Appendix 3-D

Tailings Storage Facility Design Report

AJAX PROJECT

Environmental Assessment Certificate Application / Environmental Impact Statement for a Comprehensive Study

Status	Status Title	Check
A	Approved (No Further Action Required)	X
B	Proceed - (Revise and Resubmit Final)	
C	Do Not Proceed (Revise and Resubmit)	
I	Information Only	

Approver: [signature] Date: 27 Aug 15

KGHM review of consultant's drawings shall in no manner alleviate the consultant from their [illegible] as engineer of record. The review or approval of Consultant's drawings does not constitute [illegible] of satisfaction of waiver of any term of Consultant's work. These drawings remain the responsibility of Consultant.

Tailings Storage Facility Design Report – Rev 0

Submitted to:
KGHM Ajax Mining Inc.

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Norwest Corporation
Suite 1830, 1066 W. Hastings Street
Vancouver, British Columbia V6E 3X2
Tel: (604) 602-8992
Fax: (604) 602-8951
Email: vancouver@norwestcorp.com

www.norwestcorp.com

Author:
Christopher Klassen, P.Eng.,
Eugene Ngwenya, P.Eng.

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EXECUTIVE SUMMARY

This report provides the geotechnical design for the Tailings Storage Facility (TSF) for the Ajax Project. This design has been advanced since the feasibility study in support of the Project's Environmental Assessment application activities and in preparation for a planned BC Ministry of Energy and Mines (MEM) permit submission. The project site is located in the South-Central Interior of British Columbia, southeast of the junction of the Trans-Canada Highway No. 1 and the Coquihalla Highway (No. 5), within the Thompson Nicola Regional District. The TSF will be located approximately 1km south of the open pit, east of Lac Le Jeune Road.

The TSF is designed to permanently store approximately 440 million tonnes of tailings generated during approximately 20 years of mine operation. The TSF will comprise of four earth-rockfill dams: north, east, south and southeast embankments in order to contain tailings and supernatant water. The north and east embankments will be buttressed by mine rock storage facilities constructed against the downstream slopes of the TSF, which are incorporated in the embankment designs from start-up and throughout the life of the facility.

Key features of this design report are as follows:

- **An overview of the foundation geology on which the TSF embankments will be constructed, and the TSF basin where tailings and supernatant water will be contained.** The project area contains a variably thick surficial soil cover, up to more than 40 m thick, over slightly to moderately weathered igneous bedrock. The cover varies widely in grain size, density and thickness, but is typically sandy silt and gravelly glacial deposits, which are relatively compact to very dense overlying competent bedrock.

- **Stability analyses were completed to demonstrate adequate stability of the dams.** The calculated factors of safety based on limit equilibrium stability analysis exceed design criteria for static and pseudo-static conditions. Strength design parameters were based on field strength data from Standard Penetration Tests (SPT), which provided a useful correlation to estimate soil strength parameters but does not provide direct measurement of specific soil strength and behaviors.

- **Design of the dam structure and associated seepage drainage controls were completed.** This includes earth fill embankment zones, dam slope configurations, estimated dam fill volumes and seepage controls were developed to provide a defense against hydraulic fracture, piping, and internal erosion that can occur as a result of cracking due to differential settlement, arching in the fill, or other defects. Rock drains will also be used to convey any seepage flow from the downstream toe beneath the embankments to seepage collection ponds. Recovered seepage

water will be pumped to the Central Collection Pond (CCP) which will be pumped to the process plant as process water.

- **An assessment of Best Available Technology (BAT) and Best Available Practices (BAP) for tailings containment was completed.** Alternative assessments were completed in separate studies to identify the BAT for tailings technology and closure strategy for the Ajax Project. Results from these studies (which are provided in this document) demonstrated that BAT for the Ajax Project was thickened tailings technology and implementation of a dry cover system at closure. Thickened tailings has a higher solids content, compared to unthickened tailings technology, which will improve the physical stability of the tailings deposit itself and lower water requirements within the facility. In addition, the higher initial solids content and lower segregation potential associated with thickened tailings placement are expected to reduce the challenges and costs associated with placement of a final dry cover over the tailings deposit.

- **A tailings management plan was completed that presents the tailings and water management volumetrics and staging thought the life of the facility.** The tailings slurry shall be discharged from various points along the upstream crest of the starter embankment. The method of tailings deposition (spigotting from perimeter) is expected to result in beaches with approximately 1 to 2% slopes. The actual tailings beach slope is expected to be steeper at the crest and flatter downslope. Slope angles will vary with solids content, discharge velocity, season, and the total height of tailings. Based on results from recent tailings laboratory test work, it is expected that the slurry tailings at 60% solids content will consolidate relatively quickly to 77% solids content within the TSF. Staged TSF layouts are provided in this document for End of Year -2, -1 (start-up), 1, 2, 3, 5, 10, 15 and approximately 20 (end of mine life).

- **A performance monitoring plan was developed to assess the performance of TSF throughout construction.** This monitoring plan includes construction and instrumentation monitoring information for QA/QC to ensure construction work is in compliance with the geotechnical engineering design.

1 INTRODUCTION

KGHM Ajax Mining Inc. (KAM) retained Norwest Corporation (Norwest) to complete the TSF engineering design for the Ajax Project in support of their EA application and a planned subsequent MEM permit submission report for the TSF structure. The contract to complete the work was awarded to Norwest on May 11, 2015 under contract # KA39-KGHM-CON-000180.

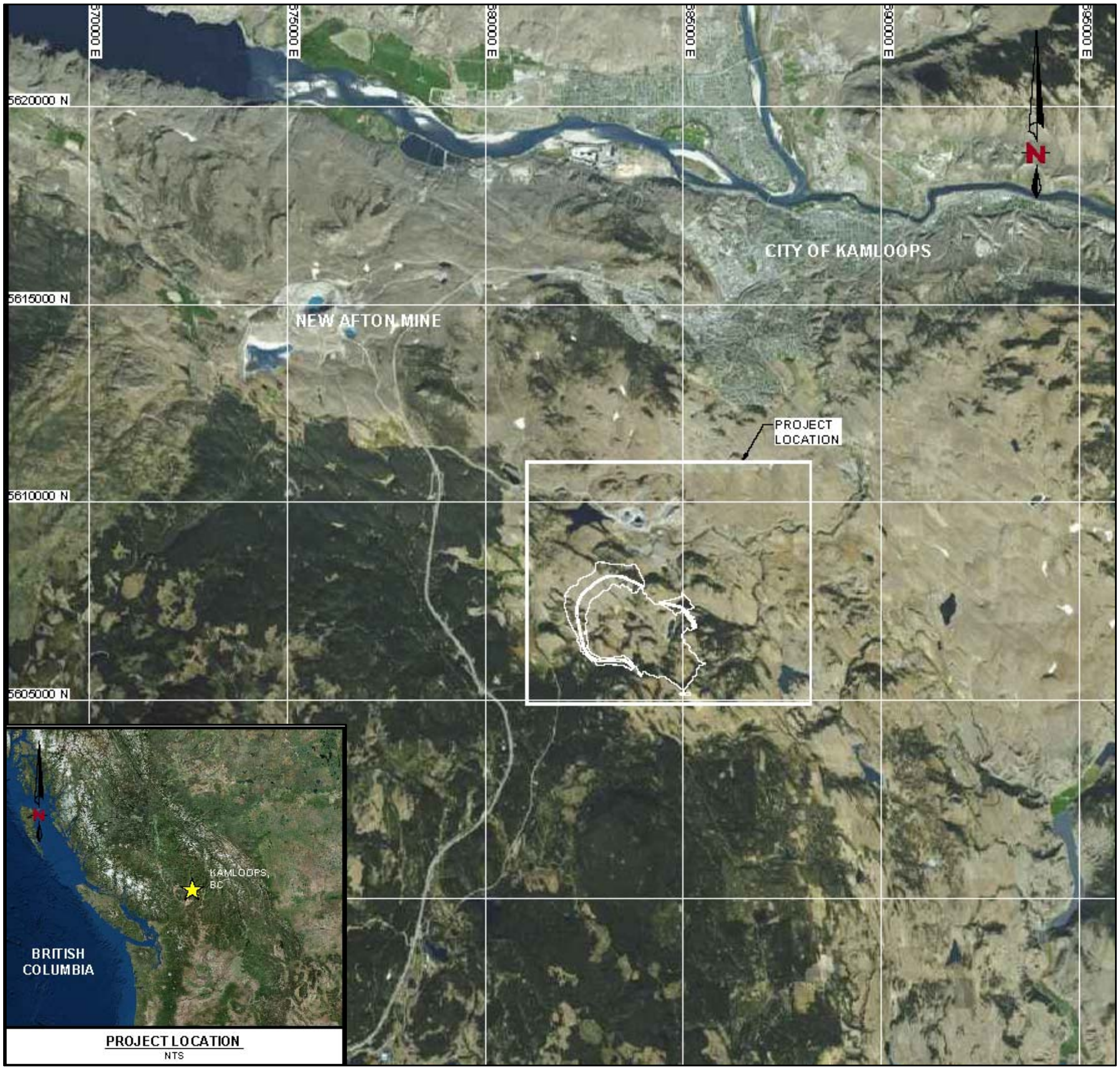
The reporting structure used to develop this TSF design report is as follows:

- Data developed by others was provided by KAM's engineering manager, Daniel Lefebvre to support the TSF design.
- The geology review, geotechnical analyses and report preparation was completed by Norwest's senior geotechnical engineer, Christopher Klassen, P.Eng.
- The hydrology review and analyses was completed by Norwest's senior water resource engineer, Eugene Ngwenya, P.Eng.
- The design provided in this report was reviewed by Richard Dawson, Ph.D., P.Eng. and Sean Ennis, P.Eng., P.E.

1.1 Project Description

The Project is located in the South-Central Interior of British Columbia, southeast of the junction of the Trans-Canada Highway No. 1 and the Coquihalla Highway (No. 5), within the Thompson Nicola Regional District. The Ajax mine will be an open pit mining using conventional truck-shovel methods. Ore production is planned at a rate of 65,000 tonnes per day and will be processed through an on-site mill and process plant. The TSF will be located approximately 1km south of the open pit, east of Lac Le Jeune Road. The location of the Ajax Project and the TSF is shown in Figure 1-1 below. A general arrangement plan of the Ajax site is shown in Drawing C180-KA39-5000-00-015.

Figure 1-1
Project Location Map



Tailings will be deposited and managed in a TSF located south of the open pit. The TSF was designed to permanently store tailings generated during the operation of the mine. The TSF will comprise of four zoned earth-rockfill dams (referred to as the north embankment, east embankment, south embankment and southeast embankment) and will be constructed using the downstream method of construction.

Integral to the embankment design will be the use of mine rock storage facilities immediately downstream of the TSF, which are incorporated in the embankment designs to buttress these structures. The West Mine Rock Storage Facility (WMRSF) will buttress the north embankment while the south mine rock storage facility (SMRSF) will buttress the east embankment. The mine rock buttresses will continually be used from start-up and throughout all stages of the north and east embankment construction.

1.2 Scope of Work

This TSF design report includes:

- A basis of design that includes a schedule for phased embankment construction and tailings storage volume estimates.
- An assessment of the foundation geology on which the embankments will be constructed, and the TSF basin where tailings and supernatant water will be stored.
- Design of the dam structure and associated seepage drainage controls. This includes dam slope configurations and estimated dam fill volumes.
- Slope stability analyses to demonstrate the embankments meet design criteria.
- Tailings management plan that presents the tailings and water management volumetrics and staging thought the life of the facility, and the corresponding staged embankment raises.
- Monitoring recommendations to evaluate the performance of the embankment throughout construction to ensure overall compliance with the geotechnical engineering design.

1.3 Sources of Information

A summary of the key reports, information and data used for this report is presented in Table 1-1.

Table 1-1
Summary of Key Sources of Information

Knight Piésold Limited, 2014. Tailings Storage Facility & Water Management Preliminary Design Report. VA101-246/26-1 Rev A.
Knight Piésold Limited, 2015. Report on Laboratory Geotechnical Testing of Tailings Materials and Tailings Consolidation Modelling. VA101-246/26-13 Rev A.

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1.4 Acronyms

AIR	Application Information Requirements/
BAT	Best Available Technology
BAP	Best Available Practices
CDA	Canadian Dam Association
CPT	Cone Penetration Testing
EA	Environmental Assessment
EAO	Environmental Assessment Office
EDGM	Earthquake Design Ground Motion
EIS-G	Environmental Impact Statement Guidelines
FOS	Factor of Safety
ICOLD	International Commission of Large Dams
IDF	Inflow Design Flood
ITRB	Independent Tailings Review Board
KAM	KGHM Ajax Mining Inc.
MAC	Mining Association of Canada
MCE	Maximum Credible Earthquake
PFMA	Potential Failure Modes Analysis
PMF	Probable Maximum Flood
PET	Potential Evapotranspiration
MAP	Mean Annual Precipitation
MRSF	Mine Rock Storage Facility
QA	Quality Assurance
QC	Quality Control
QPO	Qualified Performance Objectives
SI	Slope Inclinometer
SP	Standpipe Piezometer
SPT	Standard Penetration Test
TSF	Tailings Storage Facility
VWP	Vibrating Wire Piezometer

2 BASIS OF DESIGN

The basis for TSF design is summarized in this section. Supporting documents used to develop the basis of design are provided in Appendix A

2.1 Canadian Dam Association Guidelines (2014)

The North and East TSF dams are classified as "VERY HIGH" consequence category under the Canadian Dam Association's (CDA's) "Dam Safety Guidelines" (2014). According to CDA, the consequences of failure include loss of life of 100 or fewer and very high critical economic losses. Losses may include, but are not limited to, significant fish or wildlife habitat, infrastructure damage, loss of mining equipment, ore sterilization, and loss of tailings containment.

The South and Southeast TSF dams are classified as "SIGNIFICANT" consequence category. This classification shows that consequences of failure include an unspecified loss of life with no significant economic losses. Losses may include, but are not limited to, deterioration of fish or wildlife habitat, infrequently used transportation routes, and loss of tailings containment.

Despite the differences in TSF dam consequence classifications, the TSF dams for the Ajax Project were designed for the most extreme events (i.e. "EXTREME" consequence category), which is the highest design standard defined by CDA Dam Safety Guidelines:

- <u>An Inflow Design Flood (IDF) is the Probable Maximum Flood (PMF).</u> The PMF is defined as the most severe flood that may reasonably be expected to occur at a particular location. Flood protection is provided in the TSF design to accommodate the PMF above the tailings and supernatant pond level at each stage plus a minimum 2 meter allowance for wave run-up protection.

- <u>An Earthquake Design Ground Motion (EDGM) resulting from a 1/10,000 year event</u>. The EDGM used for the design of the TSF embankments is 0.34g, which corresponds to a 10,000 year return period earthquake. This EDGM value was based on seismic studies completed by Knight Piesold Ltd. (KPL), which were developed using a seismic hazard calculation obtained from Natural Resources Canada; the values are in accordance with the 2010 National Building Code. (See Table 3-5).

2.2 Environmental Assessment Office Requirements (2015)

The Environmental Assessment Office (EAO) issued a letter to KAM that outlined key design considerations to be included in the TSF design report as part of the EAO requirements for the pre-application stage of the EA, and the revised Application Information Requirements (AIR) under Section 16 of the Environmental Assessment Act (EAO, 2015). These design considerations are based on recommendations from the Independent Expert Engineering Investigation and Review Panel on the breach of the Mount Polley tailings facility (Independent, 2015).

EAO's design considerations, which are addressed in this report or in a separate study, are as follows:

A description and an assessment of alternative means of undertaking the proposed project with respect to options for tailings management that considers technology, siting and water balance.

The assessment must present and compare best practices and best available technologies for tailings management for the project, along with options for managing water balance to enhance safety and reduce the risk (likelihood and consequence) of a tailings dam failure during all phases of mine life (construction, operations, closure, post-closure). The assessment must present and compare technically and economically viable engineering solutions that are available to adequately address site conditions.

The assessment must provide a clear and transparent evaluation of the factors that supported the selection of the most suitable option. Factors that will be taken into consideration in the evaluation include safety, technical and financial aspects, and implications for environmental, health, social, heritage and economic values. The assessment must consider these factors in relation to tailings management options in both the short and long-term context. Life cycle cost assumptions (construction, operations, closure, post-closure) must be included in the analysis of options.

Separate studies were completed to address the design considerations outlined by EAO:

The Best Available Technology (BAT) and Best Available Practices (BAP) for tailings technology, siting and water management and TSF closure strategy for the Ajax Project were completed. Results from these studies are provided in separate documents (Norwest, 2015 and KPL 2014) but are highlighted in Section 4.5 and 9.1 of this report.

An assessment to develop options to enhance safety and reduce the risk (likelihood and consequence) of a tailings dam failure during all phases of mine life for the Ajax Project was completed. Results from the Potential Failure Modes Analysis (PFMA) were completed as part of the Dam Breach study, which is presented in a separate document (Norwest, 2015) but are highlighted in Section 6.5 of this report.

2.3 Application Information Requirements/Environmental Impact Statement Guidelines

This report meets the design requirements of the Ajax Project Application Information Requirements/Environmental Impact Statement Guidelines (AIR/EIS-G), (British Columbia Environmental Assessment Office, 2015), Section 3.7 Tailings Management. Additional supporting studies have been conducted to address specific requirements of the AIR/EIS-G, such as hydrogeological modelling and geochemical characterization of tailings materials, and are contained in separate sections of the AIR/EIS-G.

The list below contains the requirements of the AIR/EIS-G with regards to the TSF. These requirements are met within this report, or contained or expanded upon within the referenced section of the Environmental Assessment Application/Environmental Impact Statement (EA Application/EIS).

- Information on siting considerations and constraints, surface area and height, foundation characteristics and geohazards in the area (Norwest, 2015 and Knight Piesold Ltd, 2013).
- Characterization of the overburden material beneath the TSF footprint including composition, distribution, thickness and hydraulic conductivity properties (Norwest 2015).
- Identification of areas of bedrock outcrop beneath the TSF footprint (Norwest 2015).
- Geotechnical characterization of any materials to be used in the construction of the TSF (Norwest 2015).
- A hydrogeological assessment of the area around the base of the TSF (EA Application/EIS Section 6.6 - Groundwater Quantity).
- Embankment design criteria in accordance with the Canadian Dam Association Dam Safety Guidelines (Norwest, 2015).
- Description of tailings geochemical characteristics (including results of static and kinetic leach tests, and metal leaching potential) (EA Application/EIS Section 3.3 - Site Geochemistry).
- Description of any structures designed to divert water from entering the TSF (EA Application/EIS Section 11.2 - Environmental Management and Monitoring Plans).

- Description of evaporation from the tailings water including quality and quantity (EA Application/EIS Section 6.3 - Surface Water Quality, and 6.4-Surface Water Quantity, respectively).

- Description of the chemical composition of binding agents used in the tailings process (EA Application/EIS Section 3.0 - Detailed Project Description).

- Description of tailings water seepage (potential flows, direction of flow, quality, prevention and planned management strategies) and surface drainage including their collection; (EA Application/EIS Section 6.6 – Groundwater Quantity and 11.2 – Environmental Management and Monitoring Plans).

- Description of mitigation measures relating to fugitive dust emissions and aesthetic impacts; (EA Application/EIS Section 11.2 – Environmental Management and Monitoring Plans).

- Description of conceptual instrumentation and monitoring of the TSF during operations (Norwest, 2015).

