Stormwater Management and Drainage System Design

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Attachments

1. Introduction

The Mary River Project is a proposed iron ore mine and associated facilities located in northern Baffin Island, in the Qikiqtani Region of Nunavut. The Project involves the construction, operation, closure, and reclamation of a 18 million tonne-per-annum open pit mine that will operate for 21 years. The high-grade iron ore to be mined is suitable for international shipment after only crushing and screening with no chemical processing facilities. A railway system will transport ore from the mine area to an all-season deep-water port and ship loading facility at Steensby Port where to ore will be loaded into ore carriers for overseas shipment through Foxe Basin.

The project consists of the construction, operation, closure, and reclamation of an open pit mine and associated infrastructures for extraction, transportation and shipment of iron ore from two newly constructed ports at Milne inlet and Steensby inlet. After crushing and screening, iron ore will be transported from the Mine Site to the Ports for shipment.

The development requires managing stormwater runoff and flow by a well designed stormwater management system to reduce impacts of the development on the environment.

This design memo describes the stormwater water management and drainage system for the Mine Site, the Milne Port and the Steensby inlet.

2. Design Criteria and General Design Considerations

2.1 Objectives

The objectives of the design for the stormwater management and drainage are to provide: i) a safe and efficient stormwater drainage scheme that will minimize disruptions to the mine and operations (including construction) during wet weather periods, while minimizing the potential for negative impacts to the environment in the event of an uncontrolled release of stormwater runoff, ii) intercept and divert clean stormwater from undisturbed areas, and iii) provide peak flow reduction to mitigate flooding of the downstream areas.

2.2 Design Criteria

2.2.1 Surface Drainage

The general criteria for the stormwater management system is described below. Where applicable the criteria described correspond to that described in the Civil Design Criteria.

- All interior site grading and roads will be designed to provide continuous overland flow without erosion to a drainage ditch system.
- Provision must be made to ensure that there is a safe flow path for events up to the 1 in 10-year event, such that the runoff will not flood key mining areas, cause significant erosion, pick up excessive contaminants or cause other significant problems.

2.2.2 External Surface Drainage

Additional criteria for drainage of the external area are as follows:

- Run-off from undisturbed areas surrounding the mine site should be collected in clean-water perimeter ditches and diverted around and / or through the site perimeter.
- To the extent possible, these perimeter ditches will be designed to discharge at locations that best retain the characteristics of the existing (i.e., pre-development) natural drainage patterns.
- Clean water diversion ditches shall be designed to convey the 100-year flood event.

2.2.3 Stormwater and Sediment Ponds

Stormwater management ponds are designed to:

- Safely pass the Inflow design flood that meet CDA dam safety guidelines
- Reduce flooding in the downstream area
- Remove sediment concentration to meet the 15 mg/L discharge standard
- Be stable under design earthquake conditions
- Be stable under worst load conditions as required by CDA dam safety guidelines.

2.3 Dam Safety Assessment

The stormwater and sediment management ponds need embankment structures to create the required storages. These embankment structures meet the definition of dams (2 meters of height and retains more than 30,000 m^3 of water) and hence must follow the dam safety guidelines of the Canada Dam Association (2007). A dam classification is needed to determine many of the design parameters (such as the inflow design flood (IDF), and the design earthquake (DE)). The detailed dam safety assessment will be discussed in Appendix A.

3. Stormwater / Sediment Management and Drainage Systems

3.1 Mine Site

The general layout of the mine site development is presented in drawing no. H337696-4210-10-014-0001. The mine site stormwater management system includes dirty flow collecting ditches, clean water diversion ditches, and stormwater / sediment ponds. There are two main areas where stormwater management systems are required. One area is the treatment of stormwater and sediments surrounding the waste rock stockpile north of the main pit, and the other area is the treatment of stormwater and sediments surrounding the ore stockpile platform. The following sections discuss the two area’s specific features.

3.1.1 Waste Dump Stockpile Area

Figure 3-1 shows the ditches and stormwater ponds for the treatment of the storm water runoff from the waste dump stockpile area. From Figure 3-1, the waste dump stockpile is surrounded by runoff collecting ditches. The ditches have four segments.

