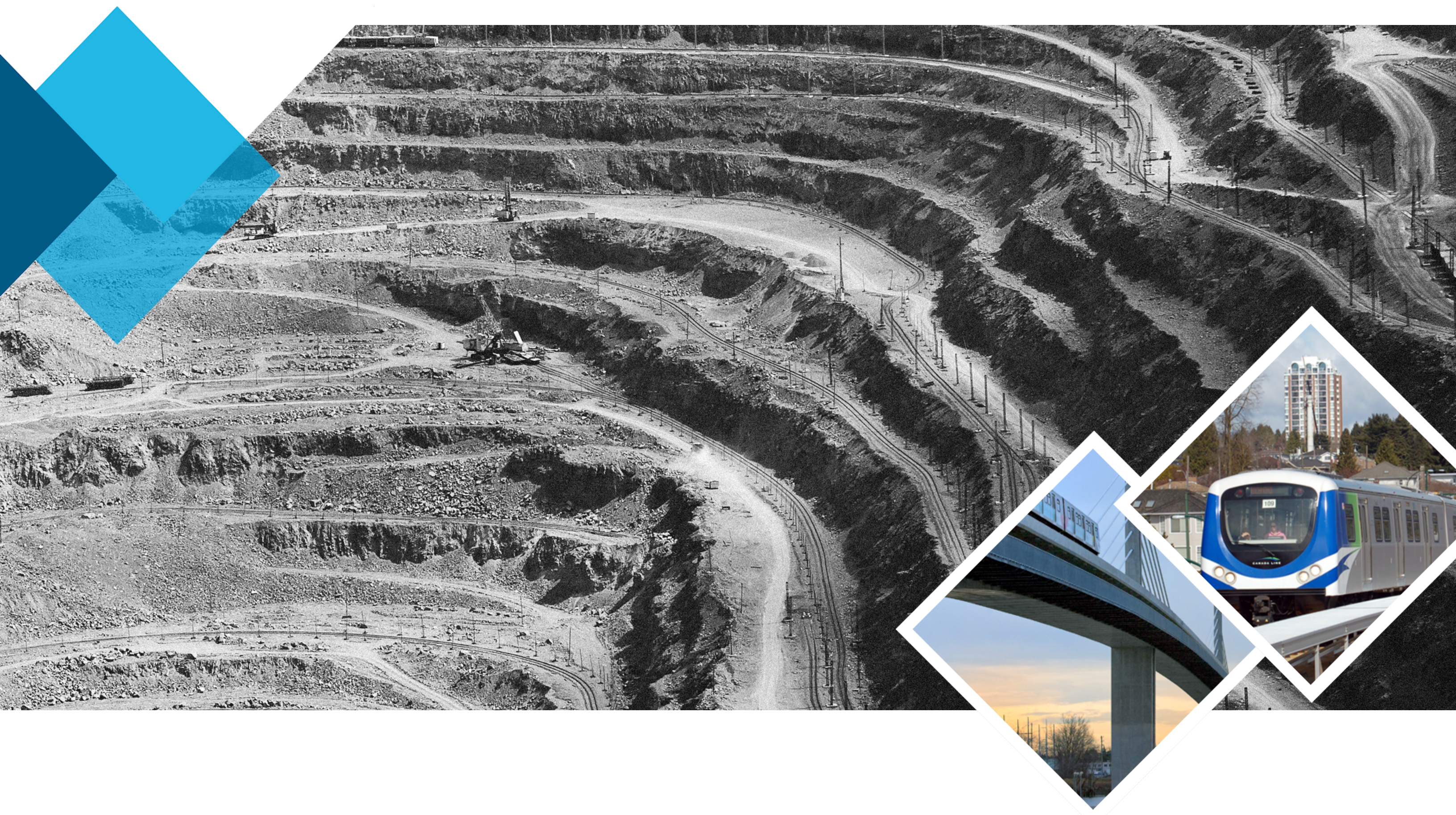
Detailed engineering of water management and geotechnical infrastructures at Amaruq

Design report of Whale Tail Dike

Agnico Eagle Mines Limited



Mining & Metallurgy

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Montreal, May 10th 2018

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Subject: Detailed engineering of water management and geotechnical infrastructures at Amaruq
Design report of Whale Tail Dike
Our file: 651298-2700-4GER-0001-01
AEM file: 6118-E-132-002-TCR-007

Mr. Bolduc,

Mr. Lavallée,

We are pleased to submit the final version of the report mentioned in the above subject.

Do not hesitate to communicate with the undersigned should you have any questions regarding the content of this report.

Truly yours,

SNC LAVALIN INC.

Yohan Jalbert, Eng.
Project Manager
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YJ/bsp

List of Revisions

Revision				Revised pages	Remarks
#	Prep.	App.	Date		
PA	AA/YJ		26/04/2018		
PB	AA/YJ	LM/GH	02/05/2018	All	Issued for client comments
00	AA/YJ	LM/GH	07/05/2018	All	Final version
01	AA	YJ	10/05/2018	Appendix A	See drawings

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Table of Contents

Page

List of Tables

List of Figures

List of Appendices

1.0 INTRODUCTION

1.1 Context

Agnico Eagle Mines Limited, Meadowbank Division ("AEM") is proposing to develop Whale Tail Pit, a satellite deposit on the Amaruq property, as a continuation of current mine operations and milling at the Meadowbank Mine. The Amaruq property is a 408 km² site located on Inuit Owned Land, approximately 150 km north of the Hamlet of Baker Lake and approximately 50 km northwest of the Meadowbank Mine in the Kivalliq region of Nunavut (Figure 1-1). The property was acquired by AEM in April 2013 and is subject to a mineral exploration agreement with Nunavut Tunngavik Incorporated.

The Meadowbank Mine is an approved mining operation and AEM is looking to extend the life of the mine by constructing and operating Whale Tail Pit, which is located on the Amaruq exploration property.

Figure 1-1 : Meadowbank and Amaruq site location

1.2 Project description

A scoping study of the Whale Tail Pit Project was initiated in January 2015 by AEM. SNC-Lavalin Inc. (SNC-Lavalin) was retained for the engineering of the geotechnical and water management infrastructure of the project (SNC-Lavalin, 2015a). The site layout was jointly developed by the SNC-Lavalin and AEM teams. A permitting engineering report followed the scoping study and was completed in early 2016 to develop the water management infrastructure (SNC-Lavalin, 2016a).

As part of this infrastructure, there is an important dewatering dike that is required to enable mining of the open pit located in the north part of Whale Tail Lake. This dike, named Whale Tail Dike (WTD), is located on a shallow plateau of the lake floor with an approximate 2 m depth of water. This plateau is located between deeper sections of the lake with water depths of about 12 m. Once in operation, the downstream side of the dike will be dewatered and the upstream side of the dike will allow a 3.5 m raise of the water level prior to being discharged by gravity towards Mammoth Lake via a new blasted channel located west of the property: the South Whale Tail channel. This channel will be built between existing A20, A45 and Mammoth lakes and will reroute 2,400 ha of watershed. The most recent layout of the mine site is presented in Figure 1-2.

It should be noted that the raising of water elevation will change the thermal regime of the flooded lands and could degrade the permafrost, especially for the Whale Tail Dike area.

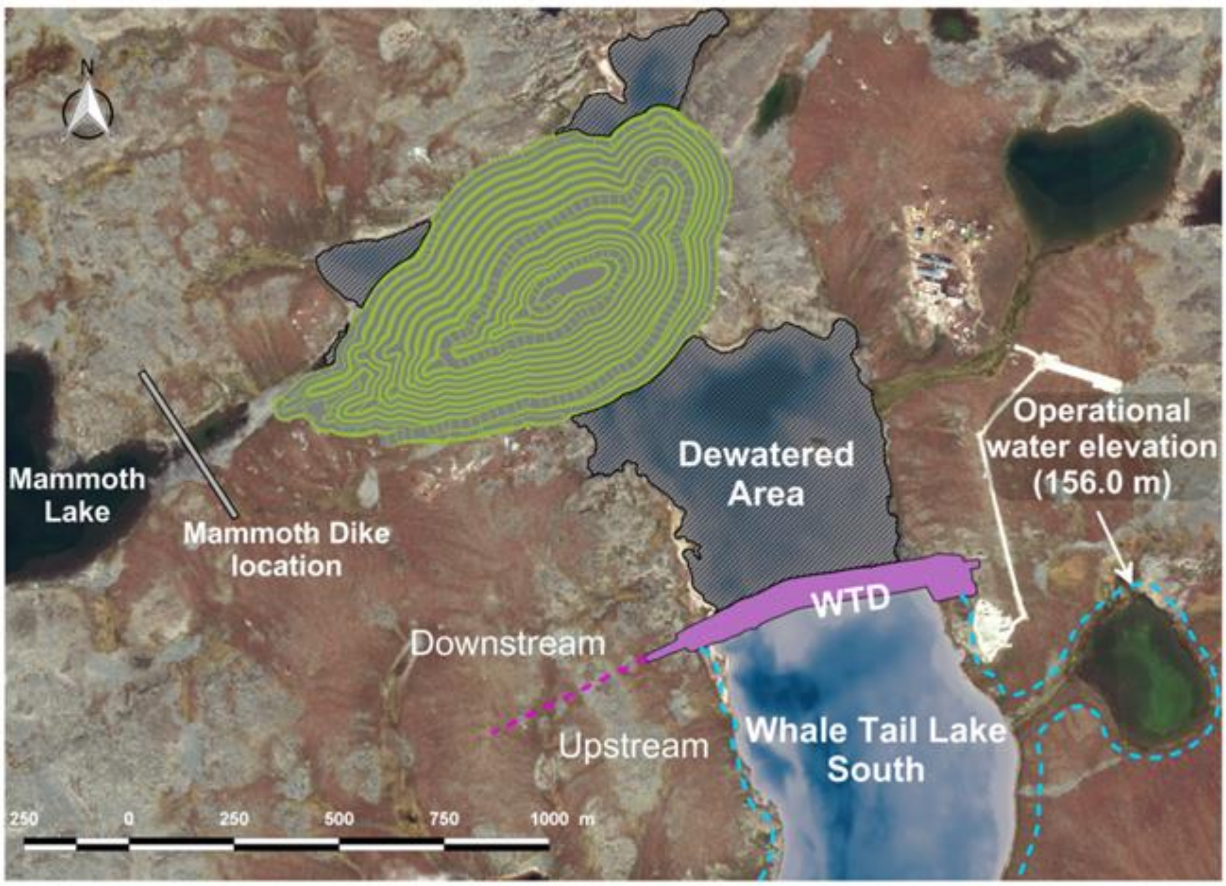


Figure 1-2 : Amaruq project mine layout

Since the completion of the permitting level study in early 2016 (SNC-Lavalin, 2016a), the available information on the evolution of WTD design was reviewed. The initial design of the dike was with a Cement-Soil-Bentonite (CSB) slurry cutoff wall. Since the soil present on site consists of till that is mainly frozen and of variable composition, the initial design was modified into a Cement-Bentonite (CB) cutoff wall. Despite the high cost to import cement, this technique was chosen due to its flexibility towards the uncertainties related to various stages of the construction.

In 2017, a contingency plan was presented for the cutoff technique of WTD for the case where the PFS design (cement bentonite cutoff trench) experiences construction delays which may extend the work into the cold season. This contingency plan involved the design of a more robust cutoff which can be adapted to an extended construction season (SNC-Lavalin, 2017c and 2017f). Finally, at the completion of these preliminary studies, AEM decided that Whale Tail Dike will be designed using a cutoff wall consisting of secant piles.

It is noted that Whale Tail Dike is designed following the extensive expertise developed by AEM and their dike superintendent during the construction of the previous dewatering dikes. This work was recorded in previous Meadowbank reports from Golder and Associates (Golder) and is also reflected in the current design of Whale Tail Dike. The main objective of this study is therefore to document the detailed engineering of Whale Tail Dike using secant piles as a cutoff wall.

2.0 REGULATION, CODES, GUIDELINES AND STANDARDS

The standards and codes presented in Table 2-1 are to be followed during the construction of WTD.

Table 2-1: Standards and codes

Activity/Tests	Norm/Code
Soils – Grain size analyses	ASTM D422 - 63(2007), Standard Test Method for Particle-Size Analysis of Soils.
Weight of bentonite mud and cement-bentonite slurries	ASTM D4380-84, Standard Test Method for Density of Bentonite Slurries.
Marsh cone viscosity of muds and cement bentonite slurries	ASTM D6910-04, Standard Test Method for Marsh Funnel Viscosity of Clay Construction Slurries.
Field vane shear test of cement bentonite mix	ASTM D2573, Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.
Bleeding test of cement bentonite mix	ASTM C940 – 16, Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory.
Pinhole test to determine erodability cement bentonite mix	ASTM D4647/D4647M-13, Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test.
Unconfined compressive strength test of cement bentonite mix	ASTM D2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.
Permeability test of cement bentonite mixes	ASTM D5084-16a, Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.
Concrete : constituents and execution of the works	CAN/CSA A23.1/A23.2-M, Concrete materials and methods of concrete construction.
General use (GU) Cement	CSA-A3001, Cementitious materials for use in concrete.
Grout mix	ASTM C150/C150M-11, Standard Specification for Portland Cement.
	ASTM C494, Standard Specification for Chemical Admixtures for Concrete.
	CSA-A23.1, Concrete Materials and Methods of Concrete Construction
	CAN/CSA-A3001, Cementitious Materials Compendium.
	CSA-A23.2-1B, Viscosity, Bleeding, Expansion and Compressive Strength of Flowable Grout.
Bedrock	ISRM (1981), Determination of some engineering properties of weak rocks.

3.0 AVAILABLE INFORMATION

Table 3-1 summarizes the studies performed throughout the previous phases of the project. These documents serve as reference for the detailed design of Whale Tail Dike. These documents are also listed in Section 18.0 - References.

Table 3-1 : Main reference documents

Document	Objective
Water management multiple account analysis. 627215-1000-40ER-0003 (SNC-Lavalin, 2015c)	An option was selected based on four (4) aspects: society, environment, economy and viability. The selected option consisted of isolating the pit area located in Whale Tail Lake with Whale Tail Dike and Mammoth Dike, and raising the water level of the Whale Tail Lake by 3.5 m to reroute water flow towards the Northwest passage through a channel.
Scoping study report of Amaruq project. 627215-0000-40ER-0001 (SNC-Lavalin, 2015a)	Identification of the surface infrastructure required for water management. Definition of the local terrestrial environment, hydrology and watersheds of the site in order to identify surface infrastructure required for water management.
Geotechnical investigation report. 627215-0000-40ER-0002 (AEM, 2015)	Review, analyze and assess all data from 2015 geotechnical investigations, including spring field investigation and borrow source and contours.
Field work 2016. 640387-1000-4GER-0001 (AEM, 2016)	Factual report gathering all the data from the 2016 field work related to the construction of the water management infrastructure. Quality and quantity of sand, gravel and till-like materials for future use is discussed, as well as bedrock mapping to optimize the cutoff alignment, assess the length of WTD and determine the dimensions of the cutoff wall. Three campaigns were completed: a drilling campaign with a tamrock drill rig, a diamond drilling campaign, and a borrow search with lab tests results.
Stability analysis of WTD. 640387-2000-4GER-0001 (SNC-Lavalin, 2017a)	Static and pseudo-static stability analyses to assess the stability of WTD at the end of construction and during operations of the Whale Tail Pit.
Thermal analysis at WTD. 640387-2000-4GER-0002	Preliminary thermal analyses to study the effect of the construction of WTD and the raising of the water level of the lake on the thermal regime of the

Document	Objective
(SNC-Lavalin, 2017d)	foundation over a 20 year period.
Sensitivity analysis of seepage through the foundation of WTD. 640387-4000-4WER-0001 (SNC-Lavalin, 2016b)	Seepage analyses to evaluate the range of possible groundwater flow seeping at the foundation of the WTD during the operational phase of Whale Tail Pit.
WTD Construction technique optimization. 640387-6000-40ER-0001 (SNC-Lavalin, 2017c)	Factual information of the existing conditions and strategies at a scoping engineering level of detail permitting the selection of the best and most adaptable technology for the construction schedule of WTD. Different construction techniques applicable to WTD are also presented, including secant piles with the cast in place method.
Permitting Level Engineering Report for Geotechnical and Water Management Infrastructure. 627215-1000-40ER-0004 (SNC-Lavalin, 2016a)	Design of all the components of the water management infrastructure for the Whale Tail Pit project including channels, ponds and retaining structures for the option selected during the second trade-off study. Hydrology study included with an update of the data used for flood routing, water balance and channel design computations. Water management study including watershed areas and water management strategy is also presented in this report.
2017 Geotechnical Investigation. 645003-2000-4GER-0001 (SNC-Lavalin, 2017a)	Information collected from diamond drilling campaign (soil collection, bedrock coring, water tests, geo-camera, and thermistors installation).
Update of the hydrogeological seepage analysis of WTD. 645003-1000-4WER-0001 (SNC-Lavalin, 2017b)	This study presents an update of the results presented in the previous study (640387-4000-4WER-0001) and an estimation of the possible range of groundwater flows that could be expected.
Whale Tail Dike Secant Pile Cutoff Wall Preliminary Design. 645003-3000-4GER-0003 (SNC-Lavalin, 2017f)	Technical note presenting the prefeasibility study of secant piles as an alternative cutoff wall for WTD.
Design criteria – Basins and pumps.	This document presents the criteria used for the design of the water management infrastructures such as dikes, basins, ditches and pumps.

Document	Objective
651298-8000-40EC-0001 (SNC-Lavalin, 2018c)	
Thermal analysis at Whale Tail Dike. 651298-2000-4GER-0001 (SNC-Lavalin, 2018a)	Update of the thermal analyses to study the effect of the construction of WTD and the raising of the water level of the lake on the thermal regime of the foundation over a 20 to 50 year period.

4.0 DESIGN BASIS, CRITERIA, PARAMETERS AND INVESTIGATIONS

4.1 Design basis and criteria

The design basis and criteria are presented in detail in document number 651298-8000-40EC-0001 (SNC-Lavalin, 2018c). The following Tables 4-1 and 4-2 summarize the main design criteria for WTD.

Table 4-1 : Dam classification identified for Whale Tail Dike

Risk Type	Dam Class	Comments	Reference
Population	**High**	Workers in the pit located downstream of the dike are assumed to be permanent population at risk of 10 or fewer workers in the area.	Dam Safety Guidelines (CDA, 2007, rev 2013)
Economy	**Low**	Limited infrastructure downstream of the dike.	Dam Safety Guidelines (CDA, 2007, rev 2013)
Environment	**Significant**	Marginal loss of habitat in case of flooding and sediment transportation: the area was already flooded before construction of the infrastructure.	Dam Safety Guidelines (CDA, 2007, rev 2013)
Summary: **HIGH**			

Table 4-2 : Design criteria for WTD

Use	Water type	Classification (CDA, 2014)	Inflow Design Flood	Water Level (m)		Crest Elevation (m)
				Normal	Design Flood	
Dewatering dike	Non-contact	High	1/3 between 1000-year and PMF[1]	156.0	157.0	159.0
Note 1: PMF means Probable Maximum Flood						

Whale Tail Dike is classified as “high” based on the classification proposed by the Canadian Dam Association (CDA, 2014) in terms of consequence of failure. Based on this dam classification, the return period of the design earthquake is 1:2,500 years. Whale Tail Dike is also considered as a “Large Dam” since the structure will impound more than 3Mm³ of water based on the classification of the International Commission on Large Dams (ICOLD). The design flood water level elevation of 157m in Table 4-2 is from flood routing analyses given in the permitting level report (SNC-Lavalin, 2016a).