- Proposed development stages including closure information (progressive reclamation etc.); (EA Application/EIS Section 3.18 – Closure and Reclamation).

- Direction of groundwater flow; (EA Application/EIS Section 6.6-Groundwater Quantity).

- Management plan of control of dust/debris and other material; (EA Application/EIS Section 11.2 – Environmental Management and Monitoring Plans).

- Comprehensive drainage plan design and plan around the TSF (EA Application/EIS Section 11.2 – Environmental Management and Monitoring Plans).

2.4 Design and Operating Design Criteria

Site specific design and operating criteria for the TSF are summarized in Table 2-1.

Table 2-1
Design and Operating Criteria for the TSF

Item	Design criteria
Mine Production	Total Ore Milled = ~ 440 Million tonnes
	Mill throughput = 65,000 tpd (@92% availability and 60% solids)
	Mine Life is approximately 20 years
Dam Location	Latitude 50.587700, Longitude -120.403900 Easting 683700, Northing 5606900 (NAD 83 UTM Zone 10N)
Site Elevation	Varies (934 – 1,080 masl)
TSF Dam Consequence Category[1]	VERY HIGH (North and South Embankments) SIGNIFICANT (South and Southeast Embankments)
Maximum Design Earthquake (MDE)	Maximum Credible Earthquake (MCE) = 1 in 10,000 year, which corresponds to an EDGM = 0.34g
Inflow Design Flood (IDF) [1]	Probable Maximum Flood (PMF)
TSF Dam Storage Capacity	Total Storage Volume is approximately 321.5Mm^3 This includes approximately 275 Mm^3 of tailings (at a dry density of 1.6 tonnes/m^3) + 2.4Mm^3 of supernatant pond + 4.1 Mm^3 (design flood) + 40 Mm^3 freeboard allowance.
Design Freeboard	Sufficient freeboard to accommodate Inflow Design Flood above maximum supernatant pond level at each stage plus allowance for wave run-up protection
Dam Crest Width	39m (This includes 35m running width and 4m for safety berms)
Maximum Dam Height	122m (maximum elevation = 1056 masl)
Design discharge	None
Seepage Control Measures	Upstream seepage control zone of the dam Compacted till blanket Rock drain to convey any seepage downstream to collection ponds.
North Embankment Seepage Collection Pond 1 and 2	Sediment removal (Year -2 and Year -1 construction) = 1 in 10 year, 24-hour event Runoff storage (Year 1 onwards) = 1 in 200-year, 24-hour flood
Dam Fill Materials	Till Blanket with Seepage Cutoff
	Filter and transition zones
	Compacted Mine Rock
	Mine Rock Buttress
	Mine Rock Storage Facility
Geotechnical Instrumentation Monitoring	Inclinometers, vibrating wire and standpipe piezometers.
	Flow monitoring for embankment and foundation drains
Closure	Dry cover system
	Spillway at closure only, which will be located at the SE end of facility.

1. Canadian Dam Association (CDA) "Dam Safety Guidelines" (2013)
2. KPL. Hydrometeorology Report (VA101-246/33-3 Rev.3)
3. KPL. Report on Laboratory Geotechnical Testing of Tailings Materials an Tailings Consolidation Modelling (VA101-246/26-13 Rev.A)
4. BGC Engineering Inc. (BGC) water quality modelling assumptions.

2.5 Stability Design Criteria

The TSF embankment dams will be designed to meet minimum required factor of safety criteria as shown on Table 2-2. These factors of safety (FOS) criteria are based on the revised 2007 Canadian Dam Association Dam Safety Guidelines (CDA, 2013).

Table 2-2
Slope Stability Design Criteria

Phase	Minimum Factor of Safety Criteria[1]
End of Construction	1.3
Long Term	1.5
Seismic (Pseudo-static Conditions)	1.0

Note:
1. As provided in the Canadian Dam Association Dam Safety Guidelines (CDA, 2013).

Stability analysis completed for this report includes:

- Static conditions at the end of construction phase for the starter embankment was checked for downstream stability with full reservoir conditions. Opportunities for improved slope stability using a mine rock buttress were also evaluated.
- Seismic conditions to evaluate the impact of the EDGM (i.e. 0.34g) on the integrity of TSF embankments.

2.6 Seepage Design Criteria

Seepage criteria to be used in the TSF design are as follows:

- A steady state average gradient cannot exceed 10%.
- Flow rate of the rock drains has a capacity of 10 times the steady state seepage flow rate through the dam to convey any seepage flow from the downstream toe to seepage collection ponds.

3 SITE CHARACTERISTICS

3.1 General

The Ajax Mine Site is accessed by the old Afton mine haul road, which crosses the Lac Le Jeune Highway approximately 8.3 km south of the intersection of Lac Le Jeune Road and Copperhead Drive off of Highway 1, west of Kamloops, BC. The coordinates for the centre of the Ajax Project area are approximately 50°58' north latitude and 120°40' west longitude and is located on mineral titles reference map M09I068 (National Topography System (NTS) 92I/9) in the Kamloops mining division. The Ajax property is situated south of Kamloops, BC (Figure 4.1).

Currently, the common land use in the area is ranching. Surface rights are privately owned, and the main water bodies in the area are Jacko, Inks, and Wallender lakes. The lakes are reserved for ranching and recreation. The Ajax area consists of rolling grasslands with timber at the higher elevations. Elevations range from 800 to 1,100 masl. Sugarloaf Hill is the prominent landform in the area and has an elevation of 1,130 masl. The area has been glaciated and numerous drumlins are present. At lower elevations, the vegetation is typically bunchgrass, sagebrush, and prickly pear cacti. Higher elevations commonly sustain growths of lodge pole pine, douglas fir, and ponderosa pine.

The watersheds beneath the proposed tailings storage facility drain towards Jacko Lake and Peterson Creek, which is the predominant body of water at the project site. Peterson Creek flows out of Jacko Lake to the northeast and eventually makes its way through the Kamloops city center and into the South Thompson River. The watersheds to the south and east of the proposed TSF drain into Humphrey Creek which is a subsidiary stream of Peterson Creek. Inks Lake lies to the northwest of the open pit and is a proposed habitat compensation area.

3.2 Climate

The climate of the Ajax mine site is typical of the dry BC Interior with generally low total precipitation, high evaporation, and correspondingly low streamflow rates. Lying within the rain shadow of the Coast Mountains, this area has a semi-arid steppe climate characterized by generally cool dry winters and hot, dry summers, with low humidity. Convective storms are frequent in the summer months, and as a result precipitation is generally highest in June and July (KPL, 2015).

Meteorological data have been collected at the site since August 2010 and include records of temperature, relative humidity, precipitation, and wind speed and direction. Mean annual precipitation for the site has been evaluated by KPL (2015) who analyzed active and inactive regional climate stations throughout the area, several of which have two decades or more of data, as well as data collected at the two climate stations that were installed on site.

Based on this analysis, KPL (2015) estimated average annual precipitation for the site at 336 mm, distributed as summarized in Table 3-1. This annual precipitation applies to an elevation of 950 m. Approximately 30% of the annual precipitation is estimated to occur as snow.

Table 3-1
Average Monthly Climate Data for Ajax

Month	Average Temperature (°C)	Average Rainfall (mm)	Snow Water Equivalent (mm)	Average Precipitation (mm)	PET (mm)
January	-4.5	2.3	21.1	23.4	0
February	-2.4	2.9	11.6	14.5	1
March	1.5	4.2	7.7	11.9	12
April	6.5	16.3	2.8	19.2	40
May	11.1	32.8	0	32.8	77
June	14.7	43.4	0	43.4	103
July	18.7	42.4	0	42.4	130
August	17.8	32.0	0	32.0	114
September	12.9	35.8	0	35.8	71
October	5.6	13.2	2.3	15.5	28
November	-0.1	5.6	22.4	28.0	3
December	-4.9	3.7	33.3	37.0	0
Average/Total	**6.4**	**235**	**101**	**336**	**579**

Reference: (KP, 2014b)

Annual potential evapotranspiration (PET) has been estimated by KPL (2015) at 579 mm; monthly values are shown on Table 3-1. In addition, sublimation between November and February has been estimated at 28 mm (KPL, 2015). Wet (i.e., above average) and dry (i.e., below average) annual precipitation for various return periods is summarized in Table 3-2.

Table 3-2
Wet And Dry Year Annual Precipitation at Ajax

Return Period (years)	Precipitation (mm)	
	Dry	Wet
10	259	413
20	238	434
50	490	459
100	197	475
200	182	490

Reference: (KP, 2015).

The nearest active manual snow survey station is the Highland Valley station (ID No. 1C09A), which is operated by the Ministry of Forest, Lands and Natural Resource Operations (FLNRO) This station is located 40 km southwest of the study area at an elevation of 1475 m and has been in operation since 1966. Long-term normals for this station show a majority of the snowpack melting in April and May. The normal snow-water equivalence on April 1 is 83 mm compared to 20 mm on May 1 and 3 mm on May 15(BGC. 2015).

3.3 Hydrology

The climate of this region is characterized by the generally low annual precipitation and high evaporation resulting in relatively low inflow rates into water bodies or impoundments. Rainfall runoff values within the proposed watershed are relatively low compared to most other areas of BC due in part to the extremely dry and absorbent soils.

3.3.1 Precipitation

The long-term Mean Annual Precipitation (MAP) is estimated to be 336 mm, with approximately 30% of the annual value expected to fall as snow. The extreme rainfall values, including the Probable Maximum Precipitation (PMP) for the project area, are summarized in Table 3-3.

Table 3-3
24-Hour Extreme Rainfall Values

Return Period (Years)	Rainfall Amount (mm)
2	27.6
5	37.0
10	43.2
15	46.8
20	49.2
25	51.1
50	57.0
100	62.8
200	68.5
PMP	219.1

Reference: (KPL, 2015)

3.3.2 Runoff

Average monthly runoff depths for the 950 m elevation band represents runoff conditions for the various mine facilities, (e.g. Open Pit, TSF), are shown in Table 3-4 (BGC, 2015). Average annual runoff (including groundwater recharge) in the mine site area is estimated at about 23 mm. A site catchment plan is provided in Appendix A.

Table 3-4
Average Estimated Monthly Runoff for Mine Site (Elevation 950m)

Month	Runoff Depth (mm)
January	0.3
February	0.3
March	2.3
April	6.5
May	7.9
June	2.9
July	0.9
August	0.5
September	0.5
October	0.6
November	0.2
December	0.3
Total	23.1

Reference: (BGC, 2015)

3.3.1 Evaporation

Annual evaporation and sublimation (603 mm) exceeds annual precipitation (336 mm), for a net deficit of -267 mm acting on standing surface water bodies. Average annual runoff (including groundwater recharge) in the mine site area is estimated at about 23mm (Table 3-4).

3.4 Geology

The Ajax Project is located within the Thompson River watershed and the Thompson-Okanagan Plateau Ecoregion. Retreat of glacial ice during the Pleistocene resulted in a landscape of gently rolling plateaus, incised river valley systems and large glacial lakes such as Kamloops Lake and Okanagan Lake. The peaks in the vicinity of the Ajax property consist of rock including areas of outcrop, and the valleys are characterized as morainal deposits consisting of drumlinized glacial till. Remnant glaciolacustrine deposits occur just north of the pit and coarse colluvium deposits occur near Sugarloaf Hill. Numerous authors have reported on the geology of the Ajax area (Ross, 1993; Ross et al., 1995; Logan et al., 2007), which was used for this report.

The mineralization in the Ajax area is associated with structural corridors of highly fractured sections of Sugarloaf and Sugarloaf Hybrid phases of the Iron Mask Batholith. Chalcopyrite is the dominant copper mineral and occurs as veins, veinlets, fracture fillings, disseminations, and isolated blebs in the host rock. Concentrations of chalcopyrite rarely exceed 5%. Accessory sulphide minerals include pyrite, magnetite, molybdenite, and occasionally bornite. A variety of steeply dipping, unmineralized dykes up to 5 m wide intrude the main rock types. Dykes are composed of aplite, monzonite, latite, and fine-grained mafic rocks.

The regional geology of the Ajax area is dominated by the Upper Triassic Iron Mask batholith. The batholith is approximately 5 km wide, 20 km in length, and trends northwest through the region. The Iron Mask batholith intruded a sequence of Nicola Group flows and volcaniclasitc rocks of mafic and intermediate composition. Near the contact with the Iron Mask batholith, the Nicola Group rocks are commonly basalt to andesite flows and flow breccias. Stratigraphically above the Nicola Group is a series of serpentinized picrite basalts, which are present within the batholith and are apparently localized along major structural corridors.

3.5 Site Seismicity

The Kamloops region is characterized by a low level of historical seismicity. Seismic hazard values for the site are available from the Natural Resources Canada (NRC) website (NRC, 2010) for earthquakes up to the 1/2,475 return period. Firm ground peak horizontal ground accelerations and the associated return periods from NRC are summarized in Table 3-5 below. Seismic hazard values for greater return periods are not provided by Natural Resources Canada. A peak ground acceleration of 0.34g and Maximum Credible Earthquake (MCE) for the 1/10,000 year return period for the Ajax project was developed by KPL (KPL, 2014). For this design report, Norwest has used the same seismic hazard values and MCE determination for consistency.

Table 3-5
Ajax Seismic Hazard Values (1)

Earthquake Return Period (Years)	Annual Exceedance Probability (AEP)	Peak Ground Acceleration (g)
100	1%	0.034
475	0.21%	0.072
1,000	0.1%	0.097
2,475	0.0404%	0.138
10,000 (2)	0.01%	0.340 (2)

1. Seismic hazard values used for the TSF design are acceptable at this level of the design for evaluation of pseudostatic stability assessment of the TSF dams. Site specific studies will be completed to confirm these design assumptions for the next level of detailed design.
2. Based on values developed by KPL (KPL, 2014).

4 DESIGN OVERVIEW

4.1 TSF Design Objectives

The principal design objectives for the TSF are to provide containment for tailings and supernatant water during operations and dry land tailings containment in the long term (post-closure), and to achieve effective reclamation at mine closure. The TSF design requirements should meet the following criteria:

- Provide permanent, secure and total confinement of all tailings materials within an engineered disposal facility.
- Control seepage through the basin and embankments and removal of free-draining liquids from the TSF during operations for recycling as process water to the maximum practical extent.
- Application of Best Available Technology (BAT) and Best Available Practices (BAP) for tailings containment;
- No surface water discharge to the environment over the life of the project.

4.2 TSF Staged Development

Table 4-1 summarizes the staged TSF embankment layouts with respective tailings and supernatant water elevations for End of Year -2, -1 (start-up), 1, 2, 3, 5, 10, 15, and approximately 20 (end of mine life) and at closure. Staged embankment layouts for these years are shown on Drawings C180-KA39-5000-00-003, C180-KA39-5000-00-004 , C180-KA39-5000-00-005 , C180-KA39-5000-00-006 , C180-KA39-5000-00-007 , C180-KA39-5000-00-008 , C180-KA39-5000-00-009 , C180-KA39-5000-00-010, C180-KA39-5000-00-011 and C180-KA39-5000-00-014. A typical TSF embankment cross section with the staged development is shown on Drawing C180-KA39-5100-00-009.

Table 4-1
Annual TSF Construction and Tailings Deposition Plan

End of Year	Dam Crest Elevation				Tailings Elev.1 (m)	Pond Elev. (m)	Drawing#
	North	East	South	Southeast			
-2		-	-	-	-	-	C180-KA39-5000-00-003
-1	971	-	-	-	-	946	C180-KA39-5000-00-004
1	984	984	-	-	968	962	C180-KA39-5000-00-005
2	992	992	-	-	980.5	971	C180-KA39-5000-00-006
3	1,000	1,000	-	-	989	978	C180-KA39-5000-00-007
5	1,012	1,012	-	-	1,003	988	C180-KA39-5000-00-008
10	1,031	1,031	1,031	-	1,023	1,013	C180-KA39-5000-00-009
15	1,046	1,046	1,046	-	1,039.5	1,028	C180-KA39-5000-00-010
~20 (End of Mine Life)	1,056	1,056	1,056	1,056	1,053	1,043	C180-KA39-5000-00-011

Notes:

1. Tailings deposition (spigotting from perimeter) is expected to result in beaches with approximately 1 to 2% slopes. The tailings elevation shown in the table is the average beach slope at 1.5%

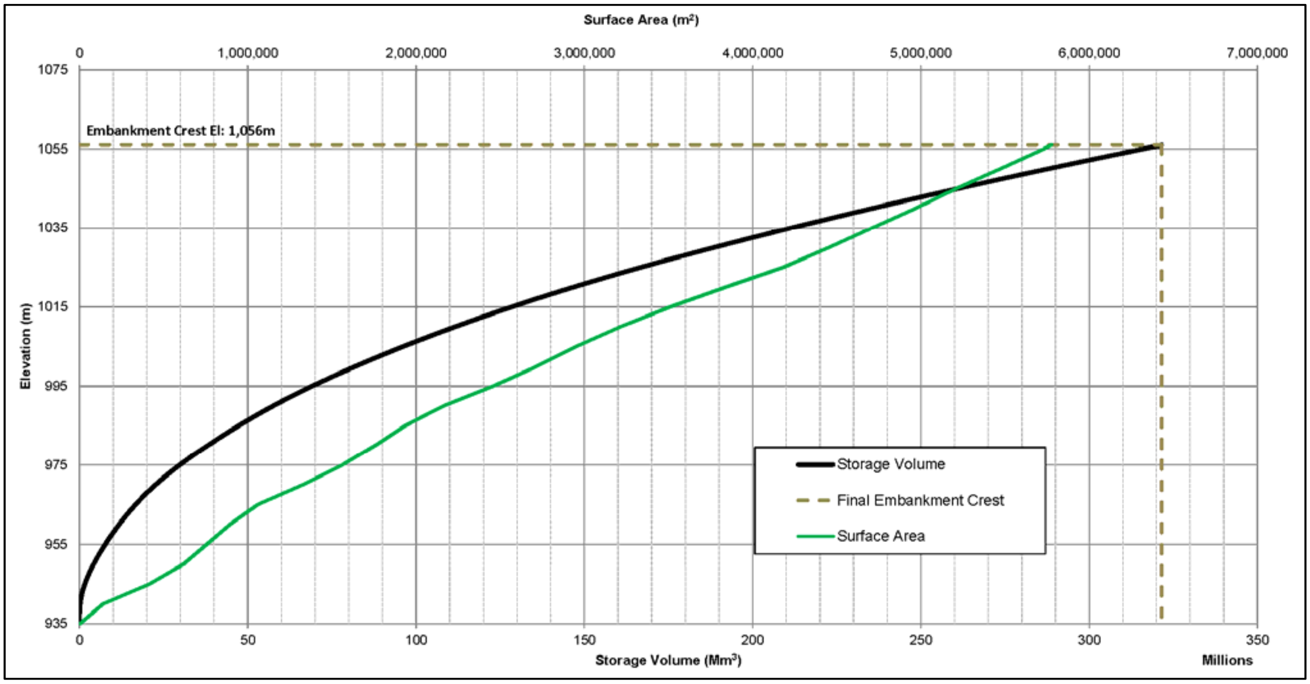
4.3 Tailings Mass Balance

Thickened slurry tailings will be pumped approximately 1,960m^3/hr to the TSF at 60% solids. Sequenced discharge from numerous locations arrayed along the crest of the dams will create a beach from the upstream crest of the dyke. The tailings solids will quickly settle to approximately 77% with tailings dry density of 1.6t/m^3 to form a beach at about 1.5% slope, with some of the fines and water carried to the supernatant pond. Water will be reclaimed from the pond for re-use in the process. A seepage collection system consisting of a rock drain will convey any seepage water into downstream seepage collection ponds, which are described in Section 7.3.1.

