with the Alaska Dam Safety Program," dated June 2005. Our work was performed in accordance with our proposal dated April 25, 2014.

Our scope of work included the following:

- A detailed review of the record documents, including previous PSI reports, design and construction documentation, the operations and maintenance manual, and other available information pertaining to the dam.
- A field inspection of the dam by a senior professional engineer experienced with dam safety including visual inspection of all dam features; documenting the site conditions by filling out the Alaska Dam Safety Visual Inspection Checklist; review of monitoring data and, discussion of operations, maintenance, and emergency response with onsite personnel.
- An engineering analysis and preparation of this PSI report.

This report includes a description of our field inspection, a detailed summary of the documents and design assumptions reviewed, the completed Alaska Department of Natural Resources (ADNR) Dam Safety forms, specific conclusions regarding the condition and safety status of the dam, and specific recommendations for additional studies, analysis, inspections, monitoring, maintenance or repairs, where applicable. Photographs from our field inspection are included in Appendix A. The ADNR forms including the visual inspection checklist and the updated project data sheet are included in Appendix B.

1.1 Location and Ownership

The Kensington Mine and the Lower Slate Lake Tailings Dam are located about 35 miles air miles south of Haines and 45 air miles north-northwest of Juneau, Alaska, as shown in Figure 1. The Kensington Mine is located in the Tongass National Forest on a small peninsula formed between the Lynn Canal and Berners Bay, an area of coastal mountains at the southern end of the Kakuhan Range. The site is accessed by floatplane, helicopter, or ferry boat. The dam and mining facilities are owned and operated by Coeur Alaska Inc. (CAK), a wholly-owned subsidiary company of Coeur Mining, Inc. (CMI) of Chicago, Illinois.

1.2 Project Description

1.2.1 Mine and Facilities

The Kensington Mine is an underground gold mine that began operations in 2010 with a planned production rate of about 730,000 tons of ore per year and an estimated project life of 10 years (CAK 2005). The mine area has two contiguous properties, Kensington and Jualin, accessed from two portals. The mine produced a record 114,821 ounces of gold in 2013 (CMI 2014).

Mined ore is processed through a gold flotation recovery circuit that includes crushing, grinding, gravity separation, floatation, thickening, and filtering. The gold concentrate is shipped off site and sold to third party smelters. The thickened waste tailings from the floatation process are either used as underground paste backfill or deposited subaqueously in the Lower Slate Lake Tailings Treatment Facility (TTF), which is located about 2.6 miles south of the mill plant (3.5 miles by road), as shown in Figure 2.

1.2.2 Lower Slate Lake Tailings Treatment Facility

As shown in Figure 3, the TTF is comprised of the Lower Slate Lake Tailings Dam, tailings delivery pipeline, water reclaim pipeworks, diversion structures, water treatment plants, and ancillary facilities. The design basis of the TTF is to provide storage of about 4.5 million tons of subaqueous tailings, and provide recycled water for use in the milling process. Daily production of tailings into the TTF is about 720 to 1,200 tons per day at 55 percent solids on average. The recycle requirements back to the mill range from 125 to 179 gallons per minute (gpm). The TTF is a zero-discharge facility.

1.2.2.1 Lower Slate Lake Tailings Dam

The Lower Slate Lake Tailings Dam is a Geosynthetic Face Rockfill Dam (GFRD) located at a natural bedrock constriction at the south end of Lower Slate Lake. The dam is currently constructed at the Stage 2 height of three proposed stages. The upstream slope of the dam is 2 horizontal to 1 vertical (2H:1V) with a bench at the Stage 1 height. The downstream slope of the dam is 1.5H:1V. The dam crest is about 480-feet long, 33-feet wide, and has a crest elevation of about 715 feet. The structural height of the dam is 63 feet from the downstream toe, and it has a hydraulic height of 77 feet at the spillway crest.

The rockfill embankment is founded on bedrock treated with 4 to 6 inches of concrete or shotcrete cover over exposed pyrite bearing phyllite to reduce oxidation and acid rock drainage (ARD). The majority of rockfill is bulk material (Zone A and A1) excavated from quarries and development rock from the underground mine. Two zoned filters were constructed along the upstream face (Zone D and F) to prevent migration of tailings and fine-grained materials into the rockfill in the event the geomembrane was compromised.

The upstream geosynthetic liner is comprised of three layers that include a 100-mil high density polyethylene (HDPE) geomembrane liner sandwiched between layers of 16-ounce non-woven geotextile (one layer of geotextile above and one below the HDPE liner). The geomembrane liner is anchored within the rockfill at the upstream crest and sealed with cement grout within an upstream cutoff trench keyed into bedrock at the toe and abutments. A three-line grout curtain (primary to tertiary grout holes) extends from 20 to 65 feet into the bedrock foundation of the cutoff trench to seal open fractures and increase the seepage flow path.

The dam embankment has a drainage collection system comprised of a series of perforated corrugated polyethylene tubing (CPT) drain pipes along the dam face and within the rockfill embankment foundation. The drain pipes vary in diameter from 4 to 12 inches and are surrounded by Zone D material that was wrapped with an 8-ounce nonwoven geotextile. The 8-inch CPT installed within the Zone D face drain layer run laterally near the upstream toe then longitudinally up near each abutment. The 4-inch CPT was installed near seeps and runs along local depressions of the foundation surface. Both the 4-inch and 8-inch CPT drain into a main 12-inch CPT outlet drain that runs longitudinally along the low part of the foundation until it terminates at a seepage collection sump downstream of the dam. The seepage collection sump is a vertical 96-inch-diameter RSC160 HDPE manhole.

The dam has an interim shotcrete covered spillway at the right (western) abutment that has a bottom elevation at 709 feet and a maximum capacity of about 1,020 cubic feet per second (cfs) with water at the dam crest elevation. The spillway is typically trapezoidal shaped, about 10-feet wide, 550-feet long, and up to 16-feet high. The spillway has a concrete and shotcrete lined plunge pool at the base to dissipate energy. The plunge pool discharges through an outlet channel back into the natural drainage course of East Slate Creek.

1.2.2.2 Tailings Delivery Pipeline

The pumped thickened tailings flow through a combination of double-walled HDPE and steel pipeline that originates at the mine process area and follows along the tailings line road to the TTF. From the 1,000-foot elevation near the Jualin millsite the tailings pipeline is a gravity feed line until it terminates into the reservoir through a floating horizontal tremie pipe for subaqueous disposal. The position of the tailings tremie spigot is adjusted routinely to control deposition of the tailings.

1.2.2.3 Water Reclaim and Treatment System

The water reclaim and treatment system is composed of a floating barge pump station that pumps reclaim water from the TTF to a reclaim water tank next to the TTF water treatment plant (WTP) where a percentage of flow is pumped to the mill to meet recycle requirements and the remaining flow is either directed to the TTF WTP or overflows and drains back into the TTF. The water reclaim system uses a 6-inch-diameter HDPE pipeline.

There are two variable frequency drive (VFD) vertical turbine pumps on the barge, one 125 horsepower (HP) and one 100 HP with one in operation and one on standby. The reclaim pumping rates range from about 500 to 800 gallons per minute (gpm), and the system has the capacity to pump up to 1,500 gpm.

1.2.2.4 Diversion Structures

Diversion structures were constructed around the catchment boundary of the Lower Slate Lake to decrease inflow from precipitation runoff into the TTF. The diversion structures for the TTF include the following:

- An intake structure at the inlet into Lower Slate Lake that diverts the natural stream flow from Upper Slate Creek around the northeast boundary of the TTF. This diversion is interpreted to be 100 percent efficient. The intake structure is a concrete gravity weir with a 26-inch HDPE pipeline that runs along the east side of the Lower Slate Lake and discharges into the flume near the Stage 2 dam abutment. The intake structure also has a gated sluice pipe to clean out debris from the intake area. The intake pipe, which is also gated, was designed to pass the peak flow resulting from 2 inches of rain falling during a 24-hour period, which translates to a flow of 18 cubic feet per second (cfs) or 8,000 gallons per minute (gpm) (KPL 2012D). The recurrence interval for this event is assumed to be annual. Some short duration flows are expected to pass over the weir during extreme runoff events and flow into the TTF.
- Two diversion structures on the north side of Lower Slate Lake that diverts natural stream flow to the intake structure. These diversion structures are thought to be about 60 percent efficient. These diversions are U-shaped metal plates embedded in stream channels with a flange adapter connected into 18-inch HDPE pipelines.
- Two diversion structures along the east side of Lower Slate Lake, which have been interpreted to be about 50 percent efficient. One is known as the “Dogleg Diversion” that collects and diverts runoff via a collection ditch and 10-inch HDPE pipeline that discharges into the flume at the Stage 2 Dam east abutment. The second diversion structure is also located east of the dam embankment and routes the natural stream flow from this upslope area. This second diversion structure is a U-shaped metal plate embedded across the creek channel with a flange adapter connected to a 6-inch HDPE pipeline. The east diversion discharges into the concrete flume box downstream of the flume.

These diversion structures mainly flow into a 26-inch HDPE water conveyance pipeline that starts at the intake structure, connects into the Parshall flume box, and discharges at a plunge pool at the entrance into the East Fork Slate Creek drainage. The East Fork Slate Creek drainage converges into the West Fork Slate Creek drainage farther downstream, and eventually reaches Berners Bay (see Figure 1).

1.2.2.5 Water Treatment Plants

There are two WTPs at the TTF, the TTF WTP located at the northeast side of the lake and the temporary TTF located on the downstream side of the dam. The TTF WTP treats reclaim water and pumps the effluent through a 12-inch HDPE at a rate up to 1,500 gpm into the second plunge pool near the East Slate Creek drainage. The temporary WTP treats drainage that is collected from the adjacent pyrite bearing phyllite stockpile. The temporary WTP discharges the effluent into a land infiltration gallery.

1.3 Hazard Potential Classification Review

The Lower Slate Lake Tailings Dam was initially given a Class II significant hazard potential classification based on a qualitative evaluation by the designer Knight Piésold Limited (KPL), and the dam is located on an anadromous fish stream. However, based on agreements with ADNR Dam Safety, the seismic and hydrologic design was increased to a Class I high hazard potential to account for the lifetime requirements for the dam after closure (KPL 2011).

2.0 DESIGN AND CONSTRUCTION HISTORY

2.1 Site Characterization

Site characterization of the Lower Slate Lake Dam are described in the Detailed Design Report (KPL 2012C), the 2011 Geotechnical Report (KPL 2012A), and the Hydrology Report (KPL 2012E).

2.1.1 Geotechnical Investigations

Geotechnical investigations prior to the Stage 2 Dam construction were performed in 2002, 2004, 2010, and 2011. These investigations are outlined below and the results summarized in the following paragraphs. The geological mapping and drill hole locations from these geotechnical investigations are summarized in Figure 4.

- The 2002 investigation was a preliminary site evaluation that included geological mapping and a seismic refraction survey along the proposed center of the dam.
- The 2004 investigation was performed to characterize the embankment foundation area and borrow sources for rockfill and concrete aggregate. The investigation included drilling six geotechnical drill holes up to 105 feet in depth and totaling 519 feet in length.
- The 2010 investigation was performed during the construction of the Stage 1 Dam embankment. This investigation included geological mapping of exposed bedrock in the dam foundation, grout trench, and rock slope adjacent to the Stage 1 interim spillway. The investigation also included logging of rock chips and rock cores, and water testing during the grout curtain installation. The grout curtain construction included drilling a total of 155 vertical and inclined drill holes to depths ranging from 20 to 56 feet, a total footage of 6,279 feet.
- The 2011 investigation was performed prior to construction of the Stage 2 Dam to collect additional subsurface geotechnical and hydrogeological information at the abutments for design of the grouting program and to confirm rock quality for the proposed rockfill materials. This investigation included six geotechnical drill holes, both vertical and inclined, that totaled 554 feet in length.

Regional site geology was described to be comprised of a sequence of metavolcanic and metasedimentary rock of Jurassic to Cretaceous age that is part of the Taku terrane. The project site is near the Gastineau Shear Zone that runs across the peninsula from the Lynn Canal to Berners Bay and along the alignment of Sweeny and Slate Creek with a northwest/southeast strike (sees Figure 1).

The investigations found overburden overlying the bedrock to be up to 5 feet thick at the dam abutments and up to 35 feet thick along the valley bottom. The overburden was composed of sand, gravel with boulders and cobbles, and fine-grained glacial deposits. The weathered bedrock layer was found to be about 7 to 10 feet thick on the steeper abutment slopes during the 2004 investigation, but was not found in the valley bottom. Weathered bedrock was found to be about 20 to 50 feet thick at the Stage 2 Dam abutments in 2011 with the depth to competent bedrock ranging from 28 to 64 feet presumably from the ground surface.

Bedrock at the site was classified as low-grade slate with quartz veins mostly a few millimeters thick during the 2004 investigation, but was found to be primarily composed of phyllites, chloritic phyllites, and mixed phyllites including carbonite quartzite in the dam foundation area during the 2010 and 2011 investigations. The general structural fabric of the bedrock had a northwest/southeast strike and dipped to the west. Graphitic slickensides on the shear planes were found to be parallel to the strike direction.

The average compressive strength of the bedrock in the foundation and borrow area was estimated to be 5,000 to 8,000 pounds per square inch (psi), which correlated to a weak to medium strength. Rock Mass Rating (RMR) values collected at 17 locations during the 2011 investigation ranged from 50 to 72 with the graphitic unit having the lowest RMR.

The permeability of the bedrock was found to range from 0 to 22 Lugeon ($22x10^{-5}$ centimeters per second) during the 2004 investigation, and was generally found to be 0 to 2 Lugeon during the 2010 and 2011 investigations. Higher permeabilities up to 100 Lugeon were encountered at eight drill holes during the grout curtain construction in 2010 (see Figure 4).

2.1.2 Seismic Hazard Analyses

The seismic hazard analyses performed included both deterministic and probabilistic methods. The results of these analyses were used to develop the design earthquakes.

The deterministic analysis calculated maximum accelerations for the 50th percentile (median) and 84th percentile, which was associated with the Maximum Credible Earthquake (MCE). Four of the most prominent seismic sources in the southeast Alaska region were considered for shallow crustal events (Fairweather-Queen Charlotte Fault, Chattham Strait Fault, Southeast Denali Fault System, and the Glacier Bay/Coast Mountains Faults) and the Alaska-Aleutian mega-thrust subduction zone. Four ground motion attenuation relationships, known as the New Generation Attenuation (NGA) relations published by Earthquake Spectra in 2008, were used to predict the maximum acceleration values for the shallow crustal quakes. The maximum accelerations for the shallow crustal events were calculated using the average of the four attenuation relationships (equal weighting). The average of two other published attenuation relationships were used to estimate the maximum acceleration for the interface subduction earthquake. Based on this analysis the MCE was determined to be a Magnitude 7.0 event located on the Chatham Strait Fault, only 5 miles away from the project site, with a maximum acceleration of 0.51g. However, a maximum acceleration of 0.6g was conservatively used based on previous design studies that used attenuation relationships published in 1997.

The probabilistic seismic hazard analysis used published seismic hazard maps for Alaska from the United States Geological Survey (USGS) to determine maximum horizontal acceleration values for return periods from 108 to 4,975 years. The maximum acceleration for a 475-year return interval was 0.128g, it was 0.255g for a 2,475-year return interval, and it was 0.326g for a 4,975-year return interval.

The Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) design events were determined using the criteria provided by in the 2005 Alaska Dam Safety Program Guidelines. A Class I (high) hazard classification potential was assumed due to the environmental impact of dam failure. Based on these criteria, the 475-year return interval event with peak ground acceleration of 0.13g was selected for the OBE with an assumed conservative 8.0 Magnitude (M8). The MCE was selected to represent the MDE with a maximum bedrock acceleration of 0.6g and assuming a 7.0 Magnitude (M7) event. Consideration of the Southeast Denali Fault System and Fairweather-Queen Charlotte Faults were also to be included in the seismic design due to their potential to generate large magnitude earthquakes (greater than 8.0 Magnitude or M8+) and associated long duration shaking.

2.1.3 Hydrology

2.1.3.1 Historical Regional and Site Specific Data

Hydrometeorological values were mainly developed from historical climate and flow data from the region, but also included site specific stream flow data on Lower Slate Creek and Johnson Creek collected since 2007. This historical regional and site specific data included:

- Sporadic precipitation measurements collected from 1995 to 1997 at the Jualin Camp just north of the Slate Creek catchment, which included only one full year of data in 1997 (total of 55.6 inches of precipitation).
- Three months of Slate Creek discharge measurements (July through September) collected during the summer of 2000 at a station downstream of the Slate Lakes and at a station upstream of the Lynn Canal. Unit area runoffs from July to September 2000 were from 4.7 to 8.8 inches downstream of Slate Lake and from 3.0 to 6.9 inches upstream of the Lynn Canal.
- About four years of discharge measurements at Slate Creek (July 2008 through 2011) and over four years at Johnson Creek (January 2007 through March 2011). The highest annual runoff was about 89 inches at Slate Creek and about 152 inches at Johnson Creek.
- About three years of sporadic precipitation and temperature measurements collected at the Slate Creek Lakes site from June 2010 to February 2012 with a full annual record during 2011. This data also included about one year of wind speed data collected from February 2011 to February 2012.
- Regional precipitation data from 13 sites in Southeast Alaska with collection intervals from 1943 to 2004. The highest mean annual precipitation was at a site in Juneau from 1949 to 1965 with about 92 inches of mean annual precipitation.
- Regional runoff data at seven creek sites in Southeast Alaska, most near or at Auke Bay, collected over a period from 4 to 30 years.

2.1.3.2 Mean Annual Precipitation and Runoff

The mean annual precipitation (MAP) for the TTF was estimated to be 80 to 85 inches by dividing the total annual runoff volume at Lower Slate Creek by the total contributing catchment area. Runoff at the TTF was estimated to be almost equivalent to the MAP assuming that an increase in precipitation with elevation is largely offset by runoff losses to evapotranspiration and deep groundwater. Although the precipitation measured at the site in 2011 (114.4 inches) was considerably higher than the estimated MAP, the site data was discounted and considered erroneous based on comparisons to stations at Auke Bay and Juneau Airport that had higher monthly precipitation values in 2010 and lower values from August 2011 onwards.

The precipitation distribution for the 85 inch MAP was estimated using the monthly precipitation patterns at Elder Rock that were considered representative for those at the TTF site. Based on this distribution the lowest monthly precipitation for the 85 inch MAP occurs in June (3.2 inches) and the highest occurs in October (16.1 inches).

Mean annual runoff at the TTF was estimated by reviewing comparable sites at Auke Creek and Lake Creek in Auke Bay that were about 30 miles from the project site and had similar drainage areas and basin elevation ranges. These sites had mean annual unit runoff values that were similar to their MAP. Therefore, the mean annual unit runoff for the TTF was estimated to be 85 inches, or equivalent to the MAP.

Runoff distribution was estimated by comparing the precipitation and runoff from Slate Creek with Auke Creek and other regional runoff data. Considering the short-term data available from the site, the data from Auke Creek was used to develop the runoff distribution. Based on this analysis, monthly rainfall runoff and snow runoff values were estimated. Snowpack accumulation was estimated to occur from October to March and contribute as runoff from April to August.

2.1.3.3 Peak Flows

Return period peak flow values were estimated from regional data by applying a power trendline to plotted log-log mean peak daily flows versus drainage areas. Two regional outliers were not included in this trendline, Auke Creek and Lawson Creek, which were justified based on lake attenuation or unrepresentative short available record periods. The mean peak flow at Johnson Creek does fit on the trendline, but the mean peak flow at Slate Creek was considerably lower, which was explained by lake attenuation, low elevation of the basin, and diversion of flows around the TTF. Factors for daily and instantaneous peak flows were also estimated for various return periods based on the regional data. Peak instantaneous flows were shown tabulated for the Upper Slate Creek Lake with an estimated peak design flow of 172 cubic feet per second (cfs) for a 2-year event and 626 cfs for a 200-year event.

These return peak flow values were also validated by KPL by comparing them to HEC-HMS modeling results using the 25- and 200-year storm events specified in NOAA Atlas 14 Volume 7, plus snow melt (KPL 2012C). The 200-year, 24-hour precipitation and snow melt values used for this analysis were 7.76 inches and 3.62 inches, respectively. Site specific temperature, precipitation, wind speed, and inflow data were analyzed and considered in the model development. Based on the modeling results, the peak daily design flows were matched assuming a runoff curve number (CN) of 60, but a CN of 80 was required to match the instantaneous peak design flows. These CN numbers were within and slightly above those recommended by the Soil Conservation Service (SCS) guidelines, which seemed to validate the regional flood frequency based values.

2.2 Dam Design

The Lower Slate Lake Dam design is primarily summarized in revision 4 of the KPL Detailed Design Report (KP 2012C), which includes aspects of the Stage 1 Dam design with revisions to account for the Stage 2 Dam raise design and responses to Alaska Dam Safety review comments. The GFRD was designed to be constructed in three stages to spread out costs during operations. The initial Stage 1 Dam was constructed to a crest elevation of 690 feet and the final third stage of the dam is planned with a crest elevation of 740 feet. The planned section of the dam showing these three stages is presented in Figure 5.

2.2.1 Embankment Design

The embankment materials were specified to meet the following criteria:

- Zone A Rockfill – Durable and reasonably well-graded rock that is 24-inch-minus with maximum 40 percent passing the U.S. #4 sieve size and maximum 5 percent passing the U.S. No. 200 sieve size
- Zone A1 Rockfill – Similar to Zone A Rockfill except the material was to have maximum 25 percent passing the U.S. #4 sieve size, and maximum 16 percent passing the U.S. #16 sieve size
- Zone D Drain – Durable gravel and sand material that is 3/8-inch-minus with maximum 15 percent passing the U.S. #100 sieve size and maximum 5 percent passing the U.S. No. 200 sieve size
- Zone F Filter – Durable gravel and sand material that is 2-inch-minus with maximum 20 percent passing the U.S. #16 sieve size and zero percent passing the U.S. No. 30 sieve size

These materials were evaluated along with the tailings to meet the filter criteria recommended in the National Engineering Handbook published by the U.S. Department of Agriculture National Resource Conservation Service.

The CPT embankment drainage pipes were sized for approximately 70 percent full flow under the assumption that their size would be reduced when buried. For the Stage 2 Dam design the 8-inch CPT drains within the Zone D filter layer would be extended from their Stage 1 Dam position.

2.2.2 Stability Evaluation

2.2.2.1 Seepage Analyses

The seepage analyses performed for the Stage 2 Dam design was done using the software program SEEP/W by GEO SLOPE International. The analyses considered the operational and post-closure cases for both Stage 2 and Stage 3 Dam design sections at the abutment and at the main (highest) section. The main section included the foundation drainage zone (Zone A1 material) and assumed the seepage collection sump was not operational. Both main and abutment sections also included a drainage zone on the downstream face of the Stage 2 and Stage 3 Dam sections. Each case considered scenarios where the geomembrane was functioning as designed or if the geomembrane had degraded and was not functioning. Additional assumptions for the Stage 2 case were an upstream water elevation of 709 feet and a minimal tailings elevation of 675 feet. The Stage 3 case assumed the maximum tailings elevation of 715 feet and lake elevations of 724 and 732 feet for the operational and post-closure cases, respectively.

Material hydraulic conductivity values were estimated from consolidation testing results (tailings), gradation using the Krumbein-Monk formula (dam embankment materials), and published literature (overburden, grout, and geosynthetics) (see Figure 6). Each analysis also subdivided the rock foundation into two layers to represent potential open joints and discontinuities in the shallow layer, and tight joints and discontinuities in the deeper bedrock. The bedrock layer thicknesses and hydraulic conductivity values for the foundation were estimated based on the Lugeon testing.

The post-closure scenario seepage analyses results for Stage 2 (geomembrane degraded) section is shown in Figure 6. The seepage results for the Stage 2 section estimated a total of about 1 gallon per minute (gpm) for the operational case (geomembrane functional) and about 3,400 gpm for the post-closure case (geomembrane degraded). The seepage exit gradient for each scenario was less than 0.1, which provided a factor of safety over 10 assuming a critical seepage gradient between 1 and 1.2.

The seepage analyses results for Stage 3 section estimated a total of about 1 gpm for the operational case and a total of 4,900 gpm for the post-closure case. Seepage exit gradients for Stage 3 were similar to Stage 2, except the exit gradient was slightly higher (less than 0.2) for the post-closure scenario at the abutment. Seepage for Stage 3 post-closure scenario was also estimated using simplified hand calculations that were found to be in agreement with the computer generated results.

2.2.2.2 Slope Stability

The static and seismic (pseudo-static) slope stability analyses were performed for the final Stage 3 Dam height considering both operations and post-closure scenarios to assess the potential worst-case conditions. The static slope stability analyses considered scenarios for pre-earthquake conditions and after an earthquake (post-liquefaction), where a decrease in tailings shear strength (residual strength) would exist during post-liquefaction conditions. This static slope stability analyses for post-liquefaction is reasonable considering that it takes several cycles of earthquake loading to mobilize liquefaction, the dam is constructed in the downstream direction with none of the dam embankment founded over tailings, and that the seismic loading would be negligible by the time the liquefaction was fully mobilized.

Similar to the seepage analyses, the stability analyses assumed the geomembrane liner was functioning as designed or had degraded, and the seepage collection sump was not operational. The tailings and water levels were also varied between scenarios to account for possible worst-case conditions. The material strength properties were modeled based on Mohr-Coulomb failure criterion and published shear and normal stress relationships (see Figure 7). The effective undrained shear strength of the tailings was modeled as a ratio to the effective normal stress with ratios of 0.25 without an earthquake and 0.05 following the earthquake (post-liquefaction). The justification for these undrained shear strength assumptions is not provided in the design documents reviewed, but they are within the range of published information (Martin, et al. 1999).

The slope analyses were done using the software program SLOPE/W by GEO SLOPE International with factors of safety computed using Spencer's method of analysis to satisfy both moment and force equilibrium. Pore pressures for the different scenarios were imported from the SEEP/W results. The maximum bedrock accelerations for the OBE (0.13g) and MDE (0.6g for M7 event) were amplified by a factor of 3 and 2 at the dam crest for the OBE and MDE, respectively, to account for potential increased ground motions within the dam embankment. An additional M8+ event that occurred at a greater distance than the MCE (as described in Section 2.1.2) was also analyzed that had a maximum bedrock acceleration of 0.21g and was amplified by 3 to estimate maximum acceleration at the dam crest. These assumed crest acceleration amplification values appear to be reasonable compared to other published values (USACE 2000, Yu, et al. 2012). The yield acceleration (acceleration that would achieve a factor of safety of one for the failure surface) was also determined for each scenario to estimate associated deformations.

The results of the slope stability analyses are summarized in Figure 7 that also shows an example of the modeled section and downstream failure surface for the post-closure scenario. The factors of safety values were the same for the static and post-liquefaction condition for each scenario, which indicated the failure surface was not controlled by the tailings condition as the critical potential slip surfaces did not pass through the tailings. The factor of safety values ranged from 1.58 to 2.66; therefore, they were all greater than the minimum factor of safety for static pre-earthquake (1.5) and post-liquefaction (1.1). In contrast, the factor of safety values for the seismic stability scenarios were reported to be all less than one, but the evaluation actually only considered deformations and not the factor of safety.

Deformations resulting from these seismically induced failure surfaces were estimated using the 1977 Makdisi-Seed method and the assumption that the average maximum ground acceleration was 50 and 85 percent of the maximum crest acceleration for the deeper downstream failures and shallow upstream failures, respectively. Based on this analysis, the estimated seismic deformations during operations would be 1 foot or less and 3.3 feet or less following closure. This magnitude of movement was considered acceptable since they would be less than the minimum operating freeboard.

2.2.3 Hydrology and Hydraulic Evaluation

2.2.3.1 Design Storms

The 1,000-year storm event was selected as the Inflow Design Flood (IDF) during operations. The IDF hydrographs were developed during the Stage 1 Dam design using similar methods as the return period peak flows (Section 2.1.3.3), except they were done using a logarithmic trendline to initially estimate peak flows from the regional data and using the software program HydroCAD to develop the hydrograph to produce these flows. This exercise assumed a CN of 90, a time of concentration of 60 minutes, and a Type 1A 24-hour rainfall of 14.7 inches to generate the hydrograph and replicate the 1,000-year storm peak flow into Upper Slate Lake, which was about 665 cfs.

