Design Report of IVR Attenuation Pond D-1 Dike

Detailed Engineering Design of Water Management and Geotechnical Infrastructures
Phase 2 - Whale Tail Project Expansion

Agnico Eagle Mines Limited - Meadowbank Division

Mining & Metallurgy

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Montreal, December 23, 2020

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Subject: Design Report of IVR Attenuation Pond D-1 Dike
Detailed Engineering Design of Surface Water Management and Geotechnical Infrastructure
Phase II - Whale Tail Project Expansion
AEM File: 6127-695-132-REP-005
Our file: 668284-5000-4GER-0001-Rev R0

Dear Sirs,

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 further questions regarding the content of this report.

Truly yours,

SNC LAVALIN INC.

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2020-12-23

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XX/xx

List of Revisions

Revision					Revised pages	Remarks
#	Prep.	Rev.	App.	Date		
R0	NQ/MDZ/MD/HT	ALN/PG	ALN	2020/12/23		Final version

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1.0 INTRODUCTION

1.1 Context

Agnico Eagle Mines Limited, Meadowbank Division (AEM) is developing the Whale Tail Project, a satellite deposit located on the Amaruq property (Kivalliq Region of Nunavut, Canada). The Whale Tail Project construction is ongoing and commercial production has begun in the third quarter of 2019. To continue mining and milling, AEM is proposing to expand the Whale Tail Project by expanding the Whale Tail pit, developing another open pit called the IVR pit and including underground mining operations. As part of the expansion project, new water management and geotechnical infrastructure shall be required for surface water management. AEM has mandated SNC-Lavalin (SLI) to carry out the detailed engineering design of the IVR Attenuation Pond including the IVR D1 Dike and its spillway.

This document presents the detailed design of IVR D-1 Dike and the associated spillway. Figure 1-1 presents the general site layout in plan view of the IVR Attenuation Pond Dike while Figure 1-2 presents the general view of Amaruq site.

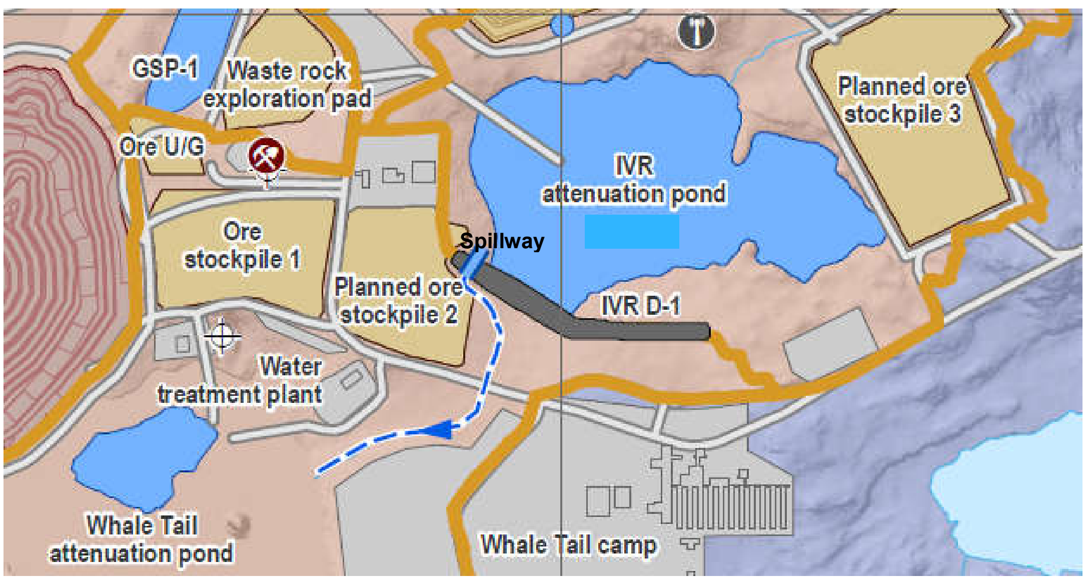
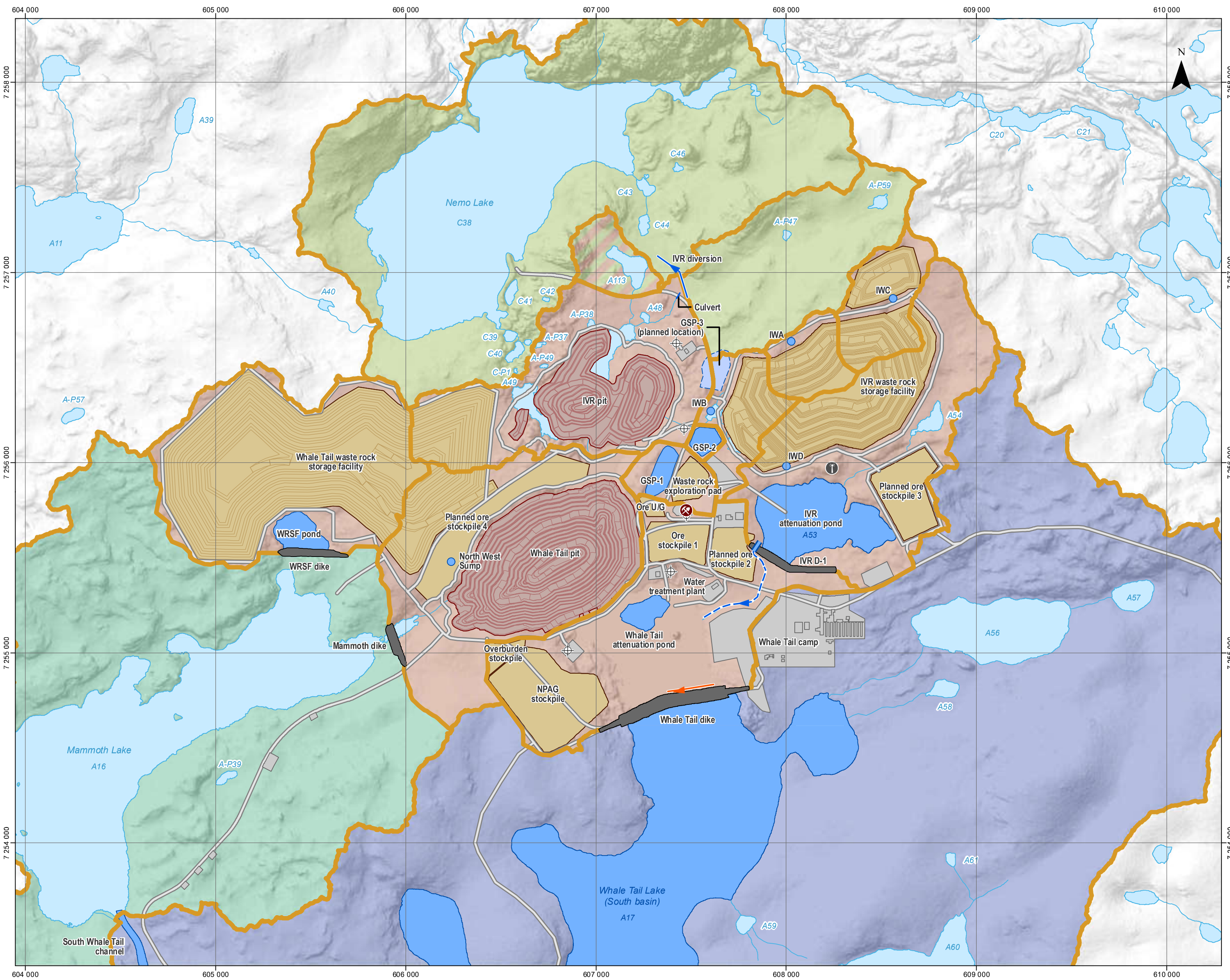


Figure 1-1: Location of IVR Attenuation Pond Dike



PROJECT COMPONENTS (PHASE 2)

- Dike
- Spillway
- Spillway outlet
- Modified waterbody or pond
- Collection ditch
- Diversion ditch
- Sump
- Open pit
- Underground mine portal
- Ventilation raise
- Storage facility
- Industrial area
- Road
- Communication tower

HYDROGRAPHY

- Watercourse
- Lake
- Modified watershed

Contact water

- Attenuation pond and affluents

Non-contact water

- Mammoth Lake
- Nemo Lake
- Whale Tail Lake (South basin)

NOTE :
Ring road around IVR WRSF shown in this figure has been approximately drawn by SNC-Lavalin. Final design will be carried out by AEM.