A depth area capacity curve that reflects the potential TSF storage volume is shown in Figure 4-1. The volumes and areas are based on topographic information provided by KAM using 3D MineSight modelling software.

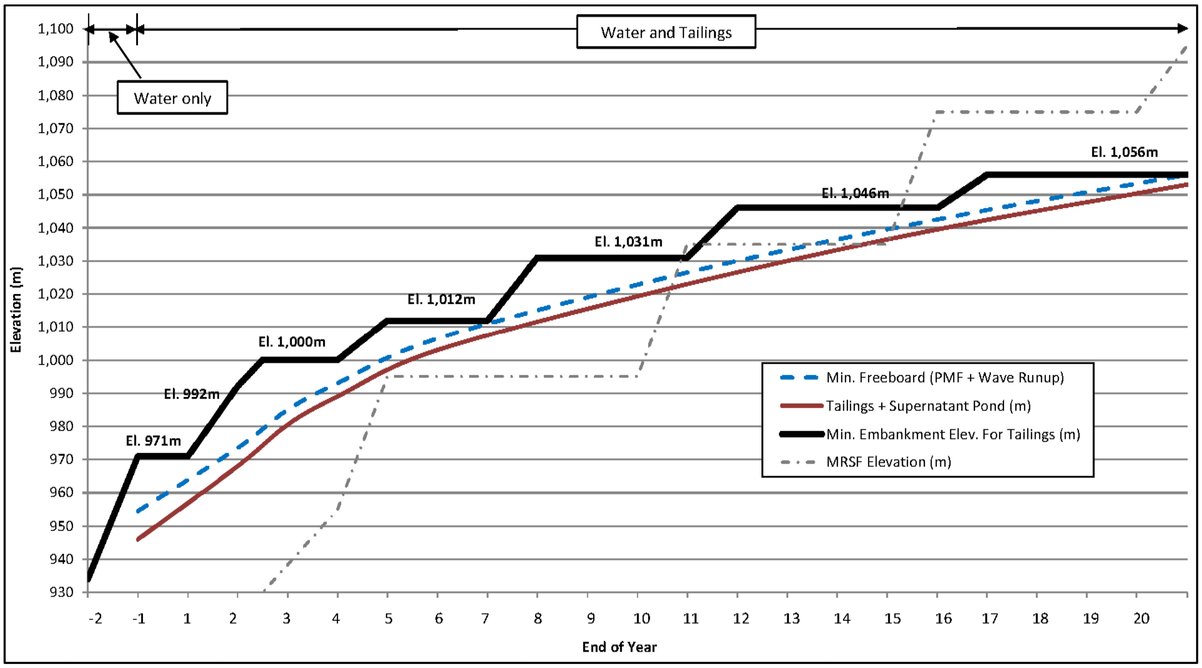
Figure 4-1

TSF Depth Area Capacity Curve



A TSF filling schedule that shows the tailings mass balance or cumulative tailings volumetrics, freeboard for PMF and wave-up and respective dam crest elevations to contain the water and tailings volumes an annual basis is shown in Figure 4-2.

Figure 4-2
TSF Filling Schedule



Notes:

1. Startup pond volume is estimated at approximately 2.4Mm^3.
2. Minimum crest elevation includes sufficient storage for tailings, pond and allowance for PMF and wave run-up
3. Tailings volume is assumed to be 65,000 tpd at a mill availability of approximately 92%.

4.4 Operational Water Balance

An operational water balance was developed for the TSF in a separate study (Norwest, 2015), which includes plant site water requirements provided by Fluor Corporation (Fluor, C147-KA39-000-55-049-PFD Rev D). Table 4-2 and Figure 4-3 provides the operational water balance for the TSF. A site wide water balance for the Ajax project was completed in a separate study by BGC (1125-006-R02-2015, June 2015).

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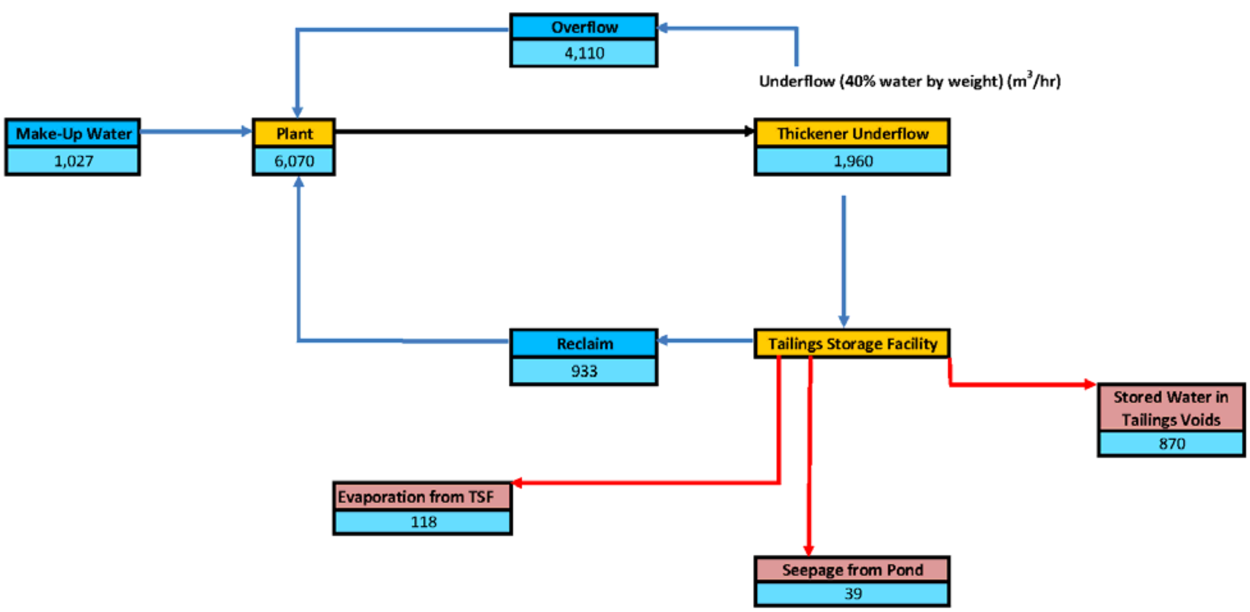
Table 4-2
Operational Water Balance

Item	Description	Estimated Water Flows (m^3/hour)
Plant Site[1,23,4]	Plant Site Requirement	6,070
	Thickener Overflow (back to Plant Site)	+4,110
	Net process make-up requirement	+1,027
	Reclaim from supernatant pond	+933
TSF	Thickener Underflow (to TSF at 60% solids)	+1,960
	Loss to tailings pore water (@77% solids)5	-870
	Loss to seepage	-39
	Loss to evaporation	-118
	Reclaim from supernatant pond	+933

Notes:
1. Plant site water requirements provided by Fluor (C147-KA39-000-55-049-PFD Rev D).
2. 2,914 tonnes/hr (fresh water tank + process water tank) (Fluor, 2015).
3. Plant requirements are 6,070m^3/hr (fresh water tank + process water tank). (Fluor, 2015).
4. No losses occur in the plant or thickener. (Fluor, 2015).
5. Tailings consolidate from 60% solids to 77% solids in less than 10 days. (Knight Piesold, 2015).

Figure 4-3
Operational Water Balance



4.5 Best Available Technology -Tailings Alternatives Assessment

Thickened tailings technology was identified as the Best Available Technology (BAT) based on a recent tailings trade-off study for detailed design for the Ajax project (Norwest and M3, 2015). The tailings trade-off study was completed based on regulatory feedback and tailings management recommendations outlined in an independent expert engineering investigation and review panel report for the TSF embankment failure at the Mount Polley mine (Independent, 2015). The expert panel from this investigation recommended that,

> "… *BAT should be actively encouraged for new tailings facilities at existing and proposed mines. Safety attributes should be evaluated separately from economic considerations, and cost should not be the determining factor*.
>
> …*while best practices focus on the performance of the tailings dam, BAT concerns the tailings deposit itself. The goal of BAT for tailings management is to assure physical stability of the tailings deposit. This is achieved by preventing release of impoundment contents, independent of the integrity of any containment structures*."

Thickened tailings addresses these concerns for the Ajax project by increasing the solids content of the tailings to improve physical stability of the tailings deposit and minimizes water requirements within the facility. In addition, the higher initial solids content and lower segregation potential associated with thickened tailings placement are expected to reduce the challenges and costs associated with placement of a final dry cover over the tailings deposit, which has been determined to be the BAT closure method for the Ajax Project.

4.6 Schedule

There are two primary construction phases through life of mine: pre-production and production. Pre-production is considered the period before mill operations (Year -2, -1) and production (Year 1 to 20) is the period during mill operations. Construction tasks for each of these two phases are as follows:

4.6.1 Pre-Production

<u>TSF starter North Embankment (Year -2, -1):</u> The starter north embankment will be constructed in 1 to 2 years prior to process plant operations. This embankment will provide capacity for approximately two years of tailings storage and will retain the start-up water pond, which is estimated at approximately 2.4Mm^3 of storage for emergency process water supply to the mill.

4.6.2 Production

Embankment raises of TSF Embankments to full height (Year 1 to approximately 20): The thickened tailings will be deposited and managed in a TSF located south of the open pit. Earth-rockfill dams will be constructed to provide tailings and water storage containment. Tailings will be discharged from the embankments that will result in a supernatant pond developing in the southeast corner of the facility. The method of tailings deposition (spigotting from perimeter) is expected to result in beaches with approximately 1 to 2% slopes. Initial tailings deposition into the TSF will be by spigotting from the face of the north embankment and then along the east embankment at the end of Year 2. Tailings will continue to be discharged along the perimeter of the north and east embankments, including the south embankment starting after year 8. At closure, all the embankments will have a crest elevation of 1,056m. Prior to closure of the TSF, the supernatant water will be pumped into the pit to expedite the establishment of a pit lake and eliminate long-term ponded water on the TSF. The tailings will be reclaimed to a terrestrial landscape, as close as practical to pre-mining conditions. Any runoff and/or flood events will be directed away from the facility into an engineered channel to Humphrey Creek, which is located in the neighboring catchment to the east of the TSF.

5 ENGINEERING GEOLOGY AND FOUNDATION CONDITIONS

5.1 General Bedrock Geology

The Ajax property and associated ore mineralization occurs at the southern contact of the northwest trending sub-volcanic Iron Mask Batholith complex. This complex is Triassic to Early Jurassic in age and is composed of a number of intrusive units, which include the Iron Mask Hybrid, Pothook, Sugarloaf, and Cherry Creek. The Triassic Nicola Group Volcanics, which includes tuffs, flows and breccias, occur south of the Iron Mask Batholith. The Nicola Group Volcanics are typically greenschist facies metamorphic grade. Diamond drilling during 2014 confirmed the presence of faults in the area. The northwest – southeast trending Cherry Creek Tectonic Zone and Edith Lake Fault Zone transect the property.

5.2 General Surficial Geology

British Columbia has undergone repeated glaciation during the Quaternary era. The Cordilleran ice sheet covered the western Canadian mountains from Alaska in the north to Washington, Idaho, and Montana at its southern most extents. The episodic growth and decay of the Cordilleran ice sheet has imposed a general order to the sedimentation, where thick valley and lowland fills contain discrete packages of glacial sediments that are separated by unconformities or thin interglacial sediment packages (Clague, 2000). A generally consistent stratigraphic order exists within the glacial sediments. Typically, a distinctive sequence of glaciofluvial and lacustrine sediments were deposited during an early phase of glaciation and was subsequently overridden and eroded by subsequent glacial advances. The development of lodgment tills were typically generated during this stage. As the glaciers retreated, interglacial fluvial and lacustrine sediments, and ablation tills were deposited (Clague, 2000).

In the Fraser and Thompson plateau, glacial advancement and retreat appear to dominantly have been in a southeast orientation. Orientation of glacially formed structures at the Ajax site general follows this southeasterly direction. The surficial deposits and landforms of the project area indicate that deglaciation occurred partially by down wasting and stagnation and partially by normal retreat.

5.3 Site Geology based on 2014 Site Investigations

The project area was significantly affected by the Quaternary glacial events, which is evident from the ridged and hummocky features formed of glaciofluvial sands and gravels and sands, non-compacted ablation till, large drumlinoid landforms dominantly formed of lodgement till, and pockets of glaciolacustrine sediments (KPL, 2014). Glaciolacustrine sediments commonly occur in Pleistocene valley fills in southern British Columbia in the Ashcroft area (Bishop et al., 2008). Review of satellite imagery of the project area relative to the 2014 drill holes and test pits that encountered clay intervals support the observation that clays typically occur proximal to valleys. A general surficial geology map of the TSF based on the 2014 site investigation work (KPL, 2014) is shown in Drawing C180-KA39-5000-00-002.

The 2014 site investigation program completed 29 drill holes and 182 test pits to characterize the geology at the proposed TSF location. A summary of the site investigation results used to characterize foundation conditions and material properties are summarized in Table 5-1, and discussed below.

Table 5-1
Site Investigation Results Summary

TSF Embankment/ Impoundment	Drill Holes	Test Pits	Lab Data						
			SOIL				ROCK		
			PSA	Atterberg	Proctor	Moisture Content	UCS	Young's Modulus	Specific Gravity
North	10	39	14	14	2	11	16	16	16
East	5	8	1	1	0	0	7	7	7
South	3	10	1	1	0	0	5	5	5
Southeast	3	3	0	0	0	0	2	2	2
Basin	8	122	14	14	4	15	1	1	1
Total	**29**	**182**	**30**	**30**	**6**	**26**	**31**	**31**	**31**

*Site investigation results are for the TSF only (Knight Piesold Ltd., 2014).

The average drill hole and test pit spacing within the footprint of the TSF embankment and basin areas are summarized in Figures 5-1 and 5-2, respectively.

Figure 5-1

Average Drill Hole Spacing for Each Embankment

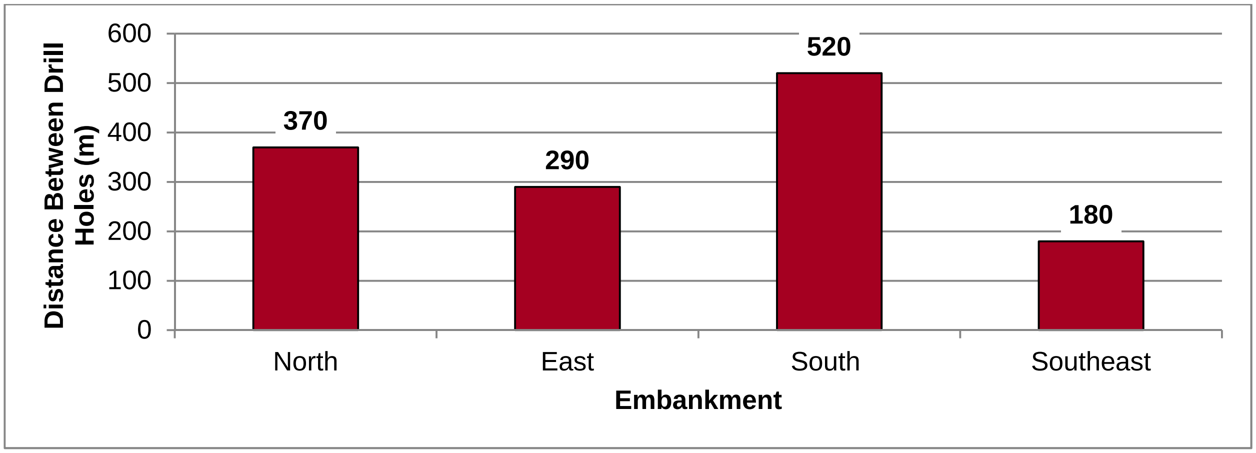
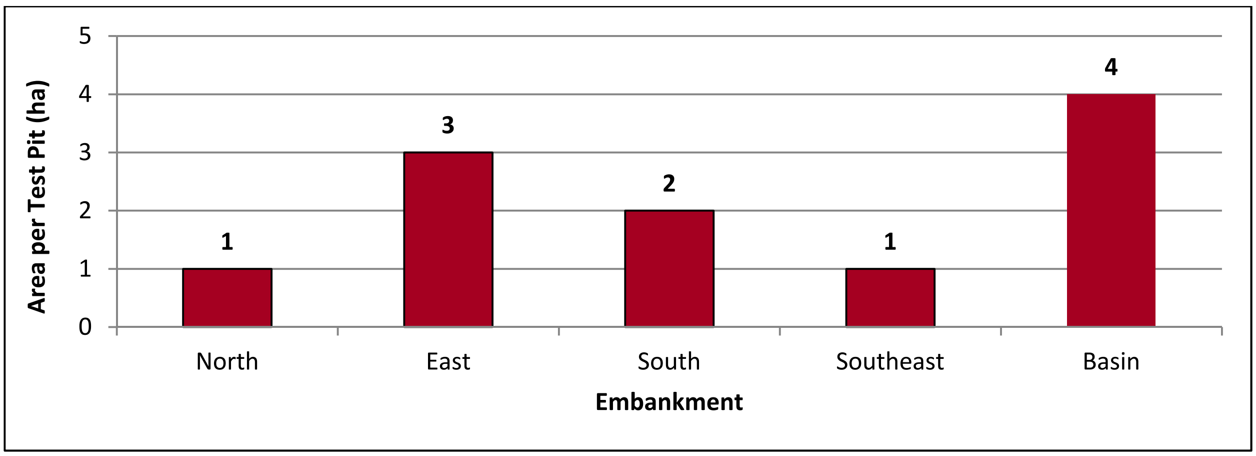


Figure 5-2

Average Test Pit Spacing at the TSF



The 2014 classification of the Quaternary material types in the project area were primarily classified based on depositional environment (i.e. glaciofluvial, glacial till, glaciolacustrine, aeolian, and colluvium). Although there is a strong correlation between material type and depositional environment, there is also significant overlap between the material types which occur within a depositional environment, and also significant variation in physical characteristics that occur within any given material type. Such variation in the physical characteristics of a material type is common with the "glacial till" classification, where till can significantly vary in composition, grain size, density, and thickness. Norwest resolved this soil material classification issue from the data collected from the 2014 site investigation program by classifying these materials based on the Unified Soil Classification System (USCS) ASTM D2488-09a. This is a system for soil mechanics and geotechnical engineering purposes. A summary of the key soil material types associated with the embankments is shown below. Table 5-2 lists the average geotechnical parameters for each of the soil material types.

- **GRAVEL, silty to clayey (GC, GW-GM, GM),** well graded, varying degree of clast angularity, varying density, commonly brown to grey, varies in moisture content depending on depth of occurrence, massive. Gravel-dominant intervals vary in thickness from ~1m (i.e. DH14-028) to >15m (i.e. DH14-036) throughout the project area. Although not common, gravel-dominant intervals are observed in each of the embankment areas. This material type is associated with colluvium, glaciofluvial, and glacial till.