The Probable Maximum Flood (PMF) was assumed for the IDF during closure. The probable maximum precipitation (PMP) was developed using a published MAP-PMP relationship chart and isohytal map of 24-hour, 10-square mile PMP (rainfall plus snowmelt) in the 1983 U.S. Department of Commerce's "Hydrometeorological Report No. 54, Probable Maximum Precipitation and Snowmelt Criteria for Southeast Alaska (HMR-54)." This information was used to establish a PMP-elevation relationship for the region and adjusting the PMP values for the orthographic conditions of Sherman Creek. The weighted average 24-hour PMP for the 1,000 elevation was estimated to be 17.26 inches. The PMP value for Sherman Creek was selected for the Slate Creek Lakes basin because of its proximity and similar physical characteristics. Snowpack, temperature, and wind speed criteria for the PMF at the site were also developed using HMR 54 along with other references to relate these values to snowmelt water equivalence. The probable maximum snow melt was estimated to be 8.0 inches and distribution over the 24-hour storm was assumed to be 15 percent during the first 6 hours, 45 percent over the next 6 hours, 25 percent over the third 6 hour period, and 15 percent over the remaining 6 hour period.

The PMF was modeled using HEC-HMS assuming no diversion ditches, a time of concentration of 30 minutes, a CN of 85, and starting flows of 64 to 76 cfs for the two lake catchments. Based on this exercise, the inflow hydrograph into the Lower Slate Lake had a peak flow of 1,256 cfs at a time of about 10 hours. The results of this modeling exercise was described by KPL as being sensitive to the storage coefficient selected and was considered conservative with the input parameters used and particularly with the PMP occurring at the same time as the probable maximum snow melt, in addition to ignoring the diversion ditches.

2.2.3.2 Stage 2 Spillway Sizing

The IDF routing for the Stage 2 Dam and spillway was also done using HydroCAD, the same input parameters to develop the 1,000-year storm of 14.7 inches, and the following assumptions:

- The outlet of the Upper Slate Lake (intake into the TTF) was modeled as a broad-crested rectangular weir that is 20 feet long and 4 feet in breadth
- The dam spillway was modeled using the Manning Equation assuming a 0.35 percent slope, a 10 foot base width, 0.5H:1V side slopes, and a Manning roughness coefficient of 0.015
- The initial water levels prior to the flood routing was at the intake weir invert (740 feet elevation) and the bottom of the spillway (709 feet elevation)
- The diversion ditches were assumed to be damaged and not functioning

The results of the flood routing exercise indicated a peak outflow of 475 cfs from Upper Slate Lake (at 10 hours), a peak inflow of 893cfs (at 9.3 hours) into the TTF, and a peak outflow of 537 cfs (at 12.6 hours) at the spillway. The high water elevation at the spillway during the peak flow was about 713.1 feet, which provides about 2 feet of freeboard below the dam crest. The maximum spillway capacity with water at the dam crest elevation is 1,020 cfs.

The spillway channel was designed with a 26 percent slope down to the plunge pool. During peak flow the water level in the drop spillway was calculated to be about 1 foot deep. The plunge pool and baffle block size was determined using methods outlined in the Design of Small Dams by the US Department of the Interior, Bureau of Reclamation. The estimated tailwater height of 12 feet was based on the 50 feet per second (fps) flow velocity in the drop spillway and calculated Froude Number of 8.

2.3 Dam Construction

2.3.1 Stage 1 Dam

The Stage 1 Dam was constructed to a crest elevation of 690 feet by Alaska Interstate Construction (AIC) between September 2009 and August 2010 (KPL 2011). Construction supervision and quality assurance (QA) was performed by KPL.

Key items completed during this construction effort included the following:

- Construction of access roads, diversion ditches, and pipelines
- Excavation of the dam footprint and a majority of the Stage 1 interim spillway down to bedrock
- Concrete and shotcrete placement at the grout trench and dam foundation

- Placement and compaction of embankment rockfill, drainage zones, and filter zones
- Installation of the embankment drainage pipeline system and seepage collection sump
- Installation of eight vibrating wire piezometers within the Zone A1 embankment foundation and four vibrating wire piezometers within the Zone A embankment fill
- Construction of the grout curtain along the upstream embankment toe
- Installation of the geotextile and geomembrane liner along upstream embankment face
- Installation of pipework and appurtenances
- Shotcrete and concrete placement for the Stage 1 interim spillway and plunge pool

The work was reported to have been done in general compliance with the drawings and technical specifications with minor modifications executed during construction to suit site conditions. All modifications were technically approved by KP and accepted by CAK. Alaska Dam Safety approved significant design changes, which were not included in the documents Golder reviewed.

2.3.2 Stage 2 Dam

The Stage 2 Dam was constructed by CAK between June 2012 and October 2012 (KPL 2012H). The construction raised the dam embankment 25-feet to a new crest elevation of 715 feet. Quality control (QC) of the earthworks and concrete was performed by MAPPA Test Labs. Northwest Linings and Geotextile Products, Inc. performed QC of the geosynthetics. QA was performed by KPL to assure the construction was performed in general accordance to the design drawings and specifications.

Key items completed during the Stage 2 Dam construction included the following:

- Foundation preparation that included bedrock exposure within the footprint near the abutments and preparing the Stage 1 Dam embankment crest and downstream face for tie in of the new embankment materials
- Concrete and shotcrete placement at the grout trench and dam foundation
- Extension of the grout curtain along the upstream embankment toe at the abutments
- Placement and compaction of embankment rockfill, drainage zones, and filter zones
- Extension of the 8-inch CPT drains within the Zone D filter layer
- Extension of the grout curtain along the upstream embankment toe at the abutments
- Installation of the geotextile and geomembrane liner along Stage 1 Dam crest bench and the raised upstream embankment face
- Excavation of the Stage 2 interim spillway and plunge pool into bedrock
- Shotcrete and concrete placement for Stage 1 interim spillway and plunge pool

The Stage 2 Dam plan view, section, and liner details from the as-built drawings are shown in Figures 8 and 9.

The work was reported to have been done in general compliance with the drawings and technical specifications with minor modifications executed during construction to suit site conditions. All modifications were technically approved by KPL and accepted by CAK. Alaska Dam Safety approved significant design changes, which included the following:

- The Stage 2 interim spillway alignment was shifted to the west and deeper into competent bedrock to reduce sliver slope excavations, improve work safety, and facilitate the required rock excavation. The design spillway grades and cross-section geometry were maintained, so there was no need to redo the hydraulic analysis.
- The upstream toe at the left (east) abutment was realigned to coincide with the as-built Stage 2 grout trench. This realignment required about 60 feet of the upstream face to be constructed steeper than the designed 2H:1V slope with the steepest realigned slope section approximately 1.2H:1V. In addition, the as-built Stage 2 grout trench was only completed to an elevation of 710.5 feet, which is 1.5 feet above the spillway invert. The grout trench had to be terminated prematurely due to conflict with the exiting flume. The Stage 2 Construction Completion Report (KPL 2012H) does not provide details as to how the liner was completed above 710.5 feet at the left abutment, but field photographs seem to indicate it was left loose and not keyed in (see Figure 10).
- The Stage 2 plunge pool outlet ended just downstream of the baffle blocks with a near vertical face due to limited space between the pool and East Slate Creek. The as-built plunge pool was deemed acceptable for its purpose even though it was not constructed according to design.
- The Stage 2 excavation and foundation treatment near the temporary WTP downstream of the dam could not be completed to the design limits.

Recommendations were made by KPL to correct issues related to the east abutment grout trench, plunge pool, and excavation and foundation treatment near the temporary WTP during the Stage 3 Dam construction.

2.4 Water Management

A water management plan was developed for the TTF to provide guidelines for control of all the water originating in, or brought into, the Lower Slate Lake catchment (KPL 2012D). The water management plan was based on the layout and design of the project facilities as presented in the 2012 KPL Detailed Design Report (KPL 2012C). As part of the water management plan, the overall, site wide water balance was modeled by Golder Associates Inc. (Golder) using the Monte Carlo simulation software GoldSim. A copy of the updated water balance model results by Golder dated December 3, 2012 was included within the appendices of the 2012 KPL Water Management Plan Report. This 2012 water balance model was updated again in 2014 (see Section 9).

The updated 2012 Golder water balance model was calibrated using the monitoring data collected from January 2012 to November 2012. The range of predicted lake elevations of the TTF were forecast using WTP discharge rates of 1,000 and 1,500 gpm. These forecast levels were plotted against the maximum lake water level that would still contain the 200-year storm below the spillway crest elevation (697.3 feet for the Stage 2 Dam interim spillway). The 200-year storm containment elevation was based on 2010 stage-storage capacity curves, the Stage 2 spillway invert elevation of 709 feet, and the 200-year storm volume of 25,100,000 cubic feet.

Based on the results of this updated water balance model, the maximum predicted lake level at a water treatment rate of 1,000 gpm would exceed the lake level required to contain the 200-year storm during the fourth quarter of 2016. The water balance model forecast that the maximum predicted lake level would not exceed the maximum 200-year storm containment elevation at a water treatment rate of 1,500 gpm. The mean and minimum predicted lake levels, which were very similar, were forecast to remain below the 200-year storm containment elevation. The mean predicted lake level is based on the average of the 100 iterations or realizations used for the model; therefore, the majority of the realization results for each time step are very similar to the minimum predicted value.

3.0 INSPECTION HISTORY

Based on the documents provided for review, the inspection history of the dam includes the first PSI performed by KPL in 2011 and inspections done by mining personnel in general accordance with the Operations and Maintenance (O&M) Manual, which includes daily, weekly, quarterly and an annual inspection and performance report performed by KPL in 2013 (KPL 2013). The annual inspection was not performed in 2012 during the Stage 2 Dam construction.

3.1 2011 PSI

The first PSI was performed by KPL on the Stage 1 Dam on June 28, 2011 (KPL 2011). During the field inspection the dam embankment, upstream liner, spillway, and downstream area were observed to be in good condition. Some signs of seepage were noted out of the shotcrete lined slope above the spillway channel with oxidation stains observed, but no flowing water. The Upper Slate Lake intake diversion structure, surface diversions, reservoir, reservoir slopes, and roads were also observed to be in "good condition."

The monitoring data review included the dam embankment and foundation piezometer data, seepage collection pump rate data, reclaim barge pump rate data, Parshall flume data, stream flow data, and weather monitoring from about January 2010 to June 2011. Highlights from the monitoring data review include the following:

- The eight piezometers in the dam foundation fluctuated in response to the pumping of the HDPE manhole sump, but the four piezometers within the embankment showed no response to the pumping as the piezometric water level was maintained below the four embankment piezometer elevations. The foundation water level exceeded the trigger elevation of 645 feet only once that was attributed to a freezing of the seepage pump pipeline during the winter and temporary shutdown of the pump system.
- Daily seepage pump rates were generally within the range of 0 and 250 gpm, but were between 350 to 450 gpm five times. The higher pump rates were attributed to higher seepage volumes while tailings were being deposited into the area between the cofferdam and dam embankment during early stages of operations, and periodic seepage pump back system shutdowns during later stages of monitoring period.
- Daily reclaim pump rates were generally within the range of 150 to 350 gpm.
- Daily diversion water flow rates in the Parshall flume were generally within the range of 1,000 to 4,000 gpm. Higher rates observed were attributed to periods of higher precipitation.

The 2011 PSI report concluded that there were no dam safety issues of any significance. However, the monitored lake levels were higher than anticipated compared to the original predictions. These higher lake levels were attributed to limited discharge of excess water from the reservoir due to water quality parameters and effluent discharge permit requirements (low TTF WTP rates). Water balance calculations indicated that the water level could return to the original predictions in the future provided the water treatment discharge rates met necessary requirements. Therefore, as a contingency to avoid encroaching on storm storage freeboard provisions the Stage 2 Dam raise construction was to be completed in 2012 instead of in 2013 as originally planned. The water treatment discharge rate requirements to return the lake level to the original predictions were not included in the 2011 PSI report.

Recommendations included installing a collection system to divert runoff from the access road near the left (east) abutment so that it does not flow over the downstream portion of the dam. Recommendations were also made to install a heat-traced, self-draining pipeline for the collection pump system to prevent freezing during winter conditions. The report noted that CAK had already planned to perform these recommended improvements.

3.2 Operations and Maintenance Inspections and Data

3.2.1 Daily, Weekly, and Quarterly Inspections

The daily, weekly, and quarterly inspection forms for the TTF from December 2013 to July 2014 were provided at our request and briefly reviewed. Our review mainly involved looking for written notes or other recorded information during periods of low or high measurements in the monitoring data (see Section 3.2.3). Daily, weekly, and quarterly inspection forms prior to December 2013 are available, but were not requested for our review as much of the information is already included in the monitoring data and was reviewed as part of the 2013 annual inspection.

3.2.2 Annual Inspection

An annual inspection of the TTF was performed by KPL in 2013 that included a visual inspection on August 7, 2013, a review and evaluation of routine inspection and monitoring reports, a summary and review of the monitoring data, a tailings survey, a review of water management, and conclusions and recommendations (KPL 2013). Highlights of the visual inspection observations are described below as well as conclusions and recommendations from the inspection. The monitoring data reviewed by KPL is also described in the following section (Section 3.2.3). The water balance was supposed to be issued as a separate document, but never was.

Similar to the 2011 PSI report, the intake diversion structure, surface diversions, reservoir, reservoir slopes, and roads were observed to be in “good condition” and did not require remedial action or additional work. Groundwater seepage from above the east side of the dam access road was observed flowing down the east grout trench and into the reservoir. A geomembrane flap was installed in this area to reduce erosion of the dam access road, as recommended in the 2011 PSI (see Section 3.1). Very minor seepage was also observed downstream of the Stage 2 crest at the contact between the Stage 2 Zone A material and the Stage 1 interim spillway. Seepage-related oxidation staining was observed on the spillway concrete lining, as well as some minor shrinkage related cracking that did not appear to affect the integrity of the lining.

A review of the water management noted that the mine filling schedule projected construction of the Stage 3 dam in 2019, but unless the treatment rate was increased construction of the Stage 3 dam may be required as early as 2015. In addition, if the surplus water in the TTF was not removed the designed Stage 3 dam elevation would not provide sufficient storage of tailings solids for the remaining life of the mine. The reason for the excess water was described to be related to effluent discharge permit requirements and water quality parameters limiting the pumped discharge of excess reservoir water at certain times.

3.2.3 Monitoring Data

Monitoring data collected at the site that was reviewed included the piezometer data, seepage pump rate data, lake levels (supernatant pond level), reclaim pump rate data, diversion flume data, flow data at Johnson Creek, air temperatures, precipitation data, and an annual tailings survey. This monitoring data is summarized in the following subsections.

3.2.3.1 Piezometer Data

The 12 vibrating wire piezometers installed in the dam embankment are used to monitor the piezometric surface during operations and their locations are shown in Figure 11. There are eight piezometers installed in the foundation (P-F-01 through P-F-08) and four piezometers installed in the dam embankment (P-E-01 through P-E-04). The foundation piezometers have a trigger elevation of 645 feet and the embankment piezometers have a trigger elevation of 665 feet. The trigger elevation indicates that “the significance of piezometer readings for dam stability should be checked” (KPL 2012F), which has been translated by CAK to mean that the cause of the high piezometer readings need to be investigated to determine their cause and then addressed as necessary. The trigger elevations are a warning of possible dam instability, but the root cause of the high readings need to be investigated first to determine what further actions are warranted. These piezometers are typically read on a weekly basis from a readout panel near the seepage collection sump. The data from the piezometers are summarized in Figures 12 to 14, and the piezometer tip elevation is shown in parentheses adjacent to the piezometer number.

Figure 12 shows the data from the four foundation piezometers installed near the 12-inch CPT outlet drain, P-F-04 and P-F-06 to P-F-08, which are shown and listed in the order from the upstream toe (P-F-04) to downstream near the seepage collection sump (P-F-08). Figure 13 shows the data from the other four foundation piezometers (P-F-01 to P-F-03 and P-F-05) that were installed away from the outlet drain and typically at higher tip elevations. As shown in Figures 12 and 13, since the 2011 PSI there have been four to five occasions (three in December 2013 and one each in January and February 2014) where the piezometric surface was above the trigger elevation of 645 feet. The piezometric surface was quickly dropped below the trigger elevation once the pumping system was in operation again, which indicates the piezometric surface was sensitive to the sump operation. A review of this data also indicates that outlet drain area is generally saturated during operations (piezometer level above the piezometer tip elevation), and the piezometric surface in the dam foundation is typically maintained below an elevation of 635 feet.

Based on our review of the weekly inspection reports, these high piezometer readings after 2011 may have been due to freezing issues at the seepage collection sump and pumping system, similar to what was described in the 2011 PSI report. We understand that CAK has installed heat traced, insulated pipelines for the seepage collection pumping system in the fall of 2012 to prevent future freezing of the discharge pipe. One freezing event did occur following installation of the heat traced, insulated pipe because the discharge end of the pipe was submerged in the lake, which froze when the lake froze. The pipe length has since been shortened to keep the discharge end above the lake level. The remaining high piezometer readings were due to operations and turning off the discharge pump. This pump shut down was done to use the sump area as a water source for the temporary WTP when the drainage from the pyrite bearing phyllite stockpile was not high enough to keep the WTP operating. This practice was recognized as an issue and has since been discontinued. Water to feed the temporary WTP is now brought from another source when necessary, and the temporary WTP will be replaced with a new WTP in 2015.

Further review of these high readings for the eight foundation piezometers indicate the highest levels occurred at Piezometer P-F-01 (elevation 652.2 to 646.1 feet) that was typically only 0.1 to 0.2 feet higher than P-F-05 and P-F-06. The lowest readings were at P-F-03, P-F-04, P-F-07 and P-F-08 that were all about 1 to 1.3 feet lower than at P-F-01. The four embankment piezometers with higher tip elevations did not show a change during these events (see Figure 15 and paragraph below), which means the water level was maintained within the foundation. Scaling off the available figures, the highest gradient between these foundation piezometers was about 0.01 feet/foot, which is very low and does not appear to pose a concern for instability when compared to the results of the static stability analyses. Groundwater levels within the embankment are expected to rise to the level of the discharge point from the sump after closure, which has been assumed to be at an elevation of 650 feet (KPL 2012C).

Figure 15 shows the data from the four embankment piezometers, which never went over their trigger elevation of 665 feet and are typically in the dry, which should not be a concern for these vibrating piezometers. Since the 2011 PSI, Piezometer P-E-03 did show a spike in the readings on April 26, 2012 (elevation of about 658 feet), but it was not reflected in the other piezometers that did not go above an elevation of 635 feet (P-F-04); therefore, the spike in P-E-03 is considered erroneous.

3.2.3.2 Seepage Collection Pump Rate and Lake Level Data

The seepage collection pump rate and lake levels are typically collected on a daily basis, except on rare occasions when access is impeded by snow depth or personnel are not available. The seepage pump rate readings, total flow and pumping rate at time of reading, are collected from an enclosed readout panel near the temporary WTP. The average daily flow rate is calculated based on the total flow and time difference between readings. The lake level is read from a staff gauge near the reclaim barge, which is occasionally resurveyed.

The seepage collection pump rate and lake level data is shown in Figure 15, which also shows the piezometer data for P-F-08 that is located near the sump. Similar to what was observed in the 2011 PSI report, the seepage pump rates tend to range between 0 and 250 gpm. The occasional spikes in the pump rate above 250 gpm are attributed to freezing pipes and pump shutdown.

As shown in Figure 15, the linear trendline for the seepage pump rate data, which is very scattered with an r-squared value of about 0.02, appears to show an increasing trend with time similar to the increasing lake level. The slope of linear trendlines for both the lake level and the seepage pumping rates are very similar looking for the data since about June 2011.

3.2.3.3 Reclaim Pump Rate Data

The reclaim pump total flow is collected on a daily basis from a readout panel in the mill control room. The average daily pump rate is calculated as the volume of flow over time between readings. The reclaim pump rate data is summarized on Figure 16 and shows that the daily pump rate has reduced with time. The reclaim pump rate averaged overall about 200 gpm since July 2010 and averaged about 163 gpm in the last year (August 2013 to August 2014). This reduction may be related to restrictions in the pipeline, as we understand the 50-HP pump in the reclaim tank was recently replaced with a 100-HP pump, but the flow rate did not increase substantially.

3.2.3.4 Diversion Flume Data

The diversion water flows are measured on a daily basis using a transducer and readout panel at the Parshall flume located near the east dam abutment. The Parshall flume is also connected to a readout at the mill control room. The Parshall flume measures flows collected from all the diversions, except the eastern diversion, and flows from the TTF WTP. The data collected includes the total volumetric flow and the flow rate at the time of the reading. Average daily flow is calculated by the volume of flow over the length of time between readings. The average daily flow rate data from the diversion flume is shown in Figure 17, which also includes the precipitation data. Precipitation data is collected from a weather station at the Jualin mine camp.

As shown in Figure 17, daily flow rates at the diversion flume have ranged between zero to over 8,000 gpm since about the time the Stage 2 Dam construction was completed in October 2012. Prior to 2012, the diversion flow rates varied within a range of about 1,000 to 6,000 gpm, similar to what was noted in the 2011 PSI report. The linear trendline for the diversion flume flows shown in Figure 17 also indicates a decrease with time, whereas the linear trendline for precipitation, although not shown, indicates a relatively flat trendline and little change with time overall.

There were 42 occasions in 2013 where the transducer data were recorded as a zero. Fifteen of these zero readings appear to be due to frozen conditions in March, but 17 of them were in August 2013 that had the same amount of rain as the previous month. Although there was no record in the data files to confirm, the low average annual daily flow rate for 2013 is attributed to issues with the transducer (assumed) or freezing conditions (noted in the some of the daily reports). Freezing conditions were also noted for the zero readings in March 2014.

3.2.3.5 Johnson Creek Data

Average daily flow rates and precipitation data is shown in Figure 18. The flow rate at Johnson Creek is monitored on a daily basis using a stream gauge to estimate depth, which is used to estimate flow rates. Based on a review of the flow rates, the rates estimated in 2013 and the last quarter in 2014 appear unseasonably high compared to past flows and similar precipitations. We understand the data from July 11, 2013 to December 31, 2013 in the database are an average of historical flows as it shows a very linear trend. We understand the transducer was not operating over this period and there were delays to discover the problem, order parts, and have it repaired.

3.2.3.6 Air Temperature and Precipitation

Air temperature and precipitation data is collected on a daily basis at the weather station and a summary of the data is shown in Figure 19. A linear trendline fit to the air temperature data indicates slightly increasing temperatures with time over the monitored period.

3.2.3.7 Tailings Survey

The 2014 tailings survey results, including the tailings bathymetry and cross sections through the lake, are shown in Appendix C. The estimated total volume measured on July 26, 2014 was 986,373 cubic yards. This was an increase of about 116,000 cubic yards since the last tailings survey on August 7, 2013 (KPL 2013). The measured rate of deposition is less than the predicted rate (see Figure 20), which may be related to increased storage of tailings underground as a cement paste.

4.0 FIELD INSPECTION

4.1 General Information

The field inspection was performed by Mr. Steve Anderson, P.E. of Golder on June 24, 2014 who was accompanied by Mr. Ed Coffland of CAK. The weather during the inspection was mostly cloudy with light rain. The air temperature was about 55 degrees Fahrenheit (ºF). Weather before the inspection was generally the same as during the inspection. Prior to the site inspection, the most recent O&M manual for the dam, dated December 7, 2012, was reviewed.

Photographs taken during the inspection are presented in Appendix A. During our visit, the Alaska Dam Safety Program's Visual Inspection Checklist was reviewed and filled out (see Appendix B). The Project Data Sheet was updated following our inspection, which is also included in Appendix B.

4.2 Visual Inspection Highlights

The field inspection included access to the reclaim barge, the intake structure, the WTPs, the access road and east diversions, the dam crest and downstream area, the interim spillway and plunge pool, and the water conveyance pipe outlet to East Slate Creek. Access to the diversion inlet north of the lake required a bear safety watch, so it was not observed. Highlights of our visual inspection with occasional reference to field inspection photographs in Appendix A are outlined below.

- The lake water level was at an elevation of about 692.5 feet and at the approximate limits shown in Figure 3. Photos 1 and 2 show a panorama of the lake from the north and south sides. The side slopes along the west side of the reservoir appeared stable with no signs of sloughing or past instability. There was no floating trees or debris in the lake or against the log boom (see Photo 2).
- Water was observed to be flowing from the Upper Slate Lake outlet culvert and the water level was about 6 inches above the invert in the culvert. The water level behind the intake structure was at an elevation of about 938.5 feet (Photo 4). The concrete intake structure appeared to be in good condition with no signs of degradation or spalling. The intake and sluice gate were not operated during our inspection, but they are exercised monthly by the water treatment operating personnel.
- The reclaim barge location has been moved from what is shown in the O&M manual due to the higher lake elevation (see Photo 2). The new location of the reclaim barge is shown in Figure 3. The access road to the north diversion inlet and the unsuitable stockpile has also been raised near the bend due to the higher lake level.
- The diversion Parshall flume was observed and water was seen flowing at a depth of about 10 inches above the invert of the water conveyance pipe outlet (see Photo 7). The transducer readout panel for the Parshall flume is shown in Photo 8. The dogleg and eastern diversion pipeline were also observed. The amount of water flowing out from the dogleg diversion could not be discerned as the outlet was mostly below the water line. The outlet of the eastern diversion (see Photo 7), which also discharges on the downstream side of the flume, was flowing at an estimated rate of about 10 gpm. Based on what was observed during the field inspection and a review of the diversion flow data, the diversion system appeared to be performing adequately.

- The dam crest, upstream face, and upstream abutments appeared to be in satisfactory condition (see Photos 9 through 14). No settlements or obvious depressions were noted along the dam crest. No bulging, displacements, irregularity, or indications of slope instability were observed along the upstream abutments and dam face. The geotextile over the geomembrane appeared to be floating in areas along the submerged bench that was the upstream Stage 1 Dam crest (see Photos 11 and 12). The geomembrane in this area cannot float because of the anchor trenches installed along the bench (see the liner tie in detail in Figure 9). No signs of settlement or depressions were observed along the dam crest. The geotextile along the crest anchor trench did not appear stressed or in tension.

  The upper part of the east abutment was covered with a geomembrane that is anchored with a concrete barrier along the access road, as shown in the construction field photographs in Figure 10. There was some rock debris from the slope that has washed into the eastern grout trench (Photo 12), but none was observed at the western grout trench (Photo 13).

- The spillway, drop chute, and plunge pool appeared to be in satisfactory condition with no signs of spalling or scaling of the concrete/shotcreted surface (see Photos 14 and 15). Similar to what was observed in the 2011 PSI and 2013 annual inspection, the shotcreted rock face along the western dam abutment and spillway showed signs of past seepage due to oxidation staining. Some minor seepage was observed along the western spillway slope and very shallow ponding was observed at the spillway near the top of the chute (Photo 14). Water was also seen flowing at a rate of less than 5 gpm down the spillway chute into the plunge pool (Photo 15).

- The downstream dam face and abutments also appeared in satisfactory condition with no indications of instability (see Photos 17 to 19 and 21). Some minor seepage was observed at the base of the west downstream abutment, which is likely flowing along the buried Stage 1 Dam interim spillway chute (Photo 21).

- The seepage collection sump, pump discharge pipeline, and readout panels for the piezometers and sump pump were observed (see Photos 18, 21, and 22 to 24). The sump pump was operating at a flow rate of 2 gpm according to the readout panel (Photo 22), but no water was observed flowing from the pipe outlet near the upstream west abutment (Photo 14).

- The plunge pool at the water conveyance pipe outlet into the East Slate Creek drainage was also observed (see Photos 26 and 27). The plunge pool appears to be fairly shallow and constructed with a geomembrane liner along the bottom and channel into the creek drainage. The geomembrane is held in place by boulders along the edge and in the channel to help dissipate energy.

5.0 OPERATIONS AND MAINTENANCE REVIEW

The O&M Manual was last revised (Revision 2) by KPL in December 2012 following completion of the Stage 2 Dam construction (KPL 2012F). The O&M manual includes an introduction and sections on the project description, responsibilities, routine observation and maintenance, instrumentation monitoring, unusual occurrence and response protocols, and certification. Figures include a project layout plan, the filling schedule and staged construction for the permitted design basis, the TTF layout during different development stages, dam sections during development stages, the water management systems and instrument readout locations, the Stage 2 interim spillway discharge curve, and the filling schedule and staged construction for the current mine plan. The O&M Manual also includes as-built drawings of the dam and appurtenances, and appendices that have manufacturer information on the monitoring instruments and operations equipment as well as the inspection and incident reporting forms.

Inspections are performed daily, weekly, quarterly, and annually. The daily and weekly inspections mainly involve recording data and looking for unexpected behavior and signs of dam instability. The quarterly inspection is recorded on the weekly inspection form and involves a more thorough inspection of the dam and water management systems with additional focus on maintenance requirements. The annual inspections include a bathymetry survey of the tailings solids, focused inspection and maintenance of the barge and piping systems, and a review of instrumentation monitoring by a Professional Engineer (PE).