AGNICO EAGLE

SNC · LAVALIN

AMARUQ GOLD MINING PROJECT
Amaruq Phase II - Water Management & Infrastructure

Mine Layout Watersheds During Operation – Phase II

Sources:
Topography, PhotoSat, 2015
Project components : November 2020

Project: 668284
File: 668284-EG-001-F1-Wshd-201204.mxd

0 200 400 m
1:20 000
UTM projection, Zone 14, NAD83 (CSRS)

December 4, 2020

Figure 1-2

S:\INTC\MineAmaruq\668284 Phase 2 Detailed Engineering\Produit\001-Watershed\668284-EG-001-F1-Wshd-201204.mxd

1.2 Structure of the Report

This report presents the available site specific geotechnical and topographic information, together with interpretation of the factual data and the results of the thermal and stability analyses carried out for IVR D-1 Dike. For completeness, the report includes the technical specifications for IVR D-1 Dike in Appendix C, together with the construction drawings in Appendix D.

2.0 GUIDELINES AND STANDARDS

The design of IVR D-1 Dike is based on Canadian Dam Association (CDA), Dam Safety Guidelines and the American Society for Testing and Materials (ASTM). The Technical Specifications included in Appendix C, provide a list of the ASTM and other construction material standards relevant to the IVR D-1 Dike, hence are not repeated herein.

3.0 AVAILABLE GEOTECHNICAL INFORMATION

A total of five (5) field investigation programs were carried out at the IVR D-1 Dike proposed location. The first field program was carried out between September and October 2019 (AEM, 2019a). The program includes five (5) test pits within the projected footprint of IVR D-1 Dike and twenty-three (23) destructive boreholes with a Tamrock drill rig to infer the depth to bedrock and overburden characteristics. In May 2020, another series of fourteen (14) destructive boreholes were drilled to fill in the gap from previous investigation (AEM, 2020).

As the detailed design progressed, the IVR D-1 Dike proposed location was re-assessed. The dike was moved slightly towards south and downstream side from the original proposed location. In September 2020, a field investigation program comprised of twenty-one (21) test pits was carried either near the cut-off trench or the dike centerline. In addition, eight (8) diamond boreholes drilling were carried out in October 2020 to infer the subsurface conditions at the central part of the dike. Furthermore, in November 2020 a total of fifty-five (55) bedrock probes were put down with the objective to obtain the depth to bedrock at both abutments. The field investigation report (SNC-Lavalin, 2020c) summarizes factual data obtained from these three field supplementary investigations. The information presented herein is a summary of the subsurface conditions obtained from the field investigations. The locations of test pits, geotechnical boreholes, Tamrock boreholes as well as the stratigraphic profile along the dike centerline are shown on Figure 3-1.

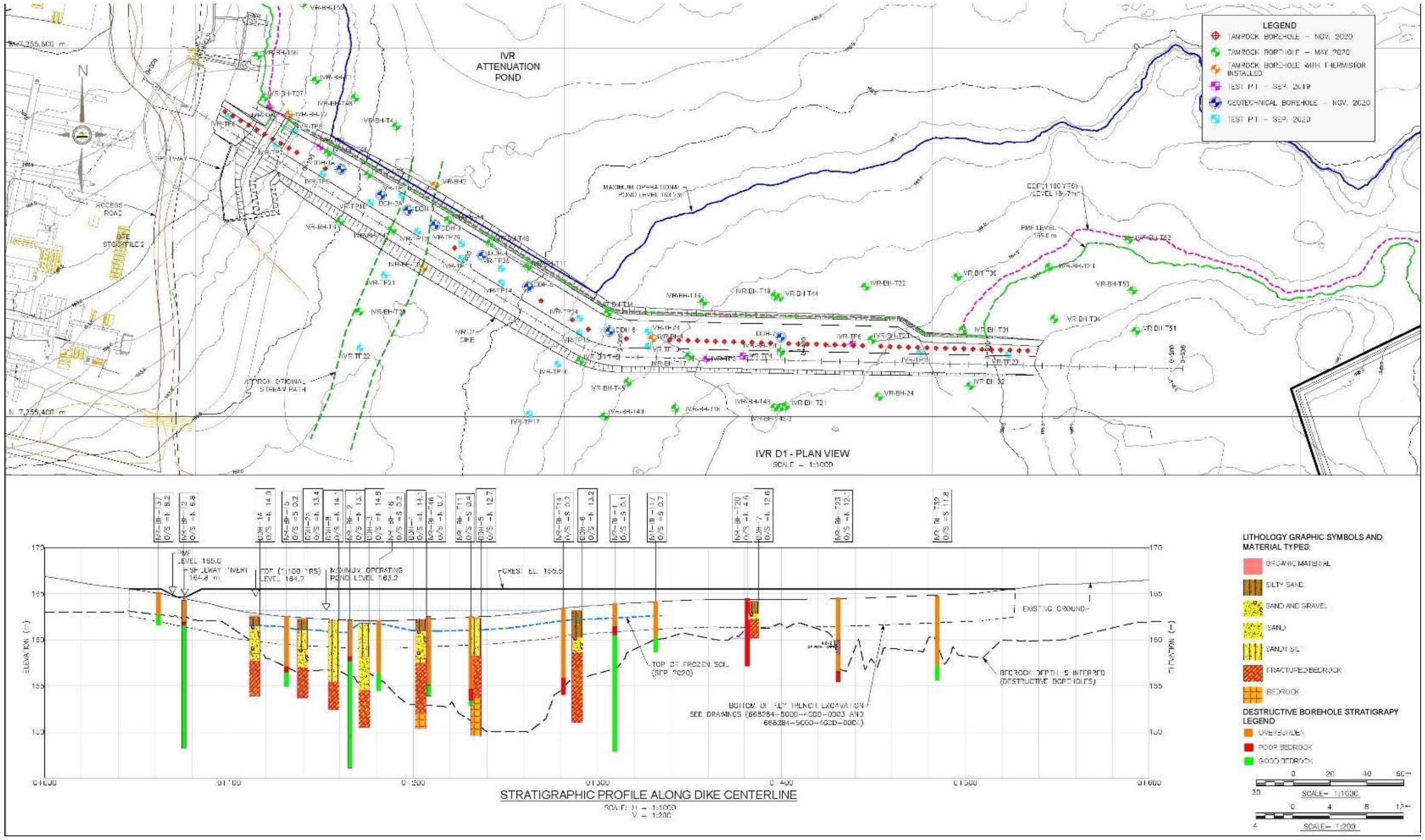


Figure 3-1: Field Investigation Location Plan and Stratigraphic Profile at IVR D-1 Dike

The overburden encountered at the IVR D-1 Dike consists of an organic layer underlain by glacial till followed by bedrock. The depth of the organic layer ranges from 0.1m to 0.3m below ground surface (mbgs). The glacial till unit includes a mixture of various sub-units which are grouped as sand to silty sand, silt to sandy or gravelly silt, gravel to sand and gravel, silt sand and gravel mixture, and sandy silt clay. The thickness of glacial till varies from 1.3 to 7.4 mbgs. Based on the test pit logs and existing thermistor data, the active layer is estimated about 1.5 to 2.1 mbgs. Ice lens were observed at the interface of soil and cobble or soil and bedrock interface. Presence of ice was noted in most test pit logs at a depth around 1.2 to 1.8 mbgs. However, no ice lenses thicker than 10 mm was observed in the soil core retrieved from the geotechnical borehole drilling. The total moisture content (moisture content plus ice content) analyzed from the samples taken from the test pits and boreholes varies from 1.9% to as high as 27%. At Borehole IVR DDH8, located in the middle of the natural outlet, the total moisture content at a depth of 1.5 and 3 mbgs is greater than 30%. Ice rich till is defined as frozen soil that contain more than 10 percent visible ice volume or moisture content is equal or greater than 30% or ice lenses thicker than 10mm. The glacial till is considered as ice-poor till except at the localized spot below 1.5mbgs at Borehole IVR DDH8.

Bedrock was encountered in the geotechnical boreholes ranging from 2.1 to 6.7 mbgs. The bedrock was described as highly fractured/completely weathered to moderately fractured with ice filled joints (10mm at IVR DDH-4 and 50 mm at IVR DDH-6). It is noted thick layer of ice of about 1.5 m thick was observed in one of boreholes drilled on the east side of the dike (IVR BH T23) at about 4.6 mbgs.

Four (4) thermistor strings were installed at depth about 12m in the ground close to the proposed location of IVR D-1 Dike to assess the thermal regime of the foundation (AEM, 2019b). The locations of the installed thermistors are shown in Figure 3-1. The results of monitoring these thermistors show that the permafrost temperature is approximately -8°C and seasonal active layer is approximately 2 m deep in till deposits.