Surface infrastructure design is based on the following available information:

- Bathymetric survey (Pt_WhaleLake_UTM14Nad83_20150902.txt and Pt_MammothLake_UTM14Nad83_20150904.txt from Nutshimit-Nippour, Groupe Conseil);
- Topographic survey (amaruq_contours.zip from Photosat, September 14, 2015);
- Exploration database for overburden depth (DDH_20161110-2013 revC.xlsx from AEM in 2016);
- OMS Manual for dewatering dikes at Meadowbank, AEM version 4, January 2015;
- The WTD will be breached at the end of the exploitation of the Whale Tail Pit. The lifetime of the dike is estimated at 20 years.

The minimum factors of safety (FOS) used for the stability analyses (Section 7.2.2) are presented in Table 4-3. The stability criteria were derived from the Canadian Dam Association (CDA, 2013 and 2014) and the selected seismic parameters were obtained from the Geological Survey of Canada (GSC, 2015).

Table 4-3 : Minimum applicable factors of safety

Loading Conditions		Minimum FOS
Static	End of construction	1.3
	Operation with steady state seepage	1.5
	Rapid drawdown	1.3
Seismic	Pseudo-static	1.0

4.2 Geotechnical parameters

The following table provides the geotechnical parameters used in the geotechnical analyses.

Table 4-4 : Geotechnical parameters of the foundation and construction materials.

Material	Bulk density (KN/m³)	Cohesion (kPa)	Friction angle (°)	Hydraulic conductivity (m/s)
Fine filter	22.2	0	32	$1x10^{-05}$
Rockfill	22.2	0	45	$1x10^{-03}$
Till and lakebed sediment	21.0	0	32	$1x10^{-06}$ m/s
Weathered bedrock	Impenetrable			$1x10^{-04}$ -$1x10^{-05}$
Bedrock	Impenetrable			$6x10^{-08}$ -$1x10^{-09}$
Permafrost	Impenetrable			$1x10^{-11}$
Notes: Refer to SNC-Lavalin, 2017a and 2017b for details. Coarse filter was considered to have the same parameters as rockfill.				

4.3 Geotechnical investigations

Several geotechnical investigations were conducted since 2015. The following investigation reports contain all the information collected from diamond and Tamrock drilling campaigns (soil collection, bedrock coring, water tests, geo-camera, and thermistors installation):

- Geotechnical investigation report, 627215-0000-40ER-0002 (AEM, 2015).
- Field work 2016, 640387-1000-4GER-0001 (AEM, 2016).
- 2017 Geotechnical Investigation. 645003-2000-4GER-0001 (SNC-Lavalin, 2017e).

It should be noted that in March 2018 an additional geotechnical campaign was performed. The information collected during this campaign will be presented in a separate document.

5.0 HYDROLOGY

5.1 Data

Available hydrometeorological and hydrometric data are:

- Hydrometeorological data from Environment Canada Baker Lake A station, located approximately 124 km southeast of the Amaruq mine site, and covering the period from 1946 to 2017. These data include total precipitation, rainfall, snowfall and snow on ground.
- Water level and streamflow measurements carried out at different locations on watersheds nearby the project site during June to September of years 2015 and 2016 (Golder, 2017). Flow measurements were made in streams covered with boulders, which affected the precision of the measured values.

5.1.1 Hydrometeorological Data

Data from Baker Lake A meteorological station is available for the period 1946-October 2017. However, years 1946-1949, 1973, 1993, and 2015 have several missing data and consequently were removed from the analysis. Missing data from other years (1950-1972, 1974-1992, and 1994-2014) were filled using the average value from available years for the same day and month. The resulting rainfall, snowfall, and total precipitation daily data series cover a total of 66 full years over the period 1950-2017 (1973 and 1993 data not used). The average annual precipitation, over the period 1950-2017, is 250 mm including, 145 mm of rainfall and 105 mm (water equivalent) of snow. This data is assumed representative of the conditions for the Amaruq mine site area.

5.1.2 Hydrometric Data

The Amaruq mine site is located on a watershed with an extensive network of lakes and interconnecting streams, with a lake to land ratio of approximately 20 %. During two summers, in 2015 and 2016, several discharge and water level measurements were made by a third party (Golder, 2017) both manually and with automated hydrometric stations. One of the automated stations was located at Whale Tail Lake outlet, and recorded data is presented on Figure 5-1.

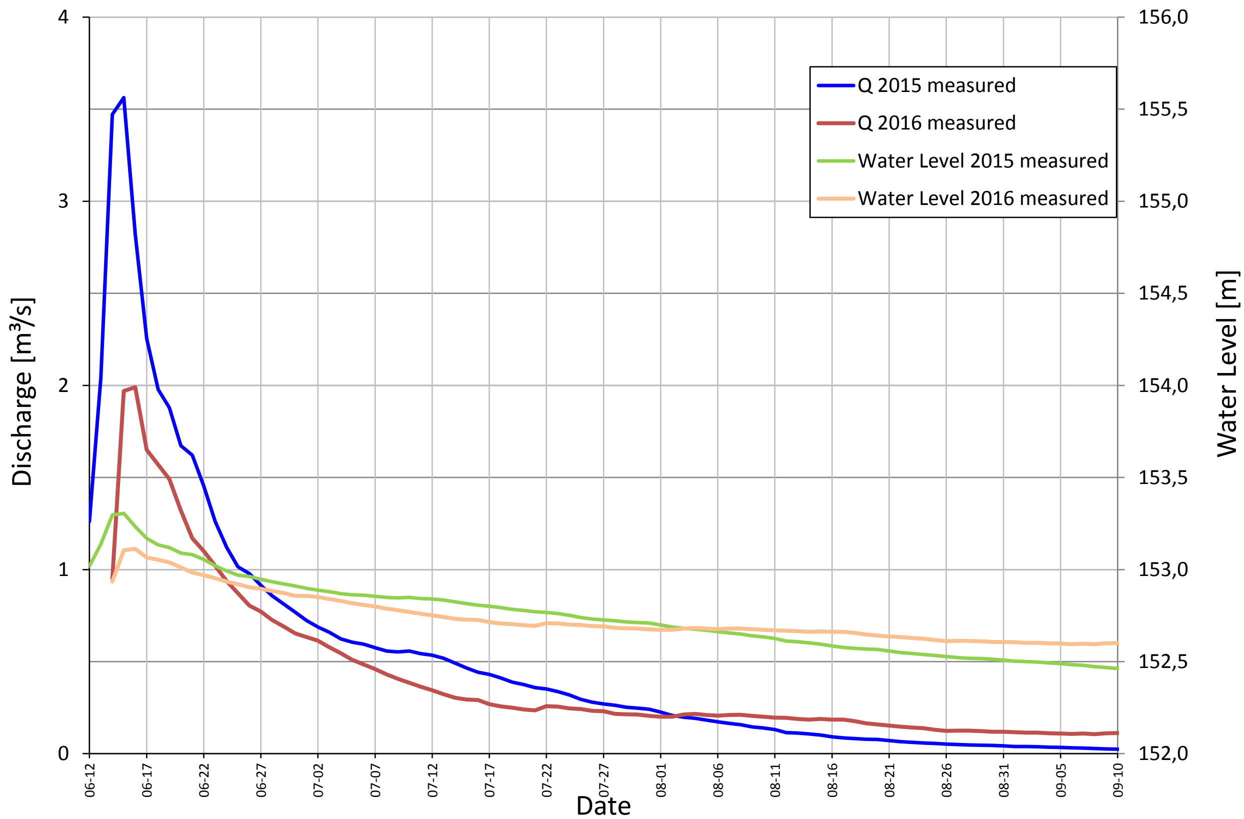
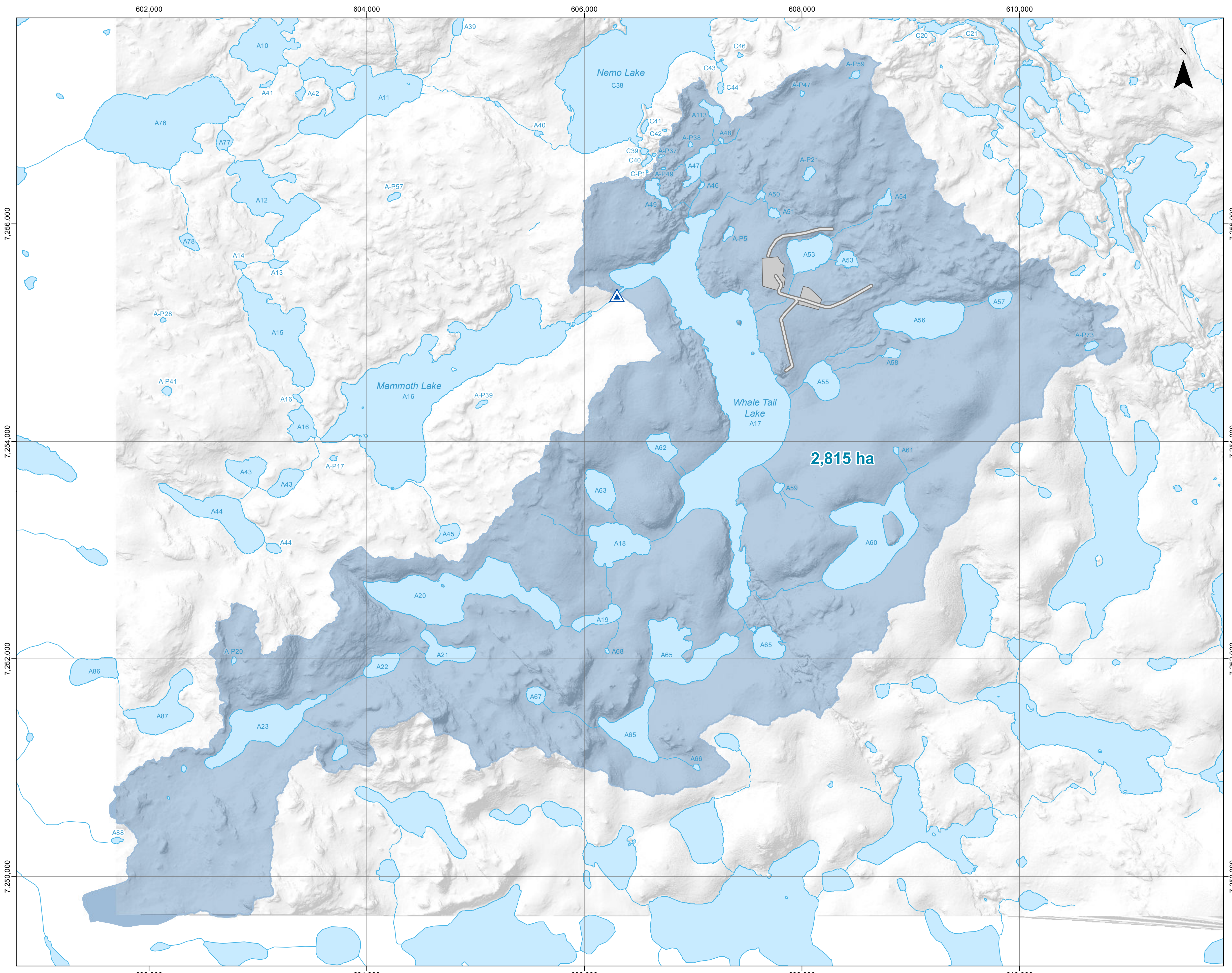


Figure 5-1: Whale Tail Lake measured outflow and water level (Golder, 2017)

5.1.3 Whale Tail Lake watershed

Whale Tail Lake watershed before construction has a drainage area of 2815 ha, as illustrated on Figure 5-2. After WTD and South Whale Tail Diversion Channel construction, an area of 2429 ha will be drained from Whale Tail South Lake to Mammoth Lake as illustrated on Figure 5-3. The watershed area on Figure 5-3 includes an area of about 60 ha comprising the watershed area of the South Whale Tail Diversion Channel which is not part of the initial Whale Tail Lake watershed.



PROJECT COMPONENTS

Existing industrial area

Existing road

HYDROGRAPHY

Watercourse

Lake

Hydrometric station

Whale Tail Lake natural watershed (2,815 ha)

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AMARUQ GOLD MINING PROJECT

Detailed Engineering of Water Management and Geotechnical Infrastructure

Whale Tail Lake Watershed
Before the Construction of Whale Tail Dike

Sources:
Topography, PhotoSat, 2015
CanVec, NRCan, 2016
Project components : March 2018

Project: 651298
File: snc651298_001_f1_WTLWshdNat_180321.mxd

0 350 700 m
1:35,000
UTM projection, Zone 14, NAD83 (CSRS)

March 21, 2018

Figure 5-2

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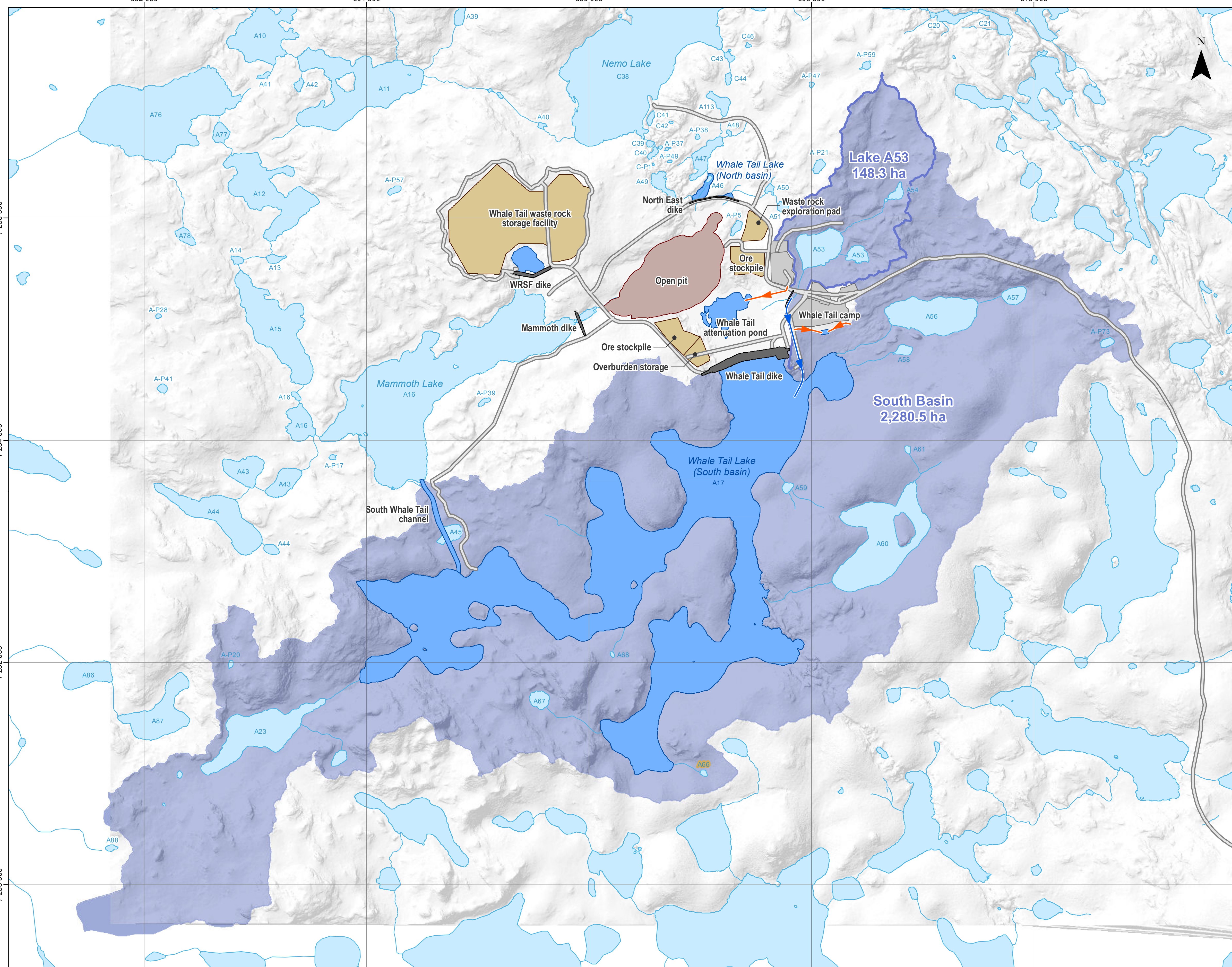

PROJECT COMPONENTS (PHASE I)

- Dike or cofferdam
- Modified waterbody
- Collection ditch
- Diversion ditch
- Open pit
- Storage facility
- Industrial area
- Road

HYDROGRAPHY

- Watercourse
- Lake
- Whale Tail Lake modified watershed (2,428.8 ha)

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AMARUQ GOLD MINING PROJECT
Detailed Engineering of Water Management and Geotechnical Infrastructure

Whale Tail Lake Watershed
After the Construction of Whale Tail Dike

Sources:
Topography, PhotoSat, 2015
CanVec, NRCan, 2016
Project components : March 2018

Project: 651298
File: snc651298_001_f2_WTLWshdMod_180327.mxd

0 350 700 m
1:35 000
UTM projection, Zone 14, NAD83 (CSRS)

March 27, 2018 **Figure 5-3**

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6.0 INITIAL CONDITIONS OF THE SITE

6.1 Water quality monitoring and management plan

In the Water Quality Monitoring and Management Plan for Dike Construction and Dewatering (AEM, 2017b), AEM proposes an action plan for Total Suspended Solids (TSS) management during the construction of WTD. This plan includes the possible winter construction of a causeway, the pumping and treatment of water impacted by the construction activities, and the installation of two rows of turbidity barrier at the downstream and upstream sides of the dike.

The strategy of this plan will mitigate the quantity of TSS released into the environment and consequently limits the impact of the construction of WTD on the aquatic fauna.

6.2 Condition of the foundation

Based on the geotechnical investigations (carried out from 2015 to 2018), the subsurface profile generally consists of sand and gravel with cobbles and boulders and/or glacial till overburden overlying weathered bedrock.

Based on the water content data from samples collected about 500 m inland east of the WTD and from the latest field investigation in the east abutment, the active layer in the east abutment area is approximately 1.5 to 2 metres thick, under which lies a layer of ice-rich soil that reaches a depth of 4.0 metres (SNC-Lavalin, 2018b). It is SNC-Lavalin's understanding that the top of the active layer has low to intermediate ice content, but is underlain by a frozen layer that contains potential segregated ice lenses, under which is ice-poor till. Gradation curves show that the ice-rich till material is silty sand with some gravel and clay. The ice-rich till can be classified as SM material according to the United Soil Classification System (USCS). The ice-poor till material is sandy gravel with some silt, and can be referred to a GW-GM material.

The bedrock encountered in the boreholes varies from greywacke (sedimentary) to diorite (intrusive). Both lithologies are deformed with an oblique foliation structure varying from weak to very strong in intensity. The most dominant structures are the foliations and then the veinlets. The Rock Quality Designation (RQD) values along the dike foundation vary significantly with a typical average in the range of 25 to 70%. The rock can be characterized as close to very close jointed with poor to fair quality based on the joint spacing and RQDs according to ISRM (1981).