The instruments and monitoring schedule are described in Section 3.2 of this report. Although noted as part of the monitoring duties in the O&M Manual, monitoring of the stream gauge at East Fork Slate Creek (downstream of the dam) has been discontinued as the Parshall flume and TTF WTP discharges combined essentially provide the stream flow data. A gauging station and transducer is maintained at the East Fork Slate Creek as a backup. Monitoring also includes updating the GoldSim site wide water balance model on a monthly basis with the precipitation, snow pack, and stream flow measurements for comparison to the predicted levels.

The O&M Manual indicates the maximum operating elevation for the Stage 2 Dam is at elevation 695 feet, which is 2.3 feet lower than what is used in the water balance (Section 2.4). We expect this O&M Manual value may include 2 feet of additional freeboard during operation, but this should be confirmed.

The filling schedule and staged construction for the current plan is presented in Figure 20 that also shows the actual lake water level with time, the surveyed tailings level for the last two years, the maximum elevation to contain the 200-year storm (697.3 feet), and the maximum elevation of the upstream east abutment grout trench. As shown in Figure 20, the tailings placement has occurred at less than the predicted rate, but the lake level is above the operating limits and is projected to reach the 200-year storm containment elevation near the end of March/beginning of April 2015 if it continues to rise at the linear trendline rate. This suggests that the planned Stage 3 Dam raise may need to be constructed in the next two years rather than summer of 2017 as originally planned.

The O&M Manual appears to be thorough and appropriate for this facility. However, the manual does not describe that the maximum elevation of the eastern upstream grout trench is 4.5 feet lower than the dam crest elevation. The O&M Manual also does not include the requirement for a PSI by a qualified professional engineer every three years in accordance to the Alaska Administrative Code 11 AAC 93.159(a) and the Class II hazard potential classification for the dam. In addition, the manual should be revised to reflect that monitoring of the stream gauge at East Fork Slate Creek is no longer performed.

6.0 EMERGENCY ACTION PLAN REVIEW

The initial emergency action plan (EAP) was issued in May 2011 and the most recent was revised for the Stage 2 Dam raise in December 2012 (KPL 2012G). The EAP includes:

- Notification flow charts
- Project description
- Emergency, detection, evaluation, and classification for non-failure emergencies, developing potential failure situations, and an imminent or actual failure in progress
- Preventive measures for overtopping, reduction in freeboard and/or loss of dam crest width, upstream and downstream slope slides, excessive seepage or leaking (piping) through embankment, and excessive embankment settlement
- General responsibilities under the EAP
- Preparedness such as emergency recognition, incident reporting, training and exercise, and supplies and resources
- Dam breach analyses and inundation maps for sunny day and flood stage dam breaks

According to the EAP, orientation and drill exercises are to be performed annually, tabletop exercises should be performed every three years, and functional exercise should be performed at the request of Alaska Dam Safety. We understand that CAK has conducted the orientation training annually from 2012 to 2014, and a drill exercise was performed in 2013. CAK has not conducted any table top exercises or functional exercises. According to the “Guidelines for Cooperation with the Alaska Dam Safety Program" (June 2005), functional exercises should be performed at the request of Alaska Dam Safety for Class I dams only; therefore, they are not necessarily required for this Class II dam. CAK is planning to perform a table top exercise on January 21, 2015, and Alaska Dam Safety should be provided written notice within seven days after the EAP exercise is completed.

A qualitative dam failure analyses was done for the dam breach analyses using the procedures outlined in the Washington State Department of Ecology Dam Safety Guidelines Technical Note 1, which is recommended by the June 2005 Alaska Dam Safety Program Guidelines document. The operating levels for the sunny day and flood stage dam breaks were 695 and 713 feet, respectively. The Fread3 equation was used to calculate the dam breach discharges with estimated maximum flow rates of 28,000 and 40,000 cfs for the sunny day and flood stage dam breaks, respectively. Downstream flooding was estimated using the Manning equation, nine cross sections downstream of the dam, and assuming zero percent attenuation. Cross sections and depth of flow are included for each break analyses and the minimum depth of flow would be 7 to 8 feet where the flood enters into Berners Bay. Flood velocities are not included.

Similar to the 2012 O&M Manual, the EAP appears to be thorough and appropriate for this facility, but it could be improved by noting the limitations of the upstream east grout trench under the project description section of the EAP. Preventive measures for excessive seepage through the embankment are included and these measures could be required if the lake level rose above the 710.5 feet elevation.

7.0 DISCUSSION OF KEY ELEMENTS OF DAM AND APPURTENANCES

The 2011 PSI report provides a good list of the key elements of the dam that are critical to dam safety. These are:

- The quality of the foundation and abutment slopes
- The structural integrity of the dam embankment materials
- The upstream geosynthetic liner, grout trench, and grout curtain
- The drainage system in the dam
- The emergency spillway
- The instrument systems used to monitor water levels within the dam, seepage pumping rate, diversion flows, and weather data

These key elements appear to have been addressed appropriately through the design, construction, and monitoring stages for this facility as described in the preceding sections.

8.0 REVIEW OF PERFORMANCE PARAMETERS

8.1 Hydrology and Hydraulics

The hydrology design parameters for the spillway and water balance were based on a review of site and regional data as well as synthetic analyses through modeling to validate peak flows, as summarized in Section 2.1.3 of this report. The regional and site specific data, analyses assumptions, and modeling input parameters and output results are included in the Hydrology (KPL 2012E) and Detailed Design (KPL 2012C) reports. Based on our review of these documents, the approach and analyses to develop the hydrology design parameters appears reasonable.

These design documents were also reviewed to assess the design storms and routing analyses to determine the spillway sizing. We also reviewed the online Precipitation Frequency Data Server (PFDS) available through the National Oceanic and Atmospheric Administration's National Weather Service (NOAA 2014) using the site coordinates. Based on this review, the online PFDS shows the 1,000-year, 24-hour storm is 9.84 inches with a 90 percent confidence interval and an upper bound of 13.1 inches compared to the 14.7 inches used for the design storm analyses (Section 2.2.3.1). This review indicates that design storm parameters are somewhat conservative, particularly since the flood route modeling assumed the diversion ditches were not operating during the floods and analysis of the outlet of Upper Slate Lake did not consider the culvert through the access road.

8.2 Stability

The permeability, density, and strength values for the materials used in the static (pre- and post-earthquake) and seismic (pseudo-static) slope stability analyses appear reasonable as well as the analyses methods for the various evaluation scenarios. According to the Federal Guidelines for Dam Safety (FEMA 2005), evaluation of seismic stability deformation using the 1977 Makdisi-Seed method is appropriate since the post-liquefaction (post-earthquake) factors of safety were significantly above one.

The design earthquakes assumed for the seismic slope stability analyses were reviewed using the dam coordinates (latitude 58.80732 degrees, longitude -135.0386 degrees) and the 2007 USGS probabilistic seismic hazard maps for Alaska (USGS 2014). The 2007 USGS probabilistic seismic hazard maps for Alaska give 0.171g and 0.419g PGA values for the 475-return and 2,475-return events, respectively. This review suggests that the OBE PGA used for design (0.13g) may have been underestimated for the seismic stability analyses, or the seismic hazard maps have since been revised. This higher OBE PGA of 0.171g is still lower than the yield accelerations, which ranged from 0.2g to 0.45g; therefore, the factor of safety is still above one with minor deformations. While the details of the deterministic analyses were not included in the reference documents reviewed, the MCE PGA appears to be appropriate. We recommend a detailed review of the seismic hazard deterministic and probabilistic analyses during the Stage 3 Dam including updated seismic data from the USGS.

9.0 UPDATED WATER BALANCE

As part of the 2014 PSI work effort, we reviewed the updated water balance prepared by Golder. The technical memorandum describing the updated water balance is attached in Appendix D. The updated 2014 water balance model was calibrated to the operational data between January and December 2013. The TTF WTP rate from August 2010 to July 31, 2014 is shown in Figure 5-2 of the 2014 Golder Technical Memorandum in Appendix D. This data indicates that the TTF WTP rate has ranged from zero to over 1,500 gpm with an average of about 600 gpm since operations began, but in the last year or so the average WTP rate has decreased to about 500 gpm.

Assuming the TTF WTP rate can be maintained at 1,500 gpm from August 2104 to January 2015 and follow the predicted TTF WTP rate thereafter (a range from about 200 to 1,400 gpm), the model forecasts that the mean lake level can be kept below the 200-year storm containment elevation and the currently planned stage construction can proceed (Figure 5-1 of Appendix D). If the TTF WTP rate can be maintained at 1,000 gpm the model predicts the mean and minimum lake levels will stay below the 200-year storm containment elevation, but the maximum predicted lake level will exceed the Stage 2 Dam 200-year storm containment elevation in the fourth quarter of 2014 (Figure 5-3 of Appendix D). The model predicts that if the TTF WTP rate can be maintained at 1,500 gpm the predicted maximum lake level will exceed the 200-year storm containment elevation during a short period during the fourth quarter of 2014 and first couple months of 2015 until it drops below the 200-year storm containment elevation (Figure 5-4 of Appendix D).

Similar to the results of the lake level trendline forecast results shown in Figure 20, the water balance modeling results indicate that if the TTF WTP rates cannot be kept at a minimum 1,000 gpm, the Stage 3 Dam raise will need to be constructed in the next several years to keep the lake level below the 200-year storm containment elevation, as required by permits. As a result of the rising lake level, we understand CAK plans to design the Stage 3 Dam in 2015 with construction in 2016.

10.0 CONCLUSIONS ON DAM SAFETY AND FUTURE PERFORMANCE

Based on our review of the design, construction, and operations documents as well as our site visit, the following conclusions can be made concerning the safety of the dam and its future performance:

- The dam crest, upstream face and downstream face, and abutments appeared to be in satisfactory condition during the field inspection with no indications of instability or settlement. The geotextile over the geomembrane appeared to be floating in areas along the submerged bench that was the upstream Stage 1 Dam crest, but it did not appear to be a cause for concern. Some minor seepage (about 5 gpm) was observed at the base of the right (west) downstream abutment with flows likely travelling on the buried Stage 1 interim spillway chute.
- The spillway, drop chute, and plunge pool appeared to be in satisfactory condition with no signs of spalling or scaling of the concrete/shotcreted surface. The shotcreted rock face along the western dam abutment and spillway showed signs of past seepage due to oxidation staining, and some minor seepage was observed along the western spillway slope that were flowing down the spillway chute.
- Based on our field inspection and review of the monitoring data, the diversion system and Parshall flume appear to be well maintained and are performing well. The water flows through the Parshall flume occasionally freeze during cold temperatures.
- Based on our review of the piezometric data, the piezometric surface in the dam foundation has risen above the 645 feet trigger elevation several times since the 2011 PSI and after CAK installed an insulated, heat traced discharge pipe for the seepage collection system to prevent freezing. The four higher embankment piezometers did not register a change during these foundation piezometer trigger events indicating the water level was maintained within the foundation materials only. The estimated highest gradient between the eight foundation piezometers during these trigger events were very low (about 0.01 feet/foot); therefore, they did not pose a concern for dam instability based on the results of the static stability analyses.

  The cause of these high piezometric levels in the foundation was mainly due to improper operation practices with one occurrence due to freezing of the submerged sump pump discharge pipe outlet in the lake. The improper operation practice involved shutting off the discharge pump and using the sump area as a water supply for the temporary WTP. This practice has been discontinued and another water source is found when necessary for the temporary WTP. The freezing sump pump discharge pipe outlet has been shortened so it will remain above the lake level. Once the sump pump and/or pump discharge pipe were operating again, the piezometric surface in the dam foundation was quickly lowered to normal levels.
- The O&M Manual appears to be thorough and appropriate for this facility. However, the manual does not describe that the maximum elevation of the eastern upstream grout trench is 4.5 feet lower than the dam crest elevation, and there appears to be a discrepancy between the maximum operating elevation of 695 feet in the O&M Manual and the 697.3 feet elevation used in the water balance. The O&M Manual also does not include the requirement for a periodic safety inspection by a qualified PE every 3 years in accordance to the Alaska Administrative Code 11 AAC 93.159(a) and the Class II hazard potential classification for the dam. In addition, the stream gauge at East Fork Slate Creek, immediately downstream of the dam, is no longer monitored as the stream flow is essentially equivalent to the discharges from the Parshall Flume and the TTF WTP discharge. CAK does maintain the gauging station and transducer at the East Fork Slate Creek as a backup.

- The EAP appears to be thorough and appropriate for this facility, but it could be improved by noting the limitations of the upstream east grout trench in the project description section of the EAP. Preventive measures for excessive seepage through the embankment are included in the EAP and these measures could be required if the lake level rose above the 710.5 feet elevation.
- The key elements of the dam and appurtenances appear to have been addressed appropriately through the design, construction, and monitoring stages.
- The hydrology design parameters for the spillway and water balance were based on a review of site and regional data as well as synthetic analyses through modeling to validate peak flows. Based on our review of the design documents, the approach and analyses to develop the hydrology design parameters appears reasonable. The methods used to develop the design storms and analyses used to design the spillway also appear to be reasonable and conservative.
- The permeability, density, and strength values for the materials used in the static (pre-earthquake and post-liquefaction) and seismic (pseudo-static) slope stability analyses appear reasonable as well as the analyses methods for the various evaluation scenarios. Based on our review of the design earthquakes using the USGS seismic hazard maps, the OBE PGA has increased to 0.171g compared to 0.13g used in seismic stability design. This higher OBE PGA of 0.171g is still lower than the yield accelerations, which ranged from 0.30g to 0.45g for the operations case; therefore, the factor of safety is still above one with minor deformations. While the details of the deterministic analyses were not included in the reference documents reviewed, the MCE PGA appears to be reasonable.
- The lake level is rising at a faster rate than planned, which is mainly attributed to low TTF WTP rates. The updated 2014 water balance modeling suggests that if the TTF WTP rates can be increased and maintained at 1,000 gpm or 1,500 gpm through the end of 2014 or longer, the lake level will remain within current dam capacity until the planned Stage 3 dam construction is completed as scheduled in 2017. This modeling shows the mean and minimum predicted lake levels, which are very similar, can be maintained below the maximum 200-year storm containment elevation of 697.3 feet. The mean predicted lake level is based on the average of 100 iterations or realizations used for the model; therefore, the majority of the realization results for each time step are very similar to the minimum predicted value. The modeling suggests that there is a risk the maximum predicted lake level could rise to the maximum 200-year storm containment elevation during the fourth quarter of 2014 or later in 2015 if the increase in WTP rates described above does not occur, but we understand that this increase is within the WTP permitted capacity and achievable. We understand that CAK plans to design the Stage 3 Dam in 2015 with construction in 2016.

11.0 RECOMMENDATIONS FOR ADDITIONAL WORK

Recommendations for additional work include the following:

- Revise the O&M Manual and EAP to include a description of the limited upstream east abutment trench that it is 4.5 feet lower than the dam crest elevation. Confirm and revise as necessary the maximum Stage 2 operating elevation to contain the 200-year storm (695 feet or 697.3 feet).
- Revise the O&M manual as follows:
  - Include the need for a periodic safety inspection by a qualified PE every 3 years in accordance to the Alaska Administrative Code 11 AAC 93.193(b) and the Class II hazard potential classification for the dam
  - Revise that monitoring of the stream gauge at East Fork Slate Creek is no longer performed, as the flows are equivalent to the combined flow from the TTF WTP and the Parshall flume, but the stream gauge and transducer are still used as a backup
- As recommended in the KPL Construction Completion Report, the following issues should be resolved during the Stage 3 Dam design and construction:
  - Correct the upstream grout trench and continue it to the new design crest elevation
  - Correct the construction issue for the Stage 2 plunge pool
  - Correct the issue with the Stage 2 excavation and foundation treatment near the temporary WTP
- Perform a detailed review of the seismic hazard deterministic and probabilistic analyses during the Stage 3 Dam design including updated seismic data from the USGS.
- Since CAK has not performed a table top exercise drill as listed in their EAP, we recommend that this exercise be performed within the next year. CAK is planning to perform a table top exercise on January 21, 2015, and Alaska Dam Safety should be provided written notice within seven days after the EAP exercise is completed. The requirement for a functional exercise in the EAP can also be removed as is it not necessary for a Class II dam.

12.0 REFERENCES

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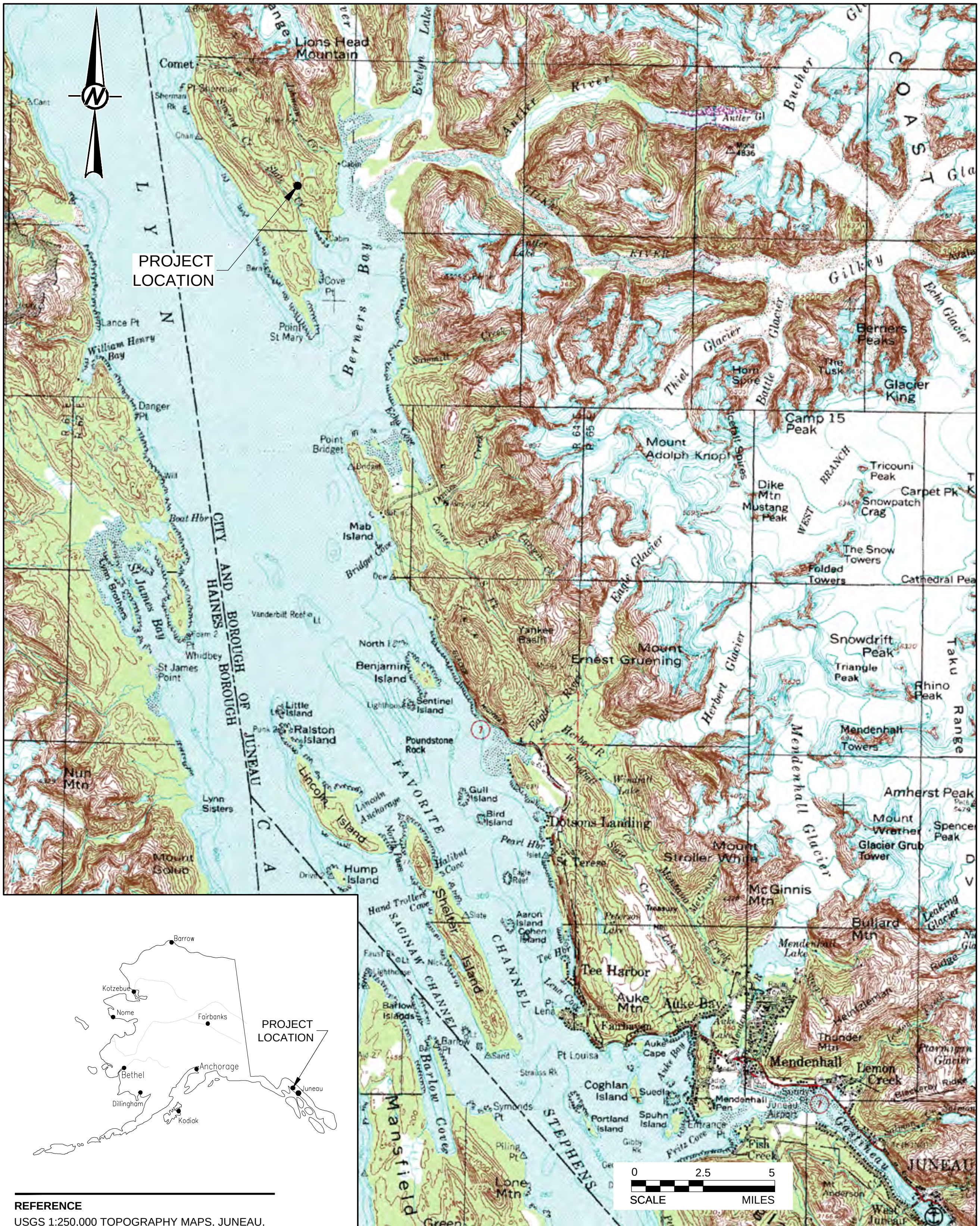
FIGURES



REFERENCE

USGS 1:250,000 TOPOGRAPHY MAPS, JUNEAU, ALASKA-CANADA, 1962 (MINOR REV. 1985)

CLIENT	PROJECT
COEUR ALASKA INC. 3031 CLINTON DRIVE, SUITE 202 JUNEAU, ALASKA 99801	LOWER SLATE LAKE DAM PERIODIC SAFETY INSPECTION KENSINGTON MINE, ALASKA

CONSULTANT		
Golder Associates	YYYY-MM-DD	2014-12-16
	PREPARED	SLA
	DESIGN	N/A
	REVIEW	SLA
	APPROVED	TGK

TITLE

PROJECT LOCATION MAP

PROJECT No.	Rev.	FIGURE
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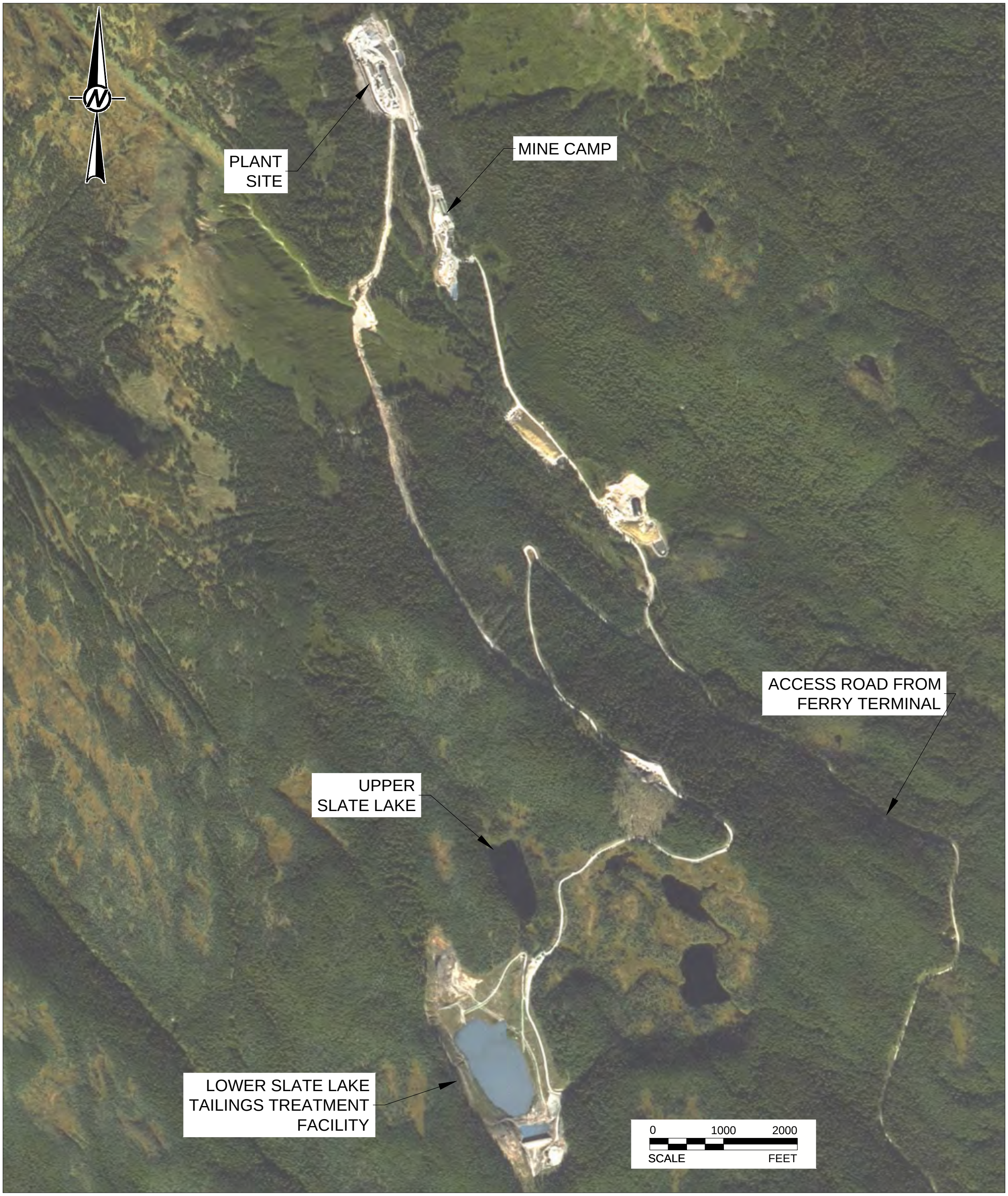

REFERENCE

BASE AERIAL IMAGERY FROM STATEWIDE DIGITAL MAP INITIATIVE (http://www.alaskamapped.org/), IMAGERY DATE UNKNOWN

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

YYYY-MM-DD	2014-12-16
PREPARED	SLA
DESIGN	N/A
REVIEW	SLA
APPROVED	TGK

PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
AERIAL VIEW OF PROJECT SITE

PROJECT No.	Rev.	FIGURE
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1 in
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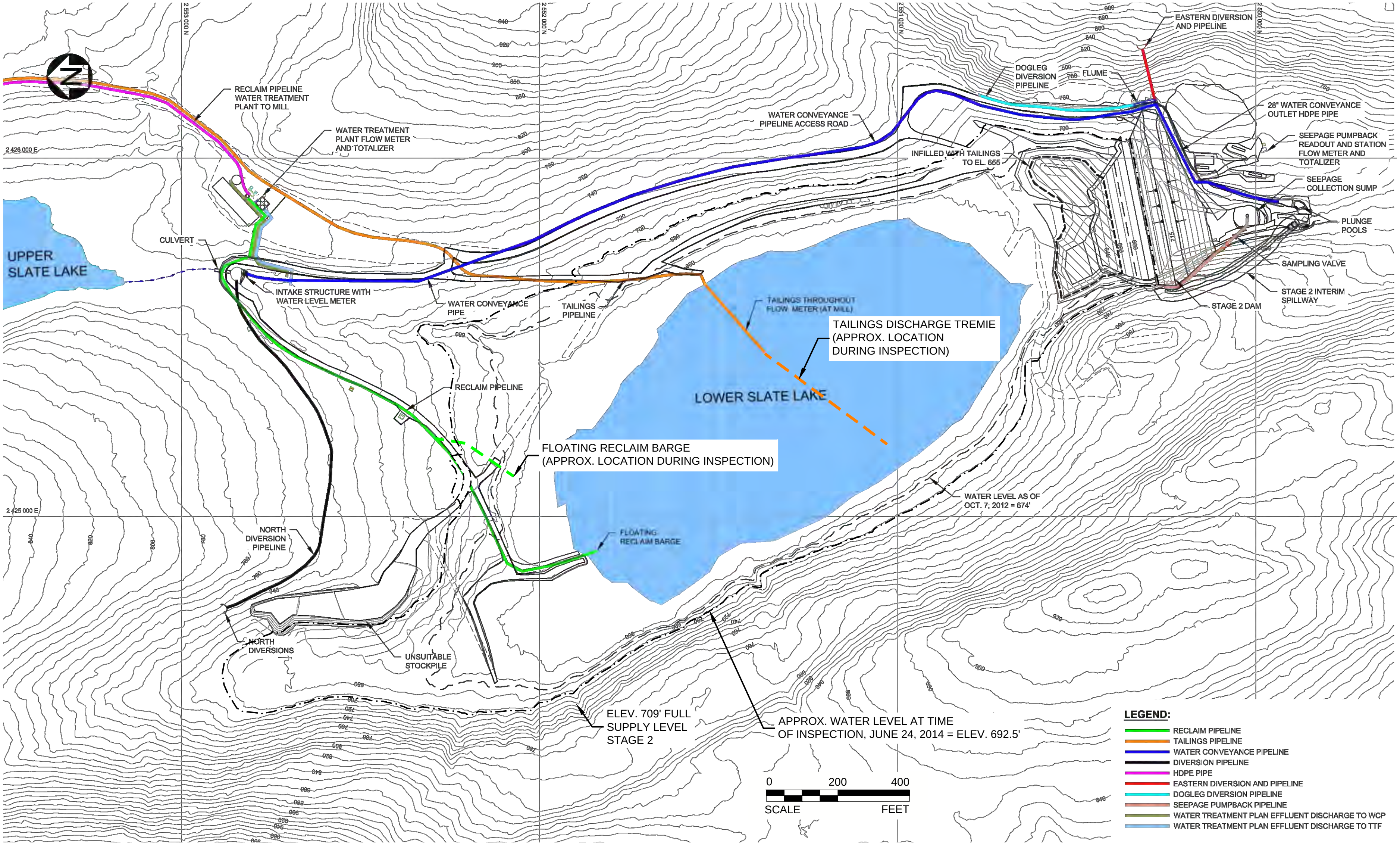