- **SAND, silty to clayey (SC, SM),** gravelly to some gravel, light brown to brown, varies in moisture content depending on depth of occurrence, massive. SM and SC units commonly occur directly below the topsoil but can be found directly above the bedrock contact. SM and SC vary in contiguous thicknesses from <1m (i.e. TP14-285) to >15m (i.e. DH14-008). Particle size analysis indicates that the clay content is higher than previously observed from the field investigations, which may indicate that a number of SM should actually be classified as SC. Silty to clayey sand is the most common soil unit, and occurs in all of the embankment areas. This material type is commonly associated with colluvium and glacial till.

- **SILT, sandy, (ML),** non-plastic to low plasticity, trace gravel to gravelly, brown, soft, dry to wet. ML is quite common in the TSF area and typically is found directly below the topsoil. ML material thickness is up to 22m thick (DH14-036), and is observed in localized areas beneath the embankment foundation at varying thicknesses. ML is typically classified as a glacial till, aeolian, and as a lacustrine sediment.

- **SILT (MH),** moderate to high plasticity, commonly contains some gravel to gravelly, light to dark brown, varies in density, commonly massive, dry to wet. MH is less common than ML and is typically found directly below ML or SM but may occur directly below the topsoil. This material was observed in the north, south, and southeast embankments ranging in thickness from approximately 2m to 14m. MH is commonly classified as glacial till and/or lacustrine sediment.

- **CLAY gravelly (CL, CH)** to trace gravel, low to high plasticity, grey to brown, varies in density, moist to wet. Where observed, commonly occurs within the top 5m of the sequence, however also occurs as intermittent layers (i.e. DH14-025). Localized clay units were identified beneath the north, south, and southeast embankment foundation at varying thicknesses that ranged from approximately 1m to 4.5m. Table 5-3 summarizes the CL and CH intervals that were observed within the embankment areas during the 2014 field investigation study. Most of the high plastic clays identified to date are located at shallow depth (<5m) and it is planned to remove these materials as part of foundation preparation.

Table 5-2

Site Investigation Summary

Soil Material Type	PSA	Atterberg Limits			Moisture Content (%)	Guelph Permeameter (m/s)	Proctor - Max Dry Density (pcf)	Proctor - Optimum Moisture (%)	SPT "N" Value Average
		Plastic Limit %	Liquid Limit %	Plasticity Index %					
GC, GW-GM, GM	n=11	non-plastic (n=6); 16 (n=11)	non-plastic (n=6); 27 (n=11)	non-plastic (n=6); 12 (n=11)	6.4 (n=11)	10^{-5} (n=1)	134.5 (n=2)	8.0 (n=2)	50+ typically refusal (n=36)
SC, SM	n=29	14 (n=29)	27 (n=29)	13 (n=29)	8.8 (n=28)	10^{-7} (n=5)	136.2 (n=2)	8.6 (n=2)	30 -50+ (n=35 out of 92; 57 refusal)
ML	n=1	20 (n=1)	22 (n=1)	2 (n=1)	7.6 (n=1)	10^{-6} (n=2); 10^{-7} (n=1)	None	None	35-50+ (n=17 out of 47; 30 refusal)
MH	n=1	39 (n=1)	53 (n=1)	14 (n=1)	43.1 (n=1)	None	None	None	30 (n=31 out of 42; 11 refusal)
CL, CH	n=9	15 (n=9)	29 (n=9)	13 (n=9)	12.4 (n=9)	10^{-7} (n=2)	130.4 (n=2)	10.2 (n=2)	22 (n=6 out of 12; 6 refusal)

1. Table 5-2 summarizes results from all of the collected data from the 2014 site investigation program. This table differs from Table 5-1, which focused on the site investigation results for the proposed TSF area only.
2. Total test count = n, which is shown in parentheses in the above table.

Table 5-3
Clay Occurrences in the Embankment Areas (2014 Site Investigation Program)

ID	Unit	Embankment Area	Depth from Surface (m)	Thickness (m)	Overlying Unit	Underlying Unit
TP14-285	CL	North	1	4.5	SW	undetermined
DH14-012	CL	North	0	4.5	N/A	SM
TP14-083	CL	North	0	2	N/A	SM
TP14-118	CL	North	1	0.5	GC	SC
DH14-025	CL	South	4	1.5	SM	SC
DH14-025	CH	South	15	1	SM	SM
DH14-025	CH	South	18	1.5	MH	SM
DH14-034	CH	South	1	3	ML	ML
DH14-036	CH	Southeast	2	2	ML	bedrock

Note: CL clays were classified in 2014 as low to medium plasticity

5.4 Key Geological Features and Units by Embankment Areas

A summary of the field and lab testing, surficial geology, bedrock geology and groundwater conditions for each embankment are provided in the following sections.

5.4.1 North Embankment

Field and Lab Testing:

A total of 10 drill holes and 39 test pits were completed. Field testing included a total of 64 lugeons and 4 falling head permeability tests in bedrock and SPT at 5ft intervals in soil at 10 drill hole locations. Laboratory index testing on soil samples included 14 Particle Size Analysis, 14 Atterberg Limits, 11 Moisture content and 2 proctors (1 standard and 1 modified). Laboratory strength testing was completed on rock samples only. These tests included 16 Unconfined Compressive Strength, 16 Young's Modulus and 16 Specific Gravity.

Surficial Geology:

The overburden thickness varies from approximately 11m to 18m in the far south, thins to approximately 2-3m in the south central area, and then progressively thickens towards the north and northeast area of the embankment to a maximum thickness of approximately 41m in DH14-008 near the Keynes Valley. The dominant material type encountered in the test pits and drill holes is silty sand (SM).

There are isolated fine grained soils identified primarily near the surface:

- A high plastic, low density (SPT "N" values between 14 and 23), silt (MH) interval was encountered at the southern end of the embankment.
- A 7m thick ML unit was identified approximately 2m deep within DH14-022.
- A thin CL layer is identified in TP14-118 that occurs at a depth of 1m.
- Further to the north, clay intervals (CL) were identified within 1m of surface in TP14-285 and DH14-012. SPT "N" values for this unit were approximately 26 and 30. Proctor tests were carried out on this material from TP14-285 indicated an average maximum dry density of 2,082kg/m^3, and an optimum moisture of 10%. In the same drill hole, a 4m thick MH interval was encountered at approximately 11m depth. The SPT "N" values for this interval ranged from 13 to 24.
- In the northeast section of the North Embankment, a 2m thick, gravelly, low plasticity clay (CL) interval was identified at surface in TP14-083. This interval had a moisture content of 6.5%, which is lower than typically observed in the area for this material type (Table 5.2).

Bedrock Geology:

The bedrock in the area of the north embankment consists of Nicola Group volcaniclastic and sedimentary rocks with an average rock quality designation (RQD) of 41% - 69% and an average rock mass rating 1989 (RMR_{89}) of 46 - 56 which corresponds to FAIR quality rock. Intact strength testing indicates a range in rock strengths between 4MPa (weak) and 162MPa (very strong).

Groundwater:

Static groundwater levels range from near ground surface to 23m below ground surface. Artesian conditions were encountered near the northeast corner of the north embankment (DH14-010). Results from in-situ hydraulic conductivity testing in bedrock indicate permeability values ranging from 10^{-6}m/s to 10^{-8}m/s.

5.4.2 East Embankment

Field and Lab Testing:

A total of 5 drill holes and 8 test pits were completed. Field testing included a total of 28 Lugeon tests in bedrock and SPT at 5ft intervals in 5 drill holes. Laboratory index testing on soil samples included 1 Particle Size Analysis and 1 Atterberg Limits. Laboratory strength testing was completed on rock samples only. These tests included 7 Unconfined Compressive Strength, 7 Young's Modulus, and 7 Specific Gravity.

Surficial Geology:

The overburden is thickest in the northern area (32m) and progressively thins to the south and southeast, where bedrock is encountered at approximately 2m depth in DH14-037 and TP14-294. No drilling was completed in the valley bottom due to access constraints but seismic refraction surveys shows soil thicknesses less than 5m (KPL, 2014).

Silty sand with gravel is the dominant material encountered in the east embankment footprint area. Gravelly ML is also observed but is less abundant. DH14-002 encountered a 14m thick interval of high plasticity silt (MH) that contains gravel at 3m depth. The lowest SPT "N" values for this material ranged from 17 to 22 at approximately 12m to 17m depth.

Bedrock Geology:

Bedrock consists of Nicola Group volcaniclastic and sedimentary rocks with an average RQD of 65% and an average RMR_{89} of 57 which corresponds to FAIR quality rock. Intact strength testing indicates a range in rock strengths between 12MPa (weak) and 107MPa (very strong).

Groundwater:

Static groundwater levels range from near ground surface to 33m below ground surface. Artesian conditions were encountered near the western slope of the valley bottom (DH14-005). Results from in-situ hydraulic conductivity testing in bedrock indicate permeability values ranging from 10-7m/s to 10-9m/s.

5.4.3 South Embankment

Field and Lab Testing:

A total of 3 drill holes and 10 test pits were completed. Field testing included a total of 9 lugeons and 4 falling head permeability tests in bedrock and SPT at 5ft intervals in soil at 3 drill hole locations. Laboratory index testing on soil samples included 1 Particle Size

Analysis, and 1 Atterberg Limit. Laboratory strength testing was completed on rock samples only. These tests included 5 Unconfined Compressive Strength, 5 Young's Modulus and 5 Specific Gravity.

Surficial Geology:

The overburden is thickest in the central section, which is approximately 29m deep (DH14-025). The overburden thins to approximately 3m in the west (TP14-121 & TP14-122) and to approximately 0.5m in the east (TP14-132 & TP14-133). The dominant material encountered in the area is sandy silt (ML) with gravel. Localized high plasticity fine grained material was identified, primarily within the top 1 to 3m below surface:

- A 3m thick interval of low density, high plasticity clay (CH) is encountered in DH14-034 at a depth of 1m. SPT "N" values range from 12 to 22 for this interval.
- A 2m thick interval of MH occurs in TP14-119 at approximately 1m from surface. This material was observed from the site investigation logs to be lacustrine sediments.
- Three additional 1 to 2m thick intervals of CH with gravel are encountered in DH14-025 at approximately 4m, 15m and 18m depths.

Bedrock Geology:

The bedrock in the area consists of Nicola Group volcaniclastic and sedimentary rocks with an average RQD of 41% and an average RMR of 46, which corresponds to FAIR quality rock. Intact strength testing indicates a range in rock strengths between 41MPa (medium strong) and 66MPa (medium strong).

Groundwater:

Static groundwater levels range 10m to 13m below ground surface in the drill holes. Two test pits (TP14-119 and TP14-120) encountered groundwater in a natural low area at approximately 3m and 5m respectively. Results from in-situ hydraulic conductivity testing in bedrock indicate permeability values ranging from 10^{-6}m/s to 10^{-8}m/s.

5.4.4 Southeast Embankment

Field and Lab Testing:

A total of 3 drill holes and 3 test pits were completed. Field testing included a total of 6 lugeons and 6 falling head permeability tests in bedrock, and SPT at 5ft intervals in 3 drill holes. No laboratory tests were completed on soils. Laboratory strength testing was completed on rock samples only. These tests included 2 Unconfined Compressive Strength, 2 Young's Modulus and 2 Specific Gravity.

Surficial Geology:
The overburden thins to the east, with thicknesses spanning from 60m in DH14-036, to 13m in DH14-038, and finally to approximately 1 to 3m in the far east section, as identified in TP14-197 and TP14-193, respectively. The dominant material type encountered in the area is silty sand with gravel (SM) based on information provided from the test pits and DH14-039. Low plasticity clay (ML) with trace to some gravel is observed in a localized area only (DH14-036 and DH14-038). A 2m thick interval of plastic clay (CH) with some gravel is identified at a depth of 2m in DH14-036.

Bedrock Geology:
The competent bedrock consists of Nicola Group volcaniclastic and sedimentary rocks with an average RQD of 31% and an average RMR^{89} of 44 which corresponds to FAIR quality rock. Intact strength testing indicates a range in rock strengths between 22MPa (medium strong) and 106MPa (strong).

Groundwater:
Static groundwater levels range 9m to 50m below ground surface in the drill holes. Results from in-situ hydraulic conductivity testing in bedrock indicate permeability values ranging from 10^{-5}m/s to 10^{-9}m/s.

5.4.5 TSF Basin

Field and Lab Testing:
A total of 8 drill holes and 122 test pits were completed. Field testing included a total of 23 lugeons in bedrock, 11 Guelph permeameters and SPT at 5ft intervals in 8 drill holes. The range of permeability data for the surficial soils ranged from 10^{-7}m/s to 10^{-5}m/s as shown on Figure 5-3.

Laboratory index testing on soil samples included 14 Particle Size Analysis, 14 Atterberg Limits, 15 Moisture Content and 4 proctors (2 modified and 2 standard). Laboratory strength testing was completed on rock samples only. These tests included 1 Unconfined Compressive Strength, 1 Young's Modulus and 1 Specific Gravity.

A total of four proctor (compaction) tests were completed: two tests at TP14-153 (GRAVEL – Glacial Till), two tests at TP14-277 (clayey SAND – Glacial Till) and two tests TP14-285 (sandy CLAY - Eolian Sediment).

Surficial Geology:

The surficial soils are glacial deposits composed of sandy-silt with thicknesses ranging from 1m to 36m. Glaciofluvial meltwater channel deposits overlie the till in some areas while eolian sediments (as identified by KPL) form a surficial cover up to 5m thick in the valley bottom and are prevalent on the northwest facing slopes. Colluvium is found on hillside spurs below bedrock outcrops and organic deposits are found in low lying, poorly drained areas.

Bedrock Geology:

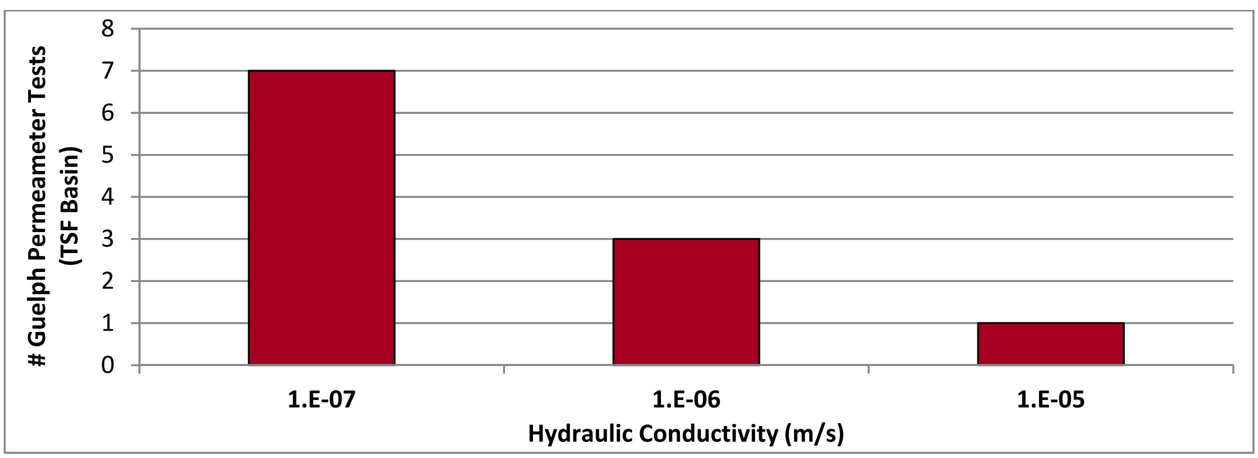
The competent bedrock in the basin area consists of Nicola Group volcaniclastic and sedimentary rocks with an average RQD of 69% and an average RMR^{89} of 56 which corresponds to FAIR quality rock. Intact strength testing indicates a range in rock strengths between 22MPa (medium strong) and 106MPa (strong).

Groundwater:

Static groundwater levels range from near ground surface to 11m below ground surface in the drill holes. Results from in-situ hydraulic conductivity testing in bedrock indicate permeability values ranging from 10^{-7}m/s to10^{-9}m/s.

Figure 5-3

Guelph Permeameter Results for the TSF Basin



6 TSF EMBANKMENT DESIGN

6.1 General

A description of the TSF dam structures and seepage drainage control measures are provided in this section. These design features are based on the tailings filling schedule (See Section 4.3 and 7) and stability analyses (See Section 6.6).

6.2 TSF Embankment Configuration

The TSF embankment configuration is summarized in Table 6-1 below and discussed in the following sections.

Table 6-1
Embankment Design Configuration

Ultimate Embankment[1]	Station	Ultimate Crest Length (m)	Maximum Embankment Height (m)	Ultimate Foundation Surface Area (ha)	Total Embankment Fill Volume (Mm^3)
North	1+550 to 5+200	3,650	122	155	71
East	0+000 to 1+450	1,450	108	27	7
South	0+000 to 1+550	1,550	42	21	4
Southeast	0+000 to 0+550	550	14	2	0.09

Note: 1. Upstream embankment slopes are 2.5H:1V and downstream embankment slopes are 3H:1V.

6.2.1 North Embankment (Station 1+550 to 5+200)

The starter embankment is comprised of the north embankment only. The starter embankment will be constructed to elevation 971m, which will have a total crest length of 1,400m and cover a foundation footprint area of 40ha. The total fill volume for this starter embankment is estimated at 4.6 Mm^3. Drawing C180-KA39-5100-00-006 shows the cross section for the starter north embankment.

The ultimate north embankment is approximately 3,650m long at its center line, has a footprint of 155ha, and a maximum embankment height of 131m. This embankment provides tailings containment on the north end of the basin above Jacko Lake and the open pit. Total fill volume is estimated to be approximately 71Mm^3. Drawings C180-KA39-5100-00-008 and C180-KA39-5100-00-007 shows the cross section and longitudinal section for the ultimate north embankment.

6.2.2 East Embankment (Station 0+000 to 1+450)

The ultimate east embankment is approximately 1,450m long at its center line, has a footprint of 27ha, and a maximum embankment height of 108m. It is situated across a small valley with steep side slopes on the east side of the TSF basin, just north of Goose Lake. Total fill volume is estimated to be approximately 7Mm^3. Drawing C180-KA39-5120-00-002 shows the cross section and longitudinal section for the ultimate east embankment.

6.2.3 South Embankment (Station 0+000 to 1+550)

The ultimate south embankment is approximately 1,550m long at its center line, has a footprint of 21ha, and a maximum embankment height of 42m. The south embankment joins with the north embankment after year 5. It is situated at the south end of the TSF basin to constrain the TSF within the KAM property boundary. Total fill volume is estimated to be approximately 4Mm^3. Drawings C180-KA39-5100-00-008 and C180-KA39-5100-00-007 shows the cross section and longitudinal section for the ultimate south embankment.

6.2.4 Southeast Embankment (Station 0+000 to 0+550)

The ultimate southeast embankment is approximately 550m long at its center line, has a footprint of 2ha, and a maximum embankment height of 14m. It is situated at the south end of the TSF basin to constrain the TSF within the KGHM property boundary. Total fill volume is estimated to be approximately <1 Mm^3. Drawing C180-KA39-5140-00-001 shows the cross section and longitudinal section for the ultimate southeast embankment.

6.2.5 Basin

The TSF Basin (Basin) is approximately 3,700m long, has a footprint of approximately 541ha. It is a large natural depression situated within the Keynes Valley and will be bounded by the four embankments.

6.3 TSF Embankment Material Estimates

The TSF Embankments will comprise of the following zones:

- Till Blanket: will be constructed from low-permeability glacial till material from pre-strip of the open pit or suitable material from the site preparation of the TSF embankment foundation. The till blanket will reduce any pore pressures in the embankment and minimize any seepage through the embankment. Prior to placement of this seepage control zone on the upstream face of the TSF, the foundation surface will include moisture conditioning and smoothing before installing a non-woven geotextile.