Televiewer surveys during the 2017 geotechnical investigation revealed 7 fractured zones and 115 open and 565 slightly open joints. The open joints had apertures ranging from 0.55 mm to 54 mm with typical average aperture of about 6.0 mm. Some of the joints classified as "open joints" by the televiewer were actually filled with soil type material or gouge since the coring procedure washed material out of the joints.

Water Pressure Test (Packer Test) results show that the hydraulic conductivity of the thawed bedrock foundation varied considerably with a geometric average of 2.9x10-03 cm/s. The overall hydraulic conductivity of the bedrock in unfrozen condition is classified as high to very high, which is consistent with the bedrock geological features from the televiewer survey.

6.3 Thermal regime of the foundation

SNC-Lavalin (2017d) showed that an open talik of a maximum depth of approximately 100 m underlies the future location of WTD. The presence of unfrozen ground under this part of Whale Tail Lake is due to a deep channel flowing from south to north in the western portion of the lake.

Recent temperature data from thermistor strings installed on the eastern part of the lake (along the WTD alignment) show that frozen ground exists in that area. It is expected that colder temperatures were monitored because of the absence of deep sections on either side of the longitudinal profile of WTD, and because of the presence of a small island near the east shore acting as a “natural thermosyphon”.

Both abutments will be initially frozen before the construction of WTD. However, the west abutment will be warmer due to its proximity to the talik. Colder temperatures were recorded for the east abutment because of its proximity to the frozen zone below the east sector of WTD.

Historical information of thermal profiles collected in the permanent and temporary thermistor strings are presented in Appendix G.

7.0 WHALE TAIL DIKE DESIGN

7.1 General

WTD is a temporary dike that will be used to dewater a portion of Wale Tail Lake. Once in operation, the downstream side of the dike will be dewatered and the upstream side of the dike will allow a 3.5 m raise of the water level at which time discharge will occur in a new blasted channel located southwest of WTD: the South Whale Tail Diversion Channel (SWTDC). Raising of water elevation will change the thermal regime of the flooded lands and will degrade the permafrost, especially at the abutments, which was accounted in the dike design.

As the other dewatering dikes constructed at Meadowbank, Whale Tail Dike will be constructed as a zoned rockfill dike with a core composed of a fine filter dynamically compacted. A cement-bentonite (CB) cutoff wall consisting of secant piles will be constructed through this dense core.

7.2 Geotechnical analyses

SNC-Lavalin completed stability and seepage analysis during the prefeasibility study, and thermal, thaw settlement and stress analyses during the detailed design. The following subsections present a brief summary of the results of these analyses. Refer to the Appendices C to F for some of the technical notes of these analyses.

7.2.1 Thermal analysis

In March 2017, SNC-Lavalin carried out a thermal analysis at WTD with the objective of assessing the effect of the construction of the dike on the thermal regime in the foundation (SNC-Lavalin, 2017d). The longitudinal profile of WTD was analyzed using a thermal modeling software. The model was calibrated using very limited factual data, but allowed an estimate to be made of the profile of the talik under Whale Tail Lake.

During the spring of 2017, a field investigation was carried out at the Amaruq site where several temporary thermistors were installed in the dike area (AEM, 2017a). This provided additional temperature data at the dike abutments, which was used to finalize the design of the dike and to optimize its construction strategy.

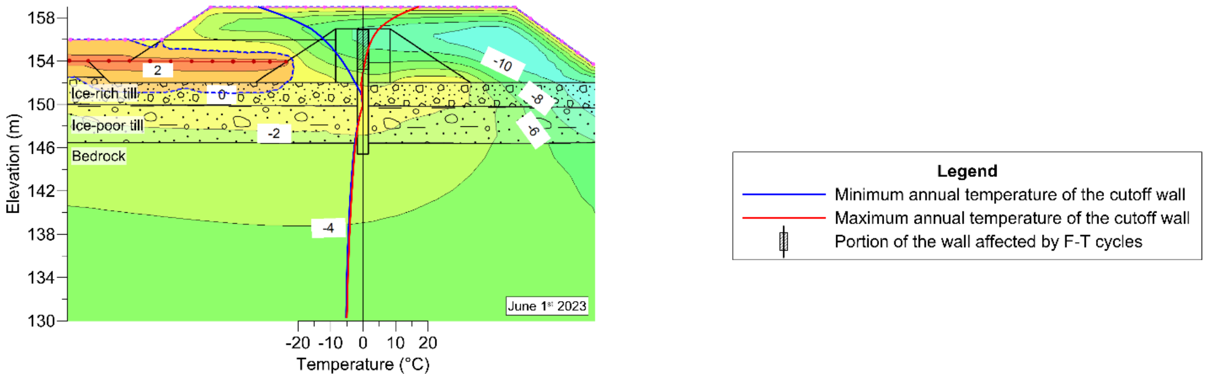
An updated report on the thermal modeling of Whale Tail Dike has been issued as part of the detailed engineering phase of the infrastructure (SNC-Lavalin, 2018a). A thermal model was built using the finite element software TEMP/W and calibrated using three thermistor strings at different locations. Three cross sections perpendicular to the longitudinal profile of the dike were analyzed for a period of 50 years: one located directly in Whale Tail Lake, and one at each abutment.

The results show that frozen soils upstream of both abutments will eventually thaw after the raise of the water level upstream of the dike. Thawing will be faster at the west abutment, where the esker upstream of the dike is expected to completely thaw after 3 to 4 years of operation. The results show that at the east abutment where a 2 m thick layer of ice-rich till is still present in the foundation, the ice-rich till layer should take approximately 30 years to thaw under the upstream slope of the dike but the core area will remain frozen. The thaw of that layer of soil is expected to cause significant settlement over time due to the high ice content of this section and there is potential for failure of the upstream slope of the dike. The thermal regime in the core area of the dike at the east abutment after 5, 20 and 50 years is shown in Figure 7-1.

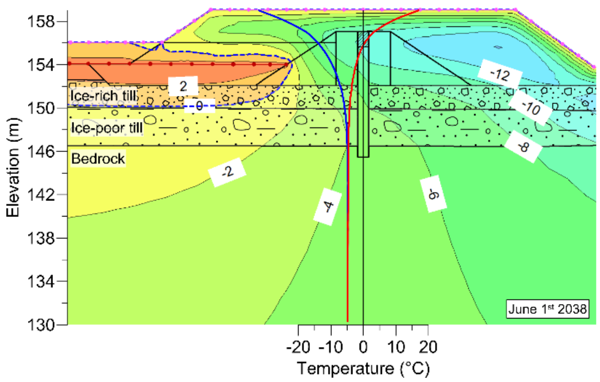
In order to prevent excessive settlement and potential instability below the upstream slope of the dike at the east abutment, the construction of a “thermal berm” upstream of the dike is recommended. This thermal berm would reduce the heat intake from the lake into the upstream and core area of the dike. The idea is to keep the water column further from the dike by using borrow material (esker-like soil) in the core of the thermal berm. The desired effect is to keep the underlying overburden in a frozen state. This would prevent thaw settlement of the ice-rich till under the upstream slope of the dike and in the area of the cutoff wall. The proposed geometry of the thermal berm and the results of the thermal model after 50 years of operation are presented in Figure 7-2. As shown most of the area below the thermal berm remains frozen thus protecting the core and the area upstream of the core from excessive settlement. Although Figure 7-2 applies after 50 years of operation, it is considered (based on preliminary modelling for earlier years of operation) that similar protection would be applicable after installation of the thermal berm.

The dimensions and geometry of the thermal berm presented in Figure 7-2 includes a factor of safety to account for the intrinsic simplifications of using a thermal model. The construction of a thermal berm should reduce the risk of significant settlement in the upstream part of the dike. Settlements near the upstream end of the thermal berm are unavoidable because of the thawing of the inferred ice-rich till layer close to the lake. However, such settlements occurring far from the cutoff wall are less critical and do not require an immediate intervention, but should be reflected in the maintenance plan.

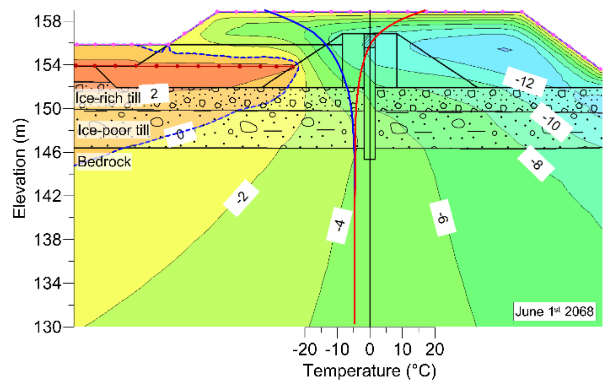
Results also show that permafrost will aggrade rapidly in the dewatered area after the construction of WTD. In the area outside of the attenuation pond, the frost front is expected to reach a depth of 50 m after 11 years of operation. Results show that the dike will become the transition between unfrozen soils (talik) upstream and frozen soils (permafrost) downstream, where most of the core of the dike should stay in the frozen state during the lifetime of the structure. The top metre of the cutoff wall would be exposed to freeze and thaw cycling throughout its operational life.



(A) Thermal regime around WTD after 5 years of operation



(B) Thermal regime around WTD after 20 years of operation



(C) Thermal regime around WTD after 50 years of operation

Figure 7-1: Modeling results: Evolution of temperature distribution around WTD at the east abutment

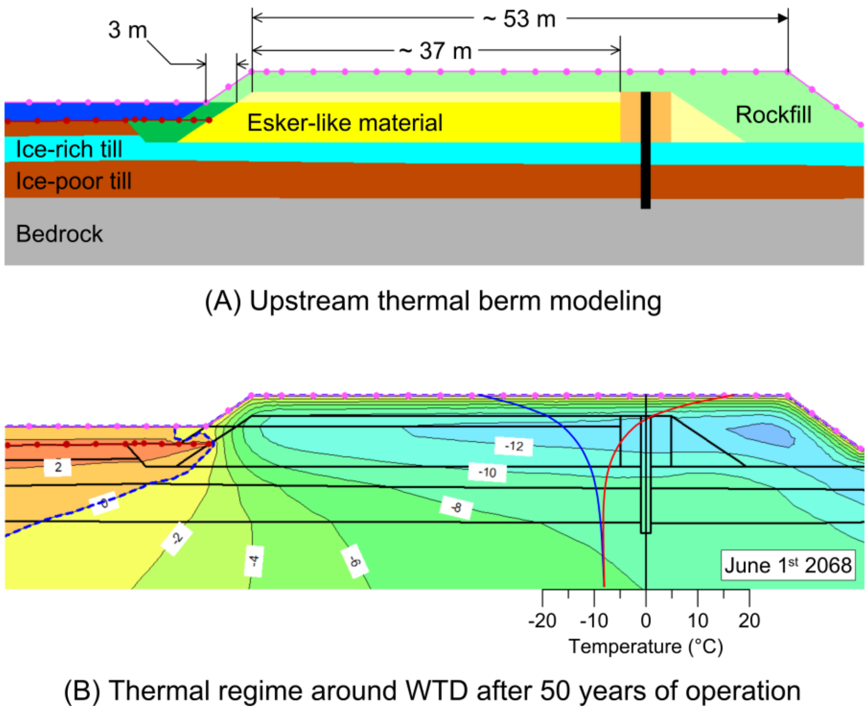


Figure 7-2: Geometry and modeling of the upstream thermal berm at the east abutment (A) and spatial temperature distribution around WTD after 50 years of operation (B)

7.2.2 Stability analysis

Stability analyses for static and pseudo-static loading conditions were performed to assess the stability of WTD at the end of construction (i.e. before the north part of Whale Tail Lake is dewatered) and during operations at Whale Tail Pit (SNC-Lavalin, 2017a). Results show that the design criteria are met for all scenarios of static and pseudo-static loading conditions.

As mentioned, the stability criteria were derived from guidelines in CDA (2013 and 2014) and selected seismic parameters were obtained from the Geological Survey of Canada (GSC, 2015).

7.2.3 Seepage analysis

Numerical simulations were carried out on two 2D cross-section models of the future WTD (SNC-Lavalin, 2017b) to calculate total groundwater fluxes under WTD during operation.

The Local WTD model was used to evaluate total seepage under WTD. Results show that the groundwater flux increases with the hydraulic conductivity of the weathered bedrock and decreases with increasing grout curtain depth. Without a grout curtain, the total seepage under WTD is between 1 300 and 2 400 m^3/day while for a 15 m deep grout curtain the total seepage is between 900 and 2 000 m^3/day. In general, the difference between having a 15 m grout curtain versus no grout curtain is about 400 m^3/day.

This flow will be reporting to the attenuation pond and will have to be treated prior to being discharged to the environment.

7.2.4 Thaw settlement analysis

Following the construction of WTD, the upstream operational water level is expected to rise by 3.5 m. Because the topography surrounding the shore of the Whale Tail Lake is gently sloping, a large portion of the west and east abutment of the dike will be flooded. Thermal analyses (Section 7.2.1) show that the heat coming from the water of the lake will thaw part of the upstream area below the dike at both abutments during the operational life of the mine (see Figure 7-1).

The west abutment of the dike will be built through an esker, where the material is poorly graded sand. This material is considered thaw stable and will not cause significant settlements following the construction of the dike. Therefore no remedial measures are required to mitigate thaw settlement at the west abutment. However, the overburden at the east abutment is much thicker and is composed of an ice-rich till layer underlain by an ice-poor till stratum. Water content measurements carried out near the industrial site showed a maximum gravimetric water content close to 150 %, which corresponds to a saturated volumetric ice content of over 80 %. Thawing of the ice-rich till layer could lead to excessive settlements under the rockfill embankment.

Thaw settlement analyses were carried out at the east abutment. Three different methods were compared in order to compute the thaw settlement following the penetration of the thaw front in the ice-rich overburden. Results show that thaw settlement will occur gradually over time following the thawing of the ice-rich till layer. The upstream slope of the dike is expected to settle by approximately 1.0 m after 30 years of operation, which corresponds to complete thawing of the ice-rich till layer. Subsequent thawing of the underlying ice-poor till layer should not induce significant settlements.

Further discussion on the effects of thaw settlement are given in Section 7.9. A complete technical note of the thaw settlement analysis can be found in Appendix C.

7.2.5 Stress analysis

The secant pile cutoff wall is a key component of the WTD for providing seepage control. Construction of the WTD and the operation will change both the thermal and stress conditions of the foundations resulting in deformations in the embankment that would induce stresses in the cutoff wall. The cutoff wall needs to be designed to sustain these stresses caused by the deformation of the embankment. Therefore, stress analyses were carried out for the WTD to evaluate the stress-strain response of the CB cutoff wall.

The analyses were carried out based on available geotechnical information, one-dimensional thermal analyses results, as well as assumptions where applicable. This section presents a summary of the stress-deformation analyses. The stress-deformation analyses are presented in more detail in Appendix D.

Two-dimensional finite element stress-deformation analyses were carried out to assess the stresses and deformations in the CB cutoff wall of the WTD using SIGMA/W (version 8.16), a computer program developed by Geo-Slope International Inc. Two cross-sections, one at the east abutment (east abutment section) and the other one in the middle of the WTD alignment (centre section) where talik is present were selected for the stress analyses. Figures showing these cross-sections (in plan and cross-section) are presented in Appendix D. A brief description of each cross-section as well as the analyses results is shown in Table 7-1.

It should be noted that three different CB mixes were developed in the SNC-Lavalin laboratory (refer to section 7.10.3). The stress analyses were performed for only two CB mixes:

- Mix 1: Highest unconfined compressive strength (644 kPa).
- Mix 3: Lowest unconfined compressive strength (124 kPa).

Only the factor of safety (FOS) for Mix 1 is presented in Table 7-1 since the analyses for the cutoff wall with Mix 3 indicated that the predicted maximum compressive stress is higher than the unconfined compressive strength of Mix 3.

The analyses indicate that differential settlement due to complete thawing of the ice-rich till below the upstream shell of WTD at the east abutment induces significant tensile stresses in the CB cutoff wall which may impact the integrity of the cutoff wall. Hence, it is recommended to provide a thermal berm at the east abutment to prevent thawing of the ice rich till zone.

Table 7-1: Scenarios and results of the stress analyses

Section	Scenario	Results
East abutment section: 2 m thick ice-rich till left in place (least favorable foundation condition since thawing of the ice-rich till may result in differential settlement of the dike).	Thermal cover to protect the frozen ground from thawing placed near the end of winter. The surcharge pressure due to the weight of the dike will result in settlement causing stresses on the CB cutoff wall.	The tensile stresses and the deflection in the cutoff wall are insignificant. FOS for compressive stress for Mix 1 = 2.59.
	No thermal cover. It is conservatively assumed that the ice-rich till and the ice-poor till underneath it will completely thaw during operation resulting in differential settlements that generate stresses on the CB cutoff wall[1].	The cutoff wall would have deflection ranging from 60 mm to 80 mm for the selected mix. FOS for compressive stress for Mix 1 = 1.79 to 1.83.
Central section: No change on the unfrozen foundation condition upstream of WTD. The talik downstream will freeze following the dewatering.	The surcharge pressure due to the weight of the dike will result in settlement of the talik. These settlements are expected to cause drag stresses on the CB cutoff wall.	The tensile stresses and deflection in the cutoff wall are insignificant. FOS for compressive stress for Mix 1 = 1.40.
Note 1: Contrary to the thermal analyses presented in Section 7.2.1, where the thawing of the ice rich till is partial, this scenario assumes a complete thawing of the ice-rich till and ice-poor till, which is a more conservative assumption.		

7.2.6 Wind and waves analysis

The wind and waves analysis for Whale Tail dike is presented in detail in Appendix E. From this analysis, it was determined that:

- The normal freeboard, corresponding to a 1000 year return period wind with a normal operation water level, is approximately 2.0 m and the required minimum dike crest elevation is 158.00 m.
- The minimum freeboard, corresponding to a 100 year return period wind when the water level is at a maximum during the design flood, is approximately 1.8 m and the required minimum dike crest elevation is 158.80 m.

The top of the secant pile wall is established to be equal to the maximum water level expected for the design flood. The dike crest is set 2 m higher than the secant pile wall to take account of the expected waves. Small infiltrations that could result from wave action below the secant pile wall are not significant. Therefore, with a design dike crest elevation of 159.0 m, Whale Tail dike is adequately designed and protected from wind and wave action (wind setup and wave runup).

The recommended riprap to protect the dike slope against a 100 year return period wind when the water is at its maximum level during the design flood, has a D_{50} of 450 mm and a minimum layer thickness of 830 mm. The rockfill proposed for the upstream shell of the dike has a D_{50} of 200 to 600 mm which is considered adequate as riprap protection, particularly since the coarser sizes will tend to preferentially accumulate on the outer slope of the upstream shell.

7.3 Source of material

The WTD will be composed of several materials (source and type): most of these materials will be produced or processed on site. According to the CDA Guidelines (CDA, 2013), the gradation of these materials should be selected to have adequate hydraulic stability against internal erosion and to maintain their hydraulic functionality.