- Segment 1 (northeast portion) collects runoff from the waste dump stockpile and carries flow to the east then to the south down to Stormwater Pond 2.
- Segment 2 (Southeast portion) receives runoff from the waste dump stockpile and flows mainly to the east and discharges into Pond 2.
- Segment 3 (Northwest portion) collects stormwater and flows to the west then to the south and releases the water into Pond 1.
- Segment 4 (South West portion) collects flows from the waste dump area and flows mainly to the west then discharges flow into Pond 1.
- Between Pond 1 and the waste dump stockpile area, there is a large area where no development is planned and there will be no disturbance to the runoff generated from the area. The water is therefore clean. The flow from this area will, however, flow down in the south direction and will be discharged into Pond 1. This will lead to unnecessary treatment of clean water by Pond 1 reducing the sediment removal efficiency or increasing the pond storage requirement. In order to avoid to treat the clean water generated by the undisturbed watershed, a clean water diversion ditch is proposed to collect the clean water generated from the natural area and divert the flow to downstream of Pond 1. The location of the clean water diversion ditch is shown in Figure 3-1.

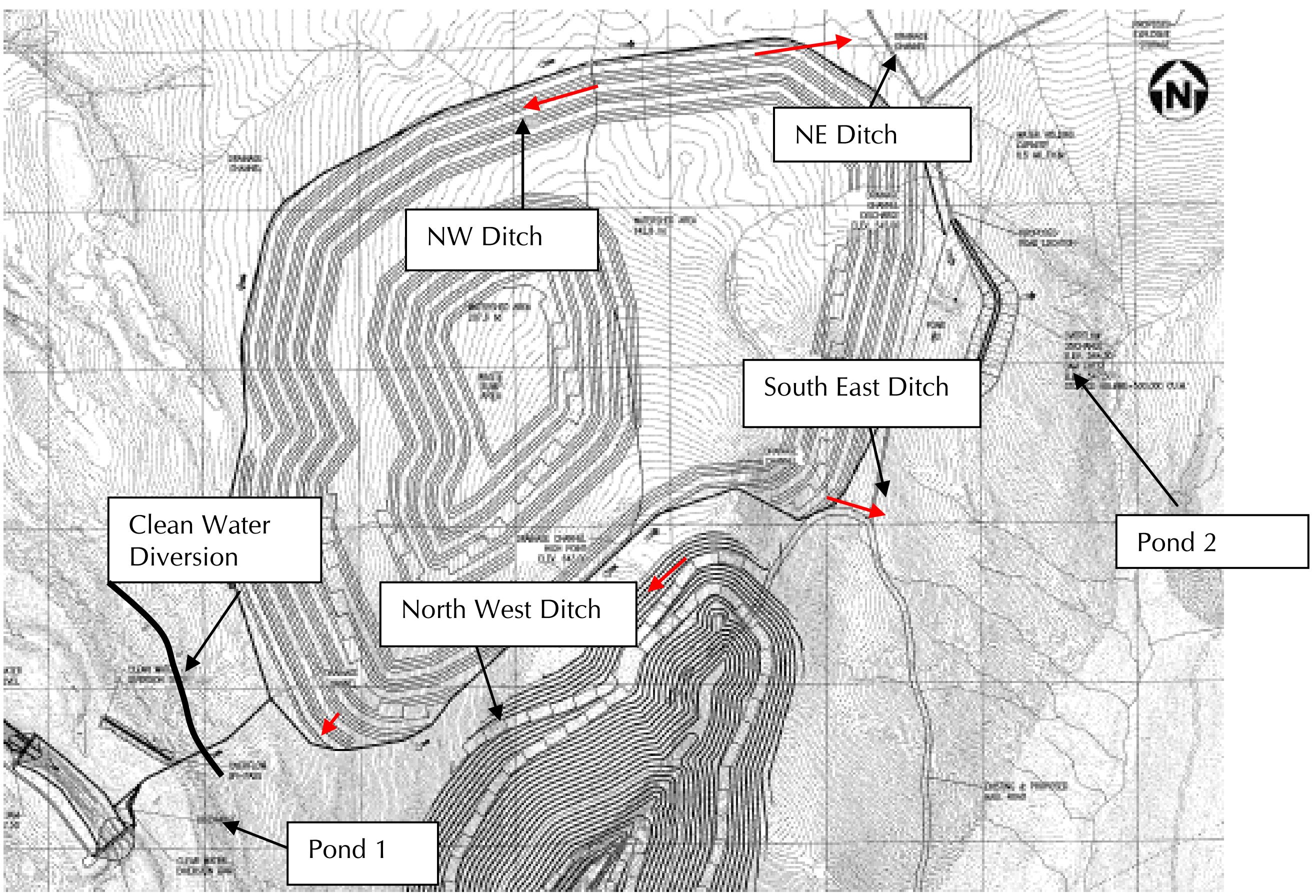

Figure 3-1: Stormwater Management System Layout - Waste Dump Stockpile Area

Two stormwater ponds are proposed to treat the stormwater for sediment removal.

- Pond 1 is located on the south west area downstream of the waste dump stockpile. This pond has two cells. Three dams will be required to form the pond cells. The pond releases flow into an existing downstream stream.