- Compacted Mine Rock: will be constructed from overburden and specific mine rock materials from open pit operations. The material placement and compaction requirements for this embankment zone will be more stringent compared to the mine rock buttress and mine rock storage facility to minimize any settlement or structural issues that may impact the upstream seepage control zone (i.e. till blanket) and will be provided as part of the detailed design work.

- Mine Rock Buttress: will be constructed from mine rock materials from open pit operations. The construction specifications for the compacted mine rock will be included as part of the detailed design work.

- Mine Rock Storage Facility: will be constructed from mine rock materials from open pit operations. The specifications for this material will be provided as part of the detailed design work.

6.4 Seepage and Drainage Controls

The seepage and drainage controls included in the TSF design are as follows:

- An upstream seepage control zone composed of low permeability engineered till (i.e. till blanket).

- A filter zone to prevent piping and internal erosion through the upstream till blanket. Currently a geotextile filter is being considered to prevent the migration of fines into the pervious downstream zones (compacted mine rock, mine rock buttress and mine rock storage facility).

- A seepage cutoff is planned beneath the dam foundation along the upstream toe or as close as possible to the dam centreline. The seepage cutoff will tie into in-situ lower permeability materials in the foundation to minimize infiltration through the in-situ soils. The foundation materials indicate that there is dense glacial till with an average in-situ permeability of approximately 10^{-6}m/s that will reduce as tailings is deposited along the upstream dam face and basin.

- Rock drains will be constructed to convey any seepage flow from the downstream toe to seepage collection ponds, and relieve any hydrostatic pressures within the embankment and downstream mine rock buttress. These drainage control features will be constructed with processed non-reactive fluvial, colluvial or selected mine rock. In order to prevent the intrusion of fine soils (silt and clay) into the rock drain, fabric geotextiles, geosynthetics (HDPE) and a granular filter zone will be employed as necessary components of the design. This design will reduce the risk of the rock drain becoming impeded or blocked and incapable of conveying the design flow. Recovered seepage water will either be pumped to the Central Collection Pond (CCP) to be pumped to the process plant as process water. Details on these seepage collection ponds are provided in Section 7.3.1.

- A minimum beach width and maximum operating water level and allowances for water storage contingency within the TSF to prevent a release of the supernatant pond in the event of a dam failure.

6.5 Potential Failure Modes Analysis

A Potential Failure Modes Analysis (PFMA) was completed in a separate study that considered different failure modes at different stages of operation and construction. This PFMA study was completed as part of the EAO requirement (See Section 2.2) and recommended by the independent expert review panel to evaluate Best Available Practices (BAP) as part of the TSF design process:

> "*At Mount Polley, the only quantitative performance objectives were those implied in its design criteria. A list of potential failure modes was compiled in the 2006 Dam Safety Report, but these were generic and not tied to specific site conditions. One of the lessons learned here is that future permit applications for TSFs must provide a more comprehensive assessment of potential geotechnical problems associated with the selected site*."

The PFMA is a systematic, proactive method for evaluating each dam structure to:

- Identify where and how it might fail (potential failure modes),
- Generally assess the likelihood of the failure occurring and the effects (consequences) of such failure, and
- Identify potential risk reduction measures that could reduce dam breach potential.

The PFMA identified a list of risk reduction measures that mitigate the failure mode by lowering probability of failure and/or reducing or eliminating the consequence altogether. Results from the PFMA are provided are in a separate study (Norwest, 2015). These measures are especially important for managing the risk of a dam breach event and therefore incorporated (where possible) as part of the TSF design provided in this report.

6.6 Stability Analyses

Stability analyses were carried out with the computer program Slope/W, which uses the method of slices to calculate the FOS in two dimensions. FOS is specified using Spencer's Method to solve for force and moment equilibrium. Cross sections were developed through the geological model at maximum dam height sections at each of the TSF embankments for use in the 2D Slope/W stability models. These cross sections, material design parameters, modelling assumptions and detailed results are presented in Appendix B.

The results of the stability analyses indicate the TSF dam configuration satisfies the stability criteria (outlined in Section 2.4) under static and pseudo-static loading conditions. Stability results for each embankment are summarized in Table 6-2. In addition to meeting the minimum design criteria, stability analyses were also carried out to evaluate for a failure surface that could lead to a breach into the impoundment; shown as Downstream (D/S) Slope – Dam Breach in Table 6-2. Due to the very large buttress that is built adjacent to the engineered fill embankment, very high safety factors are obtained for the breached condition. This means that a breach is very unlikely based on the assumed conditions.

Table 6-2
Stability Analyses Results

Phase	Embankment	Section	U/S Till Blanket	U/S Slope[1]		D/S Slope – Dam Breach	D/S Buttress[2]		Meets Design Criteria
			Static	Static	Pseudo-Static	Static	Static	Pseudo-Static	
Starter[3]	North	A	1.35	1.87	1.04	7.07	2.02	1.06	Yes
		B	1.36	1.87	1.04	4.24	1.99	1.06	Yes
Ultimate	North	A	-	1.87	1.08	3.10	1.76	1.01	Yes
		B	-	1.91	1.11	3.25	1.73	1.01	Yes
	South	D	-	-	-	2.1	-	-	Yes
	Southeast	E	-	-	-	2.39	-	-	Yes
	East[4]	C	-	1.77	1.02	3.63	1.59	1.02	Yes

Notes:

1. Stability analysis for the upstream slope was not completed for the south and southeast embankment because it was suitably buttressed by thickened tailings and it was not considered a critical failure mode.
2. The current design does not include a downstream buttress for the south and southeast embankment. For this reason, no stability analysis was completed for these areas

3. The starter embankment includes the north embankment only.
4. The downstream slope of the rock buttress for Section C was simplified as the rock buttress footprint is much larger than the embankment footprint.

7 SUPERNATANT POND AND TAILINGS MANAGEMENT PLAN

7.1 Supernatant Pond Plan

A volume of 2.4Mm^3 is judged to provide sufficient make-up water to meet 90 days of production requirements, provide contingency for water that may not be available at start-up, allow retention time for settlement of suspended solids within the pond during operations, and allow the pond elevation to stay within desired design elevations during the first year of operations (Norwest, 2015). This water will be contained by the starter north embankment. A trenched channel will be excavated within the basin (prior to plant site commissioning) to provide a surface water connection between Goose Lake and the TSF supernatant pond for the barge recycle system, during the initial production years as shown in Drawing C180-KA39-5000-00-003. This barge channel is approximately 7m deep, 12m wide at the base with 2H:1V side slopes.

As tailings are deposited into the facility, water will be released from the tailings stream during deposition and subsequent consolidation and will eventually report as supernatant water to the main tailings pond. A portion of the supernatant water will be lost due to evaporation and from seepage into the foundation. The remaining water will be available as recycle to the plant site. Deposition of tailings will be sequenced such that tailings beaches displace the supernatant pond to the south, away from the north embankment. Over time the tailings will slowly infill the barge trench towards Goose Lake. The supernatant pond level will be maintained at a level which allows for on-going operation of the reclaim water barge. Normal tailings operations will see the lowermost portion of the tailings beach submerged below the supernatant pond. Water will be reclaimed through a floating pump barge, and pumped to a reclaim tank at a high point on the reclaim water line, where it will drain via gravity to the process plant. The reclaim water line will parallel the same service road being used by the tailings distribution pipeline corridor.

7.2 Tailings Deposition Plan

The tailings slurry shall be discharged via spigots from various points along the upstream crest of the starter dam or north embankment. The tailings deposition plan for the TSF is summarized as follows:

- Initial tailings slurry discharge shall be from the north and south abutments of the north embankment to develop the beaches at these locations first and reduce seepage gradients through the abutments.

- Tailings discharge points shall be moved as required to ensure the apex elevation of the deposition fan does not violate freeboard criteria.

- The method of tailings deposition (spigotting from perimeter) is expected to result in beaches with approximately 1 to 2% overall slopes. Beach slope angles will vary with solids content, discharge velocity, season, and the total height of tailings. Based on results from recent tailings laboratory test work (KPL, 2015), it is expected that the slurry tailings at 60% solids content will consolidate relatively quickly in the TSF deposit to 77% solids content (with an estimated tailings density of approximately 1.6t/m^3).
- Supernatant pond water will be reclaimed from the TSF for use as process water for ongoing process plant operations throughout the life of mine.
- Trafficability of tailings will be poor where the tailings are saturated or nearly so. Access to the beach and pond areas shall be restricted.

A TSF filling schedule was developed that shows the tailings mass balance or cumulative tailings volumetrics, freeboard for PMF and wave-up and respective dam crest elevations to contain the water and tailings volumes on an annual basis (See Figure 4-2).

7.3 Surface Water Management

A conceptual surface water management plan was developed to manage water at the TSF in a manner that provides sufficient water to support ore processing while minimizing the potential for storm flows to cause damage to mine structures, and minimizing the potential for mining operations to cause adverse effects to downstream water quality. Water will be controlled to minimize erosion in areas disturbed by construction activities and prevent the release of sediment laden water to the receiving environment. This includes diverting and/or collecting surface water runoff, constructing sediment control ponds, and managing pump back systems.

The key facilities identified in the water management plan for the TSF include seepage collection ponds and diversion ditches. These surface water management facilities are described in the following section. A site wide water management plan has been developed for the Ajax Project in a separate study (BGC, 2015).

7.3.1 TSF Seepage Collection Ponds

The TSF seepage collection ponds are designed for ponds are designed for the following conditions;

- During construction activities, prior to deposition of tailings, the water management ponds have been sized to capture sediment, and for the 10-year, 24-hour runoff event in accordance with the Draft Guidelines for Assessing the Design, Size and Operation of Sedimentation Ponds Used in Mining (MELP

2015). During construction, the ponds will discharge to the environment. The ponds will be designed to withstand a 1 in 200-year flood event.

- The sediment ponds designed for construction sediment control will be retained for use throughout mine operation, and post-closure. These ponds will contain surface runoff to allow sediment to drop out, including contact water and any seepage fluid, which will be pumped to the Central Collection pond to be used as process water at the plant site. The water management ponds will provide storage for seasonal runoff conditions and have storm water storage in excess of the 200-year, 24-hour rainfall event.

The ponds will be constructed of earth fill/rockfill materials, and will include low permeability HDPE basin liners, as required by site conditions. The ponds will be designed with emergency spillways to discharge to the environment in the events that exceed the capacity of the ponds. The seepage collection ponds are sized based on the assumption that non-contact water diversion ditches will be in place to minimize the catchment areas contributing to the sediment/seepage collection ponds.

There are four (4) seepage collection ponds located downstream of the TSF embankments:

- <u>North Embankment Seepage Collection Pond #1</u>: This seepage collection pond is located downstream of the north embankment along the natural valley drainage to the northwest of the downstream toe. Details on the sizing calculations are included in Appendix A. Runoff from an estimated 2.7km^2 catchment that extends southwards to the end of the south embankment will drain to the pond. The containment berm will be approximately 11m high dyke with a crest length of approximately 280m. The pond is sized for the area required for sediment removal during construction of the starter embankment. The elevation at which the required area can be met gives a storage volume of 119,000m^3, which greatly exceeds the 1 in 200 year, 24-hour volume of 50,000m^3.

- <u>North Embankment Seepage Collection Pond #2</u>: This seepage collection pond is located downstream of the north embankment along the natural valley drainage to the northeast of the downstream toe. Details on the sizing calculations are included in Appendix A. The containment berm will be approximately 21m high dyke with a crest length of approximately 107m.The pond is sized for the area required for sediment removal during construction. The elevation at which the required area can be met gives a storage volume of 129,900m^3, which greatly exceeds the 1 in 200 year, 24-hour volume of 10,000m^3.

- South and Southeast Embankment Seepage Collection Pond: This seepage collection pond is contained within natural topography between the downstream toe of the South Embankment and the upslope terrain. No containment berm is proposed. A pond will form in the existing topography against the toe of the embankment.
- Central Collection Pond: Seepage from the East Embankment, as well as runoff from part of the WMRSF and SMSRF will be collected in an underdrain system that will discharge into the Central Collection Pond, which is designed by others.

7.3.2 Diversion Ditches

Runoff collection ditches will be constructed to intercept surface water runoff from areas impacted by mine operations, and route it to the collection ponds where it can be effectively managed. Separate ditches will be will be required to intercept water that has not come into contact with mining activities, and divert it to catchments downstream of mining activity.

A layout of the permanent water management structures required through the operation of the mine is shown on Drawing C180-KA39-5000-00-015. This includes the following ditches that will be used to collect contact runoff and convey it to the collection ponds described under 7.3.1 above, or to divert non-contact runoff.

- Contact Water Collection Ditches: Contact water collection ditches will be constructed to intercept runoff from the north embankment and Southeast Embankment to prevent the release of contact water into the surrounding environment. The permanent ditches will be constructed along the edge of the ultimate footprint, and sized to convey the 1 in 200 year, 24-hour peak flow to the collection ponds. Sizing of the ditches will be finalized through the next phases of design. Appendix A includes descriptions drainage areas served by the contact water ditches along the north embankment and south embankment mine rock buttresses, which discharge to Sediment Pond 1 and Sediment Pond 2. Ditches required elsewhere on the perimeter of the TSF, and directing surface contact water directly to the Central Collection Pond and the SMRSF Collection Pond are designed by others.
- Non-Contact Water Diversion Ditches: Non-contact (clean) water ditches will be constructed around the perimeter of the TSF to collect and divert surface runoff from undisturbed areas up gradient of the project footprint. This water will be directed around the TSF. A diversion ditch is proposed immediately south of the

South Embankment to intercept non-contact runoff, and convey it westwards towards the undisturbed catchment that drains to Jacko Lake. The diversion ditch will be sized for the 1 in 200 year, 24-hour event.

Collection ditches may be either temporary or permanent structures. The layouts for temporary collection ditches for construction will be designed continually to suit the changing construction footprint and non-contact areas through different stages of the TSF life. The temporary diversions required during construction of the TSF starter embankment are discussed in Appendix A, which includes sizing of Sediment Pond 1 and Sediment Pond 2, based on the footprint area that is assumed to be disturbed and contributing sediment laden runoff to the sediment control ponds. Following completion of the starter embankment and through various stages of the north embankment and WMRSF construction, the permanent contact water diversion ditch will be in place to intercept runoff from the disturbed area below the TSF embankment and convey it to the sediment control ponds. Temporary non-contact collection ditches will be constructed upstream of the southern extent of the north embankment and TSF MRSF footprint. The temporary non-contact collection ditches will be sized to convey the runoff from a 1 in 10 year return period 24-hour storm event. These collection ditches will be inspected and maintained regularly during the construction phase and any potential blockages to flow (accumulated sediment, debris, etc.) will be removed.

7.3.3 Rock drains

The rock drains were designed with capacity to manage approximately 10 times the steady state seepage flow rate through the dam to control the phreatic surface within the embankments and MRSF buttresses to the north seepage collection ponds #1 and #2, Central Collection Pond.

The rock drains will facilitate drainage at the base of the embankments/buttresses along existing topographic lows and pre-mining surface drainage courses. The lowest point in the natural watercourse will be used as the center line of each of the drain alignments. Where a defined watercourse does not exist, the lowest point in the topography should be used as the centerline of the drain.

The rock drains will be constructed by excavating a channel in the natural topography which will be lined with an HDPE liner, a base geotextile and filled with drain rock. The drain rock will be covered with a cover geotextile and a minimum of 1m of select competent non-reactive mine rock for protection from truck dumped mine rock. The first lift of truck dumped mine rock above the completed drains should be no thicker than 5m.

In order to prevent the intrusion of fine soils (silt and clay) into the rock drain, an HDPE liner, fabric geotextiles and a granular filter zone will be employed as necessary components of the design. This design will reduce the risk of the rock drain becoming impeded or blocked and incapable of conveying the design flow.

7.4 Dam Breach Analysis

A separate study was completed for the Ajax Project to evaluate potential consequences of dam failures. The results of this study are provided in a separate document (Norwest, 2015).

This study was completed in three stages as follows:

- **Stage 1: Screening Level Study:** Collect available information related to the TSF design and staging of the dams and highlight failure scenarios that could lead to a dam breach during pre-production, production and closure. Screen these scenarios to obtain a set of critical scenarios that highlight appropriate “sunny” and “rainy” day failure modes for further failure mode assessment.
- **Stage 2: Potential Failure Modes Analysis:** Complete a Potential Failure Modes Analysis (PFMA) of the highlighted cases identified in the Screening Level study (Stage 1) to identify potential mechanisms that could lead to a dam breach, generally assess the likelihood of the failure occurring and the effects (consequences) of such failure, and identify potential risk reduction measures that could reduce breach potential.
- **Stage 3: Dam Breach Evaluation:** Complete a dam breach/inundation analyses on the highlighted credible cases that could lead to a potential dam breach scenario resulting from the PFMA in Stage 2.

A list of risk reduction measures that mitigate the failure mode by lowering the probability of failure and/or reducing or eliminating the consequence altogether was completed. These measures are especially important for managing the risk of a breach event with subsequent downstream inundation potential. These mitigation measures were considered and incorporated, where possible, in the TSF dam design presented in this report.

8 PERFORMANCE MONITORING PLAN

8.1 Operations, Maintenance and Surveillance Manual

A thoroughly documented system of operation, maintenance and surveillance (OMS) of the TSF will be used to formally test design assumptions and make improvements to the design over time. The OMS manual will be developed to provide specific details of this work. The operations and maintenance procedures outlined in the OMS manual will provide appropriate levels of monitoring that are necessary and may change throughout the life of the TSF to better suit the dynamic site conditions. The OMS manual is considered a living document that will be updated as required to provide the necessary guidance to monitor the performance of the TSF. In some cases, a review of the OMS manual should be done as part of the dam safety inspections and as part of the Dam Safety Review (CDA, 2013).

The development of the OMS manual will be completed upon permit approval of the detailed design. The key elements of the OMS manual will be completed in accordance to the recommendations provided in Section 3 of the Dam Safety Guidelines (CDA, 2013) and Mining Association of Canada (MAC, 2004).

8.2 Roles and Responsibilities

- **Owner**: KAM is the dam owner. The Owner is responsible for the appointment of the project team and retains suitable companies for different aspects of the project such as construction management, design, and QA/QC. The responsibilities of the Owner may include, but are not limited to:
    - Dam safety reviews on their dams with certain classifications and at the intervals provided in the British Columbia Dam Safety Regulation (for water reservoir dams).
    - Delivery of all required land tenures and project approvals.
    - Delivery of the sites to construction management and contractors.
    - Liaison with government agencies.
    - Liaison with the local communities and land owners.
    - Communications control.

- **<u>Engineer of Record (EOR):</u>** The EOR is the qualified geotechnical engineer for the design and construction of the TSF dams for the Ajax Project. The responsibilities of the EOR may include, but are not limited to:
  - The TSF design, design changes, completion of regular field reviews throughout construction to ensure a quality end product by ensuring construction is carried out in accordance with the construction design drawings and technical specifications.
  - Review and audit of QC processes and results, daily reports, weekly construction plans and mine plans and related documentation for any deficiencies or issues.
  - Written documentation of regular field reviews in accordance to the standard regulatory requirements.

8.3 Instrumentation and Monitoring

The purpose of the geotechnical construction and instrumentation monitoring program is to:

- Confirm design assumptions and ensure performance is within expected bounds of the geotechnical design.
- Provide data for any future design changes or opportunities for optimization.
- Provide early warning of defects or impending failure.
- Predict future performance.