During the PFS, it was determined that the granular material for the fine filter will come from eskers. Therefore, during the 2016 field work (refer to report 640387-1000-4GER-0001), surface samples were taken from Esker #8, located at the west abutment of WTD. Since surface samples might not be representative of the true conditions of the material, additional sampling was proposed at Esker #8 and #9. In October 2017, sampling and field observation was conducted to collect these additional samples. Esker #8 was not accessible and therefore no samples were taken. Laboratory test results of the grain size distribution of the surface samples of Esker #8 and samples of Esker #9 are presented in Figure 7-3. It should be noted that the grain size distribution of both eskers is very similar and it could be concluded that the samples of Esker #9 are representative to the samples that can be found (at depth) in Esker #8.

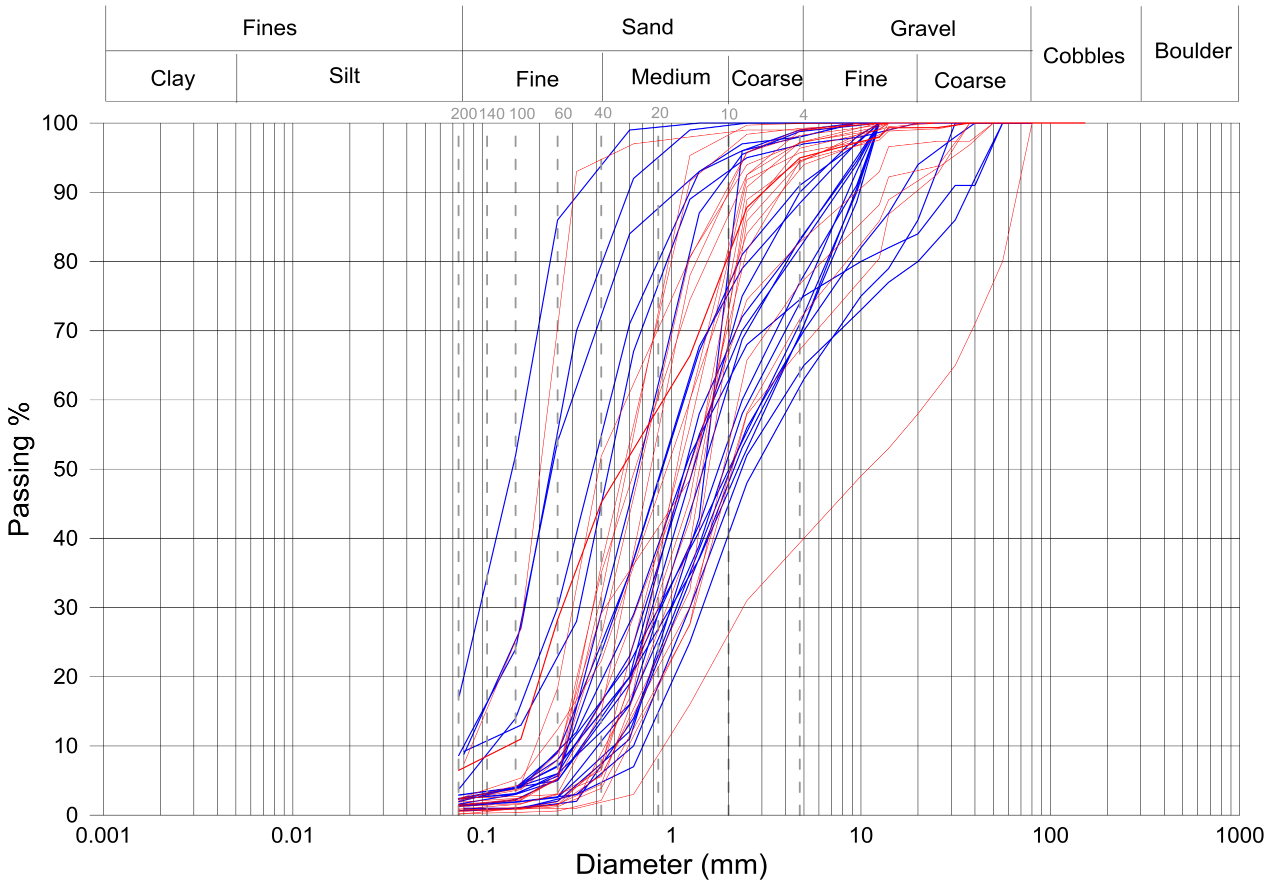


Figure 7-3 : Grain size distribution of Esker #8 (blue) and #9 (red).

Due to limited access to some of these eskers and the insufficient volume found for the construction of WTD, it was decided to produce the fine filter by crushing rock. Grain size distributions of the crushed aggregate proposed for fine filter are presented in Section 7.4.

The rockfill material will come from run of quarry and should be an appropriate material for dike construction, thus consisting of durable and non-acid generating rock.

A transition layer, identified as a coarse filter, will be placed between the rockfill and the fine filter. Grain size distributions for material proposed for coarse filter are presented in Section 7.4. Also Section 7.4 provides an assessment of filter grading suitability of the various fill zones of the dike.

7.4 Material gradation

As mentioned previously, the WTD design consists of a zoned rockfill dike constructed on the lakebed foundation with a cutoff wall acting as a seepage barrier. The cutoff wall is constructed through a central zone of granular material referred to as the fine filter zone. A coarse filter zone is provided between the fine filter and the end dumped upstream and downstream rockfill zones.

Grain size distributions of crushed aggregates produced at Meadowbank for fine and coarse filters at WTD are presented in Figure 7-4. The same production procedures are proposed for all of the fine and coarse filter materials for the WTD.

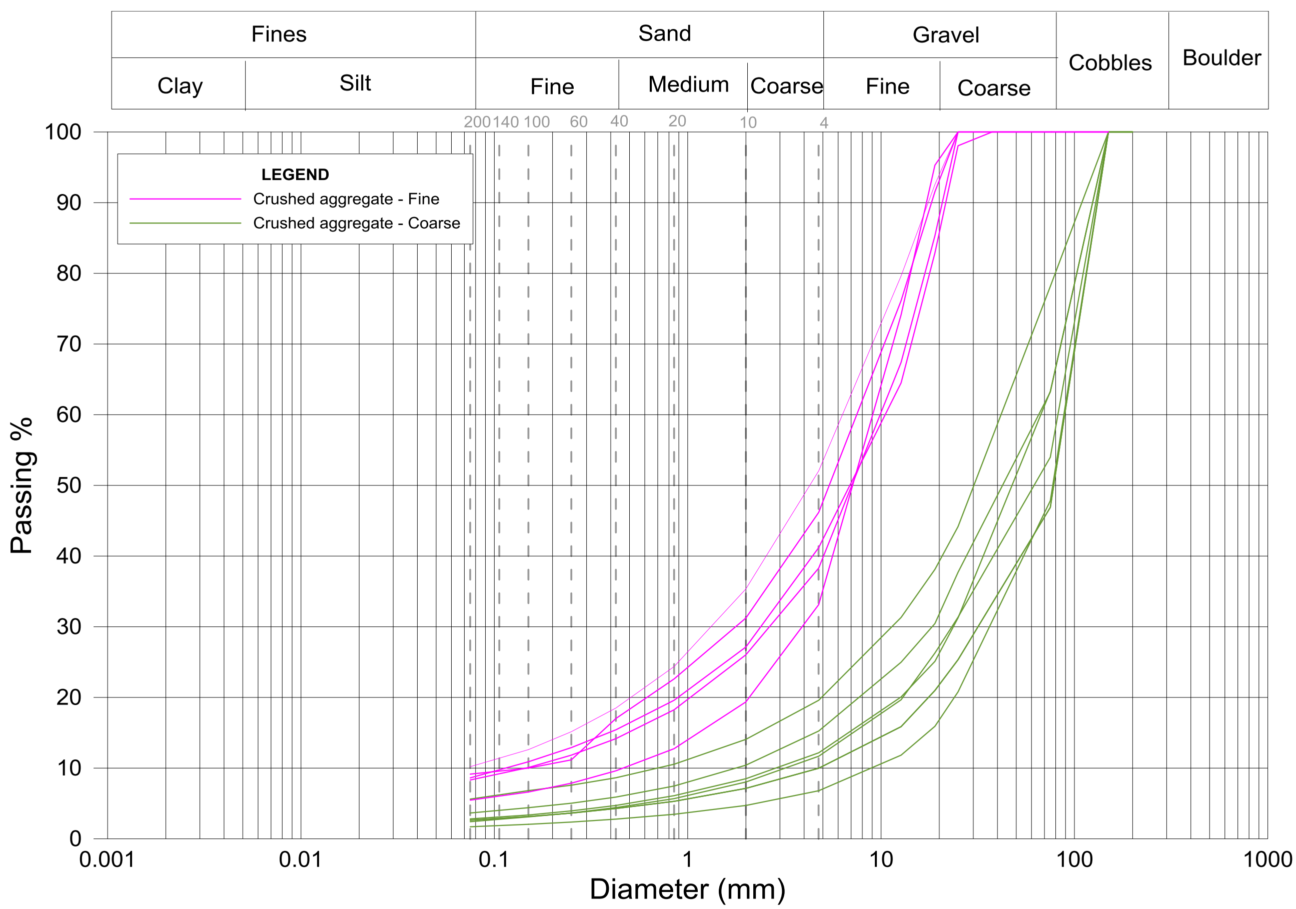


Figure 7-4 : Grain size distribution of crushed aggregates.

Based on the grain size analyses shown on Figure 7-4, gradation limits for fine and coarse filter were selected as shown on Figure 7-5. Also shown on Figure 7-5 are the gradation limits proposed for the rockfill. These gradations were evaluated with reference to requirements recommended by the US Bureau of Reclamation (USBR, 2014) and the CDA granular filter design criteria were used to verify the maximum allowable D_{15} for particle retention requirements. The proposed gradation limits were found to satisfy filter design criteria with a margin of safety against particle movement somewhat greater than required by filter design criteria. The proposed gradation limits for the WTD fill materials are shown in Figure 7-5 and Table 7-2.

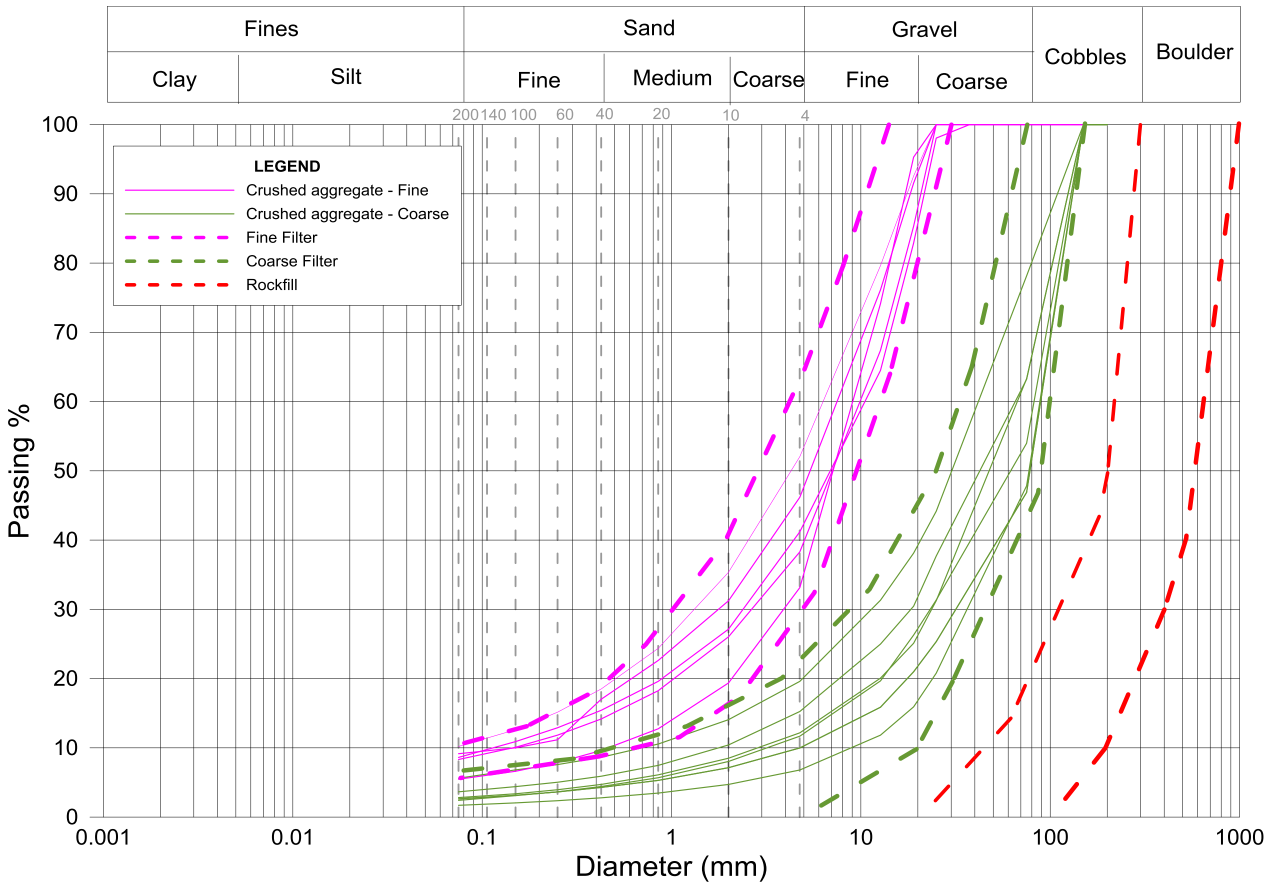


Figure 7-5 : Final gradation limits.

Table 7-2: Gradation limits of granular material for Whale Tail Dike

Sieve #	% Passing		
	Fine Filter	Coarse Filter	Rockfill
200	6 - 10	0 - 7	-
100	7 - 13	0 - 8	-
40	9 - 18	0 -10	-
20	11- 27	0 - 12	-
10	17 - 41	0 - 17	-
4	30 - 64	0 - 24	-
10 mm	51 - 88	5 - 32	-
30 mm	100	20 - 57	0 - 4
100 mm	100	60 -100	0 - 28
200 mm	100	100	10 - 50
500 mm	100	100	40 -100
1000 mm	100	100	100

Figure 7-6 illustrates a comparison of the WTD fine and coarse filter gradation limits with the corresponding Meadowbank filter gradations. It should be noted that the Meadowbank filters were also obtained from crushed rock.

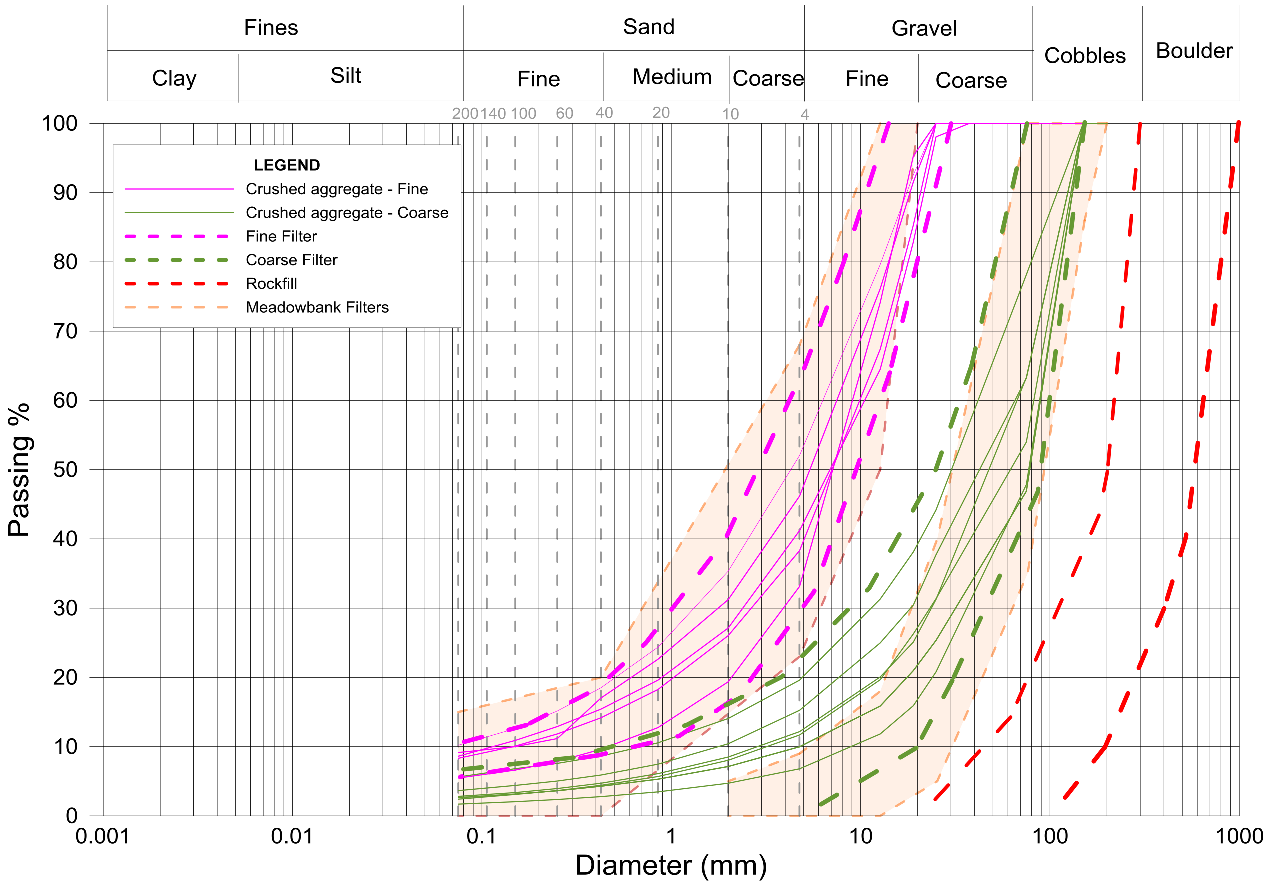


Figure 7-6 : Final gradation limits compared to Meadowbank filters.

7.5 Placement guidelines

The placement of material for the construction of WTD will be done in accordance with the same high standards used to build the existing rockfill dikes at Meadowbank. However, the placement techniques used since the construction of the East Dike in 2008 in Meadowbank, by placing material into the lake, should also be applied for fill placement into Whale Tail Lake. The placement procedure is presented in the technical specifications document for the construction of the dike (refer to Appendix B). This procedure was established in order to minimize the segregation and the loss of fines when materials are placed under water.

The maximum particle size for rockfill is 1000 mm. However, particles over 1,000 mm could be used for the embankment but should be placed on the outside shoulder of the dike whereas finer rockfill should be placed on the inside shoulder along the alignment of the cutoff.

7.6 Internal stability

The internal stability of the WTD is addressed in terms of the boundaries between fill zones, the contact between the filters and the bedrock surface, and the potential for excessive exit gradients at the contact between the downstream rockfill platform and the foundation soils.

The gradation limits of the fine and coarse filter zones satisfy filter grading rules so that the hydraulic stability between these zones is considered satisfactory. Nonetheless, placement procedures for these zones should be selected to minimize potential for segregation as recommended in Section 7.5. In addition, the proposed compaction of the fine filter zone (see Section 7.7) will increase hydraulic stability between the fine and coarse filters.

Most of the bedrock cleaning along the cutoff will be done under water, thus inspection and conventional treatment of the bedrock surface before placement of the fine filter zone is not practical. However, considering that the secant piles will extend one meter below bedrock surface, that the hydraulic gradients at the fine filter to bedrock contact for the case without grouting are generally less than 0.4 (see Figure A-16 of SNC-Lavalin, 2017b), and that the crushed aggregate fine filter is reasonably resistant to erosion, it is considered that there is sufficient safety against washing out of fine filter at the contact with open fractures at bedrock surface. In addition there is sufficient safety against excessive washing out of potential fractures in the bedrock filled with loose fine grained soil.