- Pond 2 is located to the East of the waste dump stockpile. The pond treats stormwater for sediment removal and then discharges to an existing downstream stream near the dam.

It shall be noted that the construction of the ditch and stormwater pond system for the waste rock stockpile area can be undertaken in phases corresponding to the waste rock dump development plan. Pond 1 and the runoff ditches to this pond shall be constructed before the waste rock dumping start. However, Pond 2 may not be needed until year 15 according to the current waste rock stockpile development plan. The basic criteria to determine if the construction shall be carried out is that the stormwater treatment system shall be in place once waste rock dumping begins in the affected drainage area.

The sizing of the required components (ditches and ponds will be discussed in the following sections.

3.1.2 Ore Stockpile Platform Area

3.1.2.1 Clean Water Diversion Ditch

The ore stockpile area is presented in H337697-4210-10-042-0003. The infrastructures in this area are still in the process of modifications. However, the general layout of the drainage system shall not change much from what is described in the following sections. Some changes are expected in the final design.

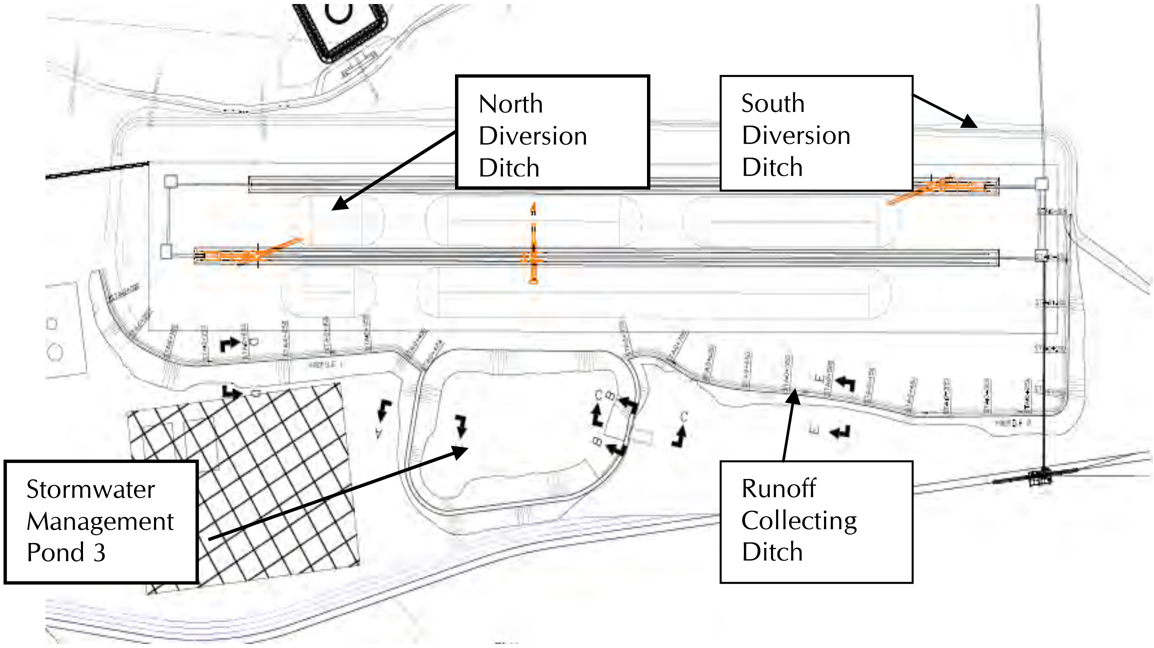

Figure 3-2: The Ore Stockpile Area (note: infrastructures in this area is still in the process of modifications and hence the ditch system may need minor modifications. But the general layout would remain unchanged)