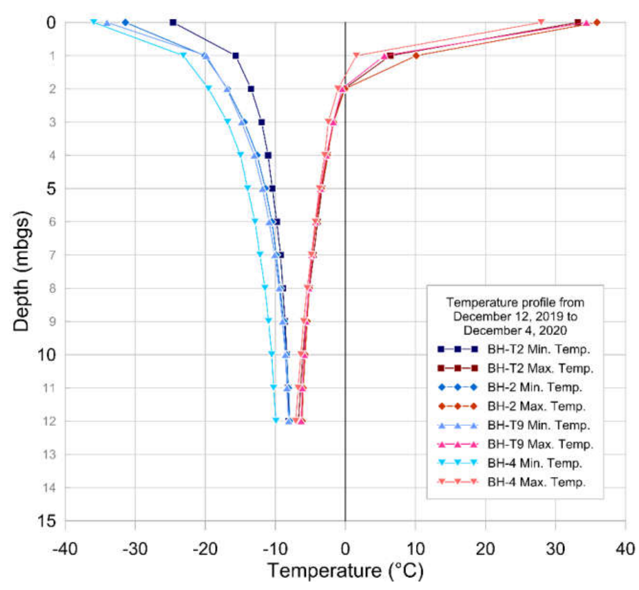


Figure 3-2: IVR D-1 Dike Ground Temperature Profile

4.0 IVR D-1 DIKE DESIGN BASIS AND CRITERIA

The information herein is a summary of the key elements of the design basis and criteria for the IVR D-1 Dike (SNC-Lavalin 2020d).

4.1 Performance Objectives

The IVR Attenuation Pond D-1 Dike shall be designed to achieve the following objectives:

> Provide necessary storage volume in the Attenuation Pond to manage the maximum operating level and contain the environmental design flood (EDF), which is the 1:100-year spring flood.

> Safely convey flood events higher than the EDF and up to the probable maximum flood (PMF) via the emergency spillway.

> The design is to have no seepage bypassing this structure under the maximum pond operating condition (El. 163.2 m).

> If there is seepage bypassing the dike, which may occur when the pond level is above the maximum operating pond level (i.e., during flood events), it can be managed by the downstream Whale Tail Attenuation Pond pumping system.

> Maintain the cut-off trench in a frozen state.

> The dike structure should be stable under all operating conditions.

> The thickness of the dike fill materials should be sufficient to maintain the dike foundation frozen. This will prevent the foundation from being subjected to freezing and thawing cycles which may result in settlement and/or deformations of the dike.

4.2 Hazard Classification

The purpose of the hazard classification (HC) is to assess the risk associated with the rapid release of large volumes of water from a potential failure of a dam/dike. The risk posed by the IVR D-1 Dike was evaluated based on life safety and sound engineering judgment in accordance to the Canadian Dam Association "Dam Safety Guidelines" (CDA, 2013 and 2014). The classification system is summarized in **Table 4-1**.

Downstream of the IVR D-1 Dike, there is a presence of the main site access road, Whale Tail Attenuation Pond, Whale Tail Pit and the Whale Tail Camp. These areas have people working on a permanent basis that could be at risk. In the event of an unexpected dike failure, there is a potential loss of life downstream of the structure. Based on this assessment, a hazard classification of "High" was adopted for this structure. However, this risk is solely contained to people working on the Mine Site and will be mitigated by the implementation of an OMS (operation, maintenance and surveillance) manual and emergency preparedness plan.

As per the CDA Guidelines (CDA, 2013), for a HC of “High”, the corresponding IDF is the 1/3 Flood between the 1:1000-Yr Flood and the Probable Maximum Flood (PMF), and the seismic criterion is the 1:2,475-year return period. For the former, since the capacity of the IVR Pond in combination with the proposed spillway size would be able to manage floods between the 1:1,000-Yr Flood and the PMF with marginal differences in water levels, the PMF was used as the reference condition to ultimately validate the design of the structures.

Table 4-1: Dam Classification Based on Dam Failure Consequences (CDA, 2007 Rev 2013)

Dam Class	Population at risk	Incremental losses [1,2]		
		Loss of life	Environmental and cultural values	Infrastructure and economics
Low	None	0	Minimal short-term loss. No long-term loss	Low economic losses; area contains limited infrastructures or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of important fish or wildlife habitat Restoration or compensation in king highly possible	High economic losses (infrastructure, public transportation, commercial facilities)
Very high	Permanent	100 or fewer	Significant loss or deterioration of critical fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses (important infrastructure or services)
Extreme	Permanent	More than 100	Major loss of critical fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses (critical infrastructure or services)

Notes:
1. Incremental compared to predicted impacts in the same natural conditions (flood, earthquake or other event) but without failure of the dam. The most severe consequence (i.e. life loss or damage) determines the category of the structure. In the case of tailings, a category of consequence must be assigned to each stage: construction, operation, closure.
2. The descriptions shown here are only indicative. See CDA (2007, rev 2013) for the complete descriptions.

4.3 Water Management

For Phase 2 of the Whale Tail Expansion Project, all contact water shall be transferred to the new IVR Attenuation Pond. The IVR D-1 Dike will be designed to provide the necessary storage volume above the maximum operating level to manage contact water and surface water produced by the EDF, a 1:100 year return period spring runoff, and safely convey the PMF via the emergency spillway.

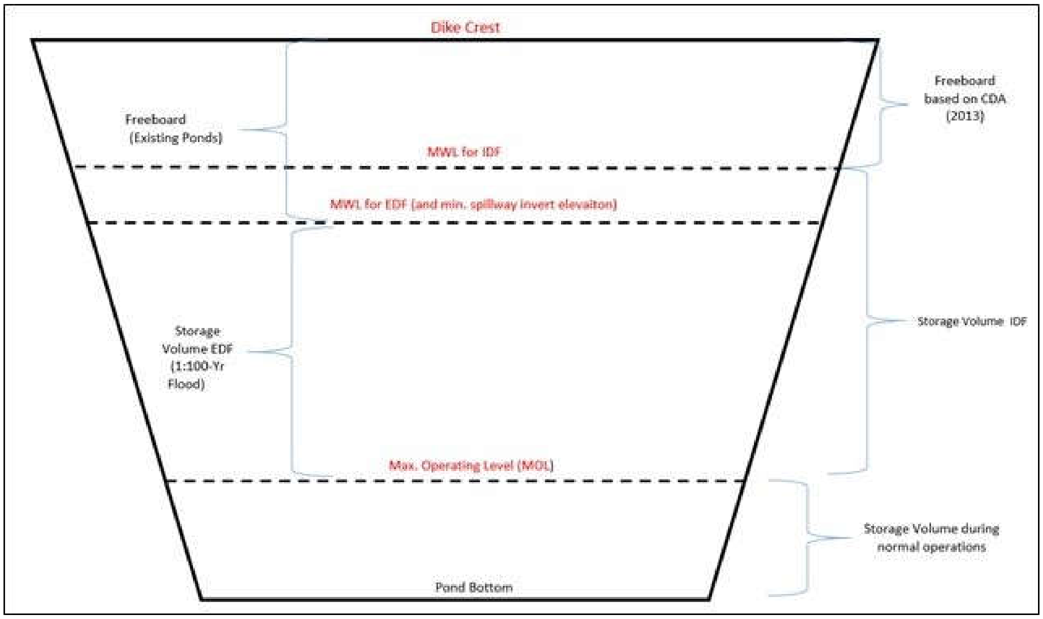
Figure 4-1 presents a schematic water levels for the Attenuation Pond. The operating levels were based on the pond capacity and pumping operations that were established using flood routing analysis during the spring freshet conditions based on the following:

- An initial water elevation in the pond equal to the maximum operating level (MOL) of 163.2 m, based on the average water balance of the site (Golder, 2019). This maximum water level is the level acceptable during winter conditions only. During summer, the normal operating water level will be set below the El. 163.2 m.
- Inflows to the IVR Attenuation Pond including pumped transfer water from Whale Tail Pit, Whale Tail Attenuation Pond, Whale Tail WRSF, IVR Pit, IVR WRSF sumps and natural runoff to the pond.
- Contact water stored in IVR Attenuation Pond is then pumped to the Water Treatment Plant (WTP) for treatment prior to discharge to Mammoth Lake or Whale Tail South Basin.

Based on this assessment, the following operating water elevations were defined:

- Environmental Design Flood level: El. 164.7 m.
- Emergency spillway elevation: El. 164.8 m.
- PMF level: El. 165.0 m.
- Minimum crest elevation of El.165.5 m to account for wave runup and wind setup (0.5 m freeboard).