The downstream rockfill platform is end dumped on foundation soils consisting of bottom sediments and/or glacial till. The contact between the rockfill and foundation soils is not filter graded and thus there is potential for piping of the foundation soils into the rockfill if the upward exit gradients at the contact exceed one. Again with reference to Figure A-16 (SNC-Lavalin, 2017b), the upward gradients are about 0.05 to 0.15 below the downstream rockfill for the case of no grout curtain, thus well within acceptable values of about 0.25 to 0.4 used in general practice (USBR, 2014b).

7.7 Dynamic compaction

After the rockfill platforms are placed, and after bedrock cleaning in the wet, the fine and coarse filters will be progressively placed by an excavator to the level of the top of the platforms (elevation 154.0 m). Therefore these fills will be in a loose condition. It is proposed to densify the fine filter using dynamic compaction by heavy tamping when the fine filter of the embankment reaches its final elevation of 157.0 m. The compaction will improve the internal stability of the fine filter, will facilitate the installation of the secant piles and will reduce the extent of settlement and deformation of the WTD during operation.

The technical specifications for the dynamic compaction are presented in detail in Appendix B.

7.8 Proposed alignment

During the scoping study of 2015, the alignment developed for WTD consisted of the shortest line from the west to the east side of the lake. In addition, it was assumed that bedrock was located at 2 m depth from the lake bottom. Following the winter 2016 field investigation with an air track drill rig (AEM, 2016), a bedrock profile was produced and lead to a modification of the alignment (2016 alignment).

In the 2017 investigation (AEM, 2017, SNC-Lavalin, 2017e), bedrock at the east abutment was not found above the design flood elevation of 157 m, at which level it was intended to key in the CB cutoff wall. Additional air track drilling campaign was then performed in November 2017 north and south of the 2016 alignment. This campaign confirmed that the bedrock was only found above elevation 157.0 m south of the 2016 alignment. Since the rock profile found on the south sector was more advantageous, the alignment was then moved south during the detailed engineering of WTD and is presented on Drawing 651298-2500-4GDD-0003. As discussed in Section 7.10.2, it was not considered necessary to extend the secant pile wall to bedrock at 157.0 m. It should be noted that an additional air track drill rig campaign was performed in February 2018 and a diamond drill campaign in March 2018, confirming once again the bedrock profile under WTD.

7.9 Foundation preparation and abutments

The foundation preparation within the lake section of the key trench of WTD consists of underwater excavation of the lakebed soils to the bedrock surface, as per the design drawings and construction specifications. Exhaustive effort shall be deployed to clean the bedrock surface as much as possible with appropriate equipment and tools. Special care should be taken for sectors where frozen soil can be encountered. An appropriate QA/QC program should limit risks of a wrong estimation of the sound bedrock that will be used as a base to define the elevation where the secant piles will have to reach.

Both dike abutments (west and east) are in a frozen condition with about 2 m of active layer. Soils overlying the bedrock were investigated in recent investigations (AEM, 2016 & SNC-Lavalin, 2017e) and revealed the presence of an esker in the west abutment and glacial till at the gentle slope at the east abutment. As mentioned previously, the raising of the water level will impact the thermal regime of the foundation. Therefore, the design strategy has considered this impact.

At the west abutment, the WTD alignment crosses an esker which extends well below lake level. Also the esker has the potential to contain ice rich zones. It is proposed to excavate the esker to about elevation 153 m at the west abutment (see Detail 1 on Drawing 0005). The bottom of the excavation would be 0.5 m above lake level and then the rockfill zones and fine filter would be placed. A coarse filter is shown on this section (Section D-D Drawing 0007) and would be placed only if required. Excavation of the esker would require drilling and blasting due to the expected frozen condition. Above elevation 153.0 m, a key trench to the bedrock will then be progressively excavated in the thawed esker to expose its surface. In addition, and to minimize the number of secant piles, it is proposed to place a CB slurry cutoff wall where bedrock surface is above elevation 155.5 m. The secant piles will overlap the cured CB slurry cutoff over 1.5 m. This elevation and overlap was determined to be an acceptable approach for sectors where a visual inspection of the exposed bedrock is possible.

The strategy for the east abutment will be different than for the west abutment. It would be beneficial to remove the active layer that contains ice rich till prior to the placement of any material. Due to the freeze and thaw process, this layer usually contains high water content and is subject to thaw settlement when ice rich material thaws. The strategy is to expose permafrost and remove the active layer during the thawing season. This would reduce expected settlement and the impact of thaw consolidation. The removal of this material will be facilitated by exposing the surface to the sun followed by progressive excavation.

At both the west and east abutments, there would be a significant thickness of frozen soil left in place. Gradual thawing of this soil would occur over the life of WTD, on the upstream side. A certain amount of total and differential settlement of the embankment can be tolerated. Performance of the secant piles is linked with these post-construction deformations which are studied in modeling analysis presented in Appendix D.

7.10 Cutoff wall design with secant piles

It should be mentioned that the design of the secant pile cutoff wall was done with the participation of Bauer Foundation Canada Inc. who was contracted by SNC-Lavalin for support on technical aspects of secant pile design and construction.

7.10.1 Drilling technique

Secant pile walls are formed by constructing overlapped concrete piles to form structural walls that resist lateral pressures and prevent groundwater through flow. Alternate primary piles (initial) are installed first with secondary piles constructed in between once the latter gain sufficient strength. Pile overlap is typically chosen to form a continuous wall with a specified minimum thickness at the contact of adjacent piles.

The columns can be constructed with soil mixing, jet grouting, augercast or drilled shaft methods. Sequenced construction of the individual columns that comprise the finished barrier helps to ensure a tight seal between columns for complete water cutoff. Secant pile walls can be constructed in a wide variety of soil conditions, including through cobbles and boulders, and can even be embedded into the bedrock with certain drilling equipment.

The cast in place technique is the most adaptable method for WTD. This method involves a rotary drill rig to install the secant piles. A casing is advanced with the drill and is cleaned out progressively during advance. The casing is seated in the bedrock prior to drilling a socket into the bedrock with appropriate drilling equipment for the rock type and conditions. The drilled bedrock socket and casing are then filled with a cement-bentonite mix material using the tremie method (refer to section 7.10.3) as the casing is withdrawn.

To ensure the continuity of the wall, continuous vertical validation on the drill casing will be done on site. A guide wall or equivalent template can also ensure continuity of the wall and is presently under consideration by AEM.

7.10.2 Dimensioning of the wall

The WTD is an 800 m long structure. Since the cutoff wall with secant piles will only be constructed between stations 0+093 to 0+810, and since the secant piles will be anchored 1 m below bedrock, the total length of the secant pile wall is 717 m with an average depth of 8.2 m[1]. At the west abutment, the secant piles extend to a point where bedrock is at elevation 155.5 m. At the east abutment the secant piles extend

[1] Based on SNC-Lavalin's bedrock profile interpretations from factual field data.

to a point where the bedrock is at about elevation 151 m. It was considered that the secant piles did not have to be extended to a higher bedrock elevation since the piles would extend beyond the upstream operating water level of 156 m and that beyond this elevation the permafrost is less susceptible to degrade. In addition, the overburden beyond the secant pile wall is a low permeability glacial till.

The dimensioning of the cutoff wall as shown on the drawings are based on the recommendations of a specialized contractor. The cutoff wall thickness at pile intersections is established at 0.8 m and 1194 piles of 1.0 m diameter spaced at 0.6 m distance are required to complete the cutoff.

The continuity of the wall shall always can be achieved with appropriate preparation and controls (QA/QC). If any gap is indicated based on quality control measurements, an extra pile will be installed between the two piles to close the cutoff wall.

Drawing 0010 in Appendix A presents a section of five adjoining secant piles to illustrate the overlap of adjacent piles. Table 7-3 presents details of the secant pile wall proposed for WTD.

Table 7-3: Summary of secant pile cutoff wall

Secant pile quantity	1 194	unit
Length of the wall	717	m
Average height	8.3	m
Maximum height	10.6	m
Total length	9 913	m
Total volume	7 791	m^3
Area of the wall(1)	5 226	m^2
Area of the secant pile(1)	5 947	m^2
Note 1: The area of the wall represents the area above the bedrock and the area of the secant pile wall represents the total area, including the piles into the bedrock.		

7.10.3 Cement and mixes

As previously mentioned, the cutoff wall of the WTD consists of self-hardening cement-bentonite slurry. The typical ingredients used are water, cement, bentonite, and additives. Additives are used to improve the workability and control the curing of the CB slurry mixes. The mix design for the composition of the CB secant pile cutoff wall of the WTD takes into consideration 3 parameters: strength, permeability, and constructability.

In order to determine a CB mix that meets the strength, permeability and constructability requirements, laboratory tests were carried out to meet the criteria presented in Table 7-4. These requirements were selected based on SNC-Lavalin experience, literature, standards and Bauer's experience in similar projects.

Table 7-4: Requirements for Cement-Bentonite Mix Design

Characteristic	Requirement
Permeability (cm/sec)	≤ 10^{-6} in 28 days of curing
Unconfined compressive strength (UCS), (kPa)	≥ 200 in 28 days of curing
Minimum Early-Strength	Minimum UCS of 50 kPa after 7 days of curing
Marsh Viscosity (seconds)	≤ 80 in 8 hours of curing
Density (g/cm^3)	≥ 1.2

It should be noted that the laboratory tests were carried out on samples prepared at warm and cold curing conditions with GU cement and sodium bentonite already ordered on site.

Preparation of the CB slurry was done in the following stages:

- Bentonite slurry was prepared by mixing the bentonite and water.
- Additive was added to the bentonite slurry.
- Cement-bentonite slurry was prepared by mixing cement with the bentonite slurry that already has the additive.

The following laboratory tests were carried out, as per the corresponding standards:

- Viscosity test using Marsh Funnel as per ASTM D6910-04 or API 13B.
- Unconfined compression test as per ASTM D2166.
- Permeability test as per ASTM D5084.
- Bleeding test as per API13B.
- Vane shear test as per ASTM D2573.
- Density testing as per ASTM D4380-84.

Details of the laboratory testing program and the laboratory test results are presented in Appendix F.

After carrying out laboratory tests on different mix ratios using different additives at both cold and warm curing condition, the mix ratio shown in Table 7-5 is considered to be the final design mix ratio. The laboratory test results for this mix (referred to as Mix 22) are presented in Appendix F.

Table 7-5: Design mix ratio

Material	Value
Cement (Cement/Water)	0.4
Bentonite (Bentonite/Water)	4.6 %
Additive* (ARBO S01P – Sodium Lignosulfonate)	0.5 %
* The additive dosage is by weight of cement.	

The mixture to be used for 1 cubic meter of slurry is:

- 875 liters of water
- 350 kg of cement
- 40 kg of bentonite
- 1.75 kg of additive (ARBO S01P – Sodium Lignosulfonate)

8.0 GROUT CURTAIN DESIGN

8.1 Context of grouting

Grouting of bedrock below WTD is under consideration as a means of reducing seepage through the dike foundation. In this regard, seepage control may be desirable to reduce the amount of seepage volume reporting to the attenuation pond. This section of the report includes a description of grout curtain design in the event that bedrock grouting is implemented.

8.2 Dike foundation seepage control

Based on the results of the geotechnical investigations carried out from 2015 to 2018 at the WTD site, the rock can be characterized as close to very close jointed with poor to fair quality based on the joint spacing and RQDs (ISRM, 1981).

A significant amount of seepage through the bedrock below the secant pile wall is expected. Some reduction of this seepage can be achieved by grouting the bedrock below the secant pile cutoff. However, it is noted that bedrock grouting is not required for hydraulic stability of WTD.

8.3 Grout curtain

The main objective of grout curtain design and construction is to achieve a grout curtain at the lowest possible cost and in the simplest possible way in terms of construction and materials used.

Based on the condition of the bedrock foundation along the WTD alignment, and the WTD design, a grout curtain will consist of one line of grout holes to be drilled when the crest of WTD is at elevation 157 m. The grout curtain will be installed from the center line of the secant pile cutoff wall with appropriate drilling tools to limit adverse consequences on the integrity of the cutoff during this activity

Grouting of rock foundations will comprise drilling and grouting of the curtain using the split space closure method. The primary hole spacing shall be 12 m. The secondary and tertiary holes at split spacing are mandatory, while quaternary holes may be required based on the results at the completion of tertiary holes or as per the Engineer's instruction.

Bedrock grouting will generally be upstage working, unless ground conditions require downstage working. If downstage working is required, it shall be instructed by the Engineer.

Grout holes shall be drilled from the crest of the dike at the center of the secant pile wall into the bedrock to the specified design depth below bedrock surface.

Grouting shall take place continuously until grout refusal occurs or the specified volume limit for that stage is reached. Grout refusal for a given stage shall be a flow rate of less than 5.0 L/min per 5 m stage length and measured over a 10 minute period (or 1.0 L/min/m for 10 minutes) at the target pressures for the stage.

8.4 Groutability of bedrock

Groutability of the bedrock foundation has been evaluated in setting up the design seepage control criteria in order to construct a cost effective grout curtain within reasonable efforts. The term groutability of rock generally includes the following aspects:

- The capacity of bedrock to take grout or as penetration of grout.
- Combined effects of fracture properties which define the maximum grain size of cement particles.
- The quality of rock in regard to easiness and possibility to seal it by grouting.
- Impact of frozen conditions of bedrock.

8.4.1 Groutability of fractured bedrock

Cement based materials are most widely used in curtain grouting in bedrock foundations. The groutability of the fractured bedrock can be assessed using the following established criterion related to the effective particle size of cement-based grout and the aperture of the open joints:

$$d_{95} \leq b/3$$

where: d_{95} is the diameter of 95% passing on the grain size distribution of cement material and b is the aperture of the fractures of bedrock.

Assuming the d_{95} of typical ordinary cements ranges from about 75 to 100 μm (or 0.075 to 0.1 mm), the minimum aperture of a fracture that can be effectively grouted with ordinary cement is estimated at about b = 0.2 to 0.3 mm.

Televiewer survey results show that the minimum aperture of the fractures measured is 0.55 mm with a typical average of 6.0 mm. Regarding grain size of particles, it can be concluded that the WTD foundation rock in unfrozen condition is groutable with ordinary cement based grout.

8.4.2 Fractured zones

Fractured zones of several hundred mm to 2 m in thickness were revealed by the televiewer imaging which may cause difficulties due to excessive intake and the loss of grout during grouting implementation.

Control grouting including using quick set grout, low mobility grout with sand fill, limiting grouting pressure, limiting grout volume and repeated grouting may be applied to limit grout losses.

8.4.3 Groutability of frozen bedrock

Fractured bedrock under frozen condition with fractures filled and bonded by ice is not considered groutable. Water pressure tests in the frozen bedrock below the WTD indicate practically no water intake during the tests.

It should be noted that excessive high grout pressure may induce the break of fractures filled and bonded by ice which may result in unwanted grout take.

Also, it should be noted that groutability of fractured bedrock in a frozen state currently should be further evaluated upon impounding which may change the condition of the thermal regime under the foundation.

8.5 Extent of grout curtain

Based on the thermal condition of the foundation bedrock along the WTD alignment, a grout curtain could be installed from approximate station ± 0+200 to station 0+520.

Requirement for grouting beyond the above section and at both abutments should be further evaluated upon reservoir impounding since this may change the thermal condition of the foundation.

8.6 Grouting equipment and material

8.6.1 Grouting Equipment

Curtain grouting consists of drilling grout holes, exploratory holes, and check holes; pressure testing, pressure washing, and injecting suspension grout under pressure; and includes furnishing of all materials, labor, and equipment as described and specified in the Project Construction Specifications. A list of main equipment required to construct the grout curtain includes but is not limited to:

- Drill Rigs and accessary to minimize damage to the secant pile cutoff during drilling;
- Grout Plants with Grout Mixer and Storage/Agitate Tank;
- Grout pumps;
- Data Collection or Recording System;
- Heating Equipment/System for Winter construction
- Site Communication Systems
- Site Laboratory and testing equipment

8.6.2 Grouting material

Curtain grouting in bedrock will be performed with the use of ordinary cement based high mobility grout (HMG) that is able to penetrate a sufficient distance into open joints/fractures to provide overlapping grouted zones over a reasonable amount of grout time.

The grout should be stable with a constant or nearly constant rheology and once in place the grout must not bleed. The grout should also have early strength such as to be resistant to leaching and wash out before hardening. The use of additive agents in cement-based grouts results in improved rheological properties of the grout mix.

The grout mix used for curtain grouting shall be typically composed of the following materials and properties:

- Water.
- Portland Cement (HE) Type 30.

- Superplasticizer: percentage by weight of cement as specified by the manufacturer.
- Viscosity modifying agent, accelerating agent, bentonite and other additives, if required: percentage by weight of cement as specified by the manufacturer.
- W/C (Water/Cement) ratio by weight: 0.6 to 0.8/1 based on pre-mix at site.
- Marsh Cone Flow: 32 to 60 sec;
- Specific Gravity: 1.60 to 1.65
- Bleeding: less than 5%;
- Unconfined compression strength (7 days): 1 MPa
- Initial set time: less than 8 hours

Grout mixes with high mobility (HMG) and low motility (LMG) shall be developed for grouting under different conditions and control of excessive grout loss.

The grout mix used to backfill casings shall have a W/C (Water/Cement) ratio by weight of 2 to 1 with 7.5% bentonite (% of cement weight) that reach 7 day UCS value of 1 MPa.

9.0 ENVIRONMENTAL CONCERNS

During Whale Tail Dike construction, contaminants and waste material (water, soil or cuttings) have to be managed according to the environmental policy of AEM.

10.0 SOUTH WHALE TAIL DIVERSION CHANNEL DESIGN

The South Whale Tail Diversion Channel (SWTDC) will be located in the south-western part of the Whale Tail Lake watershed. It will flow from Lake A20 (Whale Tail Lake) to Mammoth Lake. This channel will be the only outlet of Whale Tail Lake once the WTD is constructed and it will also act as an emergency spillway.

The SWTDC will be designed to handle extreme run-off corresponding to 1/3 from 1:1000 year return period to the PMF. This criterion was selected in accordance with CDA guidelines. The invert elevation and geometry of the channel will be selected in order to respect the operational water level (156.0 m) and the design flood water level (157.0 m) of the lake after WTD completion. The following aspects will also be included in the design:

- Ice jam performance during freshet;
- Erosion protection;
- Consideration of backwater effect from Mammoth Lake.

The length of the channel will be approximately 1000 m with a 4 m drop between Lake A20 and Mammoth Lake. The existing terrain along the channel alignment consists mainly of a boulder field. According to preliminary calculations, the width of the channel is expected to be about 30 m.

Based on the information from previous boreholes near Mammoth Lake, and the topography of the area, it is not expected that bedrock will be reached during excavation of the channel. Even though bedrock is not expected in the channel excavation, it is considered that blasting will be required for the excavation.