In this region, the area north of the ore stockpile platform will be undisturbed and hence the runoff generated from that area will be clean water. The ground elevation of the north area is higher than the ore stockpile platform. The natural flow would flow into the ore stockpile working area and causes disturbance. The extra water will eventually enter the stormwater management pond for treatment leading to larger than needed SWM storage hence increase the cost. For the purpose of avoiding problems, a clean water diversion ditch was designed to divert the flow. This ditch has two segments as shown in Drawing Number H337697-4210-10-042-0003. The North West portion flows in a northwest direction and the North East portion flows in a southeast direction and both will be discharged into nearby existing streams.

3.1.2.2 Drainage Ditch

The runoff collection ditch is designed to collect runoff from the ore stockpile platform and carry flow into SWM Pond 3 for treatment.

3.1.2.3 SWM Pond 3

Pond 3 is designed to collect dirty water generated from the ore stockpile area for treatment. After treatment the flow will be discharged into an existing stream downstream.

3.2 Steensby Inlet

3.2.1 Ore Stockpile Platform on the Island

The Steensby Inlet (drawing H-337697-4510-10-014-0001) has two main areas where stormwater and sediment treatment are required. One area is the ore stockpile platform in the island. The infrastructures of the ore stockpile platform are still in the process of being laid out and changes will be made. The basic concept shown in Figure 3-4 is for the stormwater management system of the ore stockpile platform area.

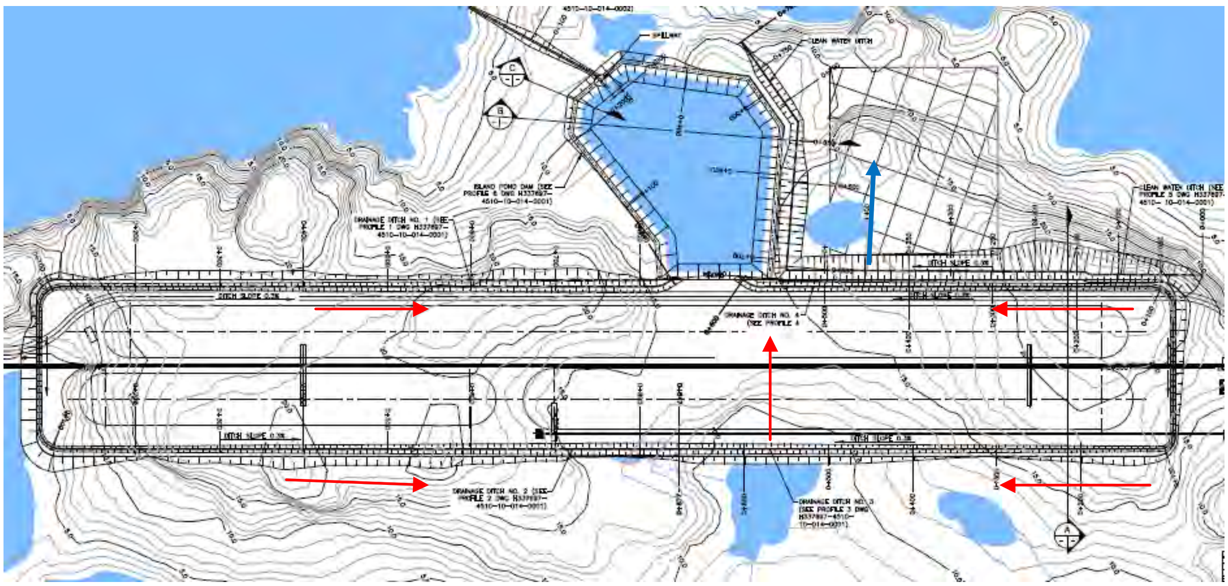

Figure 3-3: Ore Stockpile Platform Stormwater Management System

From Figure 3-3, the surrounding ditch collects the runoff generated by the ore stockpile platform area and puts it into the stormwater management pond northwest of the platform. After treatment, the flow is released to the ocean via the downstream channel. The flow arrows shown in Figure 3-3 indicate the flow collection plan.