Figure 4-2 provides the stage-storage curve for the IVR Attenuation Pond. Details on how the EDF and IDF water levels were established can be found in the Hydrological Analysis Update for Water Management Infrastructure Design Report (SNC-Lavalin, 2020a).



MOL = Maximum Operating Level
EDF = Environmental Design Flood
MWL = Maximum Water Level
IDF = Inflow Design Flood

Figure 4-1: Schematic Water Levels

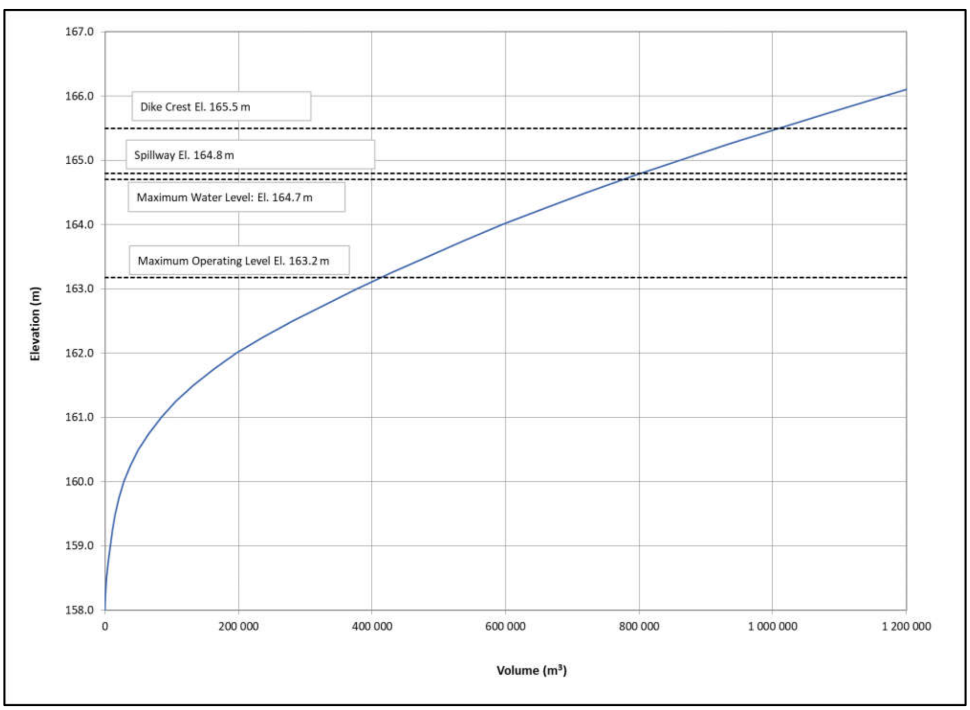


Figure 4-2: IVR Attenuation Pond Storage Curve

4.4 Design Basis for Dike Design

The following elements were considered during the design of the IVR D-1 Dike:

> Dike construction will commence by mid-February 2021 and be completed by end of April 2021;

> The operating service life of the structure is 15 years (2021-2036), after year 2036 the IVR Attenuation Pond will be backfilled;

> The IVR D-1 Dike will be constructed with non-acid generating, non-metal leaching rockfill and will be provided with a seepage barrier on the upstream side. This seepage barrier consists of Low Linear Density Polyethylene (LLDPE) liner on the upstream slope of the dike anchored in the cut-off trench in a layer of Fine Filter Amended with Bentonite (FFAB);

> The cut-off trench will be excavated to a minimum depth of 3 mbgs, based on the field investigation data, to ensure that the liner is anchored in frozen ground; and

> The IVR D-1 Dike is designed to have a frozen cut-off trench.

4.5 Design Basis for Spillway Design

The emergency spillway will allow a safe release of flood events greater than the EDF and up to the PMF. Any release from the emergency spillway shall be directed towards the Whale Tail Attenuation Pond. The emergency spillway consists of a spillway control structure, a spillway-chute (channel) and an energy dissipator.

The spillway control section consists of a trapezoidal-weir structure with a 5.0 m wide base and 10H:1V side slopes. The design is part of the flood routing analysis that is included in the Hydrological Analysis Update for the Water Management Infrastructure Design Report (SNC-Lavalin, 2020a).

Design basis for the spillway-chute and energy dissipator are listed as follows:

> Control section consist of a broad-crested weir with an invert elevation at El. 164.8 m and a dike crest elevation at El. 165.5 m.

> The emergency spillway will be designed to handle the PMF event.

> The spillway-chute will be a trapezoidal channel shape with maximum side slopes of 10H:1V at the upstream section and at 3H:1V at the downstream section (belong the dike structure); and

> An energy dissipater located at the end of the spillway-chute will be designed following the recommendations in USACE (1980) and USBR (1984). The total length of the dissipation basin will be estimated based on Type I structures USBR (1984) by applying the Momentum Equation. Erosion protection is to be provided at the end of the spillway's outlet to protect the natural terrain against erosion/scouring. The layer is to be a minimum ten (10) times the flow depth in the dissipation basin.

4.6 Stability Criteria

Based on the IVR D-1 Dike hazard classification describe in Section 4.2, the seismic parameters for a "High" dam classification were applied. The annual exceedance probability (AEP) criterion for the earthquake return period for the dike is 1:2,475 yr.

The stability criteria adopted are provided on **Table 4-3** which follows the CDA (2013) guidelines. The effect of the earthquake loading on the dike was simulated by the pseudo-static analysis method using seismic parameters for a 1:2,475 yr event. The seismic coefficient was generated using the Geological Survey of Canada's (GSC, 2014 online hazard calculator). **Table 4-4** provides the seismic coefficient generated and used in this study.

Table 4-2: Minimum Factor of Safety

Loading condition	Minimum Factor of safety
End of construction before reservoir filling	1.3
Static analysis – long-term	1.5
Full or partial rapid drawdown	1.2 to 1.3
Pseudo-static (k_h = 0.027)	1.0

Table 4-3: Seismic Coefficient

	Design Earthquake Probability of Exceedance		PGA (1) (GSC, 2015)			
	per annum	In 100 years	Firm ground (2)	Rock/Soil Boundary (3)	Hard Rock (4)	k_h (5)
IVR D-1	1/2,475	**4 %**	**0.053 g**	**0.044 g**	**0.054 g**	**0.027**

Notes:
1. Peak ground acceleration given in units of g (9.81 m²/s), where g is the acceleration of gravity.
2. NBCC 2015 Site Class C with an average shear wave velocity of 450 m/s.
3. NBCC 2015 B/C Limit with a shear wave velocity of 760 m/s. Value estimated using the conversion factors provided by GSC (2014).
4. NBCC 2015 Site Class A with an average shear wave velocity greater than 1,500 m/s. Value estimated using conversion factors recommended by Atkinson and Adams (2013).
5. Horizontal seismic coefficient. $k_h = 0.5 \times PGA_{\text{Hard Rock}} / g$

Considering the limited height of the dike in the order of 3.5.m, the use of ice-free fills and the fact that the thermal analyses show the cut-off trench will remain frozen throughout the lifespan of the dike, the effect of thaw deformation on the stability is considered negligible.

5.0 IVR D-1 DIKE DESIGN

5.1 Thermal Design

Thermal analyses were conducted with the objective of assessing the effect of the construction of the IVR D-1 Dike on the thermal regime of the dike and its foundation over the lifespan of the structure. A summary of the thermal analyses is presented below and the details of the thermal analyses are included in the Assessment of Ground Thermal Regime at IVR D-1 Dike Technical Note in which is included in **Appendix A** of this report.

Two dimensional thermal analyses were conducted using TEMP/W, a finite element computer program from the Geostudio software suite (version 8.16.2.14053, GEO-SLOPE 2016). It should be noted that this thermal modelling software does not take into account convective heat transfer.

The four (4) existing thermistor strings at the proposed location of the IVR D-1 Dike provided the ground temperature data which were used for the initial calibration of the model in conjunction with air temperature data recorded at the Amaruq weather station. The geothermal properties of the overburden (ice-poor glacial till) were based on the available geotechnical data retrieved from the field investigations, whereas the geothermal properties of the construction material were based on the as-built data available from the Phase 1 construction.

A conservative approach was followed by using the warmest year of the last decade recorded at the Meadowbank weather station for the thermal modelling of the IVR D-1 Dike while in operation. The Attenuation Pond water level was also considered at its maximum operating level of El. 163.2 m throughout the year except during the EDF event which occurs during spring / summer for a duration of 45 days where the pond water level reaches El. 164.7 m. This approach is considered to be conservative since the modelled water levels are higher than those presented in the annual water balance (Golder, 2019) which assumes a low water level during summer. The presence of a higher water level in the model translates into heat intake from the pond, which affects the thermal regime by promoting thawing of the foundation.