A geotechnical campaign will be conducted prior to construction in order to gather information about the stratigraphy along the alignment of SWTDC and to define the bedrock profile. The channel cross section will be designed taking into account the nature of the foundation and the thickness of the boulder layer. A protective riprap layer would be required and a transition layer consisting of compatible granular material might be necessary between the bottom of the excavation and the riprap protection layer. Therefore, a minimum quantity of granular material will be planned in order to account for invert levelling which will be necessary following blasting activities and transition material. The boulders removed during construction will be sorted and part of them will be reused as riprap protection.

The timeframe for the construction of the channel will be carefully planned as a function of the expected time of Whale Tail Lake filling. In order to keep a safety margin, this filling schedule will be prepared considering 1:10 years return period precipitation within the lake's watershed. The channel has to be operational before the water level reaches its invert which will be approximately at elevation 155.7 m.

In order to mitigate suspended sediment transport which will occur after the first flooding of the lake and the channel, turbidity barriers or permanent sediment settlement areas will be planned at the channel outlet in Mammoth Lake.

It is noted that the final detailed design of this system will be presented in a separate report.

11.0 CONSTRUCTION

The construction of WTD is planned between July and December 2018. Figure 11-1 presents the schedule of construction as proposed by the contractor (KCG). As shown, prior to the beginning of WTD's construction, pre-construction works are required and are similar to those involved for Bay Goose Dike South at Meadowbank in 2010. It has been agreed that 2 rows of turbidity barriers will be placed on each side of the dike. In addition, the water within the two platforms will be pumped and treated prior to being discharged to the environment.

The proposed limits of stripping and blasting, as well as the excavation of the key trench, are shown on Drawing 651298-2500-4GDD-0005. The construction stages for WTD are shown on Drawing 651298-2500-4GDD-0010, in particular for the fine and coarse filter placement, dynamic compaction of the fine filter and the secant pile wall construction.

The construction of WTD is planned to begin in summer 2018, after the spring freshet. At the beginning of July, the two initial rockfill platforms of the dike will be built.

When the rockfill platform construction extends across the entire width of Whale Tail Lake, a head differential across the rockfill will start to build. In order to limit the head differential, it is recommended to close the entire rockfill structure to the west abutment after the end of the spring freshet. The corresponding date, based on the analyses of snow cover data from Baker Lake A meteorological station covering the period between 1955 and 2017 (63 years) and taking into account the latest cases, is July 15th.

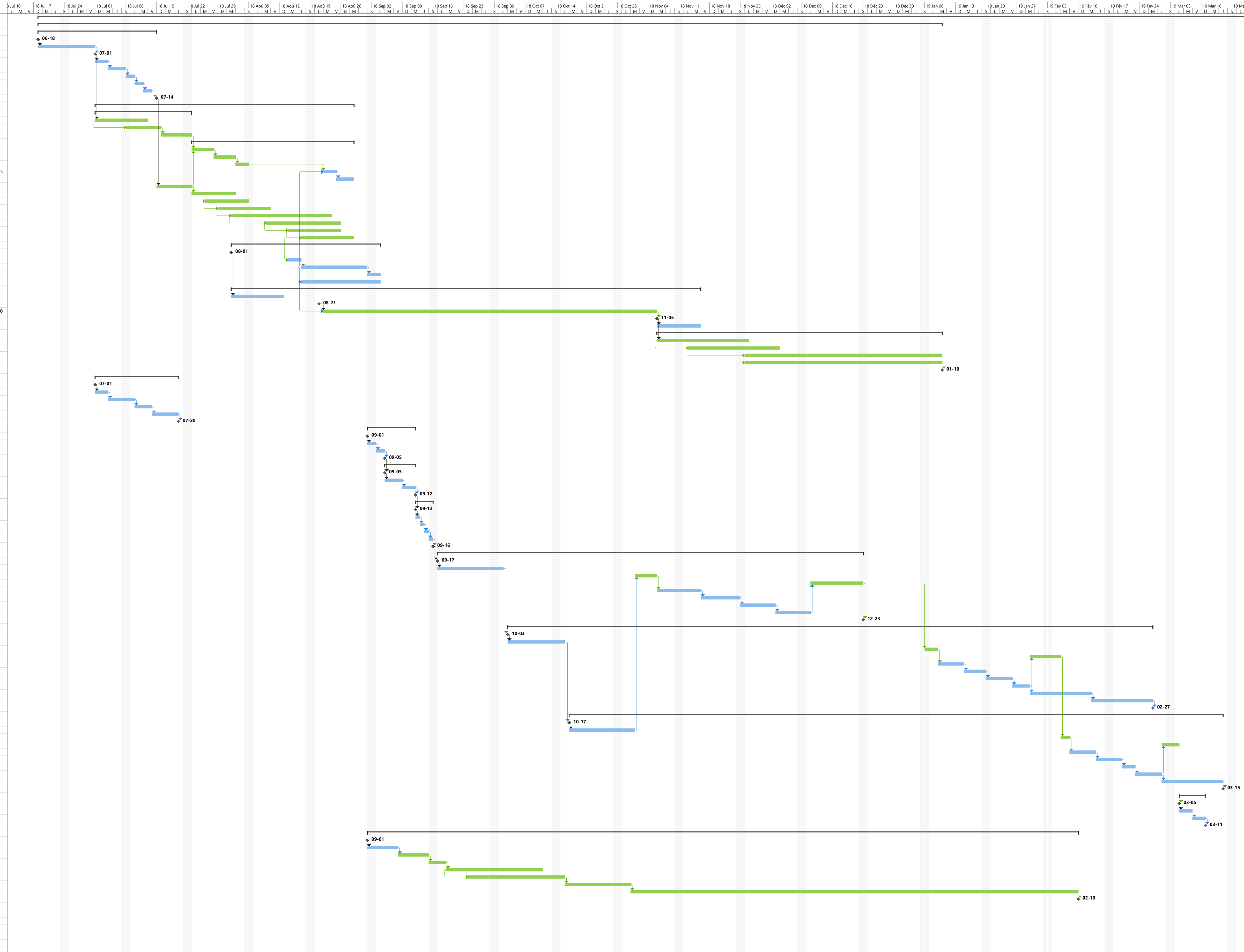
After mid-July, South Whale Tail Lake flow is relatively small. Even with no seepage assumed after Zone 1 placement and secant pile installation, no excessively high hydraulic gradients between the upstream and downstream sides of the dike are expected during the period of construction of the secant piles and grout curtain.

A sensitivity analysis was performed to assess South Whale Tail Lake water level after the dike closure. Flood routing computations were used to determine the lake water level based on the following assumptions:

- Start of the spring flood set on June 10th, which is the average freshet start date.
- Dike closed on July 15th.
- Whale Tail Lake initial water level at the start of the spring freshet: 153.0 m
- Two flood scenarios are compared: 1) 2015 (assumed representative of approximately 4-year return period condition), and 2) a flood resulting from the combination of a 10-yr return period snow cover melting during 25 days and a 2-yr return period, 72 hour duration spring rainfall happening during the last 3 days of the snow melt.

Figures 11-2 and 11-3 show the flood routing computation results for Whale Tail Lake.

N°	Nom de la tâche	Durée	Début	Fin	Quantity	Prédécesseurs
1	**AGNICO EAGLE - WHALE TAIL EARTHWORK - CONSTRUCTION**	**330 jours**	**Lun 18-06-18**	**Mar 19-05-14**		
2	**Whale Tail Dike**	**206 jours**	**Lun 18-06-18**	**Jeu 19-01-10**		
3	**Turbidity Barrier**	**26,98 jours**	**Lun 18-06-18**	**Sam 18-07-14**		
4	Start	0 jour	Lun 18-06-18	Lun 18-06-18		
5	Turbidity Barrier Preparation	13 jours	Lun 18-06-18	Dim 18-07-01		4
6	Permit Obtention	0 jour	Dim 18-07-01	Dim 18-07-01		5
7	Installation #1	3 jours	Dim 18-07-01	Mer 18-07-04	20 010 m²	6
8	Installation #2	4 jours	Mer 18-07-04	Dim 18-07-08	27 875 m²	7
9	Installation #3	2 jours	Dim 18-07-08	Mar 18-07-10	15 000 m²	8
10	Installation #4	2 jours	Mar 18-07-10	Jeu 18-07-12	3643 m²	9
11	Installation #5	2 jours	Jeu 18-07-12	Sam 18-07-14	6239 m²	10
12	End	0 jour	Sam 18-07-14	Sam 18-07-14		11
13	**Embankment**	**58,98 jours**	**Dim 18-07-01**	**Mar 18-08-28**		
14	**East Abutment**	**22 jours**	**Dim 18-07-01**	**Lun 18-07-23**		
15	Drilling&Blasting East Abutment (Within specified limit)	12 jours	Dim 18-07-01	Ven 18-07-13	32 339 m³	6
16	Excavation East Abutment (m³)	8 jours	Dim 18-07-08	Lun 18-07-16	32 339 m³	15DD+7 jours
17	Coarse, Fine Filter & Rockfill Placement	7 jours	Lun 18-07-16	Lun 18-07-23		16
18	**West Abutment**	**36,98 jours**	**Lun 18-07-23**	**Mar 18-08-28**		
19	Drilling&Blasting West Abutment (Within specified limit)	5 jours	Lun 18-07-23	Sam 18-07-28	7677 m³	24;17
20	Excavation Abutment West (m³)	5 jours	Sam 18-07-28	Jeu 18-08-02	12 685m³	19
21	Coarse, Fine Filter & Rockfill Placement	3 jours	Jeu 18-08-02	Dim 18-08-05		20
22	Slurry Wall Excavation	3 jours	Mar 18-08-21	Ven 18-08-24		21;35DD+5 jours
23	Slurry Wall Backfill	4 jours	Ven 18-08-24	Mar 18-08-28		22
24	**Rockfill placement (Bottom to 154) -2 Dozer**	8 jours	Sam 18-07-14	Dim 18-07-22	73 126 m³	12
25	**Lake Bed Mass Excavation (West to East)**	10 jours	Dim 18-07-22	Mer 18-08-01	23 292 m³	24
26	**Lake Bed Final Clean-up**	10 jours	Mer 18-07-25	Sam 18-08-04		25DD+3 jours
27	**Coarse Filter - Key Trench (Bottom to 154)**	12 jours	Sam 18-07-28	Jeu 18-08-09	14 525 m³	26DD+3 jours
28	**Fine Filter - Key Trench (Bottom to 154)**	23 jours	Mar 18-07-31	Jeu 18-08-23	56 702 m³	27DD+3 jours
29	**Fine Filter Placement (From 154 to 157)**	17 jours	Mer 18-08-08	Sam 18-08-25	34 753 m³	28DD+8 jours
30	**Coarse Filter Placement (From 154 to 157)**	12 jours	Lun 18-08-13	Sam 18-08-25	16 163 m³	29DD+5 jours
31	**Rockfill Placement (From 154 to 157)**	12 jours	Jeu 18-08-16	Mar 18-08-28	60 000 m³	30DD+3 jours
32	**Dynamic Compaction + Secan Pile Test**	**33,98 jours**	**Mer 18-08-01**	**Lun 18-09-03**		
33	Barge Material Arrival	0 jour	Mer 18-08-01	Mer 18-08-01		
34	Dynamic Compaction Setup	3 jours	Lun 18-08-13	Jeu 18-08-16		31DD-3 jours
35	Dynamic Compaction Production (m²)	15 jours	Jeu 18-08-16	Ven 18-08-31		34
36	Dynamic Compaction Demob	3 jours	Ven 18-08-31	Lun 18-09-03		35
37	Fine Filter Placement (backfill after compaction)	18 jours	Jeu 18-08-16	Lun 18-09-03	4800 m³	35DD
38	**Secant pile wall**	**106,98 jours**	**Mer 18-08-01**	**Jeu 18-11-15**		
39	Mobilisation (Mixing plant & 1st rig Set-up)	12 jours	Mer 18-08-01	Lun 18-08-13		33
40	Secant pile wall - Start	0 jour	Mar 18-08-21	Mar 18-08-21		
41	Secant pile wall - Production	76 jours	Mar 18-08-21	Lun 18-11-05		35DD+5 jours;40
42	End	0 jour	Lun 18-11-05	Lun 18-11-05		41
43	Demobilization	10 jours	Lun 18-11-05	Jeu 18-11-15		42
44	**Injection**	**65,02 jours**	**Lun 18-11-05**	**Jeu 19-01-10**		
45	Casing Installation for Injection (meters)	21 jours	Lun 18-11-05	Lun 18-11-26		42
46	Drilling Bedrock (meters)	21 jours	Lun 18-11-12	Lun 18-12-03		45DD+7 jours
47	Injection (# holes)	45 jours	Lun 18-11-26	Jeu 19-01-10		46DD+7 jours
48	Instrumentation	45 jours	Lun 18-11-26	Jeu 19-01-10		47DD
49	End	0 jour	Jeu 19-01-10	Jeu 19-01-10		48
50	**Saline Protection Channel**	**19 jours**	**Dim 18-07-01**	**Ven 18-07-20**		
51	Start	0 jour	Dim 18-07-01	Dim 18-07-01		
52	Excavation	3 jours	Dim 18-07-01	Mer 18-07-04	2580 m³	51
53	Aggregates 0-20	6 jours	Mer 18-07-04	Mar 18-07-10		52
54	Bituminous Geomembrane	4 jours	Mar 18-07-10	Sam 18-07-14		53
55	Aggregates 0-40	6 jours	Sam 18-07-14	Ven 18-07-20		54
56	End	0 jour	Ven 18-07-20	Ven 18-07-20		55
57	**WRSF Collection Channel**	**11 jours**	**Sam 18-09-01**	**Mer 18-09-12**		
58	Start	0 jour	Sam 18-09-01	Sam 18-09-01		
59	Excavation (Pond/Channel)	2 jours	Sam 18-09-01	Lun 18-09-03	1100 m³	58
60	RipRap	2 jours	Lun 18-09-03	Mer 18-09-05	400 m³	59
61	End	0 jour	Mer 18-09-05	Mer 18-09-05		60
62	**East Channel**	**7 jours**	**Mer 18-09-05**	**Mer 18-09-12**		
63	Start	0 jour	Mer 18-09-05	Mer 18-09-05		61
64	Excavation	4 jours	Mer 18-09-05	Dim 18-09-09	4400 m³	63
65	Riprap	3 jours	Dim 18-09-09	Mer 18-09-12	1300 m³	64
66	End	0 jour	Mer 18-09-12	Mer 18-09-12		65
67	**East Channel Coffer Dam**	**4 jours**	**Mer 18-09-12**	**Dim 18-09-16**		
68	Start	0 jour	Mer 18-09-12	Mer 18-09-12		66
69	Excavation	1 jour	Mer 18-09-12	Jeu 18-09-13	480 m³	68
70	Rockfill	1 jour	Jeu 18-09-13	Ven 18-09-14	1008 m³	69
71	Filters	1 jour	Ven 18-09-14	Sam 18-09-15	600 m³	70
72	Till	1 jour	Sam 18-09-15	Dim 18-09-16	320 m³	71
73	End	0 jour	Dim 18-09-16	Dim 18-09-16		72
74	**North East Dike**	**97 jours**	**Lun 18-09-17**	**Dim 18-12-23**		
75	Start	0 jour	Lun 18-09-17	Lun 18-09-17		73FD+1 jour
76	Excavation	15 jours	Lun 18-09-17	Mar 18-10-02		75
77	Rockfill -AEM - Work Platform (25%)	5 jours	Jeu 18-11-01	Mar 18-11-06		98
78	Rockfill -AEM (75%)	12 jours	Mar 18-12-11	Dim 18-12-23		82
79	Coarse Filter	10 jours	Mar 18-11-06	Ven 18-11-16		77
80	Fine Filter	9 jours	Ven 18-11-16	Dim 18-11-25		79
81	Fine Filter Amended with Bentonite	8 jours	Dim 18-11-25	Lun 18-12-03		80
82	Bituminous Geomembrane	8 jours	Lun 18-12-03	Mar 18-12-11		81
83	End	0 jour	Dim 18-12-23	Dim 18-12-23		78
84	**WRSF Dike**	**147 jours**	**Mer 18-10-03**	**Mer 19-02-27**		
85	Start	0 jour	Mer 18-10-03	Mer 18-10-03		76FD+1 jour
86	Excavation	13 jours	Mer 18-10-03	Mar 18-10-16		85
87	Rockfill -AEM - Work Platform (25%)	3 jours	Dim 19-01-06	Mer 19-01-09		78FD+14 jours
88	Rockfill -AEM (75%)	7 jours	Mer 19-01-30	Mer 19-02-06		92
89	Coarse Filter	6 jours	Mer 19-01-09	Mar 19-01-15		87
90	Fine Filter	5 jours	Mar 19-01-15	Dim 19-01-20		89
91	Fine Filter Amended with Bentonite	6 jours	Dim 19-01-20	Sam 19-01-26		90
92	Bituminous Geomembrane	4 jours	Sam 19-01-26	Mer 19-01-30		91
93	Grout Curtain	14 jours	Mer 19-01-30	Mer 19-02-13		92
94	Instrumentation	14 jours	Mer 19-02-13	Mer 19-02-27		93
95	End	0 jour	Mer 19-02-27	Mer 19-02-27		94
96	**Mammoth Dike**	**149 jours**	**Mer 18-10-17**	**Ven 19-03-15**		
97	Start	0 jour	Mer 18-10-17	Mer 18-10-17		86FD+1 jour
98	Excavation	15 jours	Mer 18-10-17	Jeu 18-11-01		97
99	Rockfill -AEM - Work Platform (25%)	2 jours	Mer 19-02-06	Ven 19-02-08		88
100	Rockfill -AEM (75%)	4 jours	Ven 19-03-01	Mar 19-03-05		104
101	Coarse Filter	6 jours	Ven 19-02-08	Jeu 19-02-14		99
102	Fine Filter	6 jours	Jeu 19-02-14	Mer 19-02-20		101
103	Fine Filter Amended with Bentonite	3 jours	Mer 19-02-20	Sam 19-02-23		102
104	Bituminous Geomembrane	6 jours	Sam 19-02-23	Ven 19-03-01		103
105	Instrumentation	14 jours	Ven 19-03-01	Ven 19-03-15		104
106	End	0 jour	Ven 19-03-15	Ven 19-03-15		105
107	**Main Camp Pond**	**6 jours**	**Mar 19-03-05**	**Lun 19-03-11**		
108	Start	0 jour	Mar 19-03-05	Mar 19-03-05		100
109	Excavation (Pond/Channel)	3 jours	Mar 19-03-05	Ven 19-03-08		108
110	Geomembrane	3 jours	Ven 19-03-08	Lun 19-03-11		109
111	End	0 jour	Lun 19-03-11	Lun 19-03-11		110
112	**South Wail Tail Diverson Channel A45 Lake - 612 m.li.**	**162 jours**	**Sam 18-09-01**	**Dim 19-02-10**		
113	Start	0 jour	Sam 18-09-01	Sam 18-09-01		
114	Turbidity Barriers 3 m x 330 m Li	7 jours	Sam 18-09-01	Sam 18-09-08		113
115	Access Road	7 jours	Sam 18-09-08	Sam 18-09-15		114
116	Platform Backfill (For D&B)	4 jours	Sam 18-09-15	Mer 18-09-19	TBD	115
117	Drill and Blast	22 jours	Mer 18-09-19	Jeu 18-10-11		116
118	Excavation	22 jours	Lun 18-09-24	Mar 18-10-16		117DD+5 jours
119	Filters	15 jours	Mar 18-10-16	Mer 18-10-31		118
120	RipRap	102 jours	Mer 18-10-31	Dim 19-02-10		119
121	Instrumentation	0 jour	Dim 19-02-10	Dim 19-02-10		120
122	**WRSF Saddle Dam**	**13 jours**	**Mer 19-05-01**	**Mar 19-05-14**		
123	Start	0 jour	Mer 19-05-01	Mer 19-05-01		
124	Excavation	2 jours	Mer 19-05-01	Ven 19-05-03		123
125	Rockfill -AEM (1)	3 jours	Ven 19-05-03	Lun 19-05-06		124
126	Coarse Filter	3 jours	Lun 19-05-06	Jeu 19-05-09		125
127	Fine Filter	2 jours	Jeu 19-05-09	Sam 19-05-11		126
128	Compacted Till	3 jours	Sam 19-05-11	Mar 19-05-14		127
129	End	0 jour	Mar 19-05-14	Mar 19-05-14		128