REFERENCE

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CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

Golder Associates

YYYY-MM-DD	2014-12-16
PREPARED	SLA
DESIGN	N/A
REVIEW	SLA
APPROVED	TGK

PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
TAILINGS TREATMENT FACILITY AND LOWER SLATE LAKE DAM LAYOUT

PROJECT No. 1404132 | Rev. 0 | FIGURE 3

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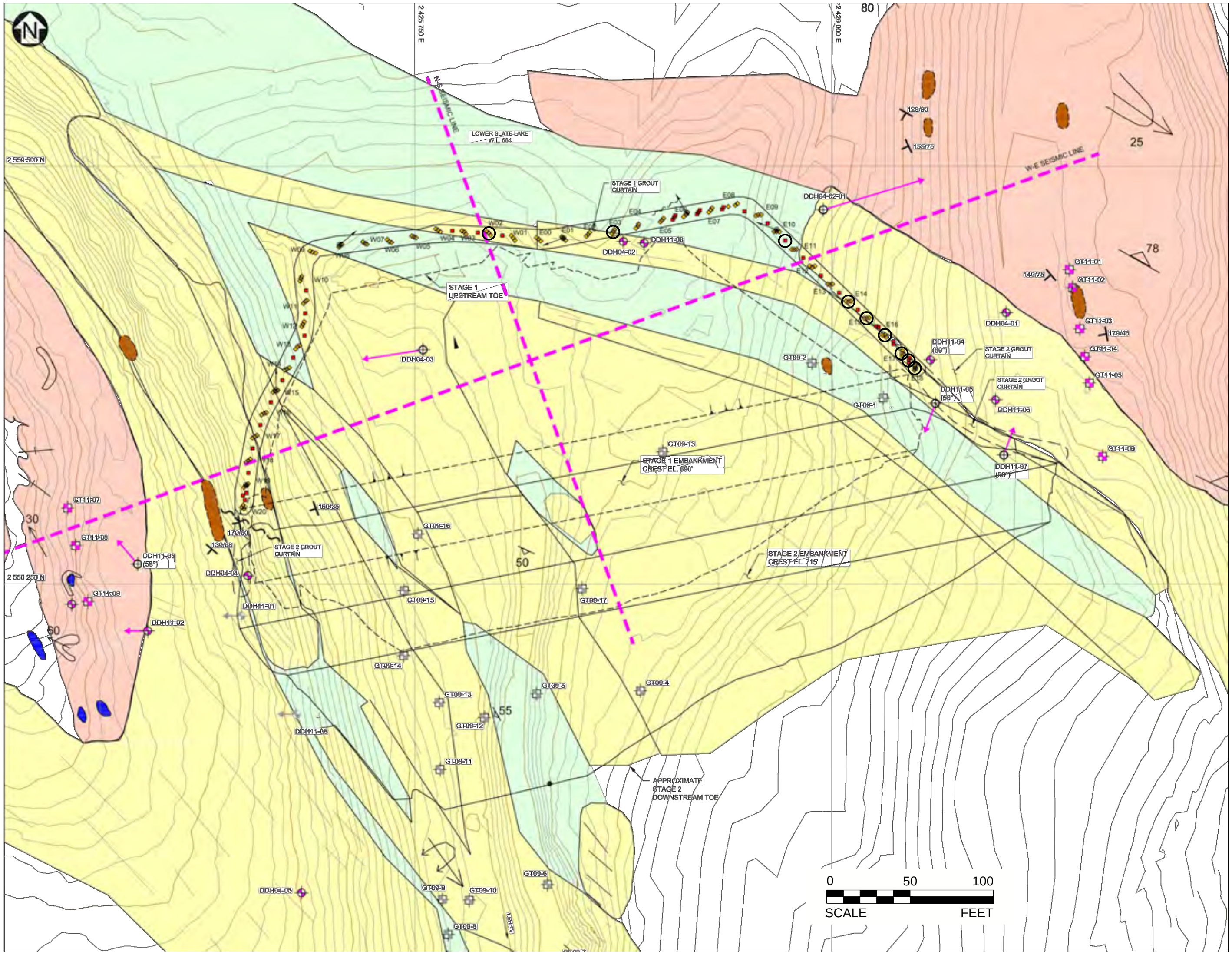

LEGEND

- BEDROCK OUTCROP EVALUATED DURING 2004 GEOLOGIC SITE EVALUATION
- 2004 GEOPHYSICAL SEISMIC SURVEY LINE
- CHLORITIC PHYLLITE
- MIXED CHLORITIC/GRAPHITIC PHYLLITE AND CARBONACEOUS QUARTZITE
- GRAPHITIC PHYLLITE
- QUARTZ VEINING
- 170/60 JOINT CHARACTERISTICS, STRIKE DIP ANGLE
- STRIKE & DIP OF FOLIATION
- CHLORITIC BED
- PYRITE
- 25 FOLD WITH AXIS AND PLUNGE
- 2010 GROUT CURTAIN CONFIRMATORY HOLES
- 2010 GROUT CURTAIN PRIMARY HOLES
- 2010 GROUT CURTAIN SECONDARY HOLES
- 2010 GROUT CURTAIN TERTIARY HOLES
- GT11-01 (ROCK MASS RATING) CLASSIFICATION LOCATION (2011
- GT09-01 (ROCK MASS RATING) CLASSIFICATION LOCATION (2009)
- MAJOR FAULT
- DDH04-02 2004 COMPLETED VERTICAL DRILL HOLE
- DDH11-02 2011 COMPLETED VERTICAL DRILL HOLE
- DDH04-05 (68°) 2004 COMPLETED DIRECTIONAL DRILL HOLE
- DDH11-05 (68°) 2011 COMPLETED DIRECTIONAL DRILL HOLE
- DDH11-01 NOT COMPLETED DRILL HOLE
- 2010 GROUT CURTAIN WATER TEST HOLES WITH > 1 LUGEON VALUES

REFERENCES

1. BASE DRAWING AND LEGEND FROM "DETAILED DESIGN REPORT, REV 4 [DRAFT]" BY KNIGHT PIÉSOLD LIMITED, DATED JUNE 11, 2012.
2. HIGH LUGEON VALUE LOCATIONS FROM "LOWER SLATE LAKE TAILINGS FACILITY, 2011 GEOTECHNICAL REPORT" BY KNIGHT PIÉSOLD LIMITED, DATED JANUARY 20, 2012.

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

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PREPARED	SLA
DESIGN	N/A
REVIEW	SLA
APPROVED	TGK

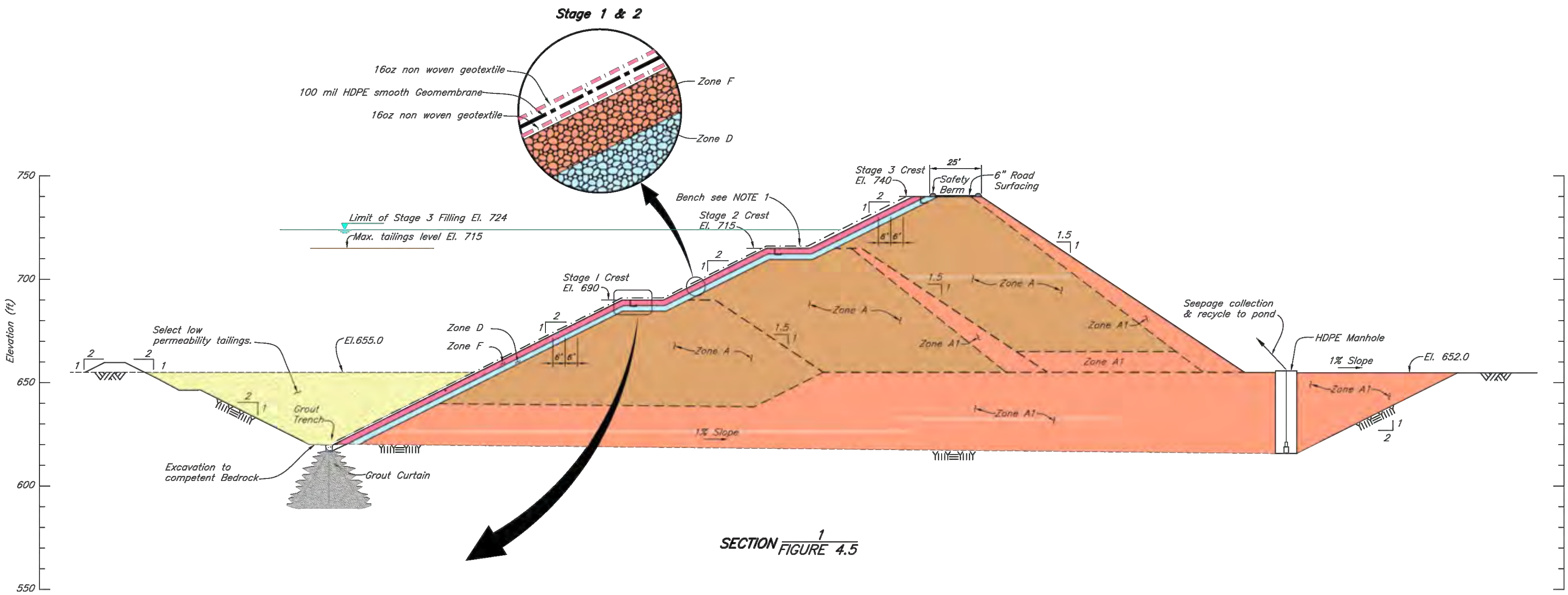
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

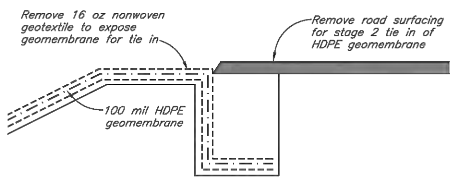
TITLE
SUMMARY OF GEOTECHNICAL INVESTIGATIONS

PROJECT No.	Rev.	FIGURE
1404132	0	4

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0

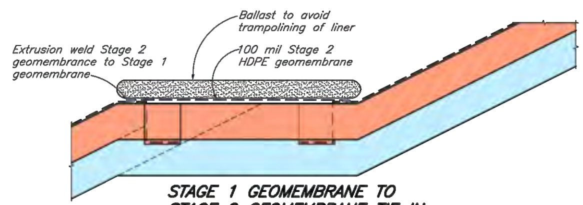
Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: Design Sections.dwg





EXISTING STAGE 1 CREST PRIOR TO STAGE 2 TIE IN

NTS



STAGE 1 GEOMEMBRANE TO STAGE 2 GEOMEMBRANE TIE IN

NTS

NOTE

1. Bench for Stage 1 and Stage 2 to be minimum 20 ft

NOT FOR CONSTRUCTION

0 40 80

SCALE FEET

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

REFERENCE

FIGURE FROM "LOWER SLATE LAKE TAILINGS DAM, DETAILED DESIGN REPORT, REV 4 [DRAFT]," BY KNIGHT PI£SOLD LIMITED, DATED JUNE 11, 2012.

CLIENT

COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

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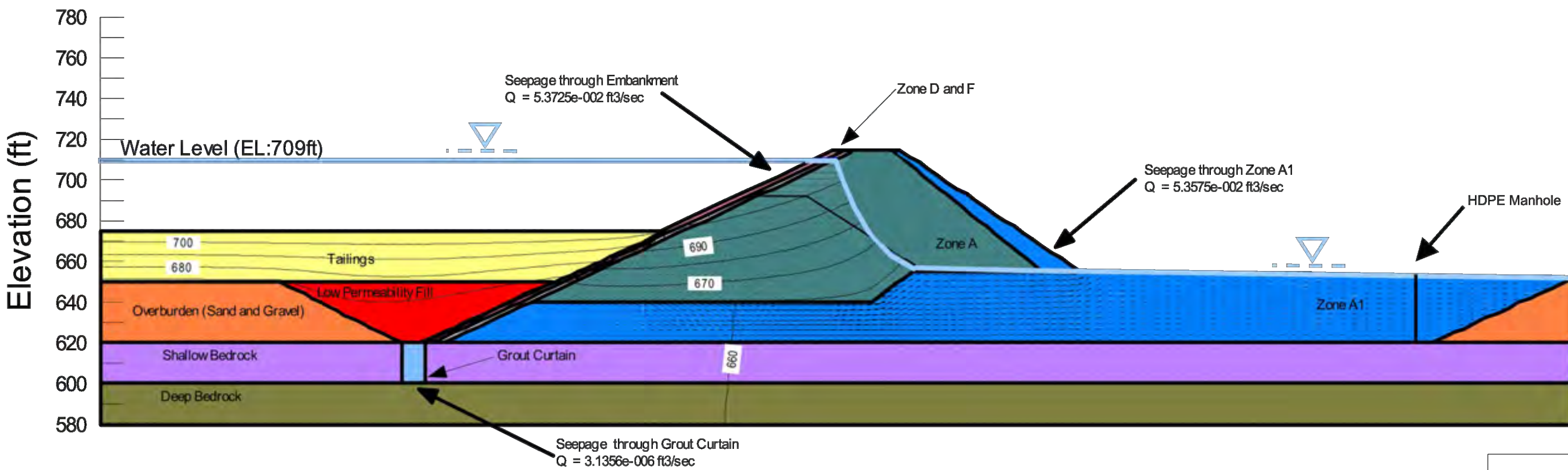
PROJECT

LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

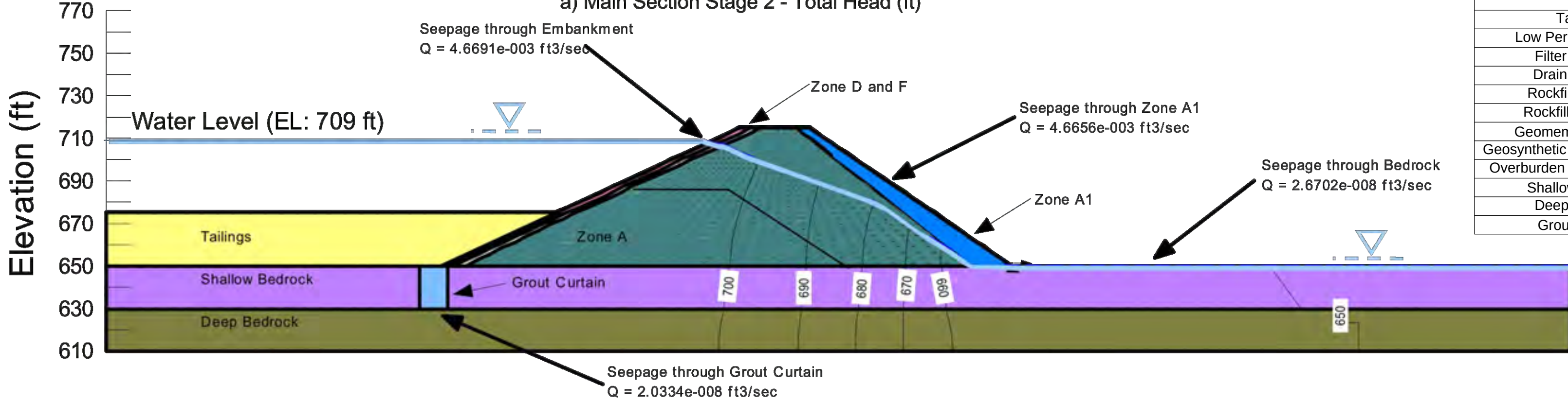
TITLE

PLANNED DESIGN SECTION FOR DAM STAGES

PROJECT No.	Rev.	FIGURE
1404132	0	5



a) Main Section Stage 2 - Total Head (ft)



b) Abutment Section Stage 2 - Total Head (ft)

MATERIAL	HYDRAULIC CONDUCTIVITY (Kh, cm/sec)	Kv:Kh
Tailings	5.0E-06	0.1
Low Permeability Fill	5.0E-06	0.1
Filter (Zone F)	1.0E-02	1.0
Drain (Zone D)	1.0E-01	1.0
Rockfill (Zone A)	1.0E-02	1.0
Rockfill (Zone A1)	2.1E+00	1.0
Geomembrane Liner	1.0E-09	1.0
Geosynthetic Clay Liner (GCL)	1.0E-06	1.0
Overburden (Sand & Gravel)	1.0E-02	1.0
Shallow Bedrock	1.0E-04	1.0
Deep Bedrock	1.0E-06	1.0
Grout Curtain	1.0E-05	1.0

Notes:
1) Dam crest at El. 715 ft, water level at El. 709 ft, and tailings level at El.675 ft.
2) Geomembrane liner degraded.
3) Water pressure directly applied on the highly permeable upstream dam surface.
4) HDPE manhole pumping ceased.

REFERENCE
FIGURE FROM "LOWER SLATE LAKE TAILINGS DAM, DETAILED DESIGN REPORT, REV 4 [DRAFT]," BY KNIGHT PIÉSOLD LIMITED, DATED JUNE 11, 2012.

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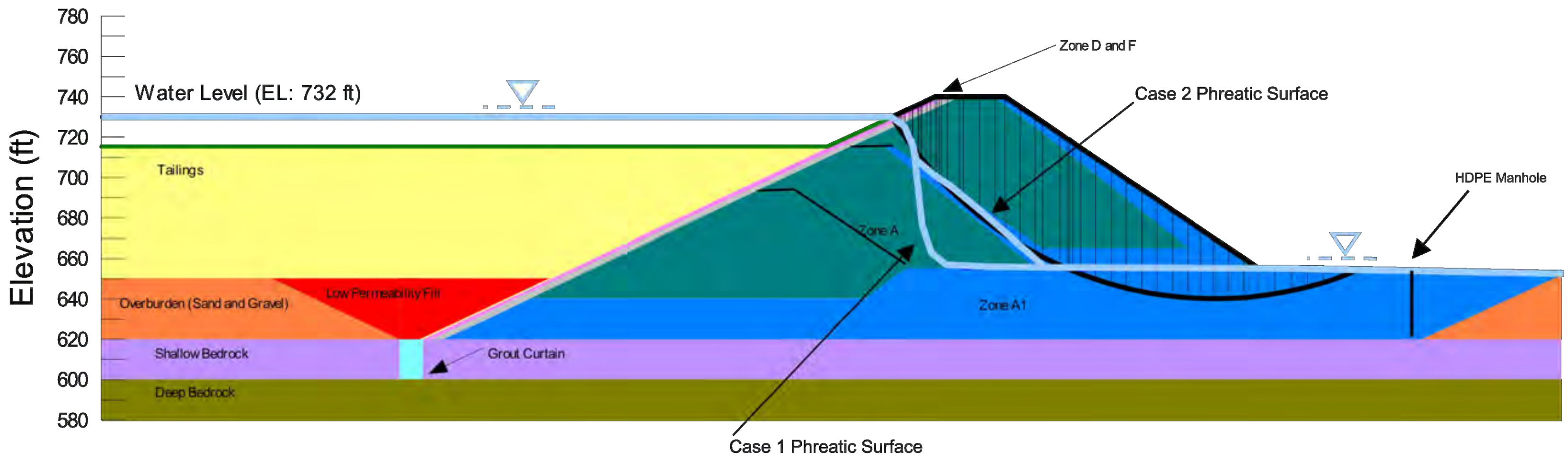
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
SEEPAGE ANALYSIS RESULTS FOR STAGE 2 DAM, POST-CLOSURE

PROJECT No. 1404132 | Rev. 0 | FIGURE 6

EXAMPLE OF SLOPE STABILITY SECTION AND FAILURE SURFACE (DOWNSTREAM SLOPE, POST-CLOSURE CASE)



ASSUMED MATERIAL STRENGTH PROPERTIES

Material	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Effective Shear Strength/Normal Stress Ratio
Tailings (Static)	115			0.25
Tailings (Post Liquefaction)	115			0.05
Filter (Zone D)	134	0	36	
Drain (Zone F)	134	0	36	
Rockfill (Zone A)	138	0	42	
Rockfill (Zone A1)	138		Leps, 1970	
Overburden (Sand & Gravel)	137	0	30	
Shallow Bedrock	146	0	45	

SUMMARY OF STABILITY ANALYSIS RESULTS

Modelling Scenario	Water Level (feet)	Tailings Elevation (feet)	Stress Conditions	Factor of Safety	Yield Acceleration	Estimated Deformation from MDE (feet)
Upstream Slope, Operation Case	709	700	Static	2.66		
			Seismic	<1	0.45g	1.0
			Post-Liquefaction	2.66		
Downstream Slope, Operation Case	724	715	Static	1.98		
			Seismic	<1	0.30g	0.3
			Post-Liquefaction	1.98		
Upstream Slope, Post Closure Case	732	715	Static	2.44		
			Seismic	<1	0.30g	3.3
			Post-Liquefaction	2.44		
Downstream Slope, Post Closure Case	732	715	Static	1.58		
			Seismic	<1	0.20g	2.3
			Post-Liquefaction	1.58		

REFERENCE

STABILITY SECTION AND TABULATED INFORMATION FROM "LOWER SLATE LAKE TAILINGS DAM, DETAILED DESIGN REPORT, REV 4 [DRAFT]," BY KNIGHT PI£SOLD LIMITED, DATED JUNE 11, 2012.

NOTES

1. LOW PERMEABILITY FILL MODELED WITH SAME STATIC AND POST-LIQUEFACTION PARAMETERS AS TAILINGS.
2. DEEP BEDROCK AND GROUT CURTAIN ASSUMED IMPENETRABLE (NEGLECTED IN MODELING).
3. UNDER OPERATING BASIS EARTHQUAKE (OBE), BEDROCK AND DAM CREST ACCELERATIONS DETERMINED AS 0.13G AND 0.39G, RESPECTIVELY.
4. UNDER MAXIMUM DESIGN EARTHQUAKE (MDE), BEDROCK AND DAM CREST ACCELERATIONS DETERMINED AS 0.6G AND 1.2G, RESPECTIVELY.
5. GROUNDWATER SURFACE AND ASSOCIATED PORE WATER PRESSURES IMPORTED FROM SEEPAGE ANALYSES.

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

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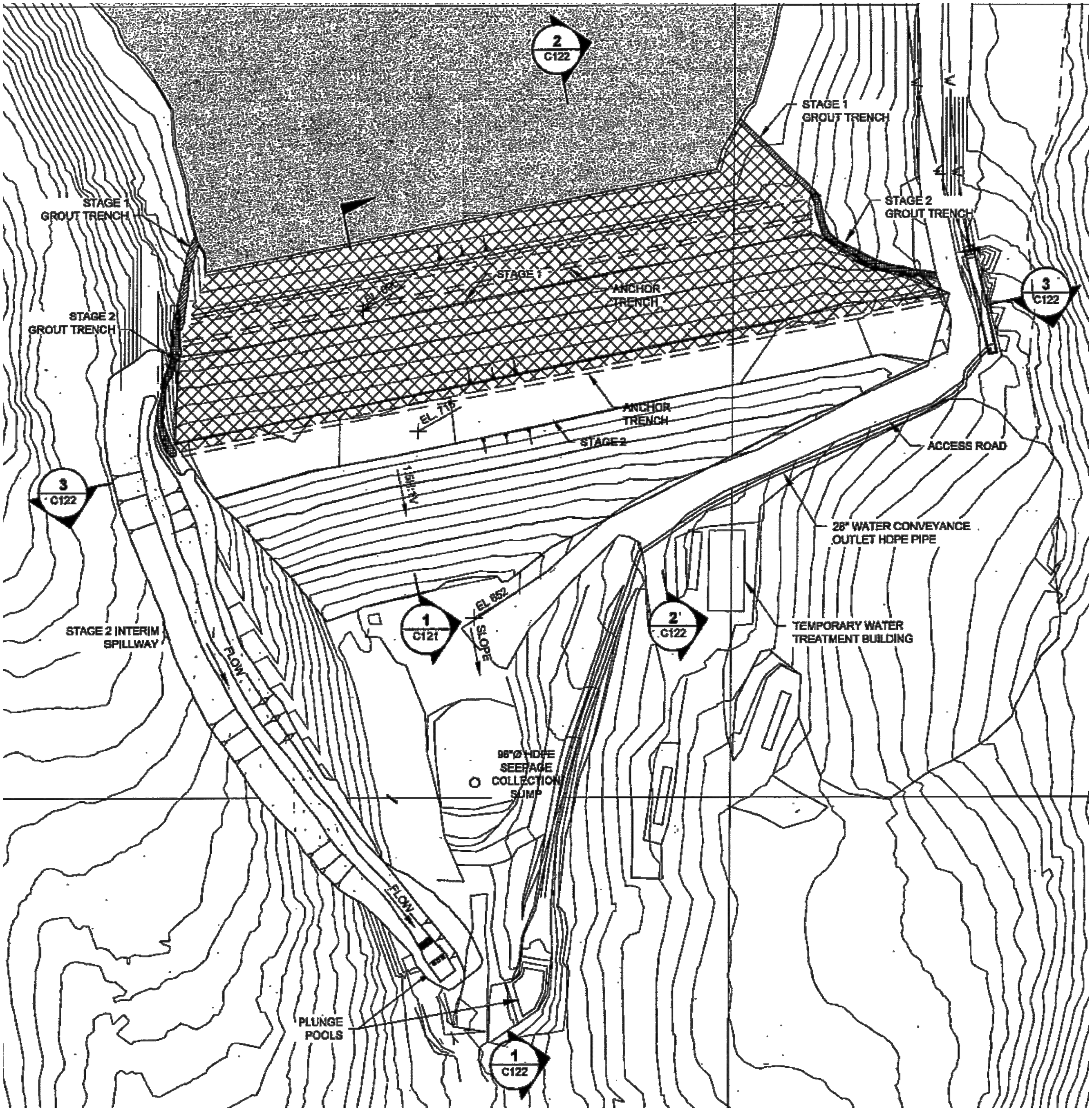
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS AND EXAMPLE OF POST-CLOSURE CASE FOR DOWNSTREAM SLOPE

PROJECT No. 1404132 | Rev. 0 | FIGURE 7



REFERENCE

AS-BUILT DRAWING FROM "CONSTRUCTION COMPLETION REPORT, LOWER SLATE LAKE TAILINGS TREATMENT FACILITY, STAGE 2," BY KNIGHT PIÉSOLD LIMITED, DATED DECEMBER 7, 2012.

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

Golder Associates

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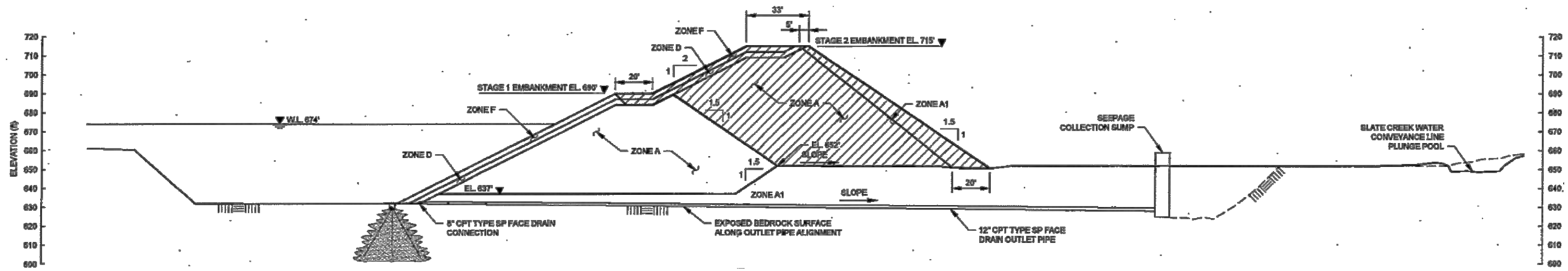
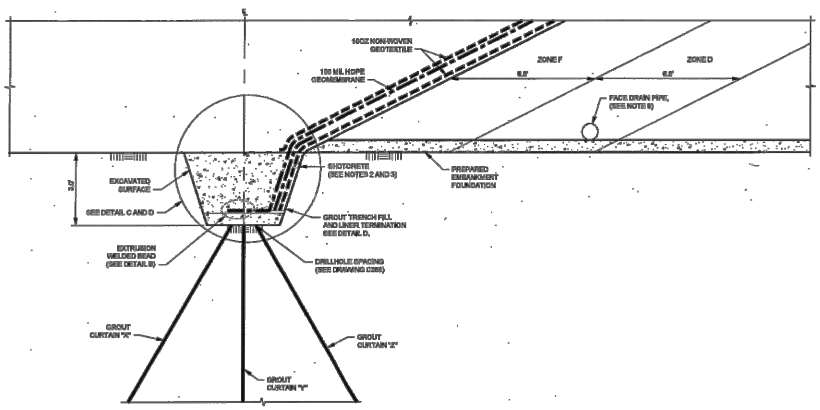
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
PLAN VIEW OF AS-BUILT STAGE 2 DAM EMBANKMENT AND SPILLWAY

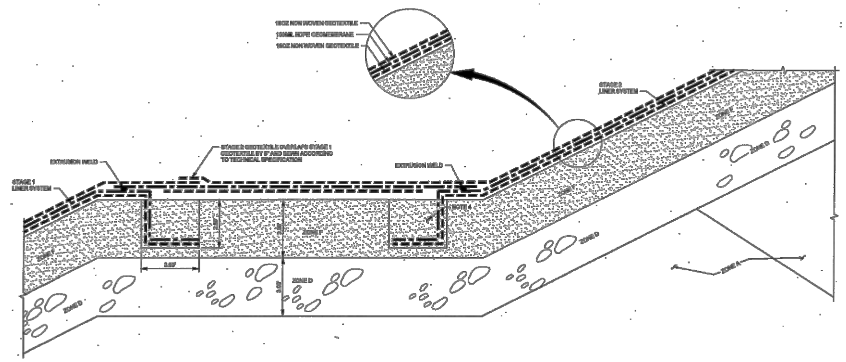
PROJECT No.	Rev.	FIGURE
1404132	0	8



AS-BUILT STAGE 2 DAM SECTION



AS-BUILT GROUT TRENCH AND LINER KEY DETAIL



AS-BUILT STAGE 1 AND STAGE 2 GEOSYNTHETIC LINER TIE IN DETAIL

REFERENCE

AS-BUILT DRAWINGS FROM "CONSTRUCTION COMPLETION REPORT, LOWER SLATE LAKE TAILINGS TREATMENT FACILITY, STAGE 2," BY KNIGHT PI£SOLD LIMITED, DATED DECEMBER 7, 2012.