The thermal modelling was conducted using transient analyses simulating 15 years of operation at different locations along IVR D-1 Dike. Two (2) cross sections and one (1) longitudinal section were studied to have a comprehensive understanding of the thermal regime across the dike footprint.

The base case thermal analysis was conducted at the critical section at station 0+172 located at the lowest natural ground elevation of 162.0 m. The selected cross section corresponds to the preliminary design developed at the early design stage where the width of the esker thermal berm was 6 m. The thermal regime after 15 years of operation is shown on **Figure 5-1**.

The results show that the connection weld of the geomembrane at elevation 161.5 m would remain marginally frozen after 15 years of operation. Permafrost degradation can also be observed upstream of the IVR D-1 Dike due to the presence of the IVR Attenuation Pond. Nevertheless, the bottom of the cut-off trench is expected to remain frozen at a temperature below -3°C.

However, uncertainty remains as to the efficiency of the esker berm to minimize the convective heat transfer towards the cut-off trench. As mentioned previously, only heat transfer by conduction is considered in the current study; if heat transfer became significant, the thermal regime estimated on **Figure 5-1** would underestimate the actual foundation and dike temperatures.

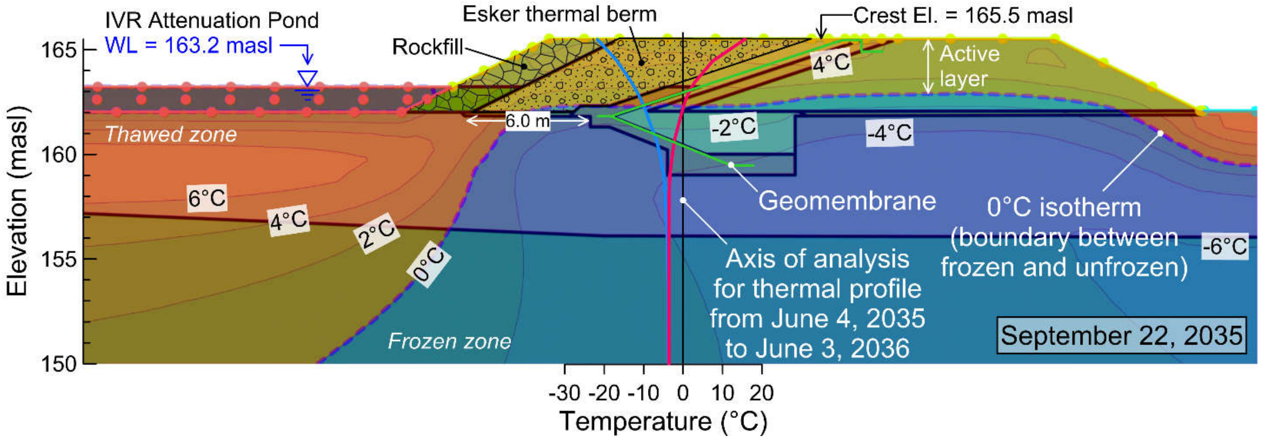


Figure 5-1: Thermal regime as of September 22, 2035 including temperature profile of the upstream berm and cut-off trench with a 6 m wide thermal berm

Because the thermal berm is entirely thawed at the end of summer and the rockfill material is pervious, the only barrier remaining between the pond and the cut-off trench is the unfrozen esker material. If the esker material has a lower fines content than expected, thus having a higher hydraulic conductivity, the effect of convective heat transfer may be more significant than anticipated in the current thermal regime assessment. To account for this uncertainty, another variation of the cross section was studied where the upstream thermal berm width of the esker was extended to 12 m, thus pushing the water pond farther away from the cut-off trench. The results are presented on **Figure 5-2**.

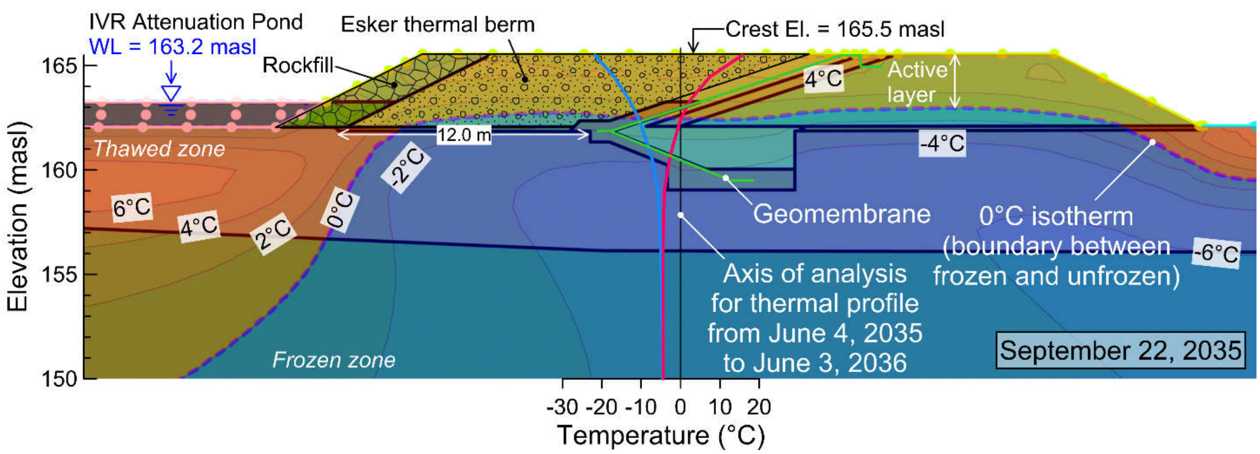


Figure 5-2: Thermal regime as of September 22, 2035 including temperature profile of the upstream berm and cut-off trench with a 12 m wide thermal berm

The difference in thermal regime between the two geometries was assessed during the last year of operation to evaluate the benefits of the construction of a wider thermal berm in the long term. By comparing the thermal regime during the last year of operation for the 6 m wide berm **(Figure 5-1)** and for the 12 m wide berm **(Figure 5-2)**, a clear difference in the thermal regime of the cut-off trench can be observed. The configuration using the 12 m wide upstream thermal berm shows lower temperatures in the cut-off trench while the base of the thermal berm remains in a frozen condition. If the esker material contains less fines than expected, which would lead to higher thermal conductivity of the unfrozen material, the risk of warming the cut-off trench by convective heat transfer from the IVR Attenuation Pond is mitigated by constructing a wider thermal berm.

The benefits or building a 12 m wide berm are then twofold: it contributes to a cooler foundation while minimizing the risk of warming the foundation and the cut-off trench due to the presence of water in the IVR Attenuation Pond.

A second cross section was studied at station 0+360 where the natural ground elevation approximately reaches 164.0 m (refer to Section D-D on Drawing 668284-5000-4GDD-0005). At this location, the water stored in the Attenuation Pond does not reach the upstream toe of the dike during normal operation. The results of the thermal modelling showed that the construction of a thermal berm at a constant elevation of 165.5 m promoted thawing of the upper part of the cut-off trench as the thickness of the thermal berm decreases when the natural ground elevation increases. However, the bottom of the cut-off trench should remain in the frozen state as there is no significant permafrost degradation from the upstream side due to the presence of the Attenuation Pond. Nevertheless, the installation of thermistor strings within the cut-off trench at different locations along the IVR D-1 Dike would be beneficial to follow up the temperature of the cut-off trench, which is a key indicator of the performance of the structure. If temperatures close to the freezing point are observed in the cut-off trench, the construction of an additional layer of esker on top of the thermal berm at the end of the following winter may prove to be a proper mitigation measure to promote cooling of the cut-off trench.

Finally, thermal analyses were carried out a longitudinal section of the west abutment along the centreline of the dike (refer to Section F-F on Drawing 668284-5000-4GDD-0005). The objective was to design the abutment in such a way that the FFAB in the cut-off trench remains frozen even when the bottom of the excavation reaches the natural ground elevation. The thermal analyses suggest that constructing a 2.5 m thick berm with rockfill material would promote cooling of the foundation and keep the FFAB at the surface marginally frozen in the abutment areas.

5.2 Seepage Considerations

Seepage analyses have not been carried out in this study for the following reasons:

> Thermal analyses (SNC-Lavalin 2020e) shows that the cut-off trench at the critical area where the water reaches the dike toe during the maximum operating level (El. 163.2 m) remains frozen all year round including during the EDF pond levels, which meets the design objective of keeping the cut-off trench frozen. At higher ground (i.e., where the ground surface is above El. 163.2 m), the bottom of the cut-off trench is expected to stay in the frozen state.