Projet : Whale Tail Earthwork - General Construction Schedule

Figure 11-1: General construction schedule

Date : Sam 18-04-07

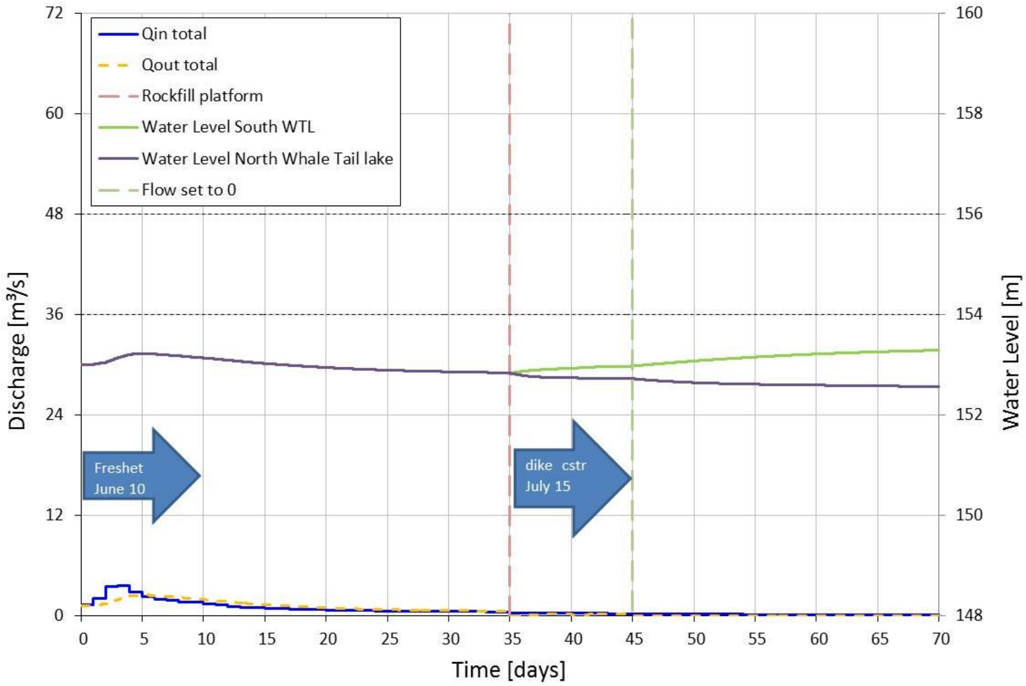


Figure 11-2: Whale Tail Lake water level (2015 spring flood)

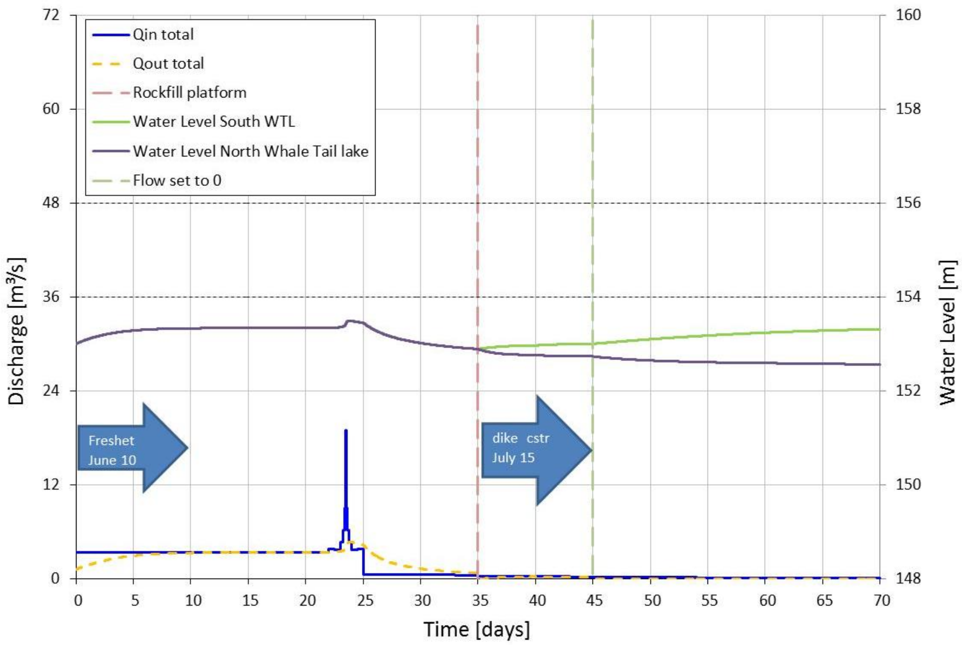


Figure 11-3: Whale Tail Lake water level (10-year snow cover)

The closure of WTD is assumed to occur on July 15th. After this date, only flow through rockfill is assumed between South Whale Tail Lake and North Whale Tail Lake. On July 25th, the flow between the two parts of the lake is set to 0 to simulate the construction of the fine filter followed by the secant pile wall.

In these conditions, as long as the dike closure happens after July 15th, the water level in North Whale Tail Lake does not exceed the rockfill platform elevation of 154.0 m. Furthermore, the gradient between the two sides of the dike is found to be less than 1.0 m. The following table presents a summary of the results in terms of water levels.

Following the sensitivity analysis, it is recommended that closure of rockfill across Whale Tail Lake occurs only on or after July 15th, when the lake level would be lower than152.9 m and on a descending trend.

Table 11-1: Whale Tail Lake water level

Water level conditions	2015 Data – 4-yr return period			10-yr snow cover melting in 25 days & 2-yr 72h spring rainfall		
	Before dike closure	South	North	Before dike closure	South	North
During freshet (maximum)	153.22	--	--	153.45	--	--
On July 15th (complete rockfill platform assumed)	--	152.83	152.83	--	152.89	152.89
35 days after dike closure (August 18th)	--	153.28	152.56	--	153.31	152.56

The technical specifications for the construction of WTD are presented in Appendix B. A summary of the quality control and quality assurance (QA/QC) procedures for the construction of WTD can be found in the technical specifications.

12.0 INSTRUMENTATION AND MONITORING PLAN

The construction of the dike will have an impact on the thermal regime of adjacent ground: permafrost will aggrade on the dewatered side of the lake while permafrost in the abutments will degrade in the flooded areas. Moreover, the calculated total groundwater flow under WTD indicates that it varies with foundation conditions as is to be expected. The seepage rate and thermal regime of the dike, among others parameters such as deformation, etc., have to be monitored to ensure proper operation of the dike. A detailed monitoring program has been developed to ensure adequate operation and performance of WTD during the lifetime of the structure.

12.1 Instrumentation

12.1.1 Thermistor Strings

A total of 15 thermistor strings is currently located in the WTD area. The temperature readings taken during the last year made it possible to assess the current thermal regime in the dike area. An open talik underlies the western part of the footprint of the WTD, whereas permafrost exists in both abutments as well as under the lake in the eastern part of the WTD footprint.

The thermal analysis carried out at WTD showed that the thermal regime of the dike area will change after construction of the dike. The main conclusions of that study are:

- Most of the core of the dike should freeze within 2 to 10 years and should stay in a frozen state during operation (ignoring the thermal effect of seepage);
- After dike construction and dewatering, the exposed bottom in the northern part of Whale Tail Lake should cool down rapidly, and permafrost should aggrade in this downstream area;
- The top 1 to 1.5 m of the cutoff wall may be exposed to freeze and thaw (F-T) cycles during the lifetime of the dike;
- The esker located on the upstream side of the west abutment should thaw in 3 of 4 years after the construction of the dike;
- At the east abutment, the ice-rich till under part of the embankment upstream of the core zone should thaw in less than 30 years.

To ensure predictable behaviour of the dike, thermistor strings must be installed within the dike to confirm that the conclusions of the thermal modeling are in link with the performance of the dike.

Thermistors installed directly in the cutoff wall would help in assessing the number of annual F-T cycles, as well as the depth of the active zone at the top of the dike. The installation of sufficiently deep thermistor strings would make it possible to assess the thermal regime under the dike as well.

In the east abutment area, maintaining the soil in a frozen condition is desirable to avoid problems due to thawing permafrost such as thaw settlements. Shorter thermistor strings in the upstream area would make it possible to monitor thermal degradation such as active layer deepening. The performance of the thermal berm at the east abutment could be monitored with thermistor strings as well. Two models of thermistor strings are presented in Figure 12-1.

The long thermistor string “Model L” has 13 beads and is 30 metres long. The first bead is located at the crest of the dike at elevation 157 m and the deepest bead is in the bedrock at elevation 127 m. It allows monitoring of the part of the dike located in the current Whale Tail Lake. According to the bedrock profile along the WTD, it is estimated that the deepest bedrock surface elevation is approximately at 148 m and the two deepest beads would be located below the 15-metre deep grout curtain. This would make it possible to assess if the foundation cools down enough to act as a hydraulic barrier which would prevent seepage, or alternatively if the water coming from the upstream part of the lake flows under the grout curtain. The installation of several thermistor strings in the cutoff wall along the dike would help characterize the thermal regime of the whole foundation, which is important for the assessment of the performance of the structure.

The short thermistor string “Model S” also has 13 beads but is half the length of the “Model L”. These strings would be located at both abutments to monitor the thermal regime in the active layer as well as within the top few metres of the bedrock. They should be installed directly along the centreline of the dike, as well as in the upstream embankment to monitor the temperature of the overburden. More specifically, the performance of the thermal berm at the east abutment should be monitored with these thermistor strings to ensure that the underlying permafrost stays frozen. The locations of the thermistor strings are shown in Figure 12-2. This figure shows only the thermistor locations along the centerline of WTD. The locations of additional “Model S” thermistors in the upstream abutment areas and at the thermal berm (if constructed) will be established later if required.

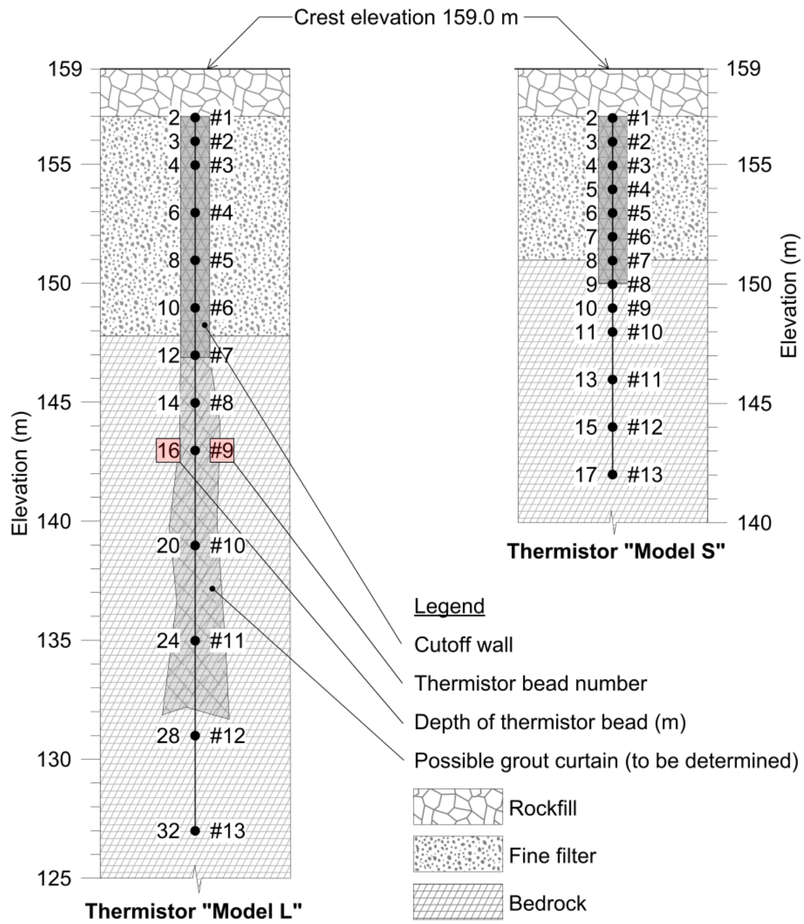


Figure 12-1: Two models of thermistor strings for the monitoring of WTD

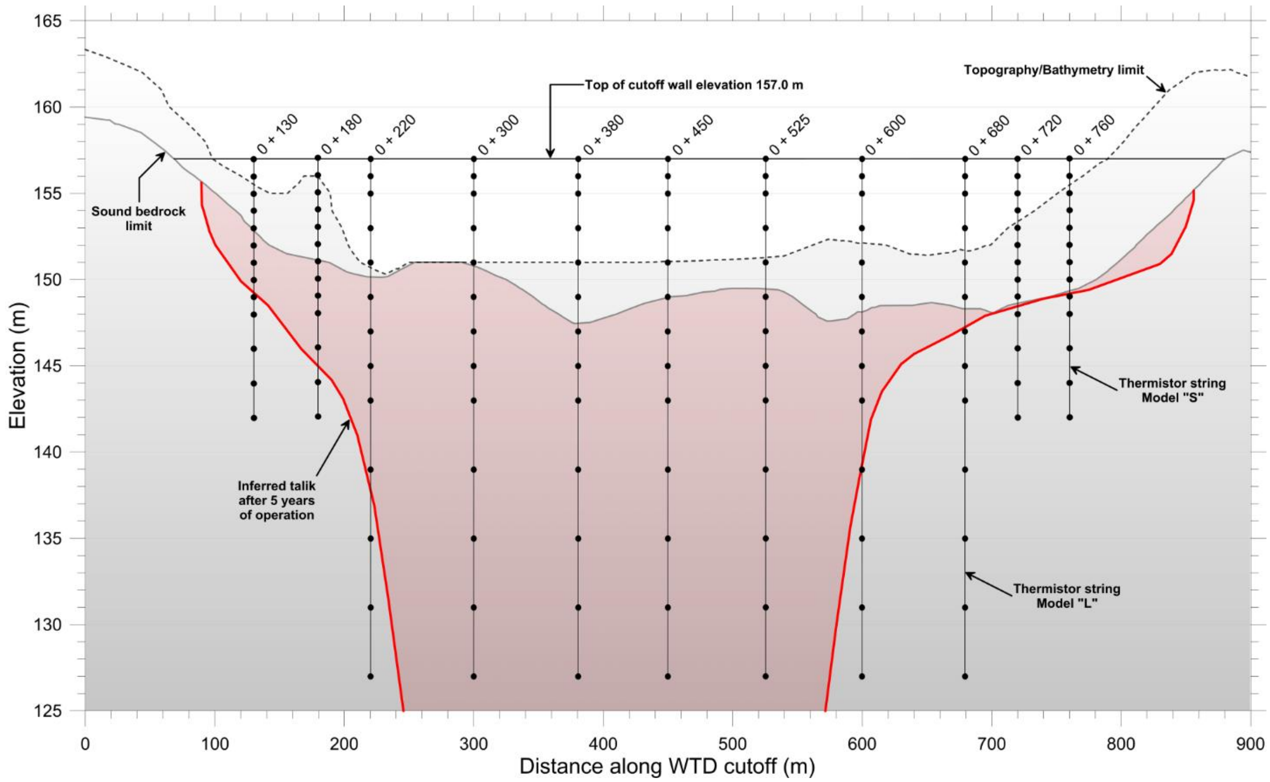


Figure 12-2: Locations of thermistor strings along the Whale Tail Dike

A total of four thermistor strings “Model S” (two located at both abutments) and seven thermistor strings “Model L” located in the dike alignment of the Whale Tail Lake would allow close monitoring of the structure. Moreover, it would make it possible to monitor the evolution of the shape of the talik with time. It is expected that the current talik zone under the dike will widen following the flooding of the east and west banks of the lake .Previous estimates (SNC-Lavalin, 2017d) show that the unfrozen zone would look like the one shown in Figure 12-2 after five years of operation. The expansion of the talik zone under the dike could have a significant impact on the seepage quantities and thaw settlement near the abutments.

The elevation of the thermistor strings is fixed throughout the dike. The depths of thermistor beads for each model are summarized in Table 12-1.

Table 12-1: Depths of thermistor beads for both models of thermistor strings

Bead number	Bead Depth (m)	
	Model “S”	Model “L”
1	0	0
2	1	1
3	2	2
4	3	4
5	4	6
6	5	8
7	6	10
8	7	12
9	8	14
10	9	18
11	11	22
12	13	26
13	15	30

12.1.2 Piezometers

In order to monitor water levels and vertical hydraulic gradients, piezometers have to be installed on both sides of the cutoff wall as shown in Figure 12-3.

As shown, a cross section includes nine (9) piezometers distributed on a 3x3 grid. They are distributed in three (3) boreholes: one located two metres upstream of the centreline (P3), a second located two metres downstream (P2) and a third located 12 m downstream of the centreline (P1). Three (3) piezometers per borehole will be installed. The shallower will be located at the surface of the rock (C), the depth of which will vary from one borehole to another. A second piezometer will be installed approximately at half-depth of the grout curtain (B). The deeper piezometer will be installed about two (2) metres below the grout curtain (A). A total of three (3) sections of nine (9) electric piezometers per section will be installed along the dike during its construction. The station of each section is summarized further in Table 12-4. Only three (3) sections of piezometers will be installed during the construction of the WTD because those instruments have to be installed in an unfrozen environment. If the saturated piezometers freeze, it damages them and they can no longer provide valuable readings. It was decided to install the piezometers in the current talik zone, where thermistor data show that the bedrock stays in the unfrozen state. Vibrating wire piezometers with a range from -100 to 350 kPa should be used.