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COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

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PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
AS-BUILT STAGE 2 DAM SECTION AND GEOSYNTHETIC LINER DETAILS

PROJECT No.	Rev.	FIGURE
1404132	0	9

Photo 38: October 7: East grout trench ready to receive concrete.

Photo 39: October 7: Liner termination at east abutment. Grout trench terminates approximately at elevation 710.5

Photo 40: October 7: Completed east grout trench.

Photo 45: October 22: Geomembrane flap installed to protect road embankment on east abutment.

REFERENCE

FIELD PHOTOGRAPHS DURING STAGE 2 CONSTRUCTION FROM "CONSTRUCTION COMPLETION REPORT, LOWER SLATE LAKE TAILINGS TREATMENT FACILITY, STAGE 2," BY KNIGHT PIÉSOLD LIMITED, DATED DECEMBER 7, 2012.

CLIENT

COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

Golder Associates

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PROJECT

LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

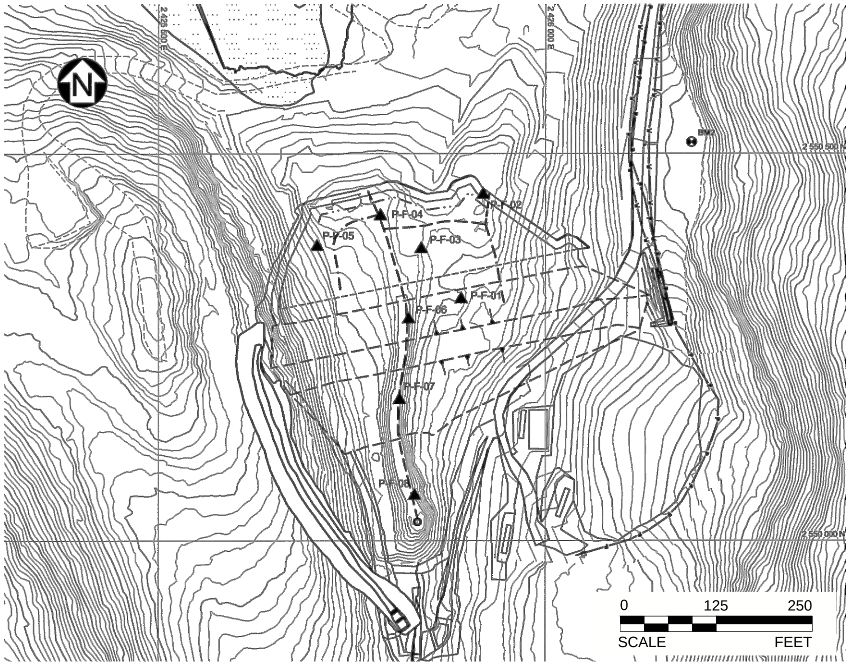
TITLE

FIELD PHOTOGRAPHS FROM STAGE 2 CONSTRUCTION SHOWING EAST ABUTMENT GROUT TRENCH

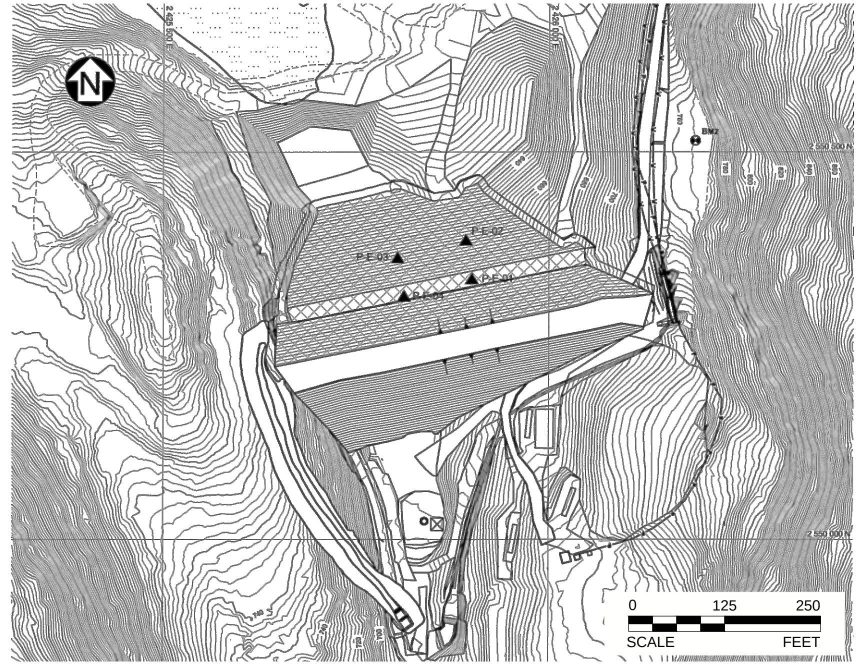
PROJECT No.	Rev.	FIGURE
1404132	0	10

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

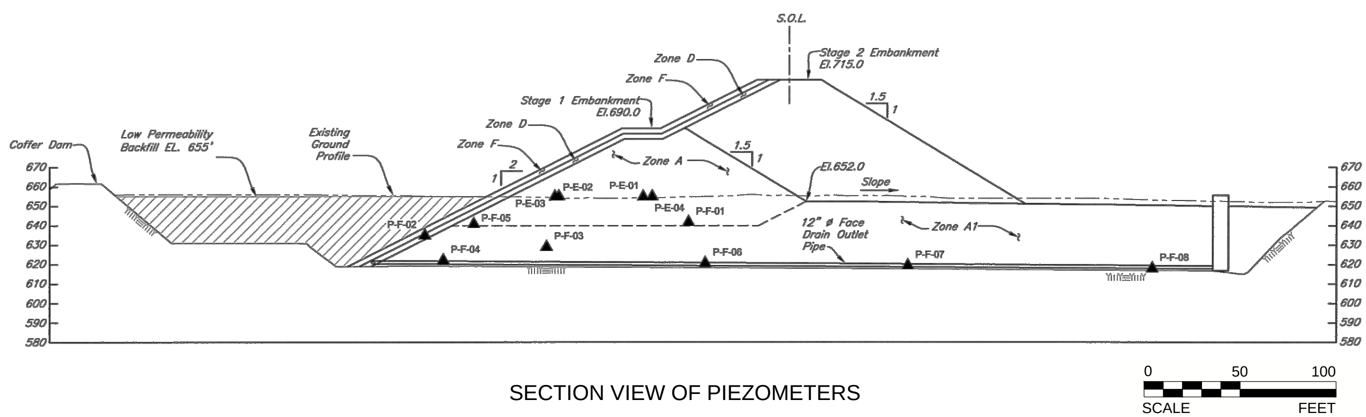
0 1 in



PLAN VIEW OF FOUNDATION PIEZOMETERS



PLAN VIEW OF EMBANKMENT PIEZOMETERS



SECTION VIEW OF PIEZOMETERS

REFERENCE
DRAWINGS FROM "OPERATIONS AND MAINTENANCE MANUAL, LOWER SLATE LAKE TAILINGS DAM NID#AK00308," BY KNIGHT PI£SOLD LIMITED, REV. 2, DATED DECEMBER 7, 2012.

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

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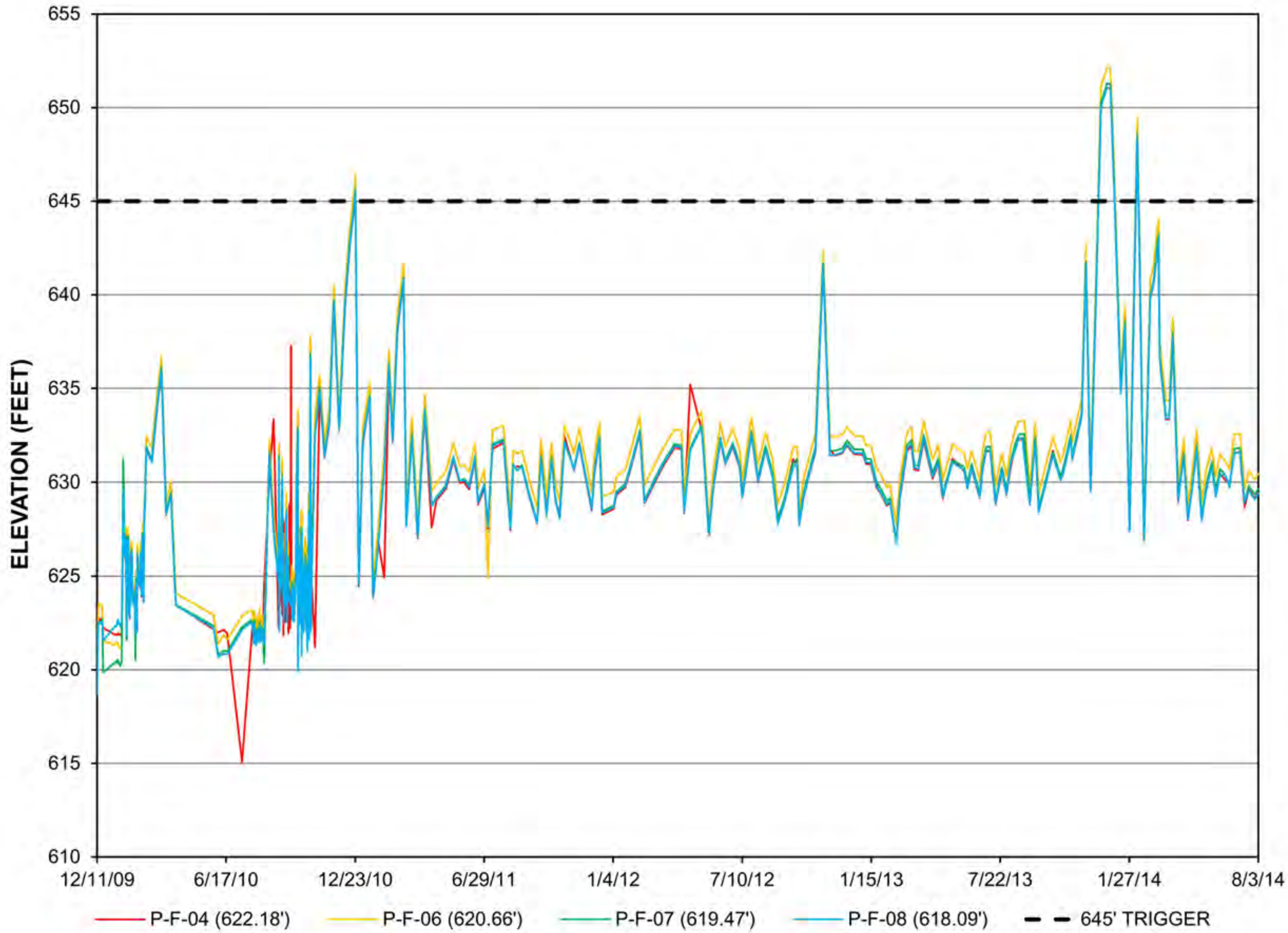
APPROVED	TGK

PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
VIBRATING WIRE PIEZOMETER LOCATIONS

PROJECT No.	Rev.	FIGURE
1404132	0	11



NOTES

1. PIEZOMETER READING FREQUENCY VARIES, BUT TYPICALLY COLLECTED ON A WEEKLY BASIS STARTING OCTOBER 25, 2010
2. PIEZOMETER TIP ELEVATION SHOWN IN PARENTHESES NEXT TO PIEZOMETER IDENTIFICATION NUMBER

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COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

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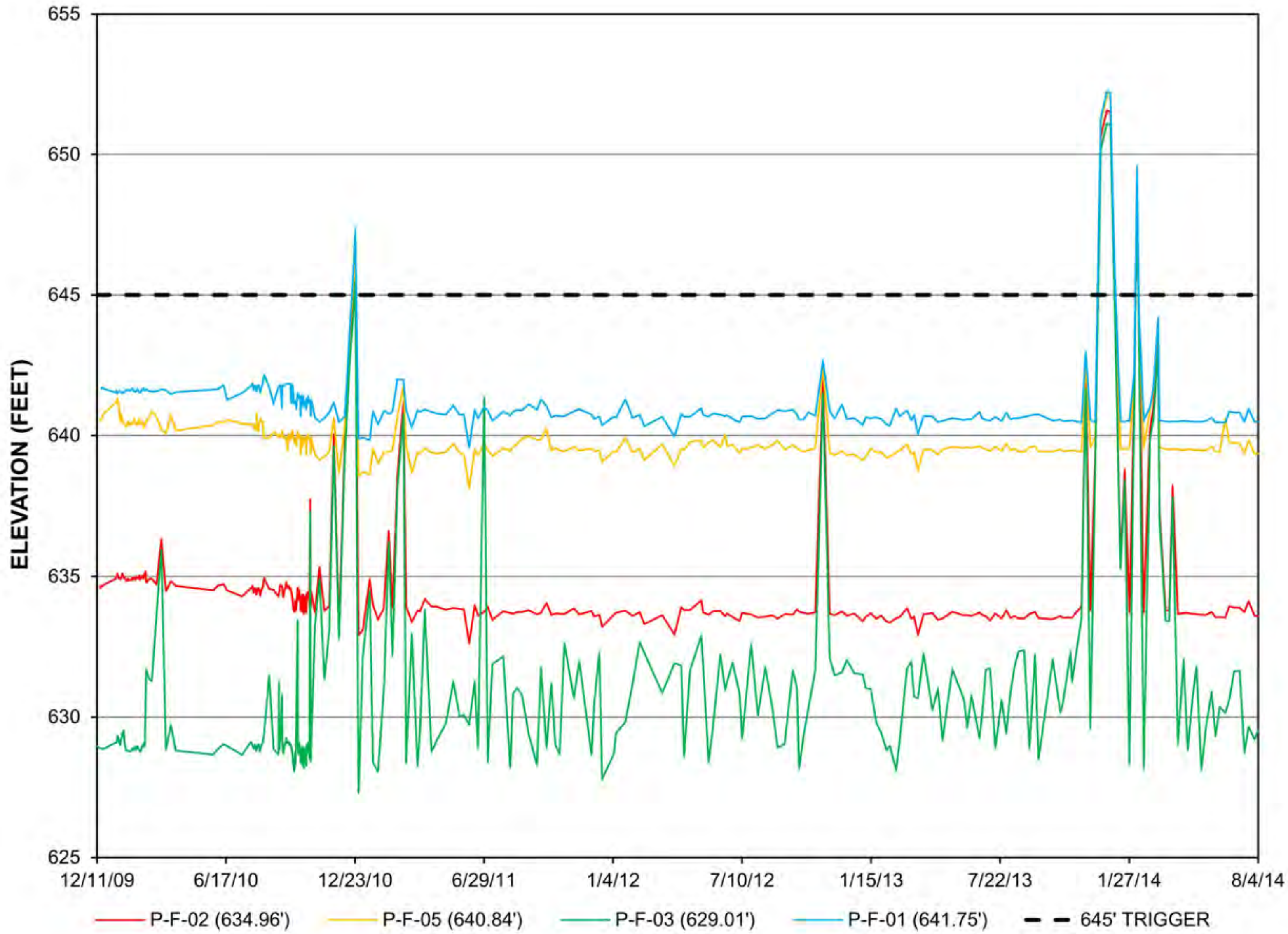
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
FOUNDATION PIEZOMETER DATA ALONG OUTLET DRAIN PIPE

PROJECT No. 1404132 | Rev. 0 | FIGURE 12



NOTES

1. PIEZOMETER READING FREQUENCY VARIES, BUT TYPICALLY COLLECTED ON A WEEKLY BASIS STARTING OCTOBER 25, 2010
2. PIEZOMETER TIP ELEVATION SHOWN IN PARENTHESES NEXT TO PIEZOMETER IDENTIFICATION NUMBER

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3031 CLINTON DRIVE, SUITE 202
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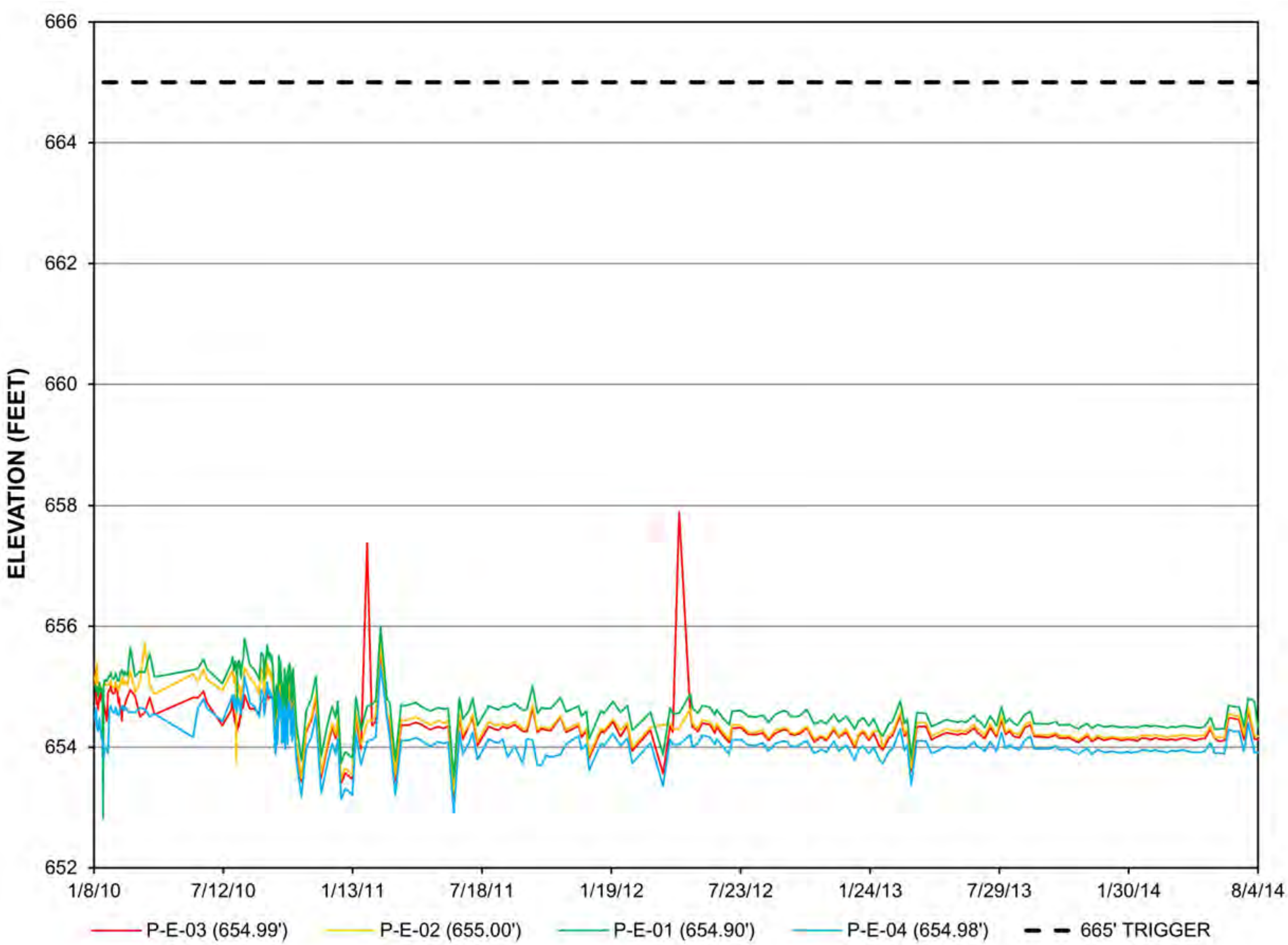
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
FOUNDATION PIEZOMETER DATA AWAY FROM OUTLET DRAIN PIPE

PROJECT No. 1404132 | Rev. 0 | FIGURE 13

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: Monitoring Data.dwg



NOTES

1. PIEZOMETER READING FREQUENCY VARIES, BUT TYPICALLY COLLECTED ON A WEEKLY BASIS STARTING OCTOBER 25, 2010
2. PIEZOMETER TIP ELEVATION SHOWN IN PARENTHESES NEXT TO PIEZOMETER IDENTIFICATION NUMBER

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3031 CLINTON DRIVE, SUITE 202
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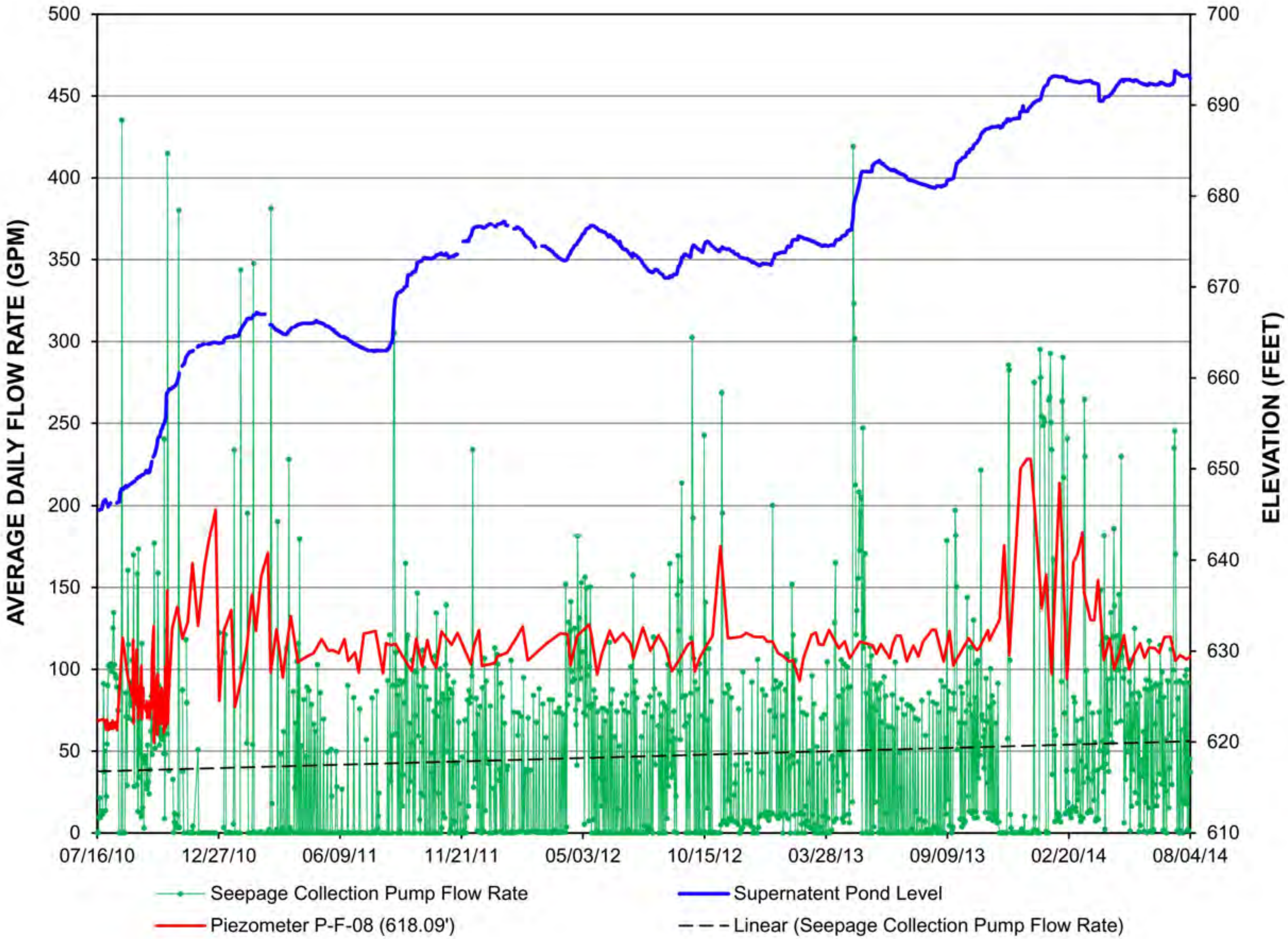
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
EMBANKMENT PIEZOMETER DATA

PROJECT No. 1404132
Rev. 0
FIGURE 14

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
0 1 in



NOTES

1. TOTAL SEEPAGE FLOW (GALLONS) AND LAKE WATER LEVEL COLLECTED ON DAILY BASIS, PIEZOMETER READING TYPICALLY COLLECTED ON WEEKLY BASIS
2. SEEPAGE FLOW RATE CALCULATED AS VOLUME OF FLOW OVER LENGTH OF TIME BETWEEN READINGS
3. PIEZOMETER TIP ELEVATION SHOWN IN PARENTHESES NEXT TO PIEZOMETER IDENTIFICATION NUMBER

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COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

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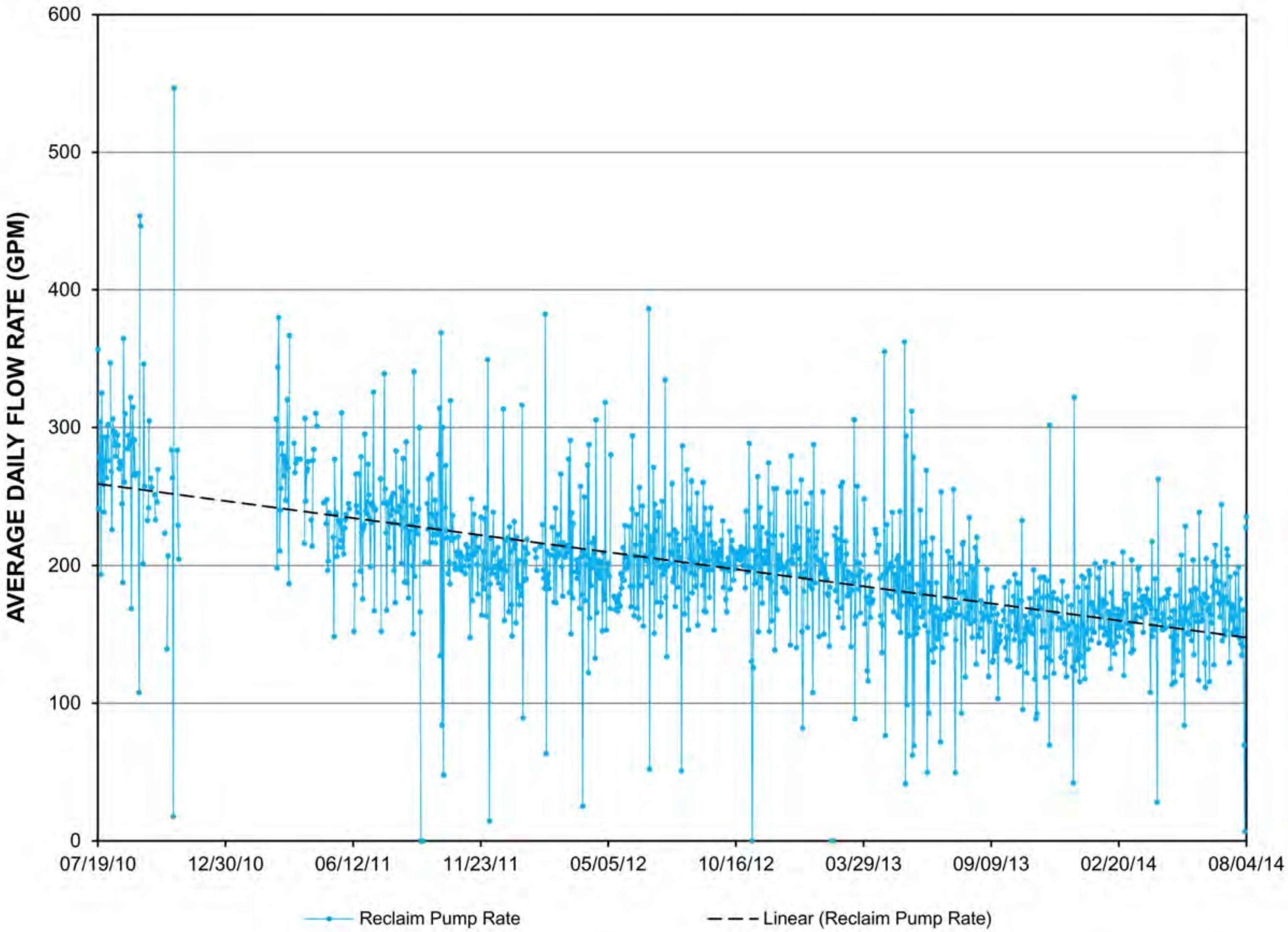
REVIEW	SLA
APPROVED	TGK

PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
SEEPAGE COLLECTION PUMP RATE, PIEZOMETER NEAR SUMP, AND LAKE LEVEL DATA

PROJECT No.	Rev.	FIGURE
1404132	0	15



NOTES

1. TOTAL RECLAIM FLOW (GALLONS) COLLECTED ON DAILY BASIS
2. RECLAIM PUMPBACK FLOW RATE CALCULATED AS VOLUME OF FLOW OVER LENGTH OF TIME BETWEEN READINGS

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT
Golder Associates

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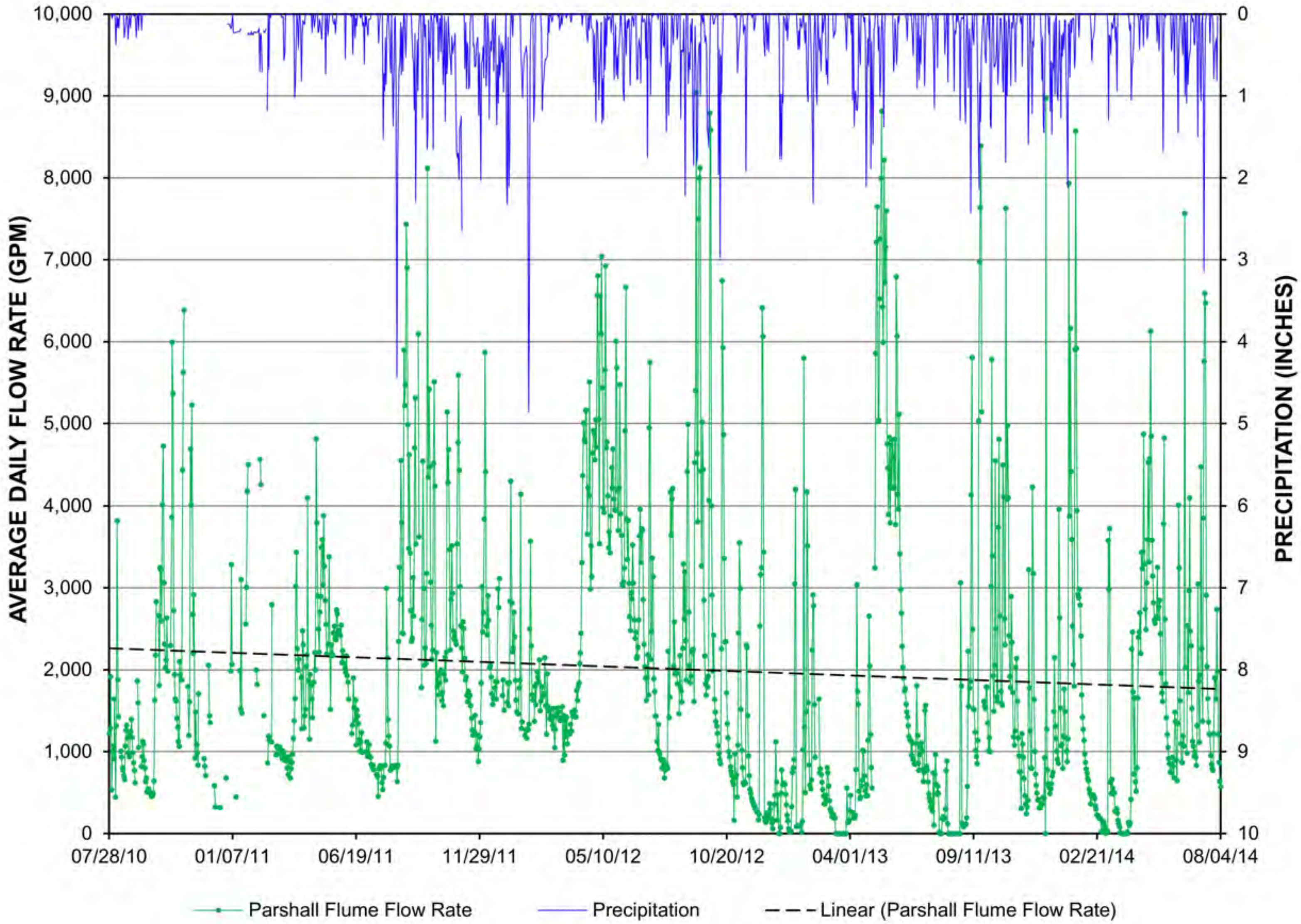
PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
RECLAIM BARGE PUMP RATE DATA

PROJECT No. 1404132
Rev. 0
FIGURE 16

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: Monitoring Data.dwg



NOTES

1. TRANSDUCER TOTAL FLOW (GALLONS) AND CURRENT FLOW RATE (GALLONS PER MINUTE) AT PARSHALL FLUME COLLECTED ON DAILY BASIS
2. AVERAGE DAILY FLOW RATE CALCULATED AS VOLUME OF FLOW OVER LENGTH OF TIME BETWEEN READINGS

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COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

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REVIEW	SLA
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PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
DIVERSION FLUME AND PRECIPITATION DATA

PROJECT No. 1404132 | Rev. 0 | FIGURE 17

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0

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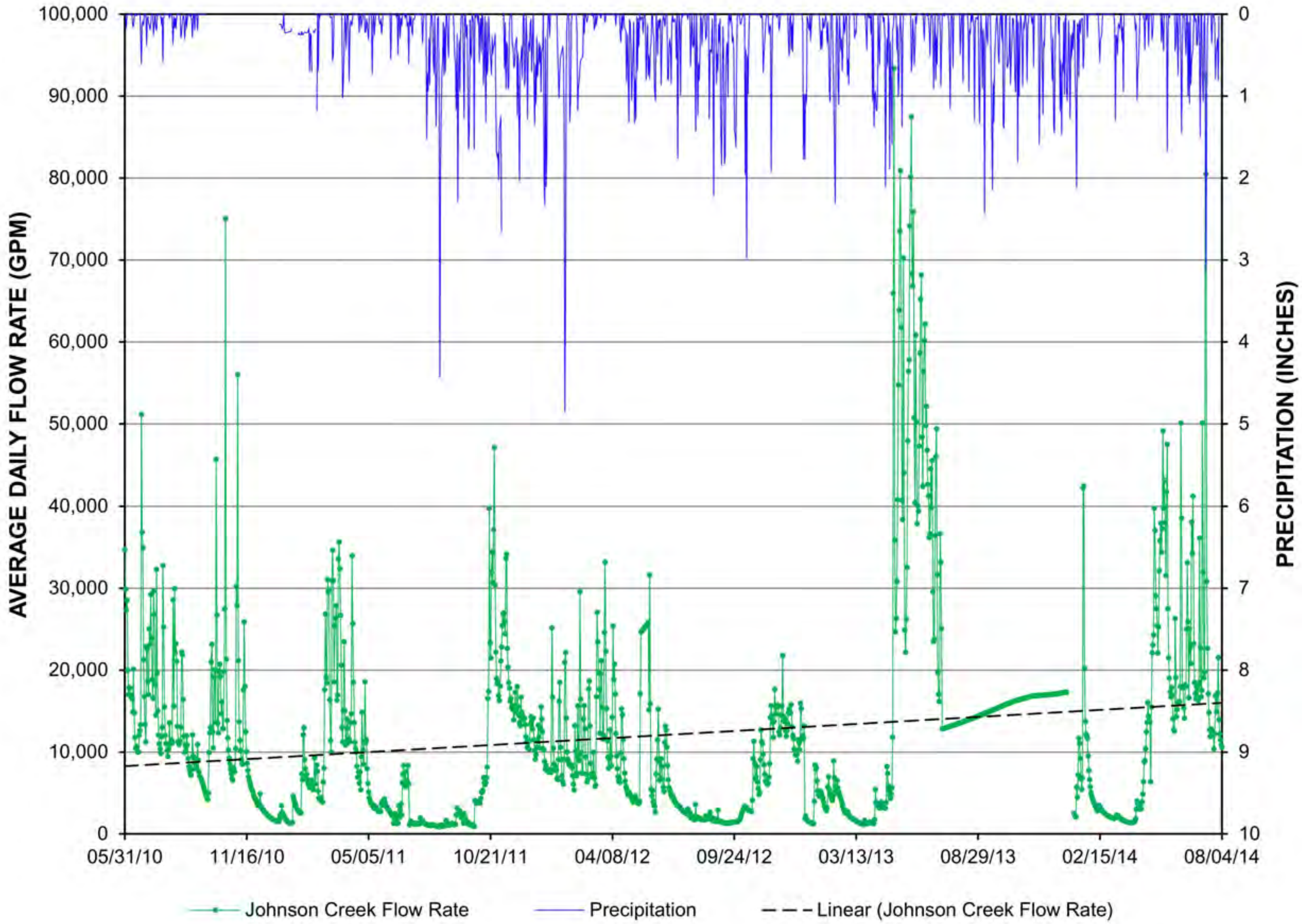

NOTES

1. STREAM GAUGE DATA AT JOHNSON CREEK COLLECTED ON DAILY BASIS
2. DATA FROM JULY 11, 2013 TO JANUARY 9, 2014 IS AN AVERAGE OF PAST HISTORICAL FLOWS

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COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

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PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
JOHNSON CREEK AND PRECIPITATION DATA

PROJECT No.	Rev.	FIGURE
1404132	0	18

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0

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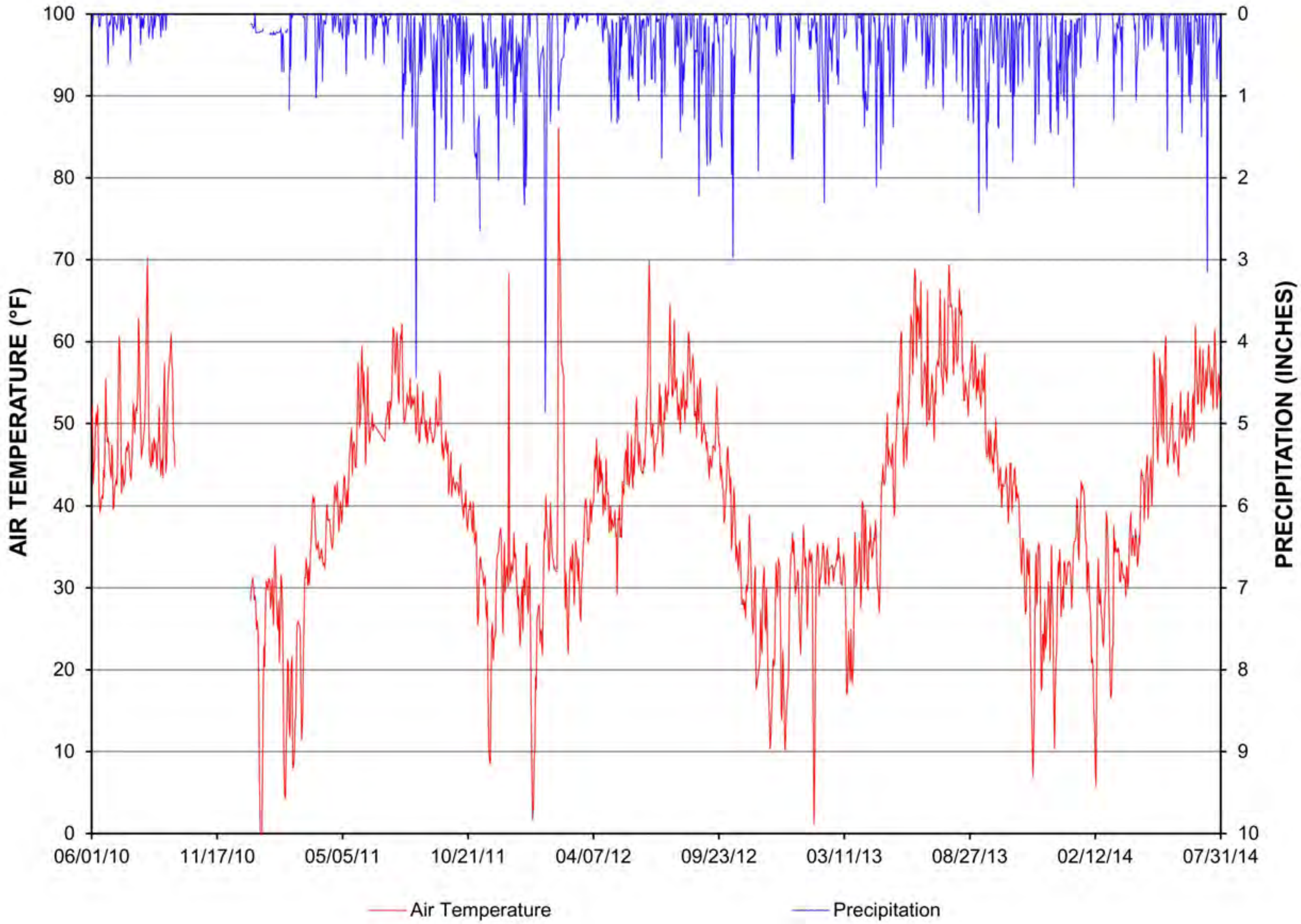

NOTE
AIR TEMPERATURE AND PRECIPITATION COLLECTED AT WEATHER STATION ON TYPICALLY A DAILY BASIS

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3031 CLINTON DRIVE, SUITE 202
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CONSULTANT

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PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
AIR TEMPERATURE AND PRECIPITATION DATA

PROJECT No. 1404132
Rev. 0
FIGURE 19

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
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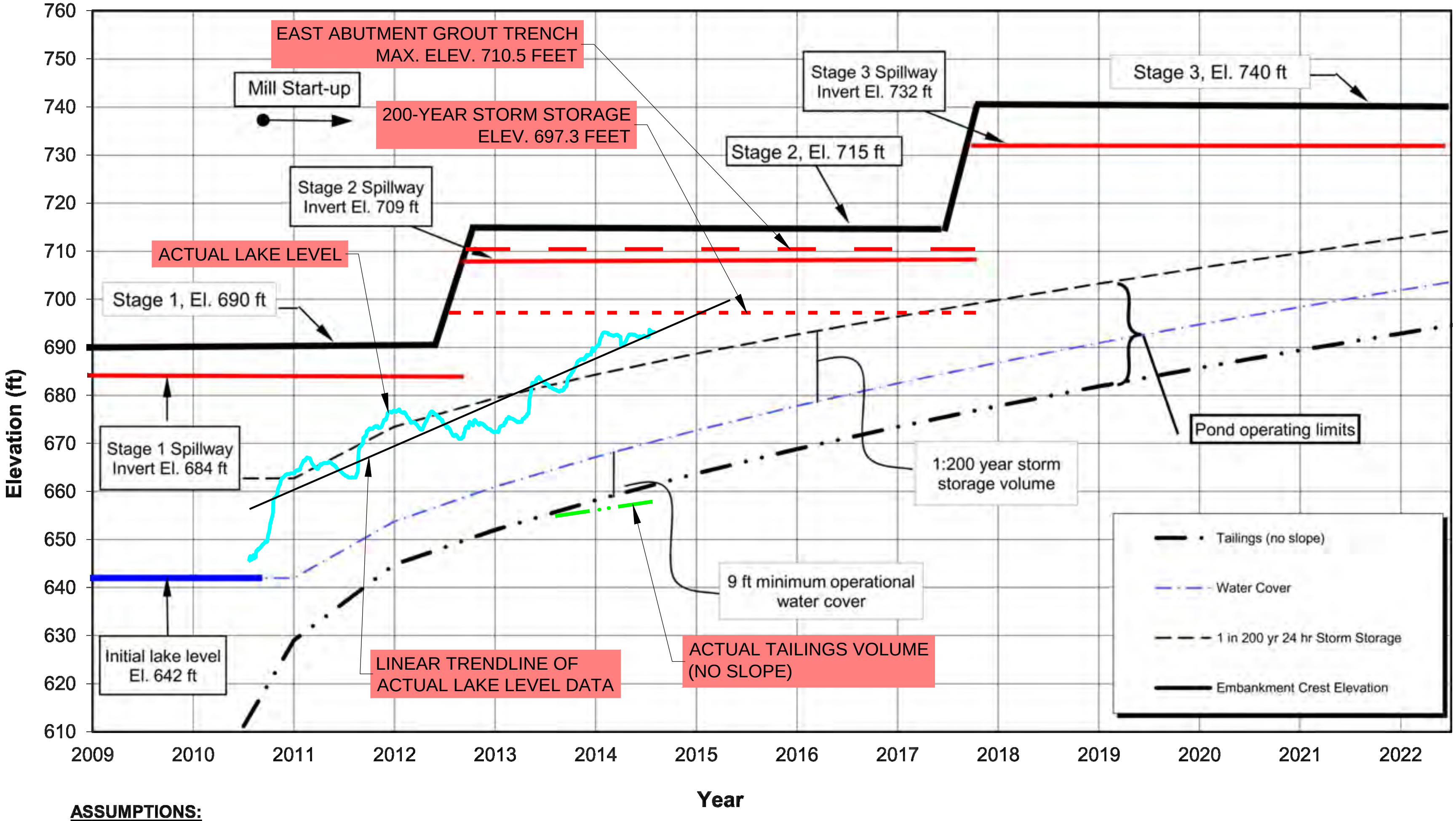

ASSUMPTIONS:

1. TAILINGS SOLIDS TO TTF FROM COEUR PRODUCTION SCHEDULE
   - MILL START UP TO JULY 1, 2011=1,202 tpd
   - JULY 1, 2011 - EOM = 770 tpd.
2. AVERAGE DRY DENSITY OF 85 pcf USED FOR OPERATIONS
3. 9 ft. MINIMUM WATER COVER ASSUMED.
4. IMPOUNDMENT INCLUDES FOR STORAGE OF THE 1:200 EVENT, WHICH CORRESPONDS TO VOLUME OF 25 MILLION ft³ (574 ACRE-FEET).
5. SIX FEET FREEBOARD ALLOWANCE FROM SPILLWAY INVERT ELEVATION TO CREST FOR STAGES 1 AND 2 AND LATER INCREASING TO 8 FEET FOR STAGE 3

REFERENCE

BASE DRAWING FROM "OPERATIONS AND MAINTENANCE MANUAL, LOWER SLATE LAKE TAILINGS DAM, NID#AK00308," BY KNIGHT PIÉSOLD LIMITED, REV. 2, DATED DECEMBER 7, 2012.

CLIENT
COEUR ALASKA INC.
3031 CLINTON DRIVE, SUITE 202
JUNEAU, ALASKA 99801

CONSULTANT

YYYY-MM-DD	2014-12-16
PREPARED	SLA
DESIGN	N/A
REVIEW	SLA
APPROVED	TGK

PROJECT
LOWER SLATE LAKE DAM
PERIODIC SAFETY INSPECTION
KENSINGTON MINE, ALASKA

TITLE
MEASURED LAKE AND TAILINGS LEVELS ON HISTORIC FILLING SCHEDULE

PROJECT No. 1404132 | Rev. 0 | FIGURE 20

APPENDIX A

SITE INSPECTION PHOTOGRAPHS

PHOTO 1: TAILINGS STORAGE TREATMENT FACILITY, LOOKING SOUTH

PHOTO 2: TAILINGS STORAGE TREATMENT FACILITY, LOOKING NORTHWEST

Golder Associates

PANORAMAS OF TAILINGS TREATMENT FACILITY

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A1

PHOTO 3: WATER TREATMENT PLANT AND HEAD TANKS, LOOKING SOUTHWEST

PHOTO 4: INTAKE STRUCTURE, LOOKING EAST

PHOTO 5: RECLAIM WATER PUMP, LOOKING SOUTH

WATER TREATMENT PLANT, INTAKE, & RECLAIM BARGE PUMP

LOWER SLATE LAKE DAM PSI, GAI#1404132

Golder Associates

FIGURE A2

PHOTO 6: WATER CONVEYANCE PIPELINE, LOOKING SOUTH

PHOTO 7: PARSHALL FLUME, LOOKING NORTH

PHOTO 8: FLUME TRANSDUCER READOUT PANEL, LOOKING EAST

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: PSI Field Photos.dwg

Golder Associates

DIVERSION PIPELINE, FLUME, & FLUME READOUT PANEL

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A3

PHOTO 9: VIEW ALONG DAM CREST FROM EAST ABUTMENT, LOOKING WEST

PHOTO 10: VIEW ALONG DAM CREST FROM WEST ABUTMENT, LOOKING NORTHEAST

PANORAMAS FROM DAM ABUTMENTS

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A4

Golder Associates

PHOTO 11: UPSTEAM STAGE 2 DAM FACE, LOOKING WEST

PHOTO 12: EAST UPSTREAM DAM ABUTMENT, LOOKING NORTHWEST

PHOTO 14: WEST UPSTREAM DAM ABUTMENT AND
SPILLWAY ENTRANCE, LOOKING NORTH

UPSTREAM DAM FACE AND ABUTMENTS

LOWER SLATE LAKE DAM PSI, GAI#1404132

Golder Associates

FIGURE A5

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: PSI Field Photos.dwg

PHOTO 14: DOWNSTREAM STAGE 2 SPILLWAY CHANNEL ENTRANCE,
LOOKING SOUTH

PHOTO 15: SPILLWAY CHANNEL AND PLUNGE POOL,
LOOKING SOUTH

PHOTO 16: DOWNSTREAM DAM AREA,
LOOKING SOUTHEAST

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: PSI Field Photos.dwg

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STAGE 2 SPILLWAY AND DOWNSTREAM DAM AREA

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A6

PHOTO 17: DOWNSTREAM DAM FACE, LOOKING NORTHWEST

PHOTO18: DOWNSTREAM DAM FACE, LOOKING NORTH

Golder Associates

PANORAMAS OF DOWNSTREAM DAM FACE

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A7

PHOTO 19: DOWNSTREAM EAST DAM ABUTMENT, LOOKING WEST

PHOTO 20: TEMPORARY WATER TREATMENT PLANT, LOOKING NORTH

PHOTO 21: DOWNSTREAM WEST DAM ABUTMENT, LOOKING NORTH

Path: \\Anchorage\Public\Geomatics\Coeur\Kensington\99_PROJECTS\14-04132 Kensington Dam PSI\02_PRODUCTION\DWG\ | File Name: PSI Field Photos.dwg

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DOWNSTREAM ABUTMENTS AND TEMPORARY WTP

LOWER SLATE LAKE DAM PSI, GAI#1404132

FIGURE A8

PHOTO 22: SEEPAGE COLLECTION
SUMP PUMP READOUT PANEL

PHOTO 23: SEEPAGE COLLECTION SUMP,
LOOKING SOUTHEAST

PHOTO 24: PIEZOMETER READOUT PANEL,
LOOKING SOUTHEAST

SEEPAGE COLLECTION SUMP AND READOUT PANELS
LOWER SLATE LAKE DAM PSI, GAI#1404132

Golder Associates

FIGURE A9

PHOTO 25: REMAINS OF STAGE 1 SPILLWAY CHUTE, LOOKING SOUTH

PHOTO 26: WATER CONVEYANCE PIPE OUTLET AND PLUNGE POOL, LOOKING NORTHEAST

PHOTO 27: ENTRANCE INTO EAST SLATE CREEK, LOOKING SOUTH

SPILLWAY 1 CHUTE AND WATER DIVERSION OUTLET

LOWER SLATE LAKE DAM PSI, GAI#1404132

Golder Associates

FIGURE A10

APPENDIX B
ADNR FORMS

VISUAL INSPECTION CHECKLIST

ALASKA DAM SAFETY PROGRAM
VISUAL INSPECTION CHECKLIST

GENERAL INFORMATION

NAME OF DAM: Lower Slate Lake Tailings Dam	POOL ELEVATION: 692.5'
NATIONAL INVENTORY OF DAMS ID#: AK00308	TAILWATER ELEVATION:
OWNER: Coeur Alaska Inc.	CURRENT WEATHER: Overcast, sprinkles
HAZARD CLASSIFICATION: I	PREVIOUS WEATHER: Overcast, sprinkles
SIZE CLASSIFICATION: N/A	INSPECTED BY: Steve L. Anderson, PE
PURPOSE OF DAM: Tailings Treatment Facility	INSPECTION FIRM: Golder Associates Inc.
O & M MANUAL REVIEWED: 12/7/12 (R2)	DATE OF INSPECTION: June 24, 2014
EMERGENCY ACTION PLAN REVIEWED: 12/7/12 (R	

ITEM	YES	NO	REMARKS
RESERVOIR			
1. Any upstream development?		X	
2. Any upstream impoundments?		X	Upper Slate Lake
3. Shoreline slide potential?		X	
4. Significant sedimentation?		X	
5. Any trash boom?	X		Timber Boom
6. Any ice boom?		X	
7. Operating procedure changes?		X	

DOWNSTREAM CHANNEL			
1. Channel			
a. Eroding or Backcutting		X	
b. Sloughing?		X	
c. Obstructions?		X	
2. Downstream Floodplain			
a. Occupied housing?		X	
b. Roads or bridges?		X	
c. Businesses, mining, utilities?		X	
d. Recreation Area?		X	
e. Rural land?	X		Forest Service
f. New development?		X	

EMERGENCY ACTION PLAN			
1. Class I or Class II Dam?	X		12/7/12 (Rev1)
2. Emergency Action Plan Available?	X		
3. Emergency Action Plan current?	X		
4. Has EAP been tested in last year?	X		Last drill 11/20/2013

INSTRUMENTATION			
1. Are there			
a. Piezometers?	X		
b. Weirs?	X		
c. Observation wells?		X	Seepage Collection Sump
d. Settlement Monuments?		X	
e. Horizontal Alignment Monuments?		X	
f. Thermistors?	X		Temperature in Piezometers
2. Are readings			
a. Available?	X		
b. Plotted?	X		
c. Taken periodically?	X		

ALASKA DAM SAFETY PROGRAM
VISUAL INSPECTION CHECKLIST

SAFETY

ITEM	YES	NO	REMARKS
SAFETY			
1. ACCESS	X		**TYPE: Gravel road**
a. Road access?	X		
b. Trail access?		X	
c. Boat access?		X	
d. Air access?		X	
e. Access safe?	X		
f. Security gates and fences?		X	Remote site
g. Restricted access signs?		X	Remote site
2. PERSONNEL SAFETY			
a. Safe access to maintenance and operation area	X		
b. Necessary handrails and ladders available?	X		
c. All ladders and handrails in safe condition?	X		
d. Life rings or poles available?	X		
e. Limited access and warning signs in place?		X	
f. Safe walking surfaces?	X		
3. DAM EMERGENCY WARNING DEVICES			
a. Emergency Action Plan required?	X		
b. Emergency warning devices required by EAP?		X	**TYPE(S):**
c. Emergency warning devices available?			N/A
d. Emergency warning devices operable?			N/A
e. Emergency warning devices tested?			N/A
f. Emergency warning devices tested by owner?			**WHEN:**
g. Emergency procedures available at dam?			N/A
h. Dam operating staff familiar with EAP?	X		
4. OPERATION AND MAINTENANCE MANUAL			
a. O & M Manual reviewed?	X		
b. O & M Manual current?	X		**DATE: 12/07/12 (Rev 2)**
c. Contains routine inspection schedule?	X		
c. Contains routine inspection checklist?	X		