- The upstream berm, the cut-off trench and downstream foundation will be instrumented with thermistor strings to monitor the thermal regime of the dike foundation. The frozen state of the cut-off trench will be monitored closely by frequent thermistor string readings; and
- If seepage is observed along the downstream toe of the dike, the water will flow towards the Whale Tail Attenuation Pond and will be sampled in compliance with the Water License 2AM-WTP1826.

5.3 Dike Materials

Only rockfill and granular fills that is Non-Potential Acid Generating (NPAG) and non-metal leaching (NML) shall be used for the construction of the IVR D-1 Dike. The NPAG/NML rock shall be sourced from waste rock material from Whale Tail Pit or IVR Pit that has been tested in the laboratory. Waste material is considered NPAG/NML when:

- It contains less than 0.1 wt.% total sulphur, regardless of its Neutralizing Potential (NP) value;
- It contains more than 0.1 wt.% total sulphur, and the calculated carbonate Net Potential Ratio (NPR) value is greater than 2; and
- The average total arsenic < 75 ppm.

The same quality assurance/quality control (QA/QC) program currently in use at Meadowbank shall be used in the sample analysis of the waste materials from Whale Tail Pit, which includes the use of certified reference materials and duplicate analyses by an accredited external laboratory.

Materials for construction of the IVR D-1 Dike and its spillway are to be as shown on the drawings and are identified by a zone number together with an abbreviated description as follows:

- Zone 1: Rockfill (minus 1,000 mm)
- Zone 2A: Fine Filter (minus 19 mm)
- Zone 2B: Fine Filter Amended with Bentonite (FFAB) (minus 19 mm)
- Zone 3: Coarse Filter (minus 150 mm)
- Zone 4: Esker

The descriptions of the fill materials and the requirements for placement and compaction are presented in the technical specifications (**Appendix C**) and are not repeated herein.

In addition, the construction requirements for the LLDPE liner, geotextile and bituminous geomembrane (BGM) are provided in the technical specifications.

5.4 Stability Analyses

5.4.1 Methodology

The slope stability analyses were carried out using a geotechnical software, GeoStudio 2016 version 8.16.5.15361. The software applies limit equilibrium formulation based on method of slices. Morgenstern-Price method has been selected for the interslice force calculations to satisfy both moment and force equilibrium for the determination of factor of Safety (FOS) against slope failure.

The analyses address both static and seismic loading conditions. For seismic stability analysis, a pseudo-static analysis was carried out. In this method, the dynamic force during an earthquake are equated to horizontal force that is expressed as the product of the seismic coefficient and the weight of the potential sliding mass.

5.4.2 Typical Section

Drawing 668284-5000-4GDD-0003, in Appendix D, shows the plan view and Drawing 668284-5000-4GDD-0005 shows sections of the IVR D1 Dike. The stability analyses have been carried out on the most critical dike section (Section BB) which has the following features:

- Height of dam about 3.5 m;
- Maximum overburden thickness of about 6 m; and
- Ground water is assumed to be on the ground surface (El. 162 m).

It is noted that the stability analyses were carried out on the former version of the typical section BB where riprap was placed on the upstream slope and on top of the thermal berm. In the final version of the typical cross section, riprap is replaced with rockfill. This modification does not have an impact on the dike stability.

5.4.3 Geotechnical Parameters

Table 5-1 presents the geotechnical parameters of the dike materials and foundation used in the stability analyses. The strength parameters for the foundation overburden are the same as those assumed for the Waste Rock Storage Facility (SNC-Lavalin, 2018). The strength parameters for frozen till were proposed using data provided in GSC (1999). The thawed till layer was assumed to be 3 m thick to be conservative. For the rockfill / coarse filter, the shear strength is assumed to be a function of the normal stress and the strength function presented on **Figure 5-3** is used. The normal shear relationship shown on **Figure 5-3** is used simply to eliminate shallow slip surfaces.

Table 5-1: Geotechnical Parameters of the Dike Materials and Foundation

Material	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Rockfill/Riprap/Coarse Filter	22.2	0	(Note 2)
Fine Filter/Esker	22.2	0	32
Thawed till	20	0	30
Ice poor till (1)	19.6	4	31.5
Bedrock	Impenetrable		

Notes:

1. Ice poor till parameters were obtained from Norman Wells pipeline (1999) report; and
2. Shear/Normal strength function 37^0 and 30^0, see the plot in red on Figure 5-1.

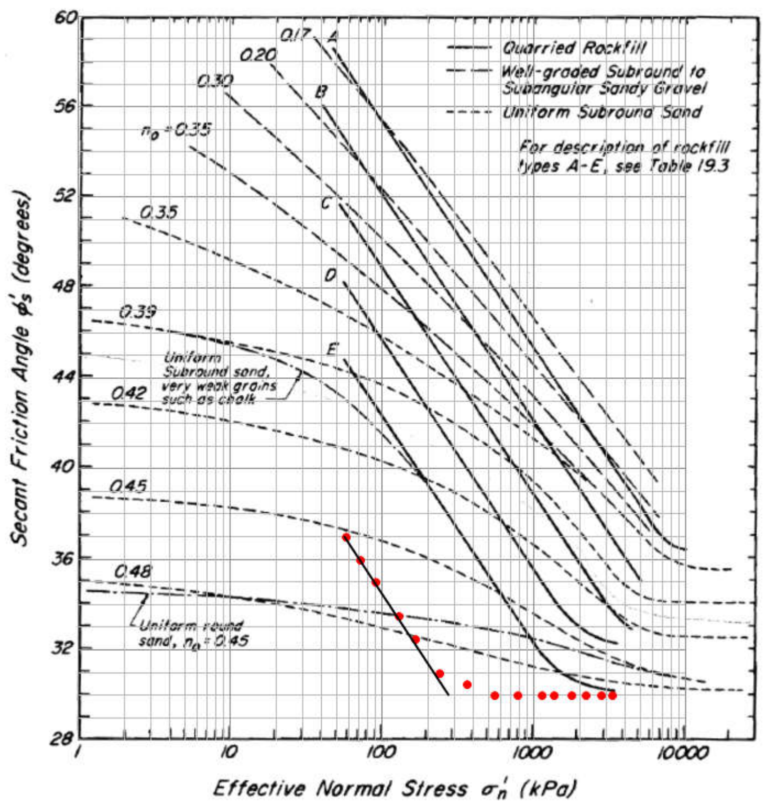


Figure 5-3: Values of Friction Angle for Rockfill/Granular Soils (Terzaghi et al, 1996)

5.4.4 Results

The results of the stability analyses are summarized in Table 5-2 and on Figures 1 to 11 in Appendix B. The results of the analyses indicate the Factor of Safety meet the required minima.

Table 5-2: Stability Analyses Results

Loading conditions	Slope	Water level	Factor of Safety required	FOS obtained	Figure
Static analysis – End of construction	Upstream	Natural ground = 162 m	1.3	1.58	B-1
	Downstream			1.88	B-2
Static analysis – long term	Upstream	Maximum operation level = 163.2 m	1.5	1.51	B-3
	Downstream			1.88	B-4
Static analysis – long term	Upstream	PMF flood = 165 m	1.5	1.74	B-5
	Downstream			1.88	B-6
Full or partial rapid drawdown	Upstream	163.2 to 162	1.2 to 1.3	1.31	B-7
Pseudo static – End of construction	Upstream	Natural ground = 162 m	1.0	1.47	B-8
	Downstream			1.75	B-9
Pseudo static – long term	Upstream	Maximum operation level = 163.2 m	1.0	1.39	B-10
	Downstream			1.61	B-11

5.5 Dike Design

IVR D-1 Dike will be composed of hard and non-acid generating rock, with the upstream slope to be lined with a textured LLDPE geomembrane embedded in the fine filter zone on the slope, and in a layer of fine filter amended with bentonite (FFAB) within the cut-off trench. The fine filter is in turn separated from the rockfill by a transition zone referred to on the design drawings as coarse filter. Non-woven geotextile will be placed above and below the LLDPE liner to prevent any potential damage from the angular materials. The maximum particle size for the fine filter was limited to 19 mm to ensure no damage to the LLDPE liner during construction. The LLDPE liner and the FFAB layer were designed as a composite lining system to retain water up to the maximum operating level of El. 163.2 m. Beyond this level, there will be no FFAB and the LLDPE will be anchored at the crest elevation of El.165.5 m.