There is a small area North West of the ore stockpile platform where flow generated will be clean water and therefore a clean water diversion ditch will be used to collect and divert the flow around the SWM pond.

The stormwater management pond is designed to treat the stormwater and sediment. The sizing of the ditches and the ponds will be discussed in the following sections.

3.2.2 *The Stormwater Management System for the Laydown and Storage Area*

This area has three components in the drainage and stormwater management system. The three components include:

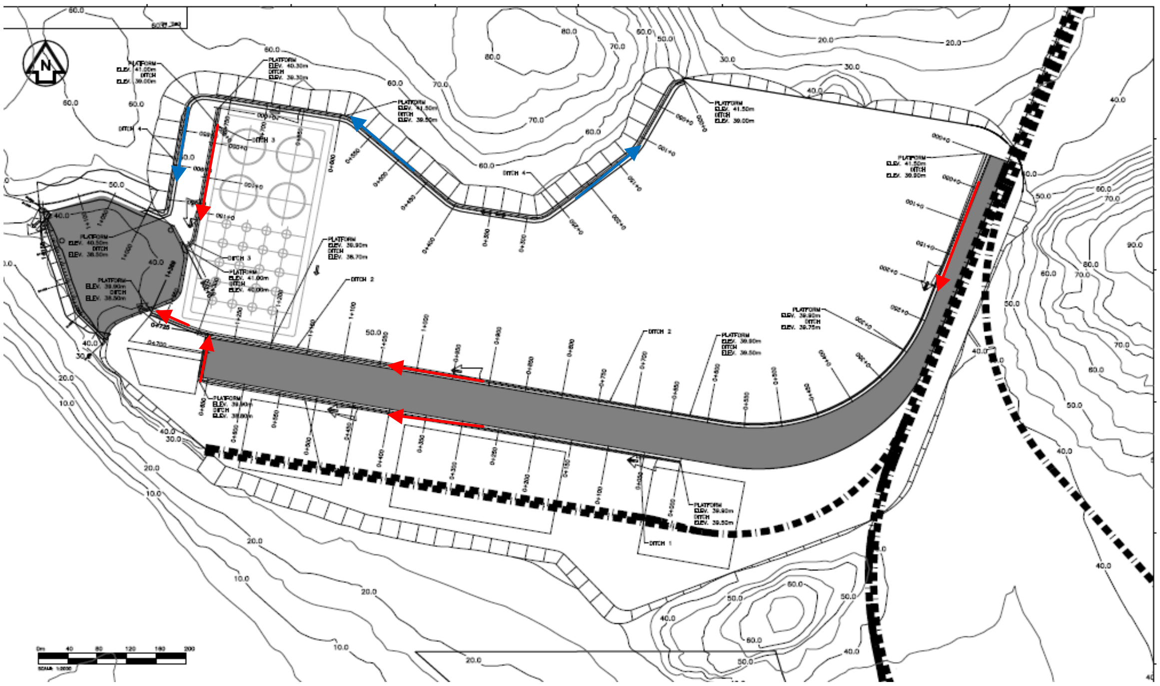

Figure 3-4: Stormwater Management and Drainage network

- Clean water diversion ditch (Figure 3-4)
  - The clean water diversion ditch has two segments. The East portion flows in a Northeast direction and discharges into a small lake north of the area. The second segment flow mainly in a West direction bypassing the stormwater management pond and directly discharges to the ocean.
- The drainage ditch collecting flow from the affected area to the pond for treatment (Figure 3-4)
- The stormwater management pond west of the area
  - After treatment, the water is released to the ocean.

3.3 Milne Inlet

The Milne inlet does not have permanent structures. The drainage work required is to collect the runoff and discharge it to the nearest streams or water courses. The area to be served is small and hence the sizes of the ditches are small. The area is shown in Figure 3-5.