ALASKA DAM SAFETY PROGRAM

VISUAL INSPECTION CHECKLIST

EMBANKMENT DAMS

ITEM	YES	NO	REMARKS
EMBANKMENT DAMS			**TYPE:** Rockfill with U/S Lined Face
1. CREST			
a. Any settlement?		X	
b. Any misalignment?		X	
c. Any cracking?		X	
d. Adequate freeboard?	X		
2. UPSTREAM SLOPE			
a. Adequate slope protection?	X		
b. Any erosion or beaching?		X	
c. Trees or brush growing on slope?		X	
d. Deteriorating slope protection?		X	
e. Visual settlement?		X	
f. Any sinkholes?		X	
3. DOWNSTREAM SLOPE			**TYPE:** Rockfill
a. Adequate slope protection?	X		
b. Any erosion?		X	
c. Trees or brush growing on slope?		X	
d. Animal burrows?		X	
e. Sinkholes?		X	
f. Visual settlement?		X	
g. Surface seepage?	X		Near right (west) abutment toe
h. Toe drains dry?	X		
i. Relief wells flowing?	X		Seepage sump pump (2 gpm)
j. Slides or slumps?		X	
4. ABUTMENT CONTACTS			
a. Any erosion?		X	
b. Seepage present?		X	
c. Boils or springs downstream?		X	
5. FOUNDATION			**TYPE:** Treated bedrock surface
a. If dam is founded on permafrost		X	
(1) Is fill frozen?			N/A
(2) Are internal temperatures monitored?			N/A
b. If dam is founded on bedrock	X		**TYPE:** Pyrite-bearing Phyllite
(1) Is bedrock adversely bedded?		X	
(2) Does rock contain gypsum?		X	
(3) Weak strength beds?		X	
c. If dam founded on overburden		X	**TYPE: N/A**
(1) Pipeable?			N/A
(2) Compressive?			N/A
(3) Low shear strength?			N/A

ALASKA DAM SAFETY PROGRAM
VISUAL INSPECTION CHECKLIST

SPILLWAYS

ITEM	YES	NO	REMARKS
SPILLWAYS			**TYPE(S):** Shotcrete chute
1. CREST			**TYPE(S):** Trapizoidal Broad Crested
a. Any settlement?		X	
b. Any misalignment?		X	
c. Any cracking?		X	
d. Any deterioration?		X	
e: Exposed reinforcement?		X	
f. Erosion?		X	
g. Silt deposits upstream?		X	
2. CONTROL STRUCTURES		X	
a. Mechanical equipment operable?			N/A
b. Are gates maintained?			N/A
c. Will flashboards trip automatically?			N/A
d. Are stanchions trippable?			N/A
e. Are gates remotely controlled?			
3. CHUTE			
a. Any cracking?		X	
b. Any deterioration?		X	
c. Erosion?		X	
d. Seepage at lines or joints?	X		Some minor seepage at west face
4. ENERGY DISSIPATERS	X		Plunge pool with baffle blocks
a. Any deterioration?			Obscured by ponded water
b. Erosion?			N/A
c. Exposed reinforcement?			Obscured by ponded water
5. METAL APPURTENANCES		X	
a. Corrosion?			N/A
b. Breakage?			N/A
c. Secure anchorages?			N/A
6. EMERGENCY SPILLWAY		X	
a. Adequate grass cover?			N/A
b. Clear approach channel?			N/A
c. Erodible downstream channel?			N/A
d. Erodible fuse plug?			N/A
e. Stable side slopes?			N/A
f. Beaver dams present?			N/A

ALASKA DAM SAFETY PROGRAM
VISUAL INSPECTION CHECKLIST

INTAKES

ITEM	YES	NO	REMARKS
INTAKES	X		
1. EQUIPMENT			
a. Trash racks		X	
b. Trash rake?		X	
c. Mechanical equipment operable?	X		
d. Intake gates?	X		Knife gate valve
e. Are racks and gates operable?			N/A
f. Are gate operators operable?	X		
2. CONCRETE SURFACES	X		Concrete broad-crested weir
a. Any cracking?		X	
b. Any deterioration?		X	
c. Erosion?		X	
d. Exposed reinforcement?		X	
e. Are joints displaced?		X	
f. Are joints leaking?		X	
3. CONCRETE CONDUITS		X	
a. Any cracking?			N/A
b. Any deterioration?			N/A
c. Erosion?			N/A
d. Exposed reinforcement?			N/A
e. Are joints displaced?			N/A
f. Are joints leaking?			N/A
4. METAL CONDUITS		X	
a. Is metal corroded?			N/A
b. Is conduit damaged?			N/A
c. Are joints displaced?			N/A
d. Are joints leaking?			N/A
5. METAL APPURTENANCES			
a. Corrosion?		X	
b. Breakage?		X	
c. Secure anchorages?	X		
6. PENSTOCKS		X	**TYPE MATERIAL:**
a. Material deterioration?			N/A
b. Joints leaking?			N/A
c. Supports adequate?			N/A
d. Anchor blocks stable?			N/A

PROJECT DATA SHEET

LOWER SLATE LAKE TAILINGS DAM
PROJECT DATA SHEET

GENERAL

Dam Name:	Lower Slate Lake Tailings Dam
NID Number:	AK00308
Hazard Class:	II (significant)
Purpose:	Tailings treatment facility
Year Built:	2010 (Stage 1)
Year Modified:	2012 (Stage 2)
Location:	58.8073° latitude, -135.0386° longitude
Reservoir Name:	Kensington Mine Tailings Treatment Facility
River or Creek Name:	East Slate Creek / Lower Slate Lake
Owner:	Coeur Alaska Inc.
Owner Contact:	Ed Coffland
Address:	3031 Clinton Drive, Suite 202 Juneau, AK 99801 Phone: (907) 523-3332 Fax: (907) 895-2866

DAM

Type:	Geosynthetic Faced Rockfill Dam (GFRD)
Core Type:	Rockfill dam with 100 mil HDPE geomembrane on upstream face and grout trench excavated into bedrock
Crest Length:	480 feet
Crest Width:	33 feet
Crest Elevation:	715 feet
Crest Height (from d/s toe):	63 feet
Hydraulic Height:	57 feet

INTERIM SPILLWAY

Type:	Trapezoid channel to plunge pool energy dissipation structure and outlet channel back into natural drainage course
Location:	Right (west) abutment
Invert Elevation:	709 feet
Top Width:	16 feet (varies along profile)
Bottom Width:	10 feet
Length:	550 feet
Discharge Capacity at Dam Crest:	1,020 cubic feet per second

CLOSURE SPILLWAY

Type:	To be constructed at closure following operations
Location:	Right (west) abutment
Invert Elevation:	732 feet
Depth:	8 feet
Top Width:	15 feet
Bottom Width:	22.5 feet
Length:	600 feet
Discharge Capacity at Dam Crest:	1,256 cubic feet per second

OUTLET WORKS

LOWER SLATE LAKE TAILINGS DAM
PROJECT DATA SHEET

Type:	Floating reclaim pump facility
Location:	North end of facility
Invert Elevation:	650 to 695 feet (floating intake)
Outlet Invert Elevation:	Tank elevation at Water Treatment Plant
Diameter:	10 inches
Length:	1,000 feet
Outlet Type:	Discharge to Water Treatment Plant
Discharge Capacity to WTP:	3.3 cubic feet per second

RESERVOIR

Normal Water Surface Elevation:	695 feet
Normal Storage Capacity:	2,300 acre-feet
Maximum Water Surface Elevation:	709 feet
Maximum Storage Capacity:	2,800 acre-feet
Max. Surface Area at Spillway Crest:	53 acres
Surface Area at Spillway Crest:	51 acres

HYDROLOGY

Drainage Basin Area:	294 acres (Catchment Areas 1a, 1b, and 1c, Lower Slate Lake) 379 acres (Catchment Area 2, Upper Slate Lake)
Average Annual Rainfall:	85 inches
Storm Storage Equivalent Runoff Depth:	10.2 inches
Storm Storage Flood Volume:	589 acre-feet
1,000 Year/24 Hour Rainfall:	14.7 inches
1,000 Year Flood Volume:	667 acre-feet
1,000 Year Flood Peak Flow Rate:	893 cubic feet per second
Probable Maximum Precipitation:	17.3 inches (over 24 hour period)
Probable Maximum Flood:	1,256 cubic feet per second
Probable Maximum Flood Volume:	1,489 acre-feet
Flood of Record:	Not available

APPENDIX C

2014 TAILINGS SURVEY RESULTS

TRANSMITTAL

TO:	Coeur Alaska Inc. 3031 Clinton Drive, Suite 202 Juneau, Alaska USA, 99801	**DATE:**	August 6, 2014
		FILE NO.:	VA101-20/28-A.01
ATTENTION:	Mr. Ed Coffland	**CONT. NO.:**	VA14-01176

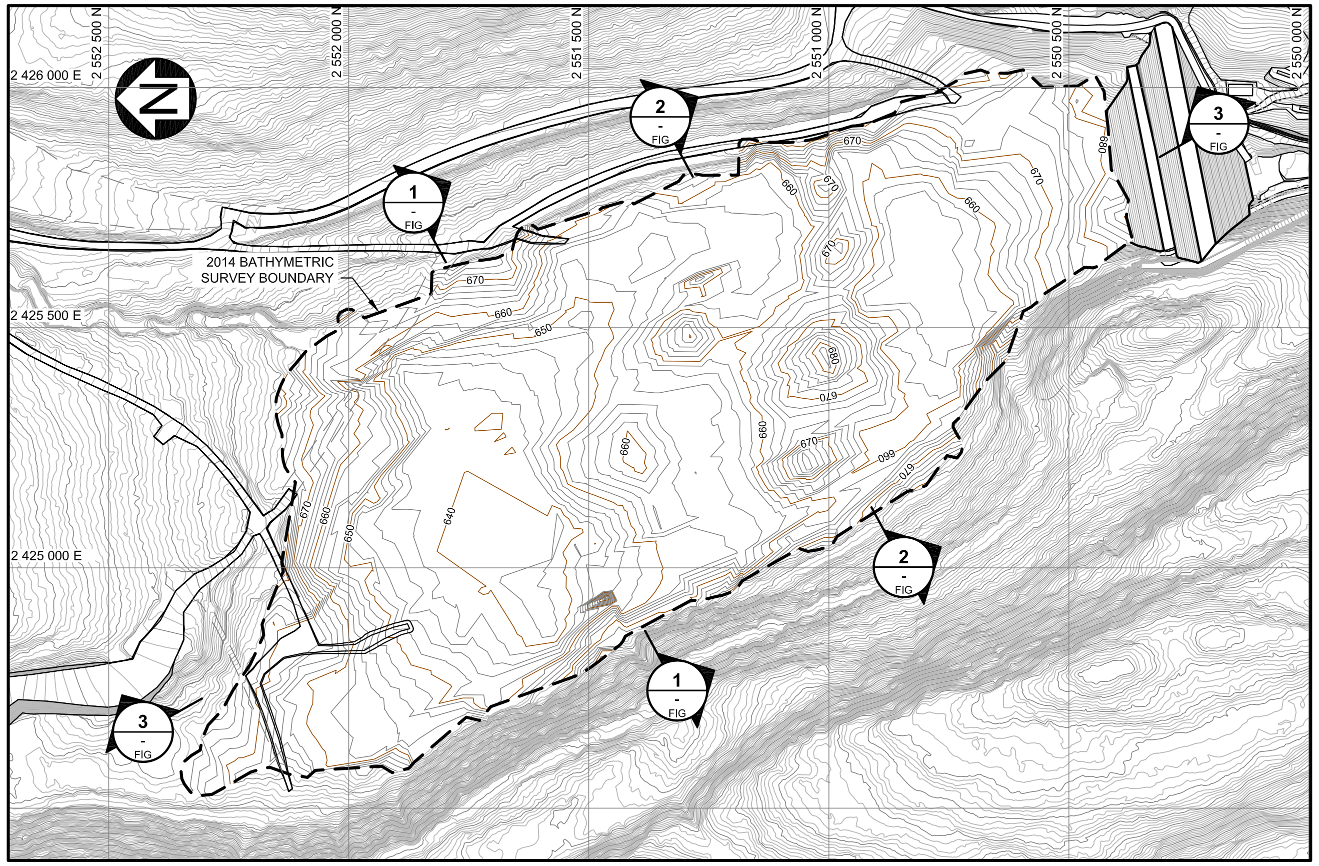
RE: Preparation Support for TTF Tailing Dam Stage 3

ITEM NO.	DESCRIPTION
1.	1 Copy of Each – Figures Figure 1 Rev 0 – tailings Treatment Facility 2014 Bathymetric Survey – Plan and Sections

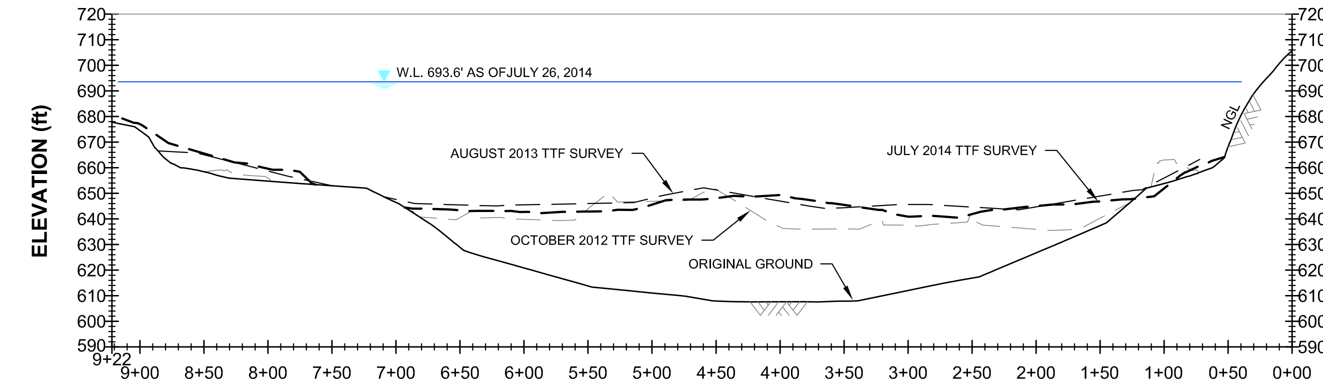
REMARKS:

Signed: Erin Legere

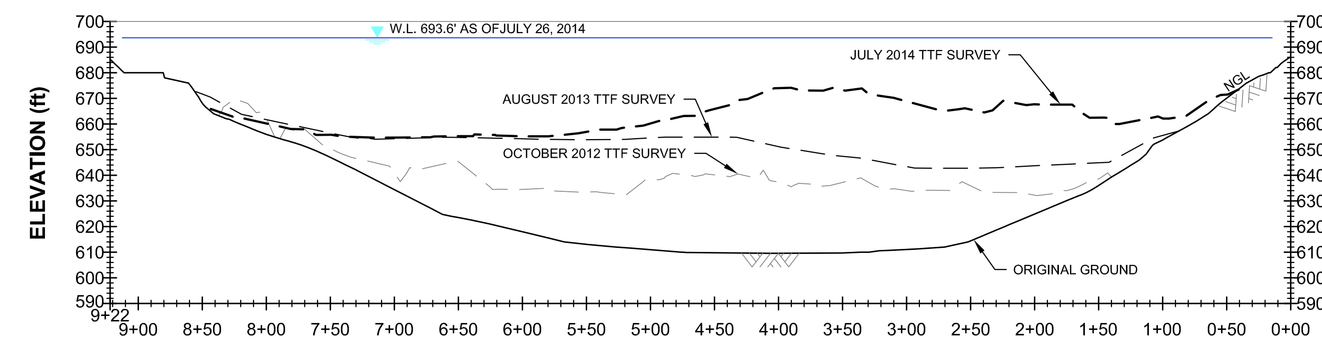
Approved: for Brad Hamilton



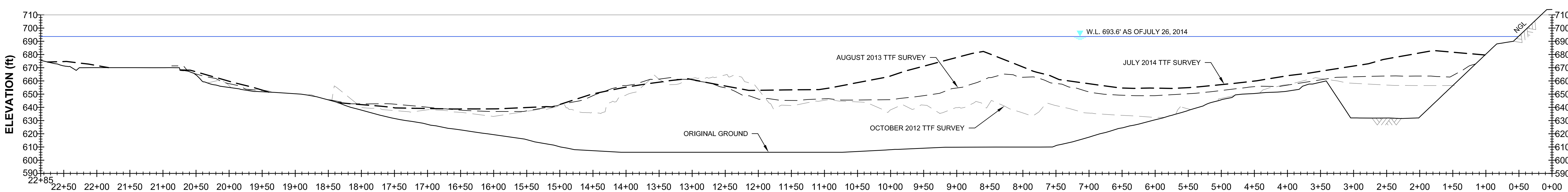
KEY PLAN
TTF 2013 AS-BUILT SURVEY
NTS



1 - FIG SECTION
HORIZONTAL: SCALE A
VERTICAL: SCALE B



2 - FIG SECTION
HORIZONTAL: SCALE A
VERTICAL: SCALE B



3 - FIG SECTION
HORIZONTAL: SCALE A
VERTICAL: SCALE B

NOTES:

1. PLAN/SECTION BASED ON INFORMATION PROVIDED BY KENSINGTON MINES, DATED JULY, 2014.
2. CONTOUR INTERVAL IS 2 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. TOTAL TAILINGS VOLUME AS OF AUGUST : 986 373 yd³

SCALE B: 150 0 150 300 450 600 750 ft
SCALE A: 75 0 75 150 225 300 375 ft

COEUR ALASKA INC.

KENSINGTON PROJECT

TAILINGS TREATMENT FACILITY 2014 BATHYMETRIC SURVEY PLAN AND SECTIONS

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-20/28	VA14-01176

FIGURE 1 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	CHK'D	APP'D
0	06AUG'14	ISSUED FOR INFORMATION	BSH	NSD	BB	KDE

APPENDIX D

2014 UPDATED WATER BALANCE

TECHNICAL MEMORANDUM

Date: September 4, 2014 **Project No.:** 073-93714-03.006

To: Ed Coffland **Company:** Coeur Alaska

From: Adrianne Yang and Mike Brown

RE: **UPDATED KENSINGTON SITE WIDE WATER BALANCE SUMMARY**

This technical memorandum provides an updated summary of the site wide water balance model that was developed for the Kensington Mine as well as the recent model calibration results related to the Tailings Treatment Facility (TTF).

1.0 MODEL DESCRIPTION

The water balance model simulates the operation of the Kensington Mine with respect to water flow rates and storage volumes in all the major components of the mine, which are shown schematically in Figure 1-1. The model tracks these flow rates and volumes on a daily time step through the life of the mine. The model incorporates measured precipitation, stream flows, and operational data, when available, during the life of the mine. As additional data are collected during mine operations, the calibration parameters included with the model can be modified as necessary. Over time, these modifications will result in improved predictions of future flows.

The mine is modeled by defining a network of water sources, storage volumes, and interconnecting conveyance features (pipes, pumps, etc.). Maximum flow rates and storage volumes are limited by the parameters selected for design. For example, the water treatment plants have a maximum treatment rate that cannot be exceeded. The operation of the mine is also modeled through the use of operating rules; e.g., the model may be configured so that the mill draws water from the portals before taking water from Johnson Creek or reclaim from the TTF.

The model simulation starts on August 1, 2010 and ends on July 1, 2022 (a 12 year mine life). The initial assumptions for operational parameters, such as milling rate, were based on the mine plan and information from Coeur Alaska (Coeur). As site specific data became available, actual values for operational parameters were incorporated into the model.

1.1 Meteorological Data

1.1.1 Precipitation

The primary driver of the model is precipitation. The site precipitation record is used for the simulation of historic conditions and model calibration. Stochastic precipitation is based on the statistics of historic site (measured at Jualin Heights) and regional precipitation.

Golder Associates Inc.
18300 NE Union Hill Road, Suite 200
Redmond, WA 98052 USA
Tel: (425) 883-0777 Fax: (425) 882-5498 www.golder.com

For projections into the future, the model can generate stochastic input precipitation data that are statistically similar to historic precipitation, including means, standard deviations, and auto-correlation. The statistics required for stochastic precipitation generation were based on the period of record at Juneau (COOP ID 504092). A comparison of historic site precipitation and the precipitation generated within the model (GoldSim) is provided in Table 1-1; the mean monthly values are the same.

Table 1-1: Mean Monthly Precipitation

Month	Historic Site Precipitation for Period of Record (in)	GoldSim Generated Precipitation for Life of Mine (in)[1]
January	5.90	5.90
February	7.05	7.05
March	4.86	4.86
April	3.8	3.80
May	4.4	4.40
June	3.21	3.21
July	4.47	4.47
August	6.62	6.62
September	10.89	10.89
October	16.07	16.07
November	9.91	9.91
December	7.81	7.81
Total	84.99	84.99

Note:
[1] This is the average of 12 years and 100 realizations, so the equivalent of 1200 years. Mean annual precipitation for individual years and single realizations (12 years) deviate from the historic mean annual precipitation, sometimes substantially.

1.1.2 Rain and Snow

The model input includes historic and projected future precipitation. This precipitation falls as snow or rain throughout the year. The model assumes that, for any given month, a constant fraction of the precipitation falls as snow and the rest falls as rain, according to the percentages shown in Table 1-2 (Tetra Tech Inc. 2004a)[1]. If precipitation falls as snow it is assumed to accumulate as a snowpack. These values result in an annual average snowpack snow water equivalent (SWE) of about 20 inches in an average precipitation year (85 inches).

[1] Tetra Tech Inc. 2004a. Kensington Gold Project Final Supplemental Environmental Impact Statement, Volume 2: Appendices A-L, dated December. Lakewood CO.

Table 1-2: Rain and Snowfall Distribution

Month	Precipitation as Rain	Precipitation as Snow
January	45%	55%
February	45%	55%
March	60%	40%
April	100%	0%
May	100%	0%
June	100%	0%
July	100%	0%
August	100%	0%
September	100%	0%
October	75%	25%
November	75%	25%
December	45%	55%

The total amount of snowfall in the year is stored as a snowpack that is melted from the beginning of April through the end of September at a constant daily rate for each of the months, as shown in Table 1-3. For example, each day in April would have a snowmelt depth that is 0.33% of the snowpack SWE that year. The sum of the percent-days is 100% in the five month period. The daily snowmelt value is added to the daily rainfall to calculate the amount of water available for runoff.

Table 1-3: Monthly Snowmelt as Fraction of Maximum SWE

Month	Snowmelt Rate (inch/day per inch of Maximum Annual SWE)
January	0
February	0
March	0
April	0.0033
May	0.0132
June	0.0113
July	0.0032
August	0.0016
September	0
October	0
November	0
December	0

1.1.3 Evaporation

Evaporation in the model is based on an annual average value of 17.1 inches for free water surfaces, which was provided in the Environmental Impact Statement for the project (Tetra Tech Inc. 2004b)[2]. The evaporation is assumed to occur during the months of April through September according to the distribution in Table 1-4.

Table 1-4: Monthly Free Water Evaporation

Month	Monthly Evaporation (inches)
January	0
February	0
March	0
April	1.5
May	3.3
June	3.7
July	3.4
August	3.7
September	1.5
October	0
November	0
December	0

2.0 TAILINGS TREATMENT FACILITY

The primary focus of the water balance model to date has been on the TTF. The volume of free water in the TTF pool is calculated in the model by adding inflows and subtracting outflows. The inflows to the TTF pool include direct precipitation, runoff from the areas surrounding the TTF and water in the tailings slurry. Slate Creek is assumed to be diverted around the TTF. Outflows from the TTF free water pool include evaporation, retained tailings pore water, reclaim water for use in the mill, and water that is treated and discharged to Slate Creek.

At the start of the model simulation, in August 2010, the TTF was assumed to be at an elevation of 646 feet, which corresponds to a volume of about 415 acre-feet. Stage-storage curves are incorporated in the model for easy comparison between measured and modeled TTF level.

2.1 TTF Runoff

Runoff flows into the TTF as a result of precipitation or snowmelt onto the four catchment areas around the TTF, listed in Table 2-1. Three of these areas are at least partially diverted around the TTF. The diversions are assumed to be 100 percent effective. The volume of runoff entering the TTF is calculated

[2] Tetra Tech Inc. 2004b. Kensington Gold Project Final Supplemental Environmental Impact Statement, Volume 1: Sections 1-10, dated December. Lakewood CO.

using the drainage basin areas, the fraction of the area that contributes to the TTF, the amount of precipitation and snowmelt, and a runoff coefficient. The runoff coefficients in the model, along with an orographic effects adjustment, are treated as calibration parameters and are discussed further in Section 4.2.

Table 2-1: TTF Contributing Drainage Basins

Basin Name	Description	Area (ac)	Percent Diverted Around TTF
TTF 1a	Northeast End of TTF	153	60%
TTF 1b	West of TTF	46	50%
TTF 1c	Lower Slate Creek Lake	94.6	0%
TTF Catchment 2	Upper Slate Creek Lake	378.7	100%

2.2 TTF Water Treatment

Water discharged to Slate Creek from the TTF is first treated in the water treatment plant. If available, the model uses historic TTF treatment rates for simulation of historic conditions. Otherwise, the model calculates the required withdrawal rate and volume, up to 1500 gallons per minute (gpm), to draw the pool down to a minimum level. Regulatory requirements require that 9 feet of water remain over the tailings at all times, and the model accounts for this requirement when determining the water withdrawal volume.

2.3 Other Outflows

The water balance model tracks solids and water separately and all of the water in the tailings slurry is initially assumed to become part of the TTF free water pool. In reality, when the slurry enters the TTF there is a rapid separation of solids and water but some water remains within the tailings slurry. After deposition, the tailings settle and consolidate over time, slowly releasing additional pore water to the TTF pool. Not all of the water in the slurry ends up in the TTF pool; some is permanently entrained in the tailings. The model calculates how much water is permanently trapped within the tailings voids, using the tailings production rate, dry density and specific gravity of the tailings provided by Coeur, and designates this volume as lost from the TTF free water pool.

The amount of reclaim water pumped from the TTF to the mill is determined by the amount of water needed in the mill for processing, limited by the use of other sources, availability in the TTF, and facility capacity. The mill uses water from Johnson Creek or, if needed, the underground mine for processes that require fresh water. Reclaim water from the paste backfill plant is used before the mill uses reclaimed water from the TTF. However, there is a limit on the amount of water that can be reclaimed from the paste backfill plant. For this reason there is usually a demand for reclaim water from the TTF.

3.0 OTHER FACILITIES

Other facilities that are included in the site wide water balance, but have not been the main focus of study, are described below.

3.1 Mill

The mill water requirements are computed based on incoming ore composition, tailings slurry water requirements, concentrate, pebble production rate, and paste backfill properties and water contents. Initial assumptions about the properties of the ore, tailings, concentrate, and paste backfill are shown in Table 3-1 and were obtained from Coeur. As historic processing information becomes available, these parameters are adjusted.

Table 3-1: Initial Production Inputs for Water Balance Model

Description	Value
Ore – Production Rate	1250 tons per day, dry
Ore - % Water (by weight)	6%
Tailings to TTF	721 – 1202 tons per day, dry
Tailings - % Water in Slurry (by weight)	55%
Tailings to Paste Backfill	0 – 480 tons per day, dry
Paste Backfill - % Water	14%
Concentrate – Production Rate	48 tons per day, dry
Concentrate - % Water (by weight)	7%
Pebble Production Rate	47 tons per day, dry
Pebble - % Water (by weight)	0%

3.2 Paste Backfill Plant

The Paste Backfill Plant became operational in May 2012. The Paste Backfill Plant further thickens and stabilizes a portion of the tailings for use as backfill in the underground mine, which reduces the amount of tailings that are sent to the TTF. Coeur is tracking the amount of tailings that are sent to the Paste Backfill Plant along with the water content of the final product. For historic computations these data are incorporated into the water balance model, but for predictive purposes, the model assumes the following:

- 51% of tailings from the Mill are sent to the Paste Backfill Plant
- The paste backfill has an ultimate moisture content of 24.8% water by weight
- The Paste Backfill Plant uses an average of 45 gpm of water from an underground sump as a required fresh water source during processing

Using these assumptions and the data collected by Coeur, the model calculates the amount of water that is recycled back to the Mill for use in processing.

3.3 Johnson Creek

Johnson Creek serves as the source for domestic water at the mining camp, gland water in the mill, and for other processes that require fresh water. The model tracks when the flow in Johnson Creek at the compliance point approaches the in-stream flow requirements, limiting withdrawals from Johnson Creek to times when the flow in Johnson Creek exceeds the in-stream flow requirements. Withdrawals from Johnson Creek are limited in quantity by available site storage, as well as meeting in-stream flow requirements.