A thermal berm consisted of compacted esker will be built against the upstream side of the dike to push water away from ponding at the dike toe and to serve as an insulation blanket to keep the foundation from permafrost degradation. Erosion protection material will be placed on the upstream slope of the thermal berm. It was planned to place rip rap initially and this has been substituted to rockfill to the absence of rip rap at the site. As part of the site surveillance program, the upstream slope will be monitored by the site operational team and repairs will be carried out, when required, if any erosion is observed at the upstream toe. Figure 5-4 presents the dike configuration taken from cross section B-B.

The liner will be keyed into a cut-off trench which will be excavated/blasted in frozen till to approximately 3 mbgs to ensure that the liner and base of the cut-off trench remains frozen. The cut-off trench will be backfilled with compacted esker. This layer will support the liner from potential differential settlement and combined with winter construction, the layer will promote freeze back to maintain a frozen condition of the foundation.

The dike crest is set at the minimum elevation of El. 165.5 m, which was determined in the Hydrological Analysis Update for Water Management Infrastructure Design Report (SNC-Lavalin, 2020a). This crest elevation provides a 0.5 m freeboard above the PMF elevation. Regular inspections will be carried out and if settlement is observed, AEM will carry out remedial works to re-establish the dike crest elevation.

The liner anchor trench at the dike crest will be exposed without cover protection to allow for flexibility in raising the dike in the future, if required, as requested by AEM.

The crest of the thermal berm (comprised of esker material) was set at a fixed elevation of El. 165.5 m to facilitate site construction. Also, the crest of the upstream thermal berm was not provided with an erosion protection cover to allow for the flexibility to raise the berm, if required, in the future to provide the necessary thermal protection. Based on discussions with AEM, it was decided to minimize the thickness of the thermal berm at the higher ground (above the maximum operating pond level of 163.2 m) and to use the observational approach to monitor the thermal regime of the dike structure. With this in mind, it is planned to install thermistors within the dike structure and foundation to monitor the ground temperature during post construction (see Section 8 for additional details regarding site instrumentation). Based on the observed temperature data, the berm may be required to be raised in the future to ensure that the cut-off trench remains frozen, thus achieving the design objective.

In addition, the thermal modelling results (see Section 5.1 and the detailed results are presented in Appendix A) confirm that the bottom of the cut-off trench remains in a frozen state, thus validating the design performance objective. However, lowering the thickness of the thermal berm in the higher ground area (i.e., reduced fill height) may impact the performance of the FFAB around the connection weld since it may be subjected to seasonal thaw and freeze cycles. The effectiveness of the FFAB to act as a low permeability element may be impacted as part of the composite lining system.

Similarly, the thermal berm at the abutments, as recommended in thermal modelling technical note (Appendix A), may be required based on the ground temperature monitoring data obtained during post construction.

Drawings 668284-5000-4GDD-0003 and 0005 show the dike in longitudinal profile and typical cross sections. Figure 5.4 presents the dike configuration taken at cross section BB.

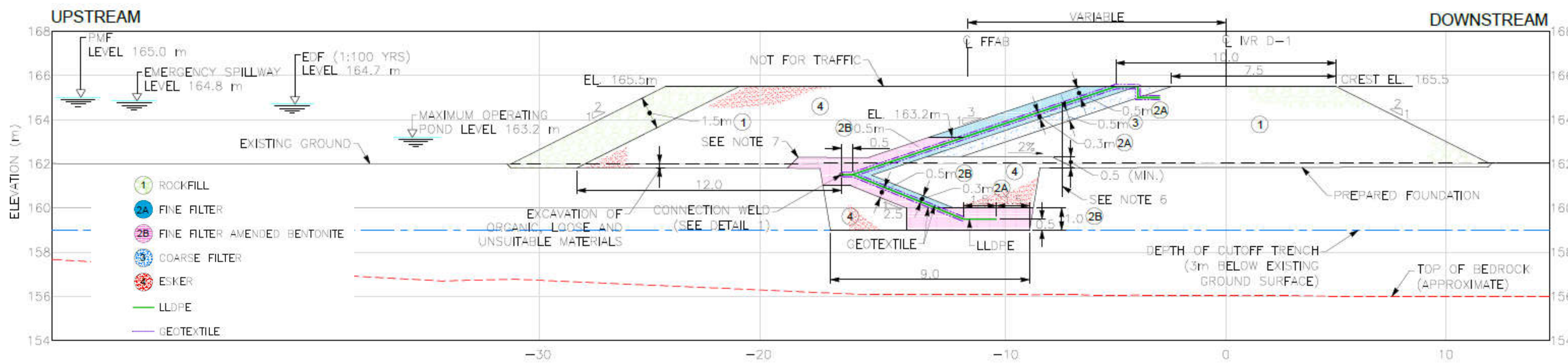


Figure 5-4: IVR D-1 Dike Configuration

5.6 Spillway Channel and Energy Dissipator Design

An emergency spillway is located at the right abutment of the dike to safely convey flood events larger than the EDF and up to the PMF. It be noted that the IDF for the spillway is the flood located 1/3 between the 1:1000 year and PMF. However, AEM has selected the PMF design criterion for the spillway design.

5.6.1 Spillway – Control Section (Weir)

As indicated in Section 4-5, a broad-crested weir of 5.0 m wide and 10H:1V side slopes was established in SNC-Lavalin (2020a) to control the outflows from the IVR-Pond. The design of this structure was based on the spring PMF which was more critical than the summer/fall PMF. The spring EDF was determined to be at El. 164.7 m and the invert elevation of the spillway was set slightly higher at El. 164.8 m.

5.6.2 Peak Outflow

As per the design approach, the spillway-chute structure shall safely convey the PMF from the IVR Attenuation Pond into the Whale Tail Attenuation Pond. Various simulations were carried out in that analysis where the peak water level in the IVR Pond resulted close to El. 165.0 m. At this elevation, the peak outflow from the weir was estimated to be approximately 2.1 m^3/s which was used for the design of the spillway-chute and the energy dissipator.

5.6.3 Spillway – Chute

To design the spillway-chute, a channel-bed slope was projected from the sill (El. 164.8 m) to the ground surface (El. 163.5 m). The slope between these two elevations would be approximately 3.1% that will allow the discharge into the existing valley away from the downstream toe of the IVR-Dike. The proposed spillway would be 5.0 m wide with side slopes of 3H:1V. The configuration of the spillway chute was oriented about 45 degrees to the southeast so that the discharge flow would not be directed to the downstream toe of the dike to avoid any potential erosion of the structure. Also, the alignment may be adjusted at the time of construction as to not interfere with the access road and downstream pipeline.

The spillway was initially proposed to be protected with rip rap material. However, following design discussions with AEM and due to material availability, it was decided to use a bituminous geomembrane (BGM) instead as the protection layer. Coletanche, the BGM supplier, indicated that the “Coletanche ES2” can sustain high flow velocities in the spillway design. A BGM was used to line spillway chute downstream of the spillway sill. This section was validated using the Manning equations for a trapezoidal channel and a Manning “n” of 0.012[1] which corresponds to BGM liners. The results of the calculations are summarized in **Table 5-3.**

The results indicate that a proposed spillway with the characteristics described above would safely convey the PMF. The minimum channel depth at the spillway-chute resulted to be 0.8 m, including a super-elevation estimated, as per recommendations stated in USACE (1994) for the channel bend. However, a minimum of

1.0 m of freeboard is recommended based on the constructability standpoint. Also, the estimated shear stress was found to be significantly below the maximum recommended value by the BGM vendor (2,000 N/m^2). Figure 5-5 presents a schematic profile-section of the spillway structure.