In this area, there are small streams. Land near natural streams will be graded to drain to the natural stream and hence no ditches are required.

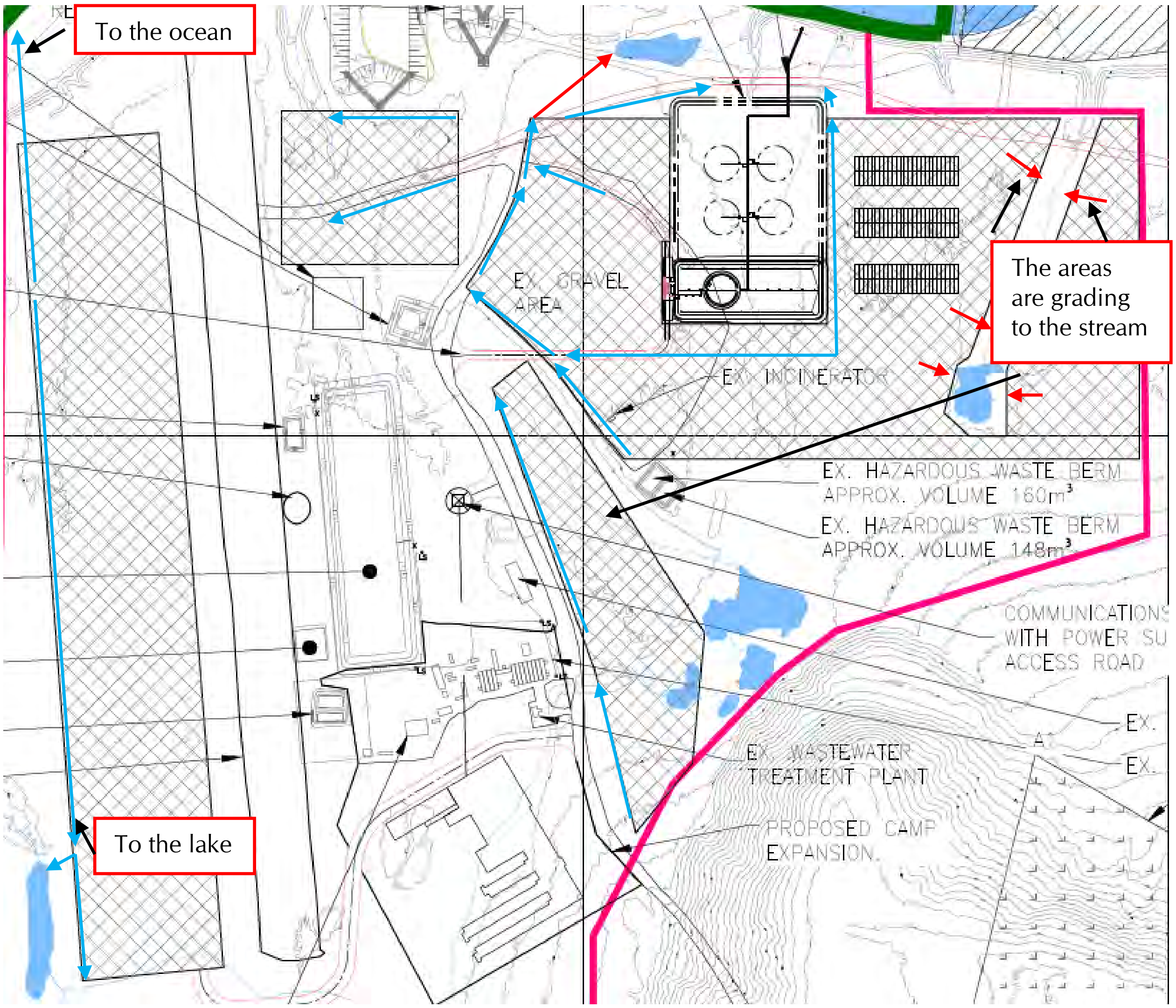

Figure 3-5: Milne Inlet Drainage Plan

4. Stormwater Pond Design

4.1 Stormwater Ponds

4.1.1 Mine site

In the Mine site, three stormwater / sediment ponds are proposed. These SWM ponds are designed to reduce peak flows, to store runoff generated in the area and to reduce sediment (TSS) concentration.