The construction and instrumentation monitoring program are summarized in the following sections. The performance monitoring plan will be designed to evaluate the following items though the life of the TSF:

- Horizontal movement of embankment and foundation soils.
- Vertical settlement.
- Pore-water pressure.
- Seepage rates.

8.3.1 Construction Monitoring

Construction monitors will be used throughout construction of the TSF to identify and document that the construction satisfies the technical specifications and to identify and communicate when non-conformances are detected throughout the construction process. Basic responsibilities of the construction monitor include:

- Carrying out quality control testing and visual inspections of all fill types/clean up activity as required by the technical specifications, as shown on the drawings.
- Collating and analyzing all quality control test results and implementation of any remedial actions that may be required or considered appropriate.
- Coordination of all quality control test results for further review and analysis.
- Preparation of daily construction reports and/or as-built construction. This will include:
    - As-built drawings and construction photographs.
    - Any potential difficulties and an assessment of any critical design items that may have arisen or may arise in the near future.
    - Safety Performance Reporting.

Recommended frequencies for visual inspections of the TSF throughout construction will include:

- Weekly or monthly routing inspections done by the Owner.
- Engineering inspections done by the EOR or designated qualified professional engineer (in accordance to EXTREME consequence category for the TSF).
- Special inspection after unusual or extreme events, as required.

8.3.2 Instrumentation Monitoring

Geotechnical instrumentation will be installed within the dam fill and foundation. The instrumentation will be monitored to assess dam performance and identify any conditions that may be different to those assumed during design and analysis.

Geotechnical instrumentation will include: vibrating wire piezometers (VWP), slope inclinometers (SI) and standpipe piezometers (SP).

A summary of instrumentation requirements will include:

- SIs to monitor any deformations in the dam fill and/or movement in the underlying foundation materials. These instruments will be installed at the downstream toe of the embankment/buttress during initial dam construction and will extend a minimum of 5m into competent foundation beneath the embankments.

- VWPs will be installed in the dam (engineered fill, filter zone) and foundation to measure piezometric levels, drain performance and in situ pore pressures during operations. The piezometer leads will be appropriately routed from these areas to read-out panels for ease of monitoring.

- SPs will be used as required downstream of the TSF embankment. The standpipes will be used to monitor water quality and as an alternative method to the VWP to measure piezometric levels.

A summary of geotechnical instrumentation planned for the TSF is summarized in Table 8-1. These instruments will be spaced at reasonable intervals that are perpendicular to the centerline of the dam and will include slope inclinometers (SI), vibrating wire piezometers (VWP) and standpipe piezometers (SP). A typical instrumentation cross section proposed at the TSF is shown on Drawing C180-KA39-5000-70-001.

Table 8-1
Estimated Quantity of Performance Monitoring Instruments

Embankment	Instrument Type [1]	Description of Typical Section[2]
North	SI	• Inclinometers will be installed at the toe of the embankment/ buttress/MRSF. • Inclinometers shall be installed five meters into competent rock.
East South	VWP	• Piezometers will be installed in the foundation beneath the ultimate crest and ultimate toe. • Piezometers will be installed between the till blanket and the compacted mine rock as the embankments are raised. • Interim piezometers may be necessary during construction.
Southeast	SP	• A standpipe piezometer will be installed at the downstream toe of the embankment/buttress.

Notes:
1. Instrument types include Slope Inclinometers (SI), vibrating wire piezometers (VWP) and standpipe piezometers (SP).
2. See drawing C180-KA39-5000-70-001 for a typical cross sectional view with estimated number of instruments.

9 CLOSURE PLAN

9.1 Closure Alternatives Assessment

Norwest completed a separate conceptual TSF closure alternatives assessment (as outlined in the CDA Guidelines) to evaluate the BAT for the Ajax Project (Norwest, 2015). The study included design criteria, trade-off advantages and disadvantages, Rough Order of Magnitude (ROM) quantities and associated costs for wet and dry closure options.

The dry cover closure option was identified as the best alternative for the Ajax Project based on the following key advantages:

Land is returned to a terrestrial landscape, as close as practical to pre-mining conditions.

- The terrestrial landscape closure option has the added benefit of lower embankment stability risks (compared to a wet closure) during the post-closure period as no water is impounded. The consequence of failure of wet cover system a more severe than for a dam containing a dry cover system. Not only would a dam breach include the water forming the wet cover, but the tailings that are contained will be more mobile because they are saturated.
- Minimal ponded water at closure. A stagnant pond at closure results in a concentration of contaminants remaining in the pond as water evaporates. Ponded water also results in increased infiltration through the base of the pond which leads to an increase in contact seepage water and an expected corresponding reduction in discharge water quality.
- Fewer OMS requirements for a dam supporting a dry cover system than a wet cover system.
- There is minimal impact to a dry cover system to variations in climate change, as compared to a wet cover system.

9.2 Capping Plan

9.2.1 Spillway and Water Management

The entire tailings surface will be covered and recontoured with an earth fill cover to minimize ingress of surface water and to pass runoff (upon meeting water quality requirements) into an engineered channel towards the south of the TSF and into Humphrey Creek. This dry closure option will be designed to minimize ponded water within the TSF footprint and infiltration into the tailings in perpetuity. Plan view, longitudinal and cross sections of the spillway and dry capping closure concept is provided in Drawing C180-KA39-5000-00-014.

9.3 Long Term Monitoring

Long term monitoring will be completed as part of the post closure period. The post closure period is defined by ICOLD (2013) as,

"the period following cessation of operation of the tailings into its final form. The post closure period is generally divided into active and passive care periods. Active care is the period when intervention and monitoring is required to achieve a final sustainable form concurrently with stabilization of the structures and environmental elements. Passive care is the period following active care during which the performance of the tailings dam is monitored to ensure its compliance with the closure objectives. This period has no time limit, but can be defined as being necessary until the tailings dam, in the opinion of the regulatory authorities, is considered to be physically, chemically, ecologically, and socially stable and no longer poses a risk to life or the environment."

Figure 9-1 provides a comparison of the phases of Tailings Management between CDA, MAC and ICOLD. The development of a terrestrial landscape or dry TSF cover system at closure will reduce embankment stability risks as no water is impounded. Long term monitoring and maintenance will be reduced accordingly.

Figure 9-1
Phases of Tailings Management between CDA, MAC and ICOLD

	CDA Mining Dams Bulletin		MAC (1998)	ICOLD (2011)		ICOLD (2013)	
Time	Site Selection and Design		Site Selection and Design	Planning		Not addressed in this ICOLD bulletin	
				Design			
	Construction		Construction	Construction			
	Operation		Operation	Operation			
	Closure	Transition	Decommissioning and Closure	Closure	Decommissioning	Post Closure	Active Care
					Remediation		
		Active Care			After Care		
		Passive Care		Long Tem Monitoring			Passive Care
	Landform (not considered a dam)		Not addressed by MAC	Not addressed by ICOLD (2006)		No longer poses a risk to life or environment	

Reference: Application of Dam Safety Guidelines to Mining Dams, CDA 2014.

10 RECOMMENDATIONS

Norwest understands that the current TSF design will be advanced to support a MEM permit level report following the EA application. The recommendations below pertain to work related to completion of permit level design activities as well as future construction works. It is recommended that the following design items and project tasks be completed to support the planned MEM permit submission and future construction activities:

Detailed Design

- Infill drilling and testing (completed).
- Detailed stability/seepage analyses
- Detailed surface water assessment
- Site specific seismic hazard assessment

Construction of Starter Embankment:

- Detailed specifications for engineered fills.
- Detailed performance monitoring specifications for construction and instrumentation that will work towards developing guidelines for monitoring trigger levels and the extent in which the observational method of dam construction is applied.

Independent Tailings Review Board

- Retain an Independent Tailings Review Board (ITRB) to review the TSF design, construction, operation and closure and for ongoing advice on tailings operations to complement KAM's internal technical audit systems.

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12 CLOSURE

This report has been prepared for KGHM Ajax Mining Inc. to provide a tailings storage facility at the Ajax Project, located near Kamloops, British Columbia. This document represents the opinion of Norwest based on information provided by KGHM Ajax Mining Inc. and observations made during limited site visits.

All geotechnical data contained herein has been prepared and directly supervised by Christopher Klassen, P.Eng., with hydrological input by Eugene Ngwenya, P.Eng. and the review carried by Richard Dawson, P.Eng. Ph.D. and Sean Ennis, P.Eng., PE. As mutual protection to KGHM Ajax Mining Inc., the public, and Norwest Corporation, this report and its figures are submitted for exclusive use by KGHM Ajax Mining Inc. We specifically disclaim any responsibility for losses or damages incurred though the use of our work for a purpose other than described in the report. Our report and recommendations should not be reproduced in whole or in part without our express written permission.

August 26, 2015

"original signed and sealed by author"

Prepared by:

Christopher Klassen, P.Eng.
Manager, Geotechnical Engineering

Eugene Ngwenya, P.Eng
Senior Water Resource Engineer

Reviewed /Approved by:

Richard Dawson, P.Eng
Senior Vice President

Reviewed /Approved by:

Sean Ennis, P.Eng, P.E.
Vice President, Mining

Appendix A
Hydrology and Surface Water Management

APPENDIX A HYDROLOGY AND SURFACE WATER MANAGEMENT

1 INTRODUCTION

This appendix provides details on the hydrology input to the design of the Tailings Storage Facility (TSF), and associated water management structures for the Ajax Project. The appendix forms part of the Preliminary Design Report.

The TSF will be designed to retain, without discharge to the environment, runoff from precipitation on the TSF footprint and the catchments that drain into it. A diversion channel and collection ponds are required to manage runoff from construction of the starter embankment. Collection ditches and ponds are required for contact runoff during operation and post-closure.

Drawings C180-KA39-5000-00-016 and C180-KA39-5000-00-017 show the catchments, TSF and water management structure general arrangements, at start-up and at end of production. Norwest is completing designs for the North Seepage Collection Pond #1 and North Seepage Collection Pond #2, as well as the collector ditches that drain into them and other surface water management structures around the perimeter of the TSF embankment. Tailings distribution and reclaim water systems are being design by others.

2 SITE CLIMATOLOGY AND HYDROLOGY DATA

The Ajax project is located within the semi-arid steppe climate of South-Central Interior of BC. The climate of this region is characterized by the generally low annual precipitation and high evaporation. The winters are usually cool and dry and the summers hot and dry, with relatively low humidity, and high evaporation rates.

The TSF design uses climatology data presented in the KP (2015) climatology report. This appendix also refers to the KP (2014) report that includes snowmelt considerations in the development of the IDF for the previous design, as well as the BGC (2015) water management plan for the project.

3 DESIGN CRITERIA

3.1 TSF Inflow Design Flood

The Inflow Design Flood (IDF) is the Probable Maximum Flood (PMF). This selection of the IDF is based on the facility's Extreme dam consequence classification, in accordance with the Canadian Dam Association (CDA, 2013) guidelines. The TSF will be designed to contain all runoff up to the PMF from precipitation on the TSF footprint and the catchments that drain into it, with no release to the environment.

The PMF is estimated as runoff from the Probable Maximum Precipitation (PMP) event. An estimate for the 24-hour PMP is provided in the climatology studies (KP, 2015). However, as the TSF will be operated without an emergency overflow spillway a longer duration PMF event is judged more appropriate. On that basis, Norwest recommends an IDF based on the 72-hour PMP event, and with the snowmelt contribution as previously assessed (KP, 2014). The recommended IDF is runoff from a 72-hour PMP + 100-year return period snowpack – average annual snowpack.

3.2 Collection Ponds

According to the Joint Application Information Requirements for Mines Act and Environmental Management Act Permits prepared by the British Columbia Ministry of Energy and Mines and the British Columbia Ministry of Environment (Draft - February, 2015), the application should;

- Provide sediment pond design consistent with the "Guidelines for Assessing the Design, Size and Operation of Sedimentation Ponds Used in Mining (Draft – MELP 2015).
- Provide hydraulic capacity and confirmation that all ditches/channels can safety convey the design flood in accordance with CDA Guidelines (minimum 1:200 years) without overtopping, side slope failure or significant erosion.

On that basis, the following design criteria was selected;

3.2.1 Sediment Control During Starter Construction

North Embankment Collection Pond #1 and North Embankment Collection Pond #2 will be designed for the following criteria during the pre-production phase:

- Sized to capture sediment (10 micron soil particle) for the 1 in 10 year, 24-hour runoff event.
- Structures in the sediment pond system will be designed to structurally withstand a 1 in 200 year 24-hour runoff event.

3.2.2 Surface Runoff Collection

During the operation and post-closure phase, the ponds will meet the following criteria:

- Sized to store the volume from the 1 in 200 year, 24-hour runoff event.

3.3 Diversion Channel and Ditches

The collector ditches will be designed for the following criteria:

- hydraulic capacity to safely convey the 1 in 200 year 24-hour flood without overtopping, side slope failure or significant erosion.

A temporary diversion channel will be required during starter dike construction. The diversion channel will be designed to the following criteria:

- hydraulic capacity to safely convey the 1 in 10 year 24-hour flood without overtopping, side slope failure or significant erosion.

3.4 Precipitation event data

3.4.1 IDF estimate

A 24-hour PMP was estimated as 219mm in the site's climatological assessment (KP, 2015). Norwest has estimated the 72-hour PMP based on the ratio between 72-hour and 24-hour PMP presented in the Hydrometeorological Report 57 (USACE, 1994), which covers part of the areas in British Columbia. Adding the estimated 72-hour PMP (339mm) to the previously assessed (KP, 2014) snowmelt contribution (108mm) gives a precipitation amount of 447mm.

3.4.2 Sediment Pond and Diversion Sizing

The 1 in 10 year, 24-hour and 1 in 200 year 24-hour precipitation depths are 43.2mm and 68.5mm, respectively (KP, 2015).

4 IDF VOLUME ESTIMATE FOR TSF STORAGE

The total area of the TSF footprint and the catchment area draining into it varies between 10.3km^2 at start-up, and up to 10.9km^2 at end of production (see Drawings C180-KA39-5000-00-016 and C180-KA39-5000-00-017). A runoff coefficient of 0.8 was assumed for the undisturbed natural catchment, and a runoff coefficient of 1.0 assumed for the TSF beach and water pond footprint. For the IDF precipitation amount of 447mm, the volume requiring storage within the TSF is estimated to range between 3.7Mm^3 and 4.4Mm^3 (a range has been calculated to reflect the changing form of the catchment over time as tailings are deposited).

An approximate average value of 4.1Mm^3 was used for developing the TSF filling schedule. At later design stages, the runoff factors can be reassessed based on proportions of undisturbed natural catchment, TSF beach and water pond for the different stages of the TSF life.

In addition to runoff that drains naturally following storm events, water will be pumped into the TSF from the North Embankment seepage collection ponds, Central Collection Pond and other collection ponds around the site. An additional allowance for 0.3Mm^3 is included for pumping from the ponds during storm events.

The planned TSF construction schedule is such that there is always excess freeboard above normal pond water level. The freeboard height is as large as 38m at end of year 1, and at its lowest, at the end of production is 13m.

5 SURFACE WATER MANAGEMENT

5.1 General

Surface water management during the pre-production, operational and post-closure phases will involve conveying water that has come into contact with construction and mining activities to collection ponds and, where possible, diverting non-contact water away from mining and construction activities, to undisturbed catchments.

In the pre-production phase, sediment from runoff generated during construction of the starter embankment will be captured at the sediment ponds, consistent with MELP (2015) guidelines. The ponds will discharge to the environment during this period.

After construction of the starter embankment, when tailings deposition and mine rock placement starts, the collection ponds will retain runoff from the embankment and mine rock buttress as well as seepage through the embankment. The runoff will be contained without release to the environment, and pumped to the Central Collection Pond or directly to the TSF. Similarly, throughout operation and post-closure, contact runoff and embankment seepage will be contained in the sediment ponds, and pumped to the Central Collection Pond.

5.2 Collection Ponds

5.2.1 Catchment and flow estimates

The selected locations for North Seepage Collection Pond #1 and North Seepage Collection Pond #2 are as shown on Drawings C180-KA39-5000-00-016 and C180-KA39-5000-00-017. Both locations are downstream of the ultimate West Mine Rock Storage Facility (WMRSF) toe, and on current natural drainage paths for flow from the proposed TSF area.

Drawings C180-KA39-5000-00-016 and C180-KA39-5000-00-017 show the catchments estimated to drain to the ponds. During starter embankment construction, a coffer dam will be constructed upstream of the starter embankment footprint, and along the north drainage that leads to North Seepage Collection Pond #2. Construction runoff retained in this upstream basin will be pumped to North Seepage Collection Pond #1.

Rainfall runoff volumes and flow rates were estimated using the software package; "Sediment, Erosion, Discharge by Computer Aided Design", version 4. (SEDCAD4™). SEDCAD is a modeling program used for design of sedimentation ponds and open channel drainage networks. SEDCAD employs the United States Natural Resources Conservation Service (NRCS), curve number method to estimate runoff. Curve numbers were estimated from area soil maps and site investigation information completed to date. Catchment characteristics such as slope and flow path were estimated from site topographic data. A Type II NRCS storm distribution was selected.

5.2.2 Pond Sizing

5.2.2.1 Starter embankment construction phase

The ponds were sized using Method A detailed in the MELP (2015) guidelines. This method was selected as it is suitable when site-specific testing information (size distribution of influent TSS, sedimentation rates etc.) are not available. Generally, the method results in a conservative design (in terms of pond size) and is recommended where there are no environmental reasons to have the smallest effective pond.

Sediment Pond Area (A) is estimated from:

$A = (Q/V)$

Q being the pond overflow rate.

V being the settling velocity (for a particle of approximately 5 to 10 micron that MEM recommends should be designed for).

Table 1 presents the estimated peak 1 in 10 year flow rates for the catchments that drain into the into the ponds during the starter embankment construction.

Table 1
Peak Flow Rates - Starter Embankment Construction Phase

Pond	Contributing Catchment	Catchment Area (km^2)	10-Year, 24 Hour Peak Discharge (m^3/s)	200-Year, 24 Hour Peak Discharge (m^3/s)
Pond #1	A-1	0.90	2.3	5.6
	C-1	1.18	2.8	6.8
Pond #2	B-1	0.61	1.7	4.0

The pond sizing calculations were completed as follows:

- The flow estimates shown on Table 1, as well as flow hydrographs, were obtained from SEDCAD calculations.
- A reservoir routing of the 1 in 10 year, 24 hour event was completed using HEC-RAS software, based on the inflow 1 in 10 year hydrograph, the pond location's elevation-storage relationship, and an initial spillway size estimated to pass the 1 in 200 year, 24 hour peak flow. This gave estimates for the 1 in 10 year overflow rate.
- The required sediment pond area was estimated based on the MELP (2015) recommended settling velocity, 2 x 10^{-5} m/s. Based on Appendix C of the MELP (2015) guidance document, a correction factor of 2.0 was applied, to account for the limitations of Stokes equation.
- From the pond elevation-area relationship, the design area and pond elevation were selected. Pond length and width dimensions arising from the topography were inspected for general agreement with the recommended 5:1 ratio.

Table 2 summarizes the results of the sediment pond sizing calculations.