3.4 Mine Portals

Underground mine discharges are estimated based on established relationships between precipitation, baseflow and historic measured mine discharge. Separate relationships were developed for the Comet and Jualin portals (Golder 2010)[3]. These relationships are used to determine how much water is sent to the Comet water treatment plant (WTP) or is collected in the mine sumps and is available for use at the mill or mine camp.

3.5 Comet Water Treatment Plant

The Comet WTP module includes one influent pond that stores water from the mine portals before it is treated at the WTP. Treated water is discharged to Sherman Creek. In the model, the Comet WTP has no maximum capacity because actual operations have shown that the WTP must treat all flows from the mine. This portion of the model has not recently been compared to actual operations at the Comet WTP.

4.0 MODEL CALIBRATION

This section describes the current calibration of the model. As production data, water treatment rates, and other measured flows, levels and volumes, such as the predicted water level in the TTF, become available; the calibration parameters are adjusted so that the outputs predicted by the model more closely match the measured values.

4.1 Snowpack

Applying the model's assumed monthly distribution of snowfall and rain to the measured precipitation in 2012 would produce a snowpack that is about 50% higher than observed. However, the Model does produce reasonable results in typical years. To provide for the anomaly in 2012, an adjustment factor was added to historical data to produce the SWE that was recorded on site on February 5, 2012 (16.4 inches). Site personnel indicated that the maximum SWE for 2012, which occurred a few weeks later, may have been up to 50% higher than 16.4 inches. The adjustment factor was varied until the modeled snowpack on February 5 matched the measured value. Using this

[3] Golder Associates Inc. (Golder). 2010. Estimated Groundwater Inflow to the Kensington Mine, Near Juneau, Alaska for the 1-in-20 year October Precipitation Conditions. Dated March 26. Redmond WA.

adjustment factor the maximum computed snowpack for 2012 was 19.6 inches, or about 20% higher than the measured February 5 value.

4.2 TTF Level

The primary focus of the water balance has been to predict water levels in the TTF. The primary calibration factors have been the runoff fractions and the orographic effect adjustment. Reliable daily precipitation measurements at the site began in January 2011, but the model simulations start in August 2010, when only monthly estimates of precipitation were available.

As more operational data was collected, it became clear that the model relies not only on accurate precipitation records, but also on accurate records of TTF reclaim water pumping rates and TTF water treatment rates. The most complete set of data for these three parameters began in January 2013, which is the starting point for the model calibration. The model is calibrated to the operational data collected between January 2013 and December 2013.

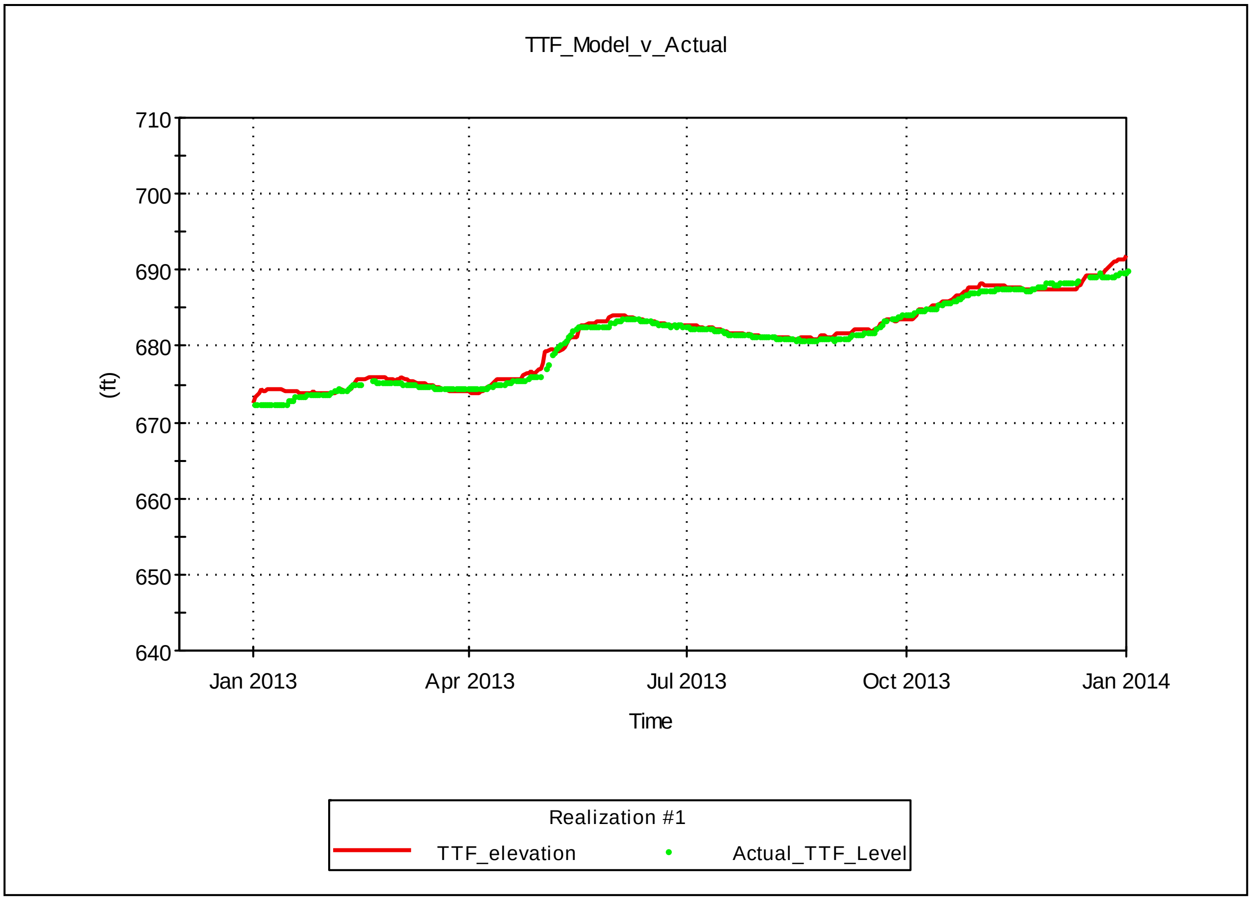
The calibration process for runoff coefficients and the orographic effect adjustment was done using the following process:

1. Search the historical data for anomalous values, such as a daily rainfall similar to storm event values or processing rates that are much higher than previously used. These values should not be removed from the historical data set, unless there is reason to believe they were erroneously recorded.
2. Run the model starting in January 2013 using the following values:
   - TTF Initial Volume: 47,434,819 ft^3 (Elevation 672.5 ft)
   - Starting Snowpack SWE: 10.79 inches
3. Adjust the calibration parameters (runoff coefficients) as needed to get best fit when comparing modeled TTF water levels to measured TTF water levels.
4. Run the model starting in August 2010 using the following values:
   - TTF Initial Volume: 18,081,360 ft^3 (Elevation 646 ft)
   - Starting Snowpack (SWE): 0.99 inches
5. Adjust the orographic effects coefficient, if needed, to get the best fit when comparing modeled TTF water levels to measured TTF water levels.

4.2.1 Results of TTF Calibration

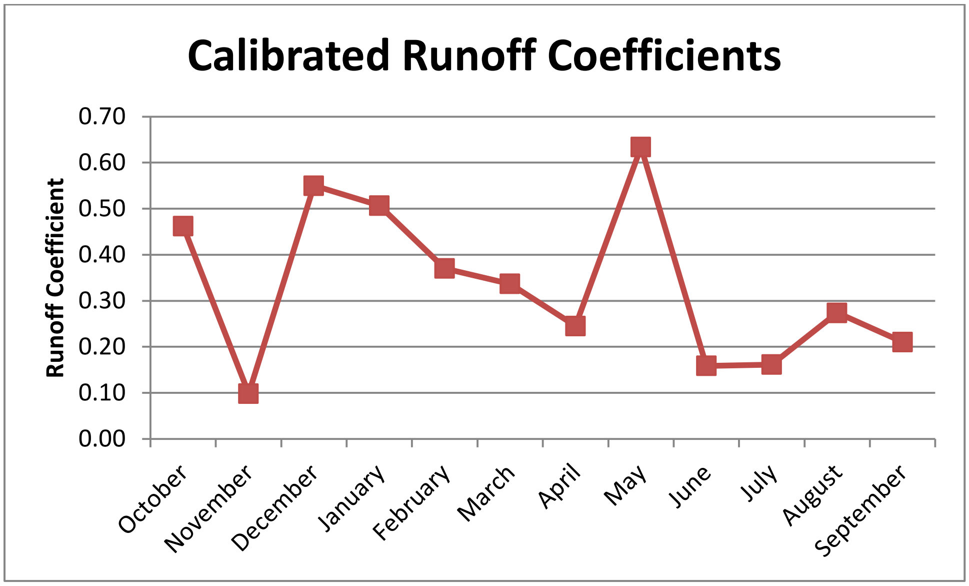
Figure 4-1 shows the results of the first three steps in the calibration process. The red line represents the modeled TTF water level and the green line represents the measured data. Generally, the calibration provides a good fit, but occasionally the model predicts a TTF water level greater than actual.

Figure 4-1: Water Balance Calibration, January 2013 through December 2013



The runoff coefficients that were used for this calibration are shown in Figure 4-2.

Figure 4-2: Calibrated Runoff Coefficients



The monthly calibration parameters from the 2013 calibration were used in the model starting in August 2010. The model was run using the estimated precipitation in 2010 and the measured precipitation in 2011 through July 2014. The resulting calibration is shown in Figure 4-3, after adjusting the orographic effect calibration parameter.

Figure 4-3: Water Balance Calibration, August 2010 through July 2014

Overall, the model shape still generally matches the shape of the TTF levels, but doesn't predict the TTF elevation accurately before June 2013, and often over predicts TTF level. The differences between the modeled and measured TTF levels are primarily due to incomplete or estimated input data, which was corrected in January 2013. Since that time the required inputs have been reliably recorded on a daily basis.

5.0 PREDICTED TTF LEVEL

Predicted TTF elevations are shown in Figures 5-1, 5-3 and 5-4. The year, shown on the horizontal axis of these figures, indicates the beginning of the calendar year. The TTF levels in these three figures through July 2014 are the modeled values that have been calibrated to historical values.

Figure 5-1 shows the mean water surface elevation in the TTF when considering a full range of precipitation scenarios, including dry and wet years. It also includes the (cumulative) tailings elevation and the maximum allowable water surface elevation that still maintains storage for the 200 year storm event below the spillway, also known as the 200 year storm elevation. The maximum (allowable) water surface elevation was computed using the planned schedule for dam and spillway raises. The TTF level draws down quickly through 2014, due to the assumptions related to the operation of the TTF water treatment plant. From August 2014 through January 2015, the model assumes that the TTF WTP operates at 1500 gpm continuously (Figure 5-2), in an effort to reduce TTF volume. The TTF WTP rate shown in Figure 5-2 is primarily operational data for periods before July 31, 2014. Beginning in August 2014, the TTF rate shown is the model predicted TTF WTP rate.

Figure 5-1: Mean TTF Pond Water Surface Elevation, Tailings Elevation, and Maximum Pond Elevation to Maintain Storage for the 200 Year Storm Event

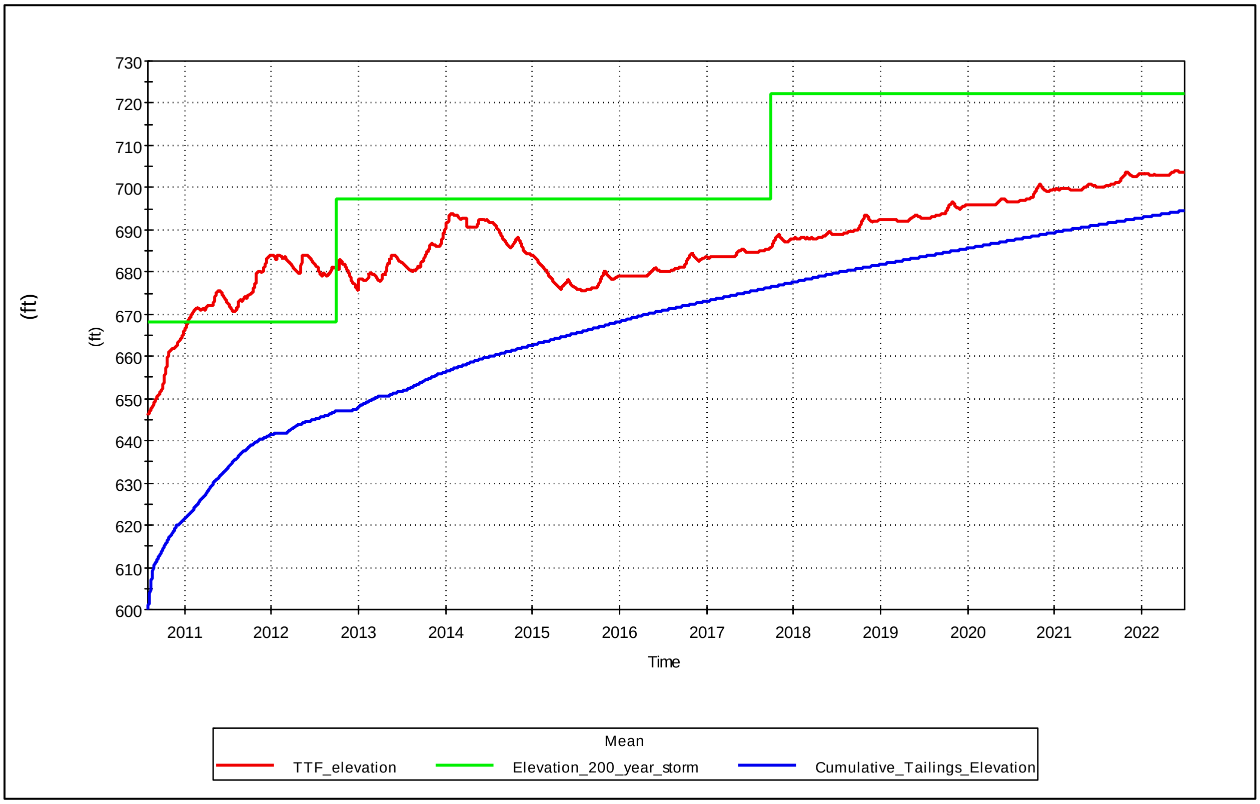
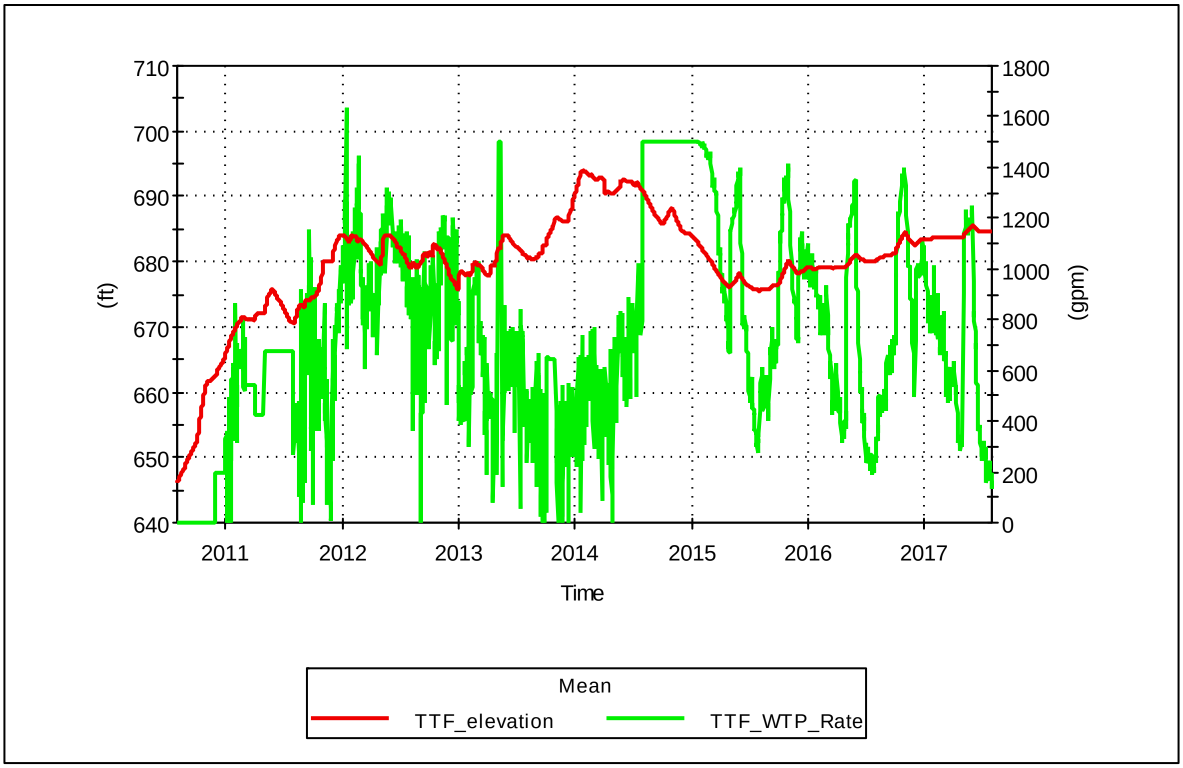


Figure 5-2: TTF Elevation and TTF WTP Rate



The range of values shown in Figures 5-3 and 5-4 represents the possible TTF elevations. The blue dashed line represents the mean TTF elevation, while the darker green represents the maximum predicted elevation and the lighter blue represents the minimum predicted elevation. The solid red line represents the maximum water surface elevation that maintains a reserve storage volume equal to or greater than the 200-year storm volume below the spillway. Figure 5-3 shows that TTF elevations are currently below the maximum water surface elevation allowable to contain the 200 year storm, but could exceed that elevation and remain high if the WTP is operating at a maximum of only 1000 gpm. Figure 5-4 shows that if the TTF water treatment rate is increased to a maximum of 1500 gpm, the maximum predicted TTF water levels could exceed the 200 year storm elevation for a short period, but would decrease rapidly.

Figure 5-3: Range of Predicted TTF Elevations at TTF Water Treatment Rate of 1000 gpm

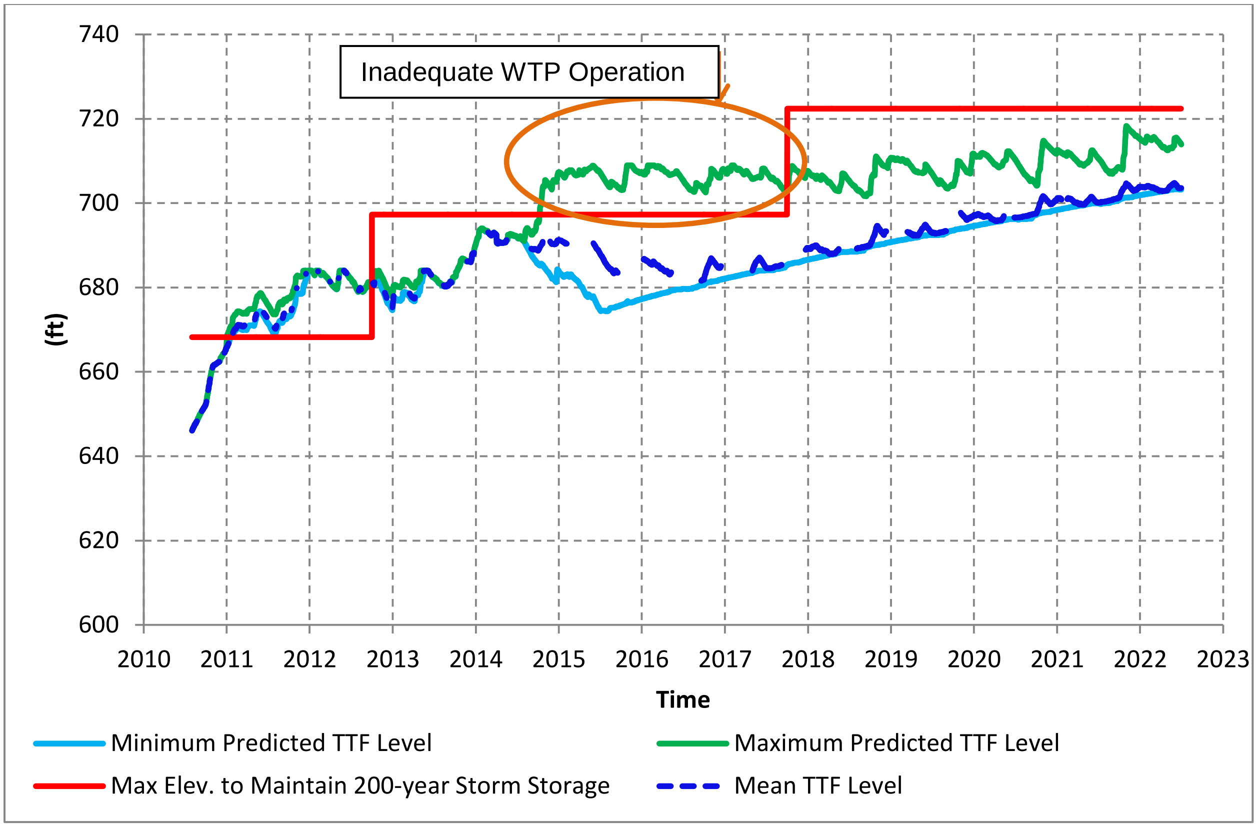
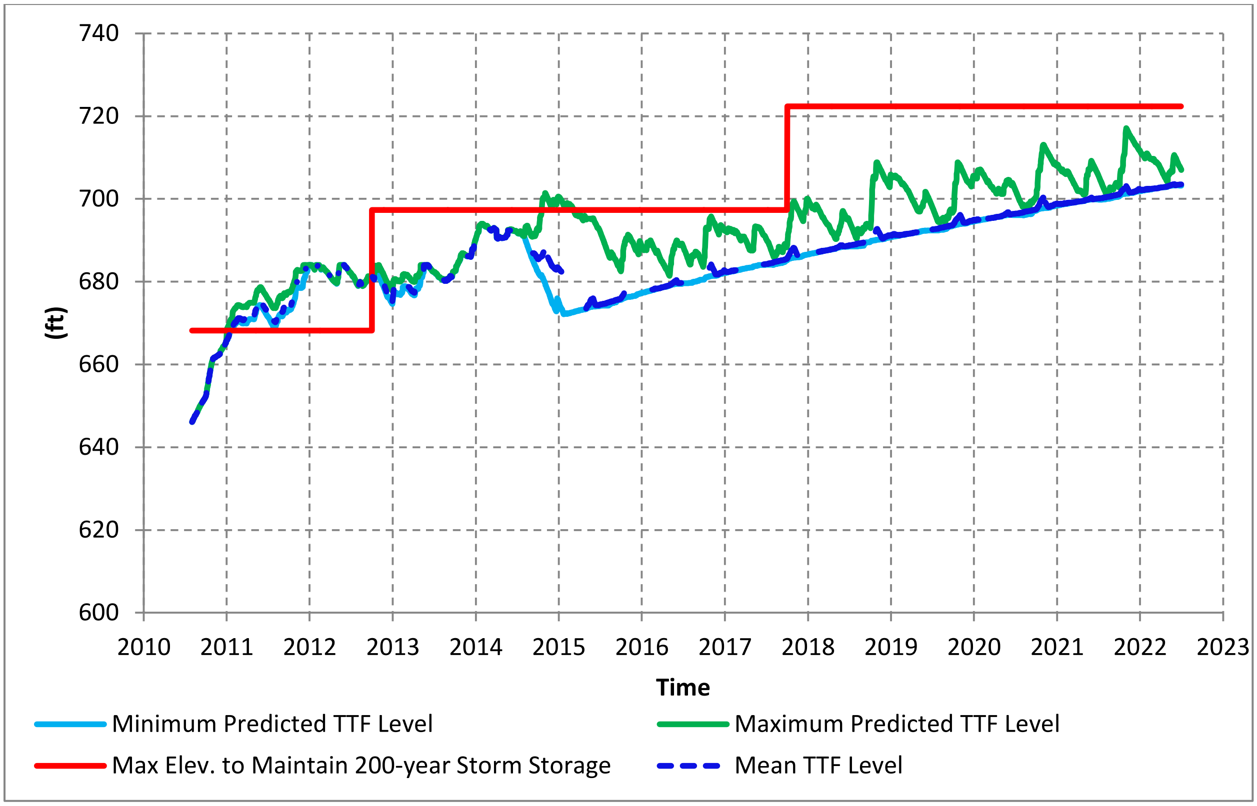


Figure 5-4: Range of Predicted TTF Elevations at TTF Water Treatment Rate of 1500 gpm



GOLDER ASSOCIATES INC.

Adrianne Yang, PE
Senior Project Engineer

Michael L. Brown, PE
Principal

AY/MLB/sb

Golder Associates

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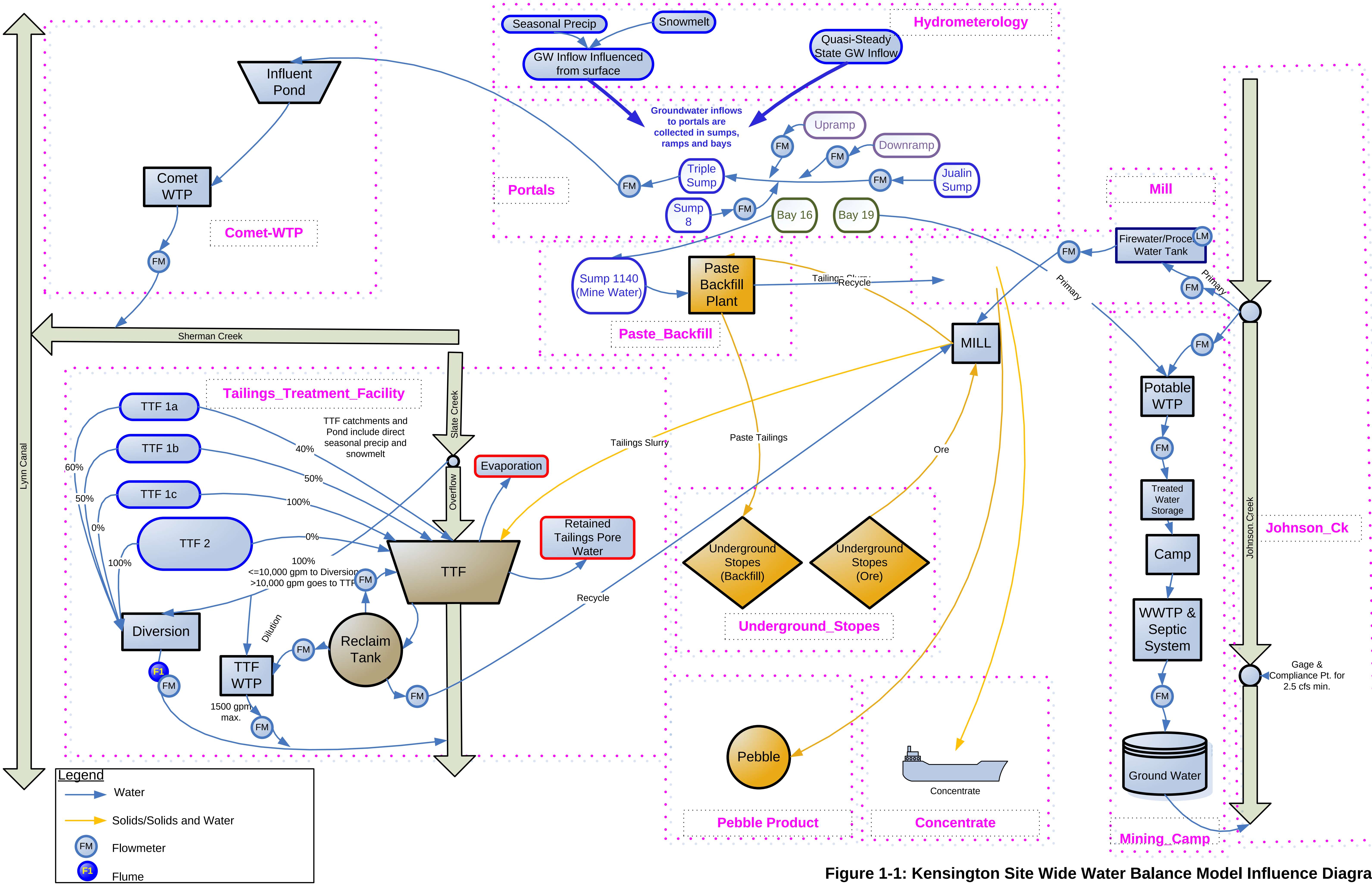

Figure 1-1: Kensington Site Wide Water Balance Model Influence Diagram

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Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Inc.
2121 Abbott Road, Suite 100
Anchorage, AK 99507
Tel: (907) 344-6001
Fax: (907) 344-6011

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