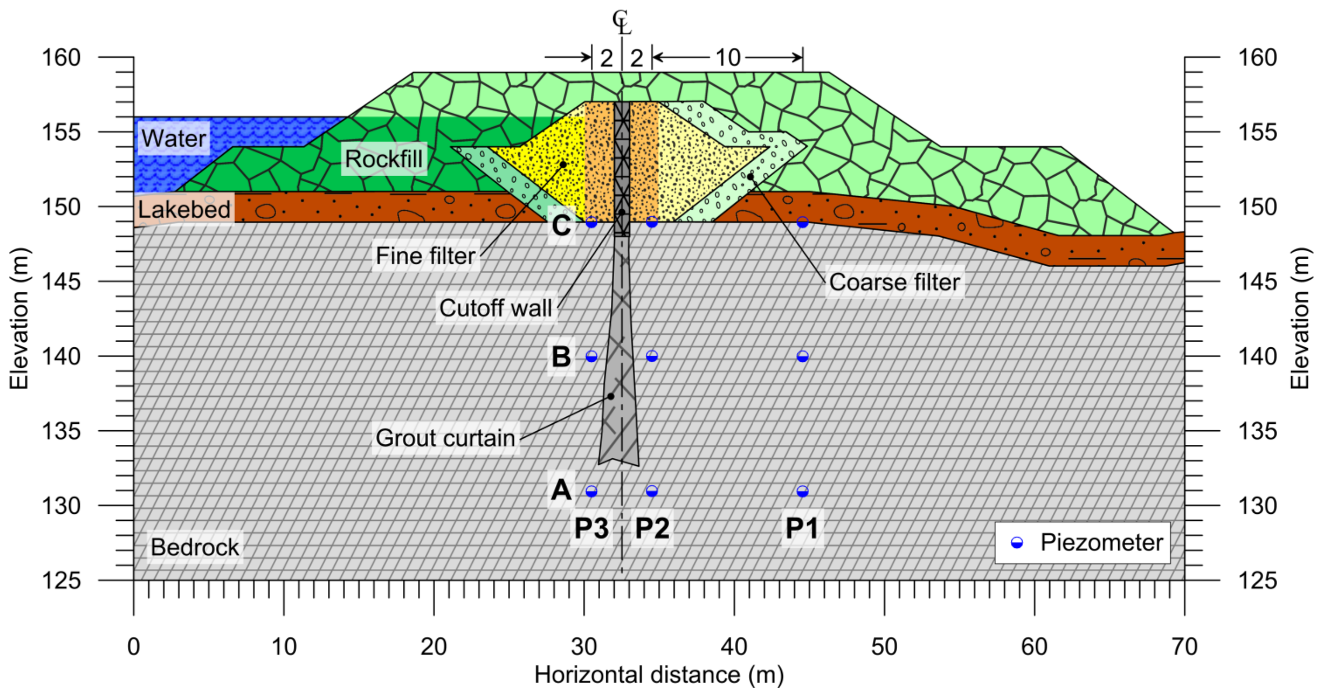


Figure 12-3: Location of multi-level vibrating wire piezometers

The multi-level piezometers shown in Figure 12-3 will be used to monitor pore pressures that may develop within the foundation and the bedrock. The data will be collected and reviewed in order to monitor the phreatic levels under the dike and verify the performance of the cutoff measures. The pore pressure data from the piezometer located just downstream of the cutoff at the contact between the fine filter and bedrock surface, will be analyzed in conjunction with the temperature data from the thermistor string located in the centerline of the dike in order verify that the seepage patterns are consistent with the expected conditions.

During the dewatering, piezometers will measure the pore water pressure changes in the dike foundation while the water level drawdown occurs downstream. Once the northern part of Whale Tail Lake is dewatered, the efficiency of the cutoff will be evaluated by measurement of hydraulic gradients in the downstream area of the foundation. In the long term, piezometers will allow the monitoring of variations in pore water pressure and hydraulic gradients under the dike.

The requirements for the piezometers are summarized in Table 12-2.

Given the harsh climate, an automated data acquisition system is deemed mandatory for the monitoring of the WTD. Data loggers will be installed in heated instrument shelters on the dike crest and all the cables will be buried for their protection against the weather and maintenance vehicles and/or trucks.

Table 12-2: Piezometers required for the instrumentation of WTD

Type	Quantity	Lead (m)
Piezometer (A)	10*	30
Piezometer (B)	10*	20
Piezometer (C)	10*	15
*One spare for each type is included in the total quantity.		

12.1.3 Survey monuments and benchmarks

In order to monitor surface movements due to thaw settlements, survey monuments have to be installed along the dike. The monuments, consisting of eleven prisms anchored to concrete blocks, will be installed after construction of the dike to monitor subsidence at those locations. The approximate locations of the monuments can be found in Table 12-4.

Two (2) survey benchmarks should be installed one on each side of the Whale Tail Lake near the dike. Settlements will be monitored with reference to these benchmarks using a total station instrument in order to quantify the amount of subsidence between the readings, which should be done on a monthly basis. Natural Resources Canada (1978) and the US Army Corps of Engineers (2012) both recommend a certain type of survey benchmark installation in permafrost areas as presented in Figure 12-4.

The most important aspect of the survey benchmarks is that they have to be anchored in bedrock so as to minimize or eliminate the effect of freeze and thaw cycles (movement of the survey benchmark). Surface movements due to the seasonal variation in the permafrost active layer have to be isolated from possible effects on the internal pipe acting as the benchmark (Tait, Brian, & Li, 2005).

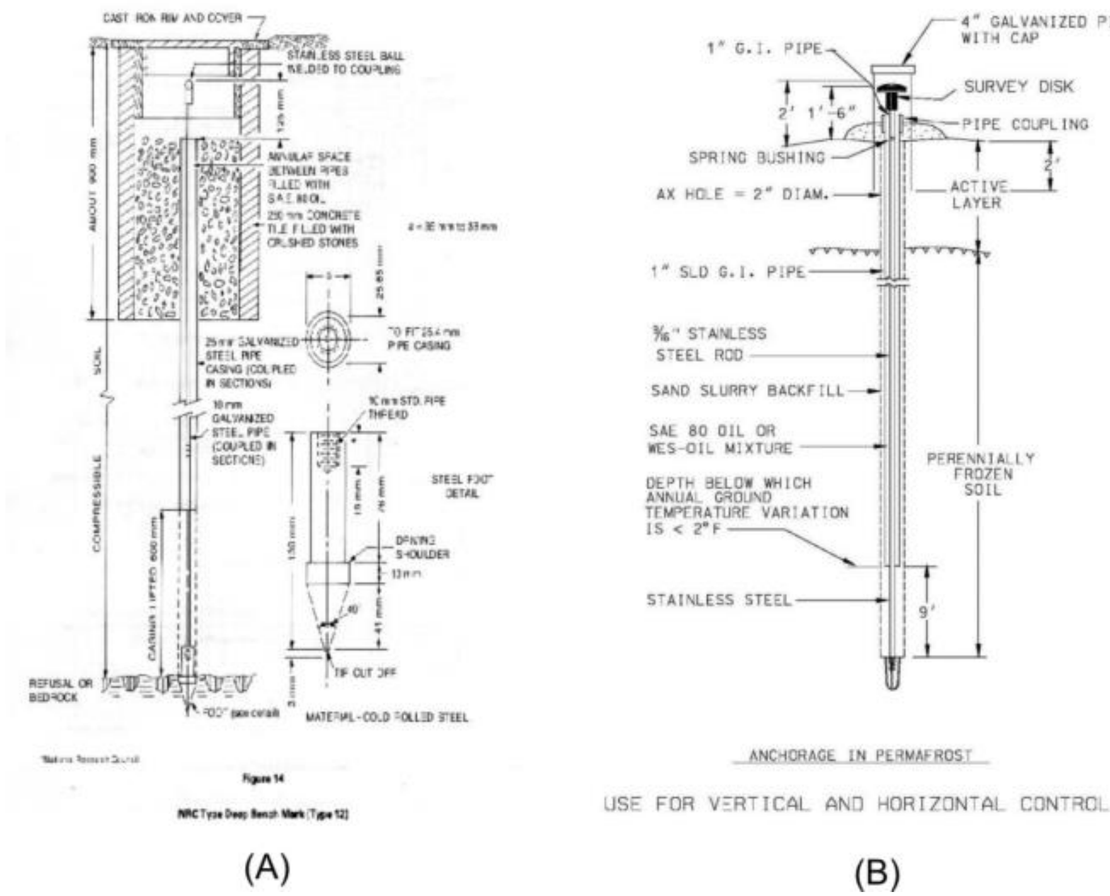


Figure 12-4: (A) Type 12 survey benchmark (after Natural Resources Canada, 1978) and (B) Type D benchnmark (after the US Army Corps of Engineers, 2012) adapted in permafrost regions

12.1.4 Inclinometers

Inclinometers will be installed at four (4) selected locations along the dike. They will be used to measure the magnitude and rate of lateral movement of the low permeability elements during dewatering and over the life span of the structure. Twenty meter long inclinometers will be installed between elevations 159 m and 139 m. They will have to be anchored at least 5 m into the bedrock and grouted using the tremie method and protected against the weather and borrowing animals with a 100 mm (4 inch) diameter casing. The locations of the inclinometers are summarized in Table 12-4, but the final locations will be determined on site.

12.2 Method and frequency of data acquisition

Given the harsh climate at the Amaruq site, automated data acquisition systems for thermistor strings and piezometers readings are deemed mandatory for the monitoring of WTD. Data loggers will be installed in shelters on the dike crest and all the cables will be carefully buried for their protection against weather and maintenance vehicles and/or trucks A frequency of acquisition of readings every 3 (three) hours would allow a safe follow-up of the behaviour of the dike. All the raw data will have to be transmitted to SNC-Lavalin by an automated satellite communication system.

Moreover, a complete weather station including the following instruments will be required for the Amaruq site:

- Barometer;
- Relative humidity sensor;
- Air temperature sensor;
- Anemometer;
- Pyranometer;
- Albedometer; and
- Precipitation sensor.

12.3 Summary of required instrumentation

All the required instrumentation for the monitoring of the WTD is summarized in Table 12-3.

Table 12-3: Quantities required for the instrumentation and monitoring of the WTD

Type of instrument	Quantity required	Comments
Thermistor strings, Model “S”	6 strings (includes 2 spares)	2 strings among the 6 require a 15-m lead with connectors.
Thermistor strings, Model “L”	7 strings	-
Piezometers	30 (includes 3 spares)	3 piezometers among the 30 require a 30-m lead.
Inclinometers	4	-
Survey benchmarks and monuments	2 benchmarks 11 monuments	To be installed in 2019.

The locations of the instruments are summarized in Table 12-4.

Table 12-4: Locations of the instruments for the monitoring of the WTD

Instrument		Station (m)
Thermistor strings, “Model S”	#1	0 + 130
	#2	0 + 180
	#3	0 + 720
	#4	0 + 760
	#5 and #6	Spare
Thermistor strings, “Model L”	#1	0 + 220
	#2	0 + 300
	#3	0 + 380
	#4	0 + 450
	#5	0 + 525
	#6	0 + 600
	#7	0 + 680
Piezometers	#1 to #9	0 + 240
	#10 to #18	0 + 380
	#19 to #27	0 + 470
	#28 to #30	Spare
Inclinometers	#1	0 + 180
	#2	0 + 380
	#3	0 + 720
	#4	0 + 760
Survey benchmarks	#1	Bedrock, west shore
	#2	Bedrock, east shore
Survey monuments	#1	0 + 130
	#2	0 + 180
	#3	0 + 220
	#4	0 + 300
	#5	0 + 380
	#6	0 + 450
	#7	0 + 525
	#8	0 + 600
	#9	0 + 680
	#10	0 + 720
	#11	0 + 760

13.0 QUANTITY ESTIMATE

Table 13-1 presents the estimated quantities of the construction materials as well as the required excavation volumes of the foundation of the WTD construction. Based on the accuracy of the data used to generate the quantities, a contingency of 20 to 40% has been added to each item.

Table 13-1: Material quantities for Whale Tail Dike

Item	Unit	MTO	Contingency (%)	Revised quantity	Details
1.0 FOUNDATION PREPARATION					
1.1 Whale Tail Dike					
West abutment					
1.1.1 Stripping/excavation for trench	m^3	4,174	20%	5,008	Bedrock profile to be confirmed following the DD campaing. Uncertainties between boreholes.
1.1.2 Blasting to el. 153.0 m (esker)	m^3	5,905	30%	7,677	Blasting limits to be confirmed on site.
Main section					
1.1.3 Excavation for trench	m^3	19,410	20%	23,292	Bedrock profile to be confirmed. Frozen soil from 0+500 to 0+700 is a source of uncertainties.
East abutment					
1.1.4 Stripping/Blasting	m^3	24,876	30%	32,339	
1.2 Collection ditch					
1.2.1 Excavation	m^3	7,362	50%	11,043	Section to be defined.
2.0 CONSTRUCTION					
2.1 Whale Tail Dike					
2.1.1 Fine filter (Zone 1)	m^3	65,325	40%	91,455	Quantity may increase due to bedrock profile (15%) and dynamic compaction (15%). Other uncertainties 10%.
2.1.2 Coarse filter (Zone 2)	m^3	25,573	20%	30,688	Quantity may increase due to bedrock profile
2.1.3 Rockfill for initial platforms to el. 154.0 m	m^3	60,938	20%	73,126	Quantity may increase if soft foundation is encountered.
2.1.4 Rockfill for dike	m^3	45,896	25%	57,370	Quantity may increase due to stripping/blasting limits/ access for equipment
2.1.5 Rockfill for safety berms	m^3	2,490	10%	2,739	
2.2 Whale Tail Dike access road					
2.2.1 Rockfill	m^3	7,843	20%	9,411	
2.3 Access road (by AEM)					
2.3.1 Rockfill	m^3	146,519	20%	175,823	
2.4 Collection ditch					
2.4.1 Riprap (assume 0.3 m thick, 3:1)	m^3	1,577	50%	2,365	Section to be defined.
2.5 South Whale Tail Channel					
2.5.1 Riprap	m^3	80,547	40%	112,765	Riprap thickness to confirm following hydraulic computations.
2.5.2 Granular material	m^3	16,000	50%	24,000	Transition material might or might not be needed on all or part of the alignment (depends on geotechnical campaign results). Note that geotextile could also be used and reduce needs for granular material.
2.5.3 General backfill	m^3	3,500	20%	4,200	Invert profile changes possible following hydraulic computations

14.0 CLOSURE

The current closure plan is to breach Whale Tail Dike after the mining operations and restore the initial hydrologic conditions of the Amaruq site once the water quality is adequate to discharge to the environment. Details of the breach will be developed before the closure stage of the project.

15.0 RISK ASSESSMENT

A risk analysis has been carried out on April 11 and 12, 2017 in Val d'Or offices of AEM. An additional meeting took place on March 22nd, 2018 to update the risk analysis. The main risks identified during this session for Whale Tail Dike construction are identified below.

Table 15-1: Main risks identified during the risk analysis session.

Risk	Cause	Mitigation plan
Unfavorable secant wall construction condition in winter	○ Task on the critical path pushes the secant wall activity to winter. ○ Deficient preparation for winter construction.	○ Mobilize in advance the equipment on site for winter work. ○ Develop a winter construction plan. ○ Coordinate meeting between stakeholders.
Material shortage.	○ Access road maintenance. ○ Material specification for quarried material not met. ○ Quantity production not met (rate of production, waste segregation). ○ Increase in NPAG need for earthwork. ○ Starter pit delay.	○ Secure alternative sources of material. ○ Ensure crushing capacity available on time. ○ Use till from starter pit for pad foundation. ○ Finish evaluation of need and sequencing for NPAG material. ○ Use Starter pit to replace NPAG for non-critical infrastructure. ○ Effective program for waste rock segregation. ○ Quarry/borrow source material characterization.
Cutoff integrity compromised during construction.	○ Poor foundation preparation resulting in slurry loss. ○ Presence of unfavorable ground condition (boulders).	○ Increase the cutoff measures to seal the foundation and the dike. ○ Develop an action plan and integrate construction design.

Risk	Cause	Mitigation plan
	○ Gap between secant piles and/or bedrock. ○ Loss of slurry during secant pile construction (high gradient in WT Lake).	○ Implement a rigorous QA/QC program ○ Contractor will mobilize equipment to mitigate risk of digging at depth.
Unfavorable change of water quality of receiving environment	○ Erosion (infrastructures + shore line). ○ Generation and deposition of dust decreasing clarity or quality of water. ○ Generation and deposition of acidifying air emissions increases acidity of water. ○ Instability (land slide) in the Whale Tail lake south basin. ○ Flooding and exposing shore to wave and ice. Slope instability in shore line associated with permafrost degradation. ○ Flooding of Whale Tail south basin with high organic matter for longer period than planned.	○ Treating the water in South Whale Tail Lake for TSS. ○ Monitoring with trigger levels and adaptive management. ○ Erosion prevention (use appropriate granular material for ditches bedding). ○ Develop freshet management plan. ○ Enforce snow removal in ditches. ○ Management of organic material in flooded basin. ○ Turbidity barrier for SWTDC outlet. ○ Study deflection dike to add setting out time.

16.0 CONCLUSION

The design of Whale Tail Dike was completed to the detailed design level of engineering following codes and standards in the industry. In this mandate, a review of the existing conditions was executed and included the monitoring data for Whale Tail Lake water level, the condition of the foundation based on the latest field investigation, and the evaluation of the thermal regime that exists in the foundation of the future dike. Based on the design developed for WTD the following conclusions can be made:

- Whale Tail Dike is classified as “high” according to the CDA guidelines (2013, 2014) and it is designed to support a flood event representing $^{1}/_{3}$ between 1,000 year return period and the probable maximum flood (Section 4.0).
- The design criteria for Whale Tail Dike are met for all scenarios of static and pseudo-static loading conditions (Section 7.2.2).

- The thermal analysis of the dike indicates that permafrost will develop with time into the embankment of the dike except for the upper portion of the cutoff (Section 7.2.1).
- The thaw settlement analysis revealed that the settlement should be limited if all the ice-rich till is removed (Section 7.2.4).
- The cutoff wall with the proposed CB mix will support the tensile stresses induced during the operation of the dike (Section 7.2.5).
- The dike will be constructed with quarry NPAG material. The gradation limits developed for the material are slightly different than that developed for Meadowbank dewatering dikes but are in accordance with the gradation results obtained from the mine for this type of material (Section 7.4).
- Foundation preparation is a critical activity for the successful performance of the dike and will be used to set the depth of secant piles prior to their construction. The west abutment located in permafrost will be blasted. At the east abutment, the strategy is to expose permafrost and remove the active layer during the summer season. This would reduce expected settlement and the impact of thaw settlement. A thermal berm will also be constructed at the east abutment upstream of the dike to reduce the heat intake from the lake into the upstream and core area of the dike (Section 7.2.1 and 7.9).
- To maintain the continuity of the secant pile wall, an accurate survey of location and verticality of piles should be implemented. If a gap between piles is expected, an extra pile should be installed to insure the continuity of the wall (Section 7.10.2).
- A significant amount of seepage through the bedrock below the secant pile wall is expected. Some reduction of this seepage can be achieved by grouting the bedrock below the secant pile cutoff and requirements for such grouting are presented in this report. However, it is noted that bedrock grouting is not required for hydraulic stability of Whale Tail Dike. The need and quantity of treatment is dependent on AEM's seepage control criteria and further cost analyses between stringent seepage control and additional pumping and water treatment efforts. If required, it is proposed to seal the foundation with a localized grout curtain where permeability of bedrock was evaluated above 10^{-5} m/s (Section 8.0).
- South Whale Tail Diversion Channel will reroute all water from the reservoir that will accumulate upstream of the dike. This channel will be designed with the same design flood as for Whale Tail Dike (Section 10.0).
- Whale Tail Dike will be fully instrumented to continuously monitor the performance of the structure. Thermistors, inclinometers, piezometers and survey monuments will be installed after the completion of the cutoff (Section 12.0).

17.0 PERSONNEL

This report has been prepared by Angie Arbaiza and Yohan Jalbert and reviewed by Les MacPhie and Getahun Haile.

We trust that this report is to your satisfaction. Should you have any question, please do not hesitate to contact us.

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