Table 2
Sediment Pond Sizing

Pond	Outflow rate (m^3/s)	Required Sediment Pond Area (m^2)	Selected Pond Elevation (m)	Available Area at Pond Elevation (m^2)
Pond #1	0.34	34,000	928	40,800
Pond #2	0.24	24,000	926.5	27,000

Based on the BGC (2015) water management plan, both North Seepage Collection Pond #1 and North Seepage Collection Pond #2 will be lined.

5.2.2.2 Operation and closure phase

Sizing of the pond under the MEM criteria is considered appropriate for the pre-production phase, when overflows from the sediment ponds will be discharged to the environment. Once tailings deposition starts, the sediment ponds will provide retention for seepage flows, and runoff from the outer embankment slope and waste rock buttress. Flow that collects in the sediment ponds will be pumped to the Central Pond, and then into the process circuit during operations (or to a treatment plant post-operations). The sizes established for the pre-production scenario will be retained for the production years and post-closure. The ponds will continue to provide sediment control facilitating pumped transfer of flows with reduced sediment load to the Central Pond.

Table 3 shows the 1 in 200 year volume estimates for the catchments that drain into the into the ponds at the end of production, and the volume at the ponds arising from sizing for sediment removal during start-up. The volume provided at start-up is many times larger than that required to retain the 200-year, 24-hour volume.

Table 3
Peak Flow Rates – End of Production

Pond	Contributing Catchments	Total Catchment Area (km^2)	200-Year, 24 Hour Volume (m^3)	Volume from Starter Phase Sizing (m^3)
Pond #1	A-2, C-2, D-2, I-2	2.07	40,500	119,000
Pond #2	B-2	0.50	7,500	130,000

Emergency spillways will be constructed at the ponds, to manage inflows in excess of the 200-year, 24 hour event during the production years and post-closure. The spillway designs will be completed during the next stages of TSF design.

5.3 Diversion Structures

The diversion channel and collector ditch network proposed for the TSF North Embankment was modelled using SEDCAD.

5.3.1 Contact Water Collector Ditches

Open channel collector ditches are proposed for the operation and closure phases, to intercept runoff and convey it to the sediment collection ponds. Drawing C180-KA39-5000-00-017 shows the proposed layout of the ditch network.

The TSF embankments and MRSF structure were divided into multiple catchment areas that discharge to the perimeter drainage system. Topography is such that at some locations, runoff would drain to the pond without need for ditches. However, at this stage, ditches are assumed to be required to manage overland flows in a controlled manner. Flow to the ditches was routed based on the proposed drainage network to the collection ponds. Peak discharges at the key points were then used for preliminary sizing of the ditch channels.

Table 4 presents the estimated peak discharges, from the SEDCAD calculations.

Table 4
Peak Flow Rates

Ditch	Catchment Areas (km2)	200-Year, 24 Hour Peak Discharge (m^3/s)
1a	1.04	3.30
1b	0.09	0.43
1c	0.18	0.87
2a	0.30	1.45
2b	0.20	0.96

The ditches were designed for the flow rates summarized above, using Manning's equation for open channel flow. At this preliminary design stage, an indicative size is estimated for each ditch based on the maximum peak flow, and the likely shallowest slope from topographic data. Design parameters for the ditches are provided in Table 5.

Table 5
Preliminary Sizing for Contact Ditches

Parameter	Ditch 1a	Ditch 1b	Ditch 1c	Ditch 2a	Ditch 2b
Bottom Width (m)	1	1	1	1	1
Side Slopes	3:1	3:1	3:1	3:1	3:1
Min. Depth of Ditch (m)	0.9	0.5	0.7	0.7	0.7
Longitudinal Slope (%)	3.4	27.0	2.0	3.0	2.0
Erosion Protection	Riprap[1]				

Notes: 1) Riprap size to be determined during detailed design phase

At this preliminary design stage, it is assumed that the ditches will be riprap lined, except where the ditch is cut in bedrock. Riprap requirements will be assessed through the next stages of design. It is noted that there are steep sections along the ditch alignments which may require additional lining and channel protection measures. The ditch alignments and sizes will be finalized at the detailed design stage.

Nominal size ditches are expected to be required at the South Embankment. Options for non-contact water diversions upstream of the South Embankment will be considered further during the next phases of TSF design. Design for runoff management at the South Embankment will then be finalized.

5.3.2 Temporary Diversion Channel

In order to reduce the amount of water reporting to the starter dyke construction area, an upstream temporary diversion channel is proposed for the construction period. The diversion channel will cut off clean water flow from sub-watersheds D-1, E-1, and F-1 as shown on C180-KA39-5000-00-016. Peak discharges at the end points of each sub-watershed were then used to design the three segments of the diversion channel. The peak flow calculations were completed using SEDCAD for the 10-year 24 hour rainfall taken from the site's climatological assessment (KP, 2015).

Table 6 presents the calculated peak discharges.

Table 6
Peak Flow Rates

Ditch	Catchment	Catchment Areas (km^2)	10-Year, 24 Hour Peak Discharge (m^3/s)
Temporary Diversion Segment 1	E-1	6.3	9.3
Temporary Diversion Segment 2	E-1 + F-1	9.2	11.3
Temporary Diversion Segment 3	E-1 + F-1 +D-1	10.8	13.5

The segments of the diversion channel were designed for the flow rates summarized above, using Manning's equation for open channel flow. Design parameters for the diversion channel are provided in Table 7.

Table 7
Preliminary Sizing for Temporary Diversion Ditch

Parameter	Segment 1	Segment 2	Segment 3
Bottom Width (m)	3.0	4.0	4.5
Side Slopes	3:1	3:1	3:1
Min. Depth of Ditch (m)	1.0	1.0	1.0
Longitudinal Slope (%)	1.0	1.0	1.0

At this preliminary design stage, it is assumed that the channel will not be lined. The channel alignment , sizes and erosion control measures will be finalized at the next design stages, taking into account confirmed construction footprints. The preliminary sizes for the channel segments assume a minimum 1% slope.

6 RECOMMENDED FURTHER WORK

This appendix provides details on the hydrology input to the design of the TSF and associated water management structures for the Ajax Project. The appendix forms part of the Draft Preliminary Design Report. The appendix will be updated with changes to the TSF design. Recommended further work on the surface water management structures includes:

- The IDF storage volume should be re-calibrated with more detailed information of runoff factors based on proportions of undisturbed natural catchment, TSF beach and water pond for the different stages of TSF construction.
- Review of the sedimentation pond capacity requirements as site specific data on particle characteristics, including settlement rates, is available.

- The design of inlets and emergency spillways that will be required at the sediment ponds during operation and closure.

- Detailed design of ancillary water management structures around the perimeter of the TSF should be completed and reviewed for integration with mine planning and process plant structures.

- Sizing and liner details (if required) of the collector ditches and diversion channel following finalization of ditch alignments.

- Preparation of a site construction water management plan should be prepared as part of the pre-construction planning activities.

7 REFERENCES

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Appendix B
Slope Stability Analyses

APPENDIX B SLOPE STABILITY ANALYSIS

1 INTRODUCTION

Stability analyses were carried out with the computer program Slope/W, which uses the limit equilibrium method of slices to calculate the FOS in two dimensions. FOS is specified using Spencer's Method to solve for force and moment equilibrium.

Cross-sections were developed through the highest portion along each of the TSF embankments for use in the two dimensional stability models. These cross-sections are presented in Figure B1.

Both starter dam and ultimate embankment configurations were analyzed. The starter dam only applies to the north embankment as the other embankments are not required until later production years. The starter dam was analyzed with a water elevation equivalent to the start-up water volume of approximately 2.4Mm^3. The ultimate dam heights were also analyzed for all of the TSF embankments with tailings at the maximum storage elevation.

2 DESIGN MATERIAL PROPERTIES

A summary of the material parameters used in this analysis are presented in Table B1. Material properties are preliminary estimates at this time. The foundation material parameters are based on field strength estimates from the 2014 site investigation program and engineering judgement for the foundation materials encountered.

Table B1
Material Parameters for Stability Analysis

Type	Material	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Cohesion (kPa)	Undrained Strength	Construction Pore Pressure
Fill	Compacted Mine Rock	21	40	0	-	-
	Mine Rock Buttress	21	37	0	-	-
	Upstream Till Blanket	20	30	5	-	R_u=0.3
	Tailings	20	30	0	-	-
	Tailings (Liquefied)	20	-	-	S_u/σ'_v = 0.1	-
Foundation	Undifferentiated Overburden	20	30	0	-	-
	Foundation Weak Layer	20	-	-	S_u/σ'_v = 0.3	-
	Bedrock	Impenetrable				

3 DESIGN BASIS AND ASSUMPTIONS

3.1 Dam Geometry

The embankment geometry used for the stability analyses is shown below:

- 3H:1V for downstream and starter dam embankment slopes
- 2.5H:1V for upstream embankment slopes
- Dam crest width is 39m
- Start-up pond water elevation: 946m (2.4 Mm^3)
- Starter Embankment crest elevation: 971m
- A mine rock buttress was included as part of the starter dam configuration. This buttress has a base width up to 400m, crest elevation 958.5m, which is approximately 2/3 the height of the starter embankment, and a downstream slope of 3:1.
- Ultimate embankment crest elevation: 1,056m
- Ultimate tailings elevation: 1,053m
- Ultimate supernatant pond elevation: 1,043m
- Mine Rock Storage Facilities (MRSF) were included as part of the ultimate embankment configuration as a mine rock buttress and a downstream slope of approximately 2.4:1.

3.2 Phreatic Surface

The phreatic surface used in the stability analysis model extends from the pond elevation to the upstream dam face within the tailings deposit and is drawn down to the original ground beneath the footprint of the embankment.

Construction pore water pressure was considered for the compacted upstream till blanket with an assumption of Ru=0.3. No construction pore water pressure changes were considered for the foundation materials as they were considered to be free-draining based on site investigation work completed to date. Undrained strength parameters were assigned to specified clay layers within the foundation profile. However, most of these high plastic clays (weak layers) identified to date are located at shallow depth (<5m) and it is planned to remove these materials as part of foundation preparation. For this reason, these weak layers were not evaluated in the stability analysis. This assumption may be revised as site specific geotechnical investigation information becomes available.

3.3 Seismic Parameters

The Kamloops region is characterized by a low level of historical seismicity. Seismic hazard values for the site are available from the Natural Resources Canada (NRC) website (NRC, 2010) for earthquakes up to the 1/2,475 return period. Firm ground peak horizontal ground accelerations and the associated return periods from NRC are summarized in Table B2.

Seismic hazard values for greater return periods are not provided by Natural Resources Canada. A peak ground acceleration of 0.34g for the 1/10,000 year return period was reported by Knight Piésold for preliminary design of the nearby tailings storage facility (Knight Piésold, 2014). The 1/10,000 year earthquake was also reported by Knight Piésold to be the Maximum Credible Earthquake (MCE) for the Ajax site.

For preliminary design purposes, Norwest have used the same seismic hazard values and MCE determination for consistency. 2/3 of PGA was used as horizontal seismic coefficient in the Pseudo static stability analysis (i.e. k_h=0.23).

Table B2
Ajax Seismic Hazard Values [(1)]

Earthquake Return Period (Years)	Annual Exceedance Probability (AEP)	Peak Ground Acceleration (g)
100	1%	0.034
475	0.21%	0.072
1,000	0.1%	0.097
2,475	0.0404%	0.138
10,000 [(2)]	0.01%	0.340 [(2)]

1. Preliminary values. To be confirmed upon site specific seismic hazard assessment.
2. Based on values used by Knight Piésold for the preliminary tailings storage facility design.

3.4 Factor of Safety Criteria

The TSF embankments will be designed to meet the minimum required FOS criteria as shown on Table B3. These factors of safety criteria are based on the revised 2007 Canadian Dam Association Dam Safety Guidelines (CDA, 2013).

Table B3
Slope Stability Design Criteria

Phase	Minimum Factor of Safety Criteria
End of Construction	1.3
Long Term	1.5
Seismic (Pseudostatic Conditions)	1.0

4 STABILITY ANALYSIS RESULTS

All four cross sections for each embankment exceed the minimum FOS requirement for both static and pseudo-static conditions (ie. end of construction and long term) for the upstream till blanket, upstream embankment/MRSF slope, downstream embankment slope (that would lead to a dam breach) and downstream buttress/MRSF slope. The results of the analyses are presented below. Table B4 shows the calculated FOS for the downstream slope of each dam section. Figures B.2 to B.9 present the stability output figures for the four TSF embankments.

Table B4
Stability Analysis Results

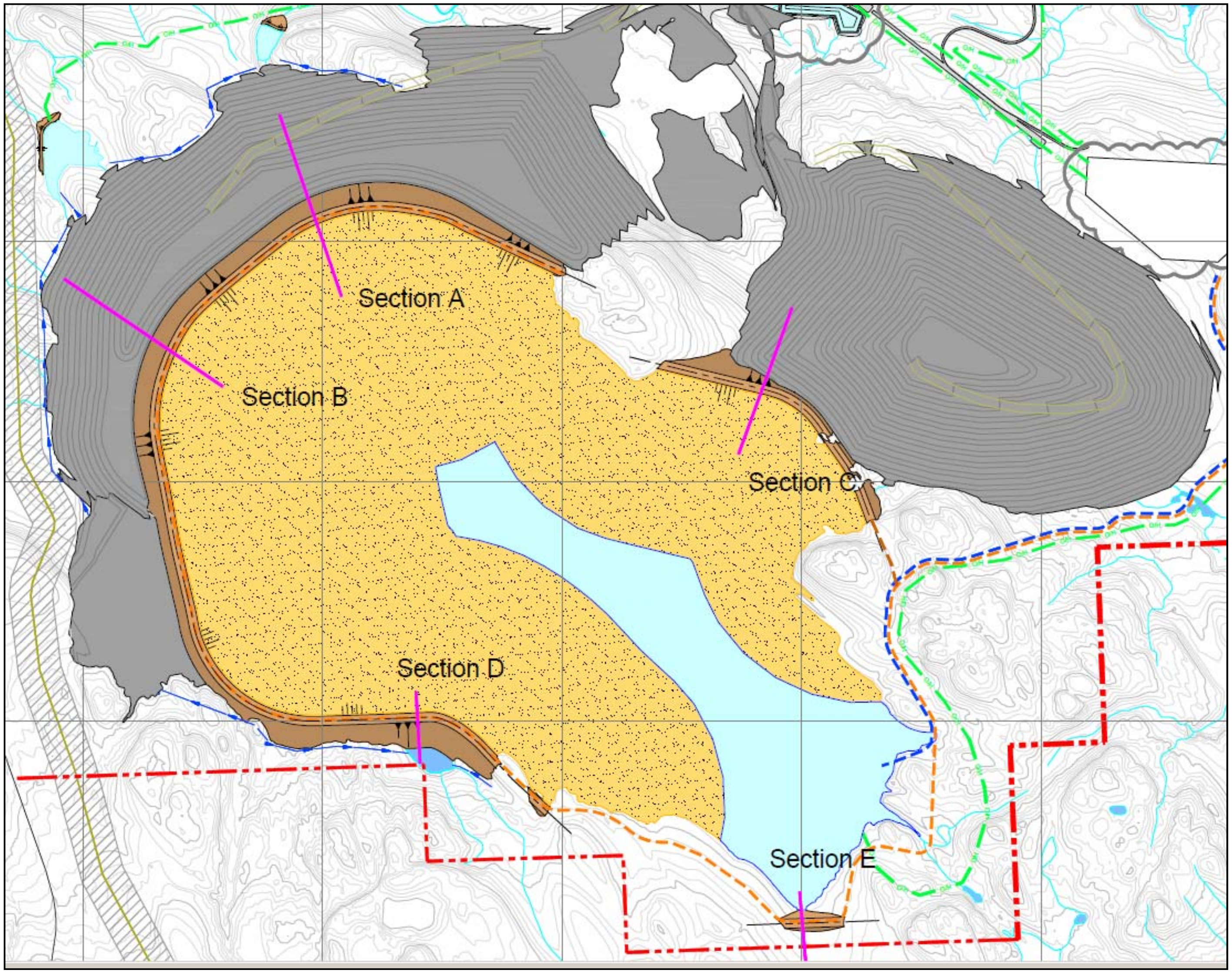
Phase	Embankment	Section	U/S Till Blanket	U/S Slope[1]		D/S Slope – Dam Breach	D/S Buttress[2]		Meets Design Criteria
			Static	Static	Pseudo-Static	Static	Static	Pseudo-Static	
Starter[3]	North	A	1.35	1.87	1.04	7.07	2.02	1.06	Yes
		B	1.36	1.87	1.04	4.24	1.99	1.06	Yes
Ultimate	North	A	-	1.87	1.08	3.10	1.76	1.01	Yes
		B	-	1.91	1.11	3.25	1.73	1.01	Yes
	South	D	-	-	-	2.1	-	-	Yes
	Southeast	E	-	-	-	2.39	-	-	Yes
	East[4]	C	-	1.77	1.02	3.63	1.59	1.02	Yes

__Notes__:

1. Stability analysis for the upstream slope was not completed for the south and southeast embankment because it was suitably buttressed by thickened tailings and it was not considered a critical failure mode.
2. The current design does not include a downstream buttress for the south and southeast embankment. For this reason, no stability analysis was completed for these areas
3. The starter embankment includes the north embankment only.
4. The downstream slope of the rock buttress for Section C was simplified as the rock buttress footprint is much larger than the embankment footprint.

5 CONCLUSIONS

The results of the stability analyses indicate the dam configuration satisfies the stability criteria under static and seismic loading conditions. In addition to meeting the minimum design criteria, stability analyses was also carried out to evaluate a failure surface that could lead to a breach into the impoundment; shown as Downstream (D/S) Slope – Dam Breach in Table B4. Due to the very large buttress that is built adjacent to the engineered fill embankment, very high safety factors are obtained for the global failure condition. This means that a breach due to global embankment failure is highly unlikely based on the assumed conditions.