Table 5-3: HEC-RAS Model Results for the Spillway-Chute Design

Spillway-Chute Design	
Inflow (m^3/s):	2.1
Max Flow Depth (m):	0.12
Max Flow Velocity (m/s):	3.4
Super-elevation (m):	0.4
Freeboard (m):	0.3
Min Channel Depth (m):	0.8
Shear Stress (N/m^2):	33.2

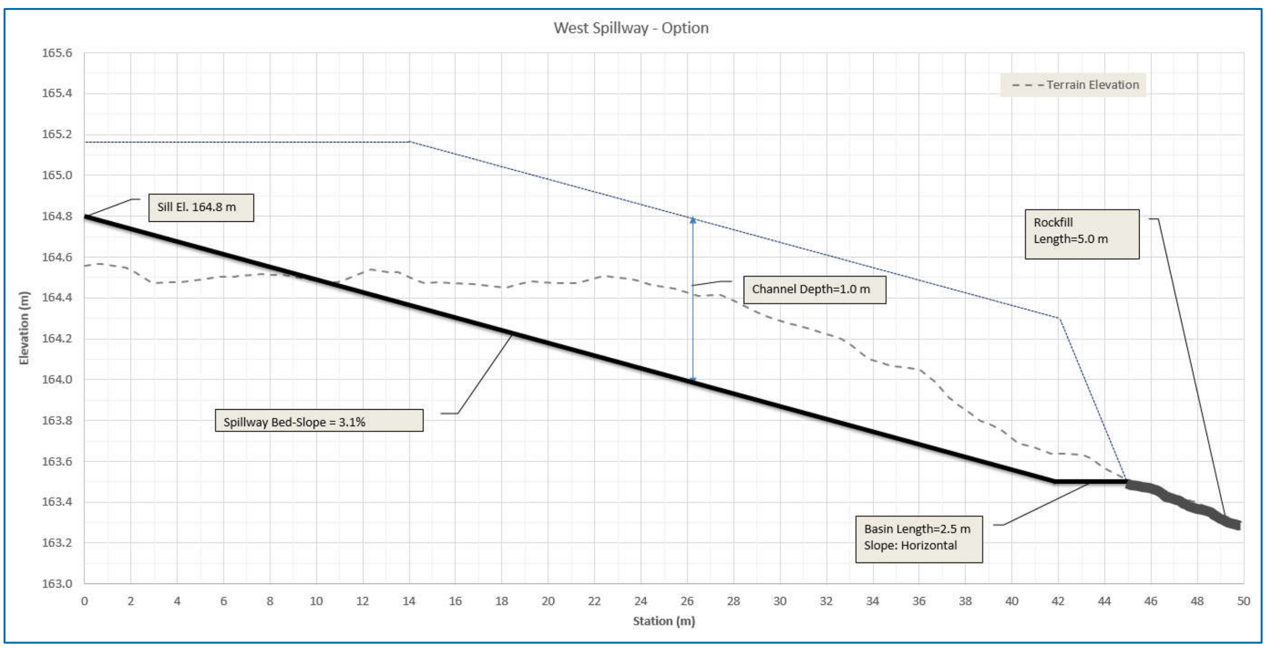


Figure 5-5: Spillway Structure

5.6.4 Energy Dissipator

An energy dissipator is proposed at the end of the spillway-chute to dissipate the flow velocities before discharging into the natural terrain, downstream of the structure, where the outflow is then directed toward the Whale Tail Attenuation Pond.

The structure consists of a dissipation basin Type I which dimensions were estimated out of the recommendations in USACE (1980) and USBR (1984). The minimum suggested length of the structure should be minimum 2.5 m. A layer of rockfill of 5.0 m long with an average size (D_{50}) of 150 mm is recommended to be placed at the outlet of the dissipator section. This is presented in Figure 5-5. It is noted that the rockfill may be subjected to erosion during the extreme flood event during the activation of the spillway, however, this would not impact the performance of the spillway and this would repaired as part of the site maintenance program implemented by AEM. Details of the above structures are presented in Drawing 668284-5000-4GDD-0006.

6.0 ENVIRONMENTAL CONCERNS

During the construction of IVR D-1 Dike, excavated materials will be managed in conformity with the environmental policy of AEM. Any water that needs to be managed during the construction of IVR D-1 Dike shall be transferred to the Whale Tail Attenuation Pond for storage and treatment.

7.0 CONSTRUCTION

The construction of IVR D-1 Dike is scheduled to start in mid-February 2021 and be completed at the end of April 2021, ahead of the spring freshet. The proposed construction sequence is shown on Drawing 668284-5000-4GDD-0008 and this may be adjusted during the time of construction based on actual site conditions. Technical details and QA/QC requirements are presented in the Technical Specifications (Appendix C).

8.0 INSTRUMENTATION

IVR D-1 Dike will be instrumented with thermistor strings to monitor the thermal regime of the dike materials, foundation with a special focus on the cut-off trench.

The design rationale for the thermistors is the following:

> Thermistor strings will be installed on the upstream side of the dike through the esker thermal berm and into the natural foundation to monitor the thaw front development.

> Thermistor strings will be installed in the cut-off trench and in the natural foundation to monitor the temperature conditions on the downstream side of the cut-off trench and the liner.

- In the downstream shell of the dike and natural foundation, thermistor strings will be installed to monitor the foundation temperature changes.

Based on the operating level levels and dike design, four sections were selected for thermal monitoring as shown on Drawing 668284-5000-4GDD-0007. These sections are the following:

- Section B-B: Deepest part of the basin within the maximum operating pond level (El. 163.2 m). This section has four thermistor strings, three (3) of them as noted above plus a fourth thermistor string are planned to be installed adjacent to the liner surface on the upstream side to observe thermal regime in this critical section.
- Section C-C: This section is at the limit of the maximum operating pond level of 163.2 m. Four thermistor strings are installed in the location as Section B-B. It should be noted that at this section the upstream berm thickness is less than the required amount for thermal protection.
- Sections D-D and E-E: These sections are located above the maximum operating pond level. Two thermistor strings were added at each section (upstream side and cut-off trench) to monitor the thermal regime of the dike in the higher ground.

Installation and monitoring of the thermistors are critical to observe the performance of the dike structure, with a focus on the cut-off trench. The cut-off trench is required to be frozen to ensure that no seepage bypasses the structure. Based on the observed monitoring data, implementation of remedial measures, such as raising the berm / dike crest, will be evaluated in the future, if required.

In addition, at each section line, it is recommended to install survey control monuments at the upstream berm and at the downstream rockfill shell to monitor the settlement of the dike structure. These will be critical at Sections D-D and E-E since the foundation may be subjected to repetitive freeze/thaw cycles since the thickness of the esker berm in this area is not sufficient. However, it should be noted that the current dike design is flexible and can accommodate design modifications in the future, if required based on the site monitoring data.

9.0 PERSONNEL

This report has been prepared by Ms. Nina Quan with contribution from Mr. Mathieu Durand-Jezequel (Section 5.1 – thermal modelling), Mr. Daniel Meles (Section 5.4 – stability analyses) and Mr. Holman Tellez (Sections 4.2, 4.3 and 5.6 – water management aspects) and reviewed by Mr. Philip Gomes and Mr. Anh-Long Nguyen.

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

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10.0 REFERENCES

AEM (2019a), Geotechnical Investigation Campaign Test Pits and Bedrock Soundings, January 2020.

AEM (2019b) IVR Dike -1 Instrumentation Report, December 2019.

AEM (2020), Excel spreadsheet, AMQ_Geotechnical Investigation D1 (Tamrock)_May 2020.xlsx.

Canadian Dam Association (2013). Dam Safety Guidelines.

Canadian Dam Association (2014). Application of Dam Safety Guidelines to Mining Dams.

GSC (1999), Geological Survey of Canada, 1999. Open File 3773. Monograph on Norman Wells pipeline – Geotechnical Design and Performance.

Geological Survey of Canada (GSC, 2014) Fifth Generation Seismic Hazard Model Input Files as Proposed to Produce Values for the 2015 National Building Code of Canada, 2014.

Golder Associates 2019 (Golder, 2019), Excel spreadsheet, 18108905_2019 Water Balance 16Apr2019-REV1.xlsx.

MTQ (2014) – Ministère des Transports de Québec Manuel de conception des ponceaux, Ouvrages routiers, Guides et manuels, Novembre 2014.

NEH, 2007 – National Engineering Handbook, Technical Supplement 14C – Isbash Method.

SNC-Lavalin (2018) - Memorandum of the WRSF Stability analysis update. 651298-6000-4GER-0002-00, October 2018.

SNC-Lavalin (2020a) - Hydrological Analysis Update for the Water Management Infrastructure Design – October 2020.

SNC-Lavalin (2020b) – Technical Specifications for the Construction of IVR D-1 Dike, December 2020.

SNC-Lavalin (2020c) – IVR Attenuation Pond D-1 Dike Field Investigation Factual Report – December 2020.

SNC-Lavalin (2020d) – Design Criteria and Design Basis for IVR Attenuation Pond Structures – December 2020.

USACE (1980) – U.S. Army Corps of Engineers – Hydraulic Design of Reservoir Outlet Works– Engineer Manual 1110-2-1602.

USACE (1994) – U.S. Army Corps of Engineers - Hydraulic Design of Flood Control Channels - Engineer Manual 1110-2-1601.

USBR (1983) – U.S. Bureau of Reclamation – Hydraulic Design for Stilling Basing for Pipe or Channels outlet, EM 25.

USBR (1984) - United States Department of Interiors Bureau of Reclamation – Hydraulic Design of Stilling Basins and Energy Dissipators, 1984.

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