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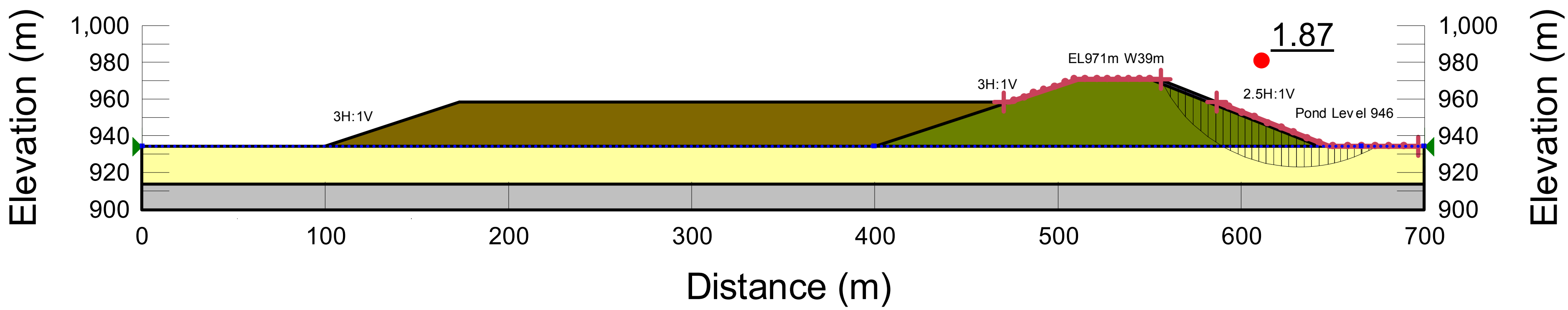
TAILING STORAGE FACILITY
CROSS SECTION LOCATIONS

NORWEST | 809-6 | FIGURE B.1 | REV. A

Material Type		Unit Weight (kN/m^3)	Strength Parameters
	Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
	Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
	Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
	Tailings	20	φ' = 30°, c' = 0 kPa
	Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
	Foundation Weak Layer	20	S_u/σ'_v=0.3
	Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	

Section A - Year -1



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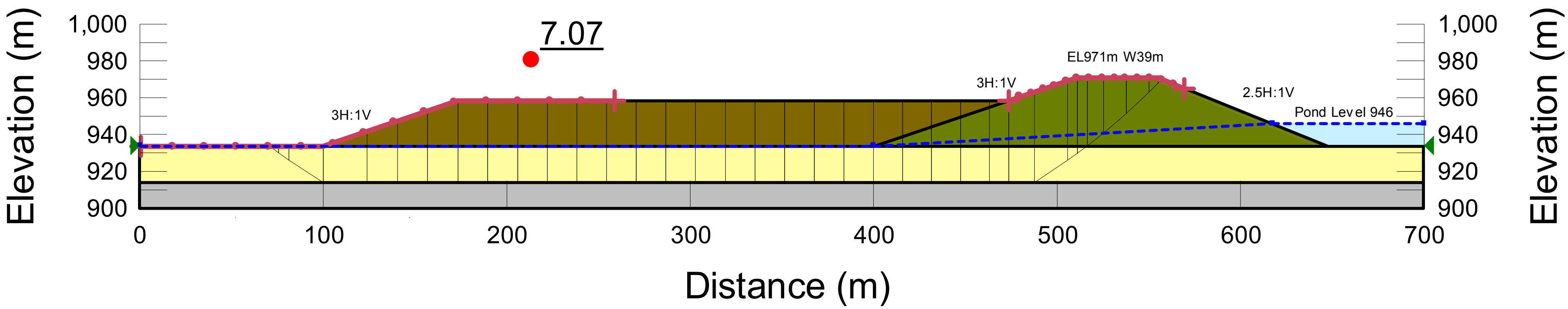
TAILINGS STORAGE FACILITY
STARTER NORTH EMBANKMENT SECTION A
END OF YEAR -1 - Upstream

NORWEST	809-6	FIGURE B.2	REV. A

	Material Type	Unit Weight (kN/m³)	Strength Parameters
	Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
	Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
	Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
	Tailings	20	φ' = 30°, c' = 0 kPa
	Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
	Foundation Weak Layer	20	S_u/σ'_v=0.3
	Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	

Section A - Year -1



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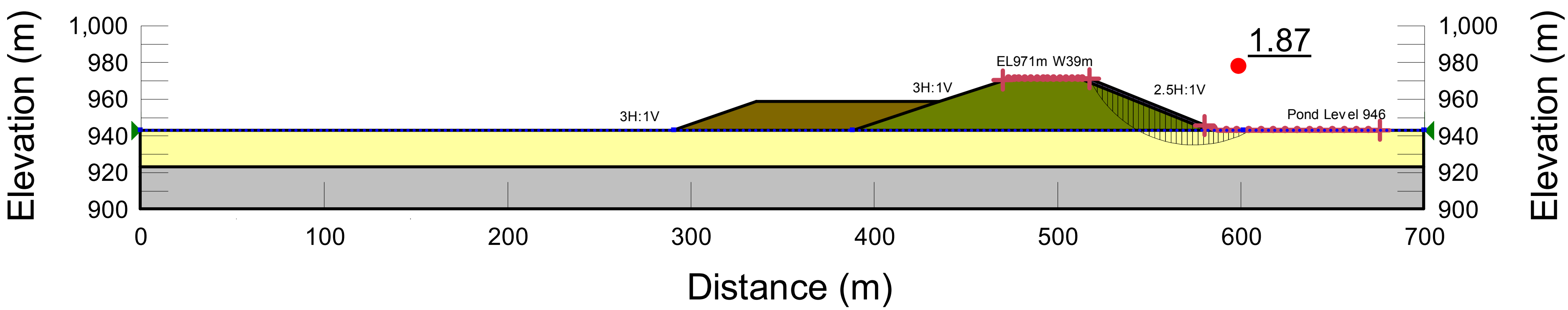
TAILINGS STORAGE FACILITY
STARTER NORTH EMBANKMENT SECTION A
END OF YEAR -1 - Downstream

NORWEST | 809-6 | FIGURE B.3 | REV. A

Material Type		Unit Weight (kN/m^3)	Strength Parameters
	Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
	Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
	Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
	Tailings	20	φ' = 30°, c' = 0 kPa
	Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
	Foundation Weak Layer	20	S_u/σ'_v=0.3
	Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	

Section B - Year -1



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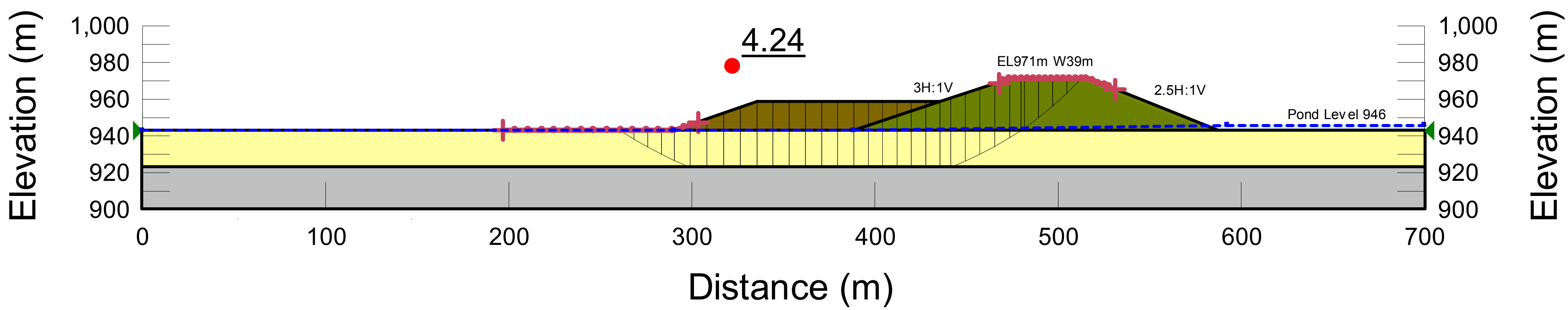
TAILINGS STORAGE FACILITY
STARTER NORTH EMBANKMENT SECTION B
END OF YEAR -1 - Upstream

NORWEST | 809-6 | FIGURE B.4 | REV. A

Material Type		Unit Weight (kN/m^3)	Strength Parameters
	Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
	Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
	Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
	Tailings	20	φ' = 30°, c' = 0 kPa
	Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
	Foundation Weak Layer	20	S_u/σ'_v=0.3
	Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	

Section B - Year -1



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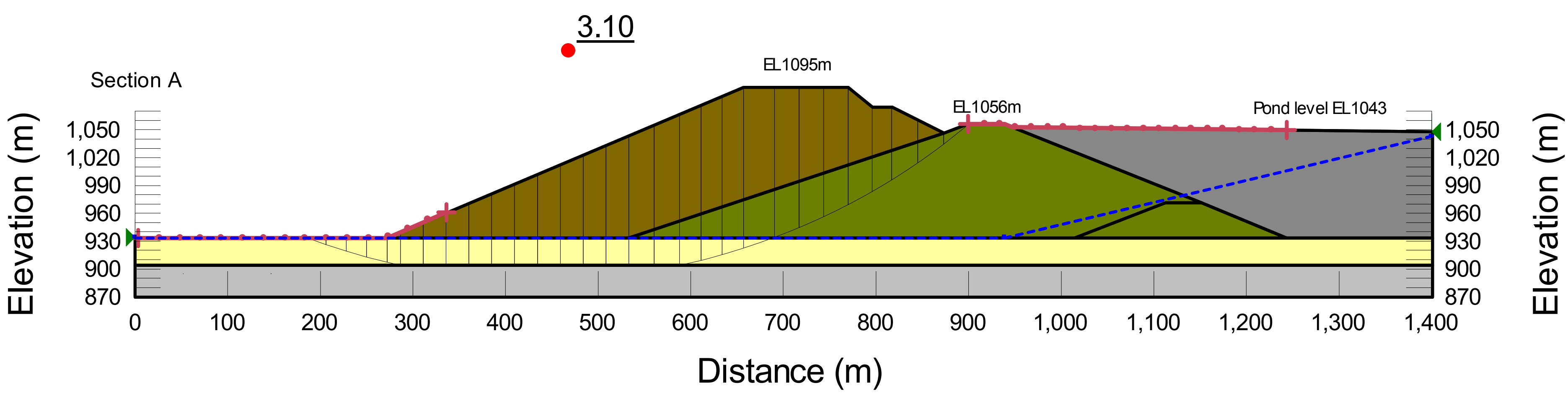
TAILINGS STORAGE FACILITY
STARTER NORTH EMBANKMENT SECTION B
END OF YEAR -1 - Downstream

NORWEST | 809-6 | FIGURE B.5 | REV. A

Material Type		Unit Weight (kN/m^3)	Strength Parameters
	Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
	Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
	Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
	Tailings	20	φ' = 30°, c' = 0 kPa
	Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
	Foundation Weak Layer	20	S_u/σ'_v=0.3
	Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	



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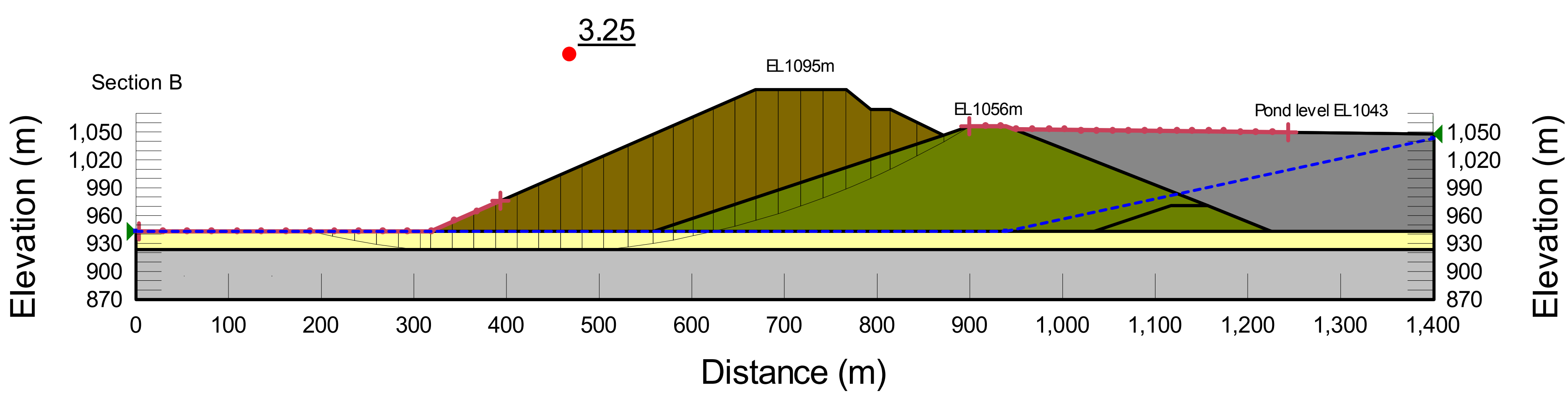
TAILINGS STORAGE FACILITY
ULTIMATE NORTH EMBANKMENT SECTION A
END OF YEAR 20

NORWEST	809-6	FIGURE B.6	REV. A

Material Type	Unit Weight (kN/m^3)	Strength Parameters
Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
Tailings	20	φ' = 30°, c' = 0 kPa
Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
Foundation Weak Layer	20	$S_u/\sigma'_v=0.3$
Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	



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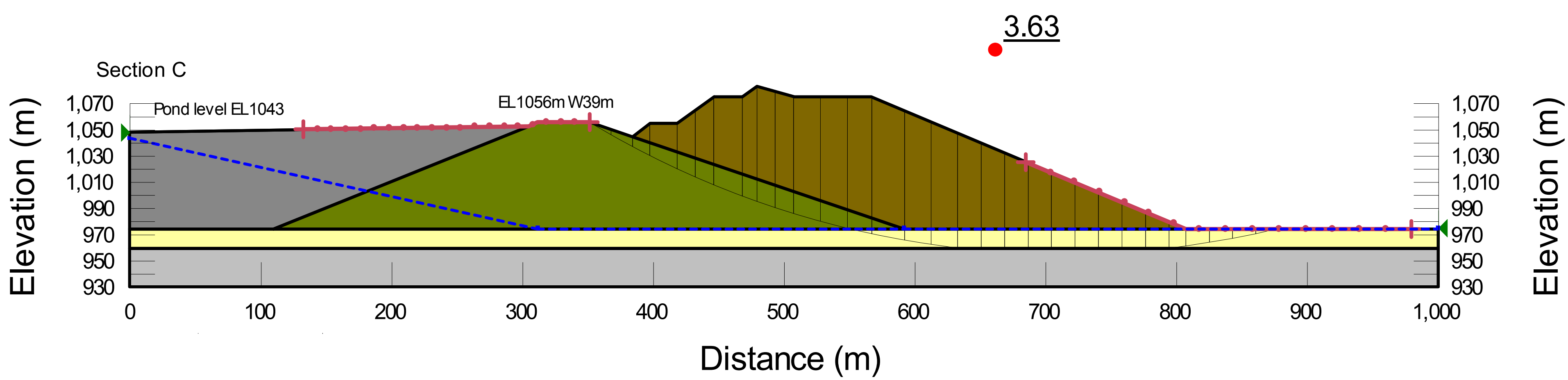
ULTIMATE NORTH EMBANKMENT SECTION B
END OF YEAR 20

NORWEST	809-6	FIGURE B.7	REV. A

Material Type	Unit Weight (kN/m^3)	Strength Parameters
Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
Tailings	20	φ' = 30°, c' = 0 kPa
Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
Foundation Weak Layer	20	S_u/σ'_v=0.3
Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	



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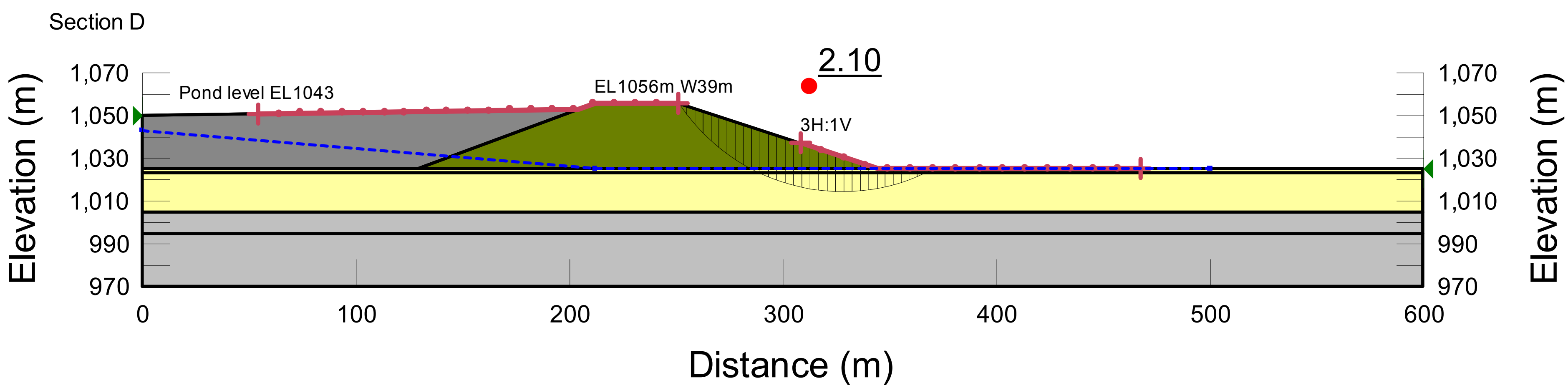
TAILINGS STORAGE FACILITY
ULTIMATE EAST EMBANKMENT SECTION C
END OF YEAR 20

NORWEST | 809-6 | FIGURE B.10 | REV. A

Material Type	Unit Weight (kN/m^3)	Strength Parameters
Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
Tailings	20	φ' = 30°, c' = 0 kPa
Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
Foundation Weak Layer	20	S_u/σ'_v=0.3
Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	



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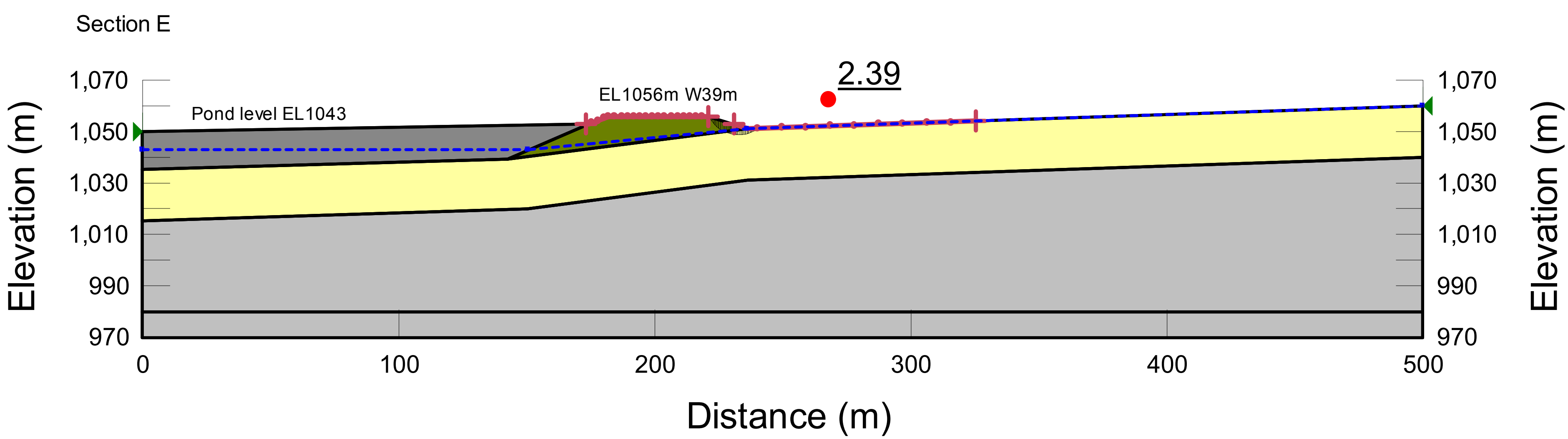
TAILINGS STORAGE FACILITY ULTIMATE SOUTH EMBANKMENT SECTION D END OF YEAR 20		
NORWEST	809-6	FIGURE B.8	REV. A

Material Type	Unit Weight (kN/m^3)	Strength Parameters
Compacted Mine Rock	21	φ' = 40°, c' = 0 kPa
Mine Rock Buttress	21	φ' = 37°, c' = 0 kPa
Upstream Till Blanket	20	φ' = 30°, c' = 5 kPa
Tailings	20	φ' = 30°, c' = 0 kPa
Undifferentiated Overburden	20	φ' = 30°, c' = 0 kPa
Foundation Weak Layer	20	S_u/σ'_v=0.3
Deep Competent Sand and Gravel Layer / Bedrock	Bedrock (Impenetrable)	



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TAILINGS STORAGE FACILITY
ULTIMATE SOUTHEAST EMBANKMENT SECTION E
END OF YEAR 20

NORWEST | 809-6 | FIGURE B.9 | REV. A