4.1.1.1 POND 1

Figure 4-1 shows the configuration of Pond 1. Pond 1 collects runoff from the waste rock dump for treatment. Pond 1 is formed by three dams. The Block dam has a crest elevation of 355 m.

This dam does not allow any flow over the embankment. Its only purpose is to block the flow. This dam has a SIGNIFICANT hazard classification and hence the inflow design flood is the 1:200 year flood (Appendix A).

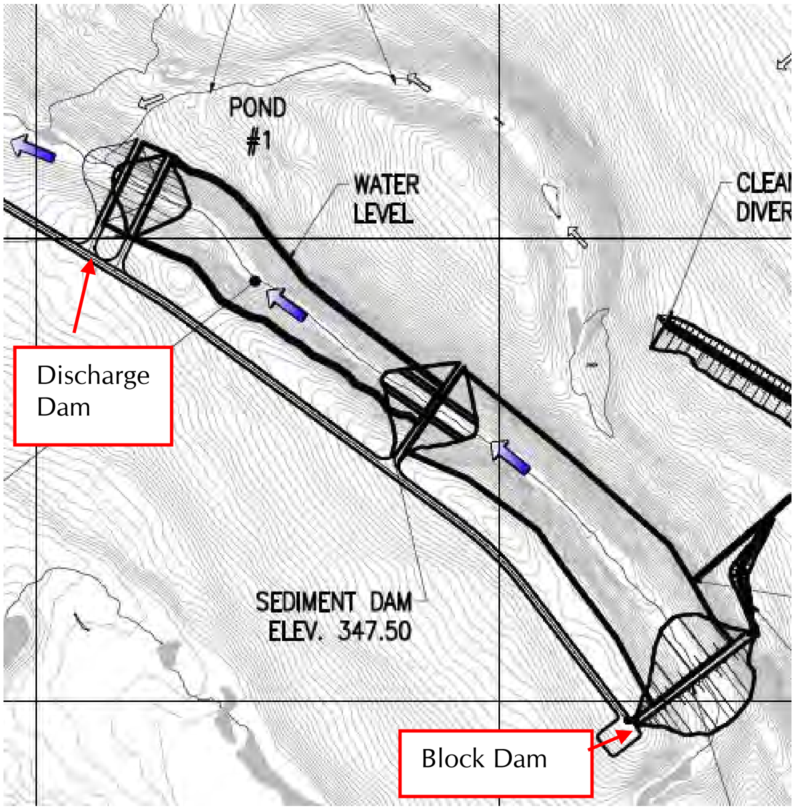

Figure 4-1: Mine site Pond 1 Configuration

The dam in the middle of Pond 1 is used for separating the pond into two cells. This dam has a crest elevation of 347.5 m. An overflow section in the middle of the dam will allow flows into cell 2. The overflow elevation is set at 344.5 m. The bottom width of the overflow weir is 10 m. The side slope of the weir is 2 (H):1 (V). The dam has a SIGNIFICANT hazard classification and the IDF is the 1:200 year flood (Attachment A).

The downstream dam has a crest elevation of 329 m. The dam has an overflow weir at elevation 326 m. The bottom width of the overflow weir is 10 m. the side slope of the overflow section is 2 (H):1 (V). This dam is classified as having a SIGNIFICANT hazard rating and the IDF is the 1:200 year flood (Appendix A). The total storage capacity of Pond 1 is approximately 0.7 million of cubic meters (MCM).

4.1.1.2 *POND 2*

Pond 2 collects runoff from the waste rock dump (east part) for sediment removal. The dam has a crest elevation of 547.5 m with an overflow weir at elevation 544.5 m. The dam height is approximately 27 m. The total volume of the pond is about 0.5 MCM. A spillway is designed to safely pass the IDF. The spillway bottom width is 10 m. The location of the spillway is on the northeast shoulder away from the dam body. The purpose is to avoid overtopping of the dam. Due to the fact that this dam is used as access road, the spillway side slope is designed to be 10 (H):1 (V) to allow road traffic. This dam has been classified as having a SIGNIFICANT hazard rating and the IDF is the 1:200 year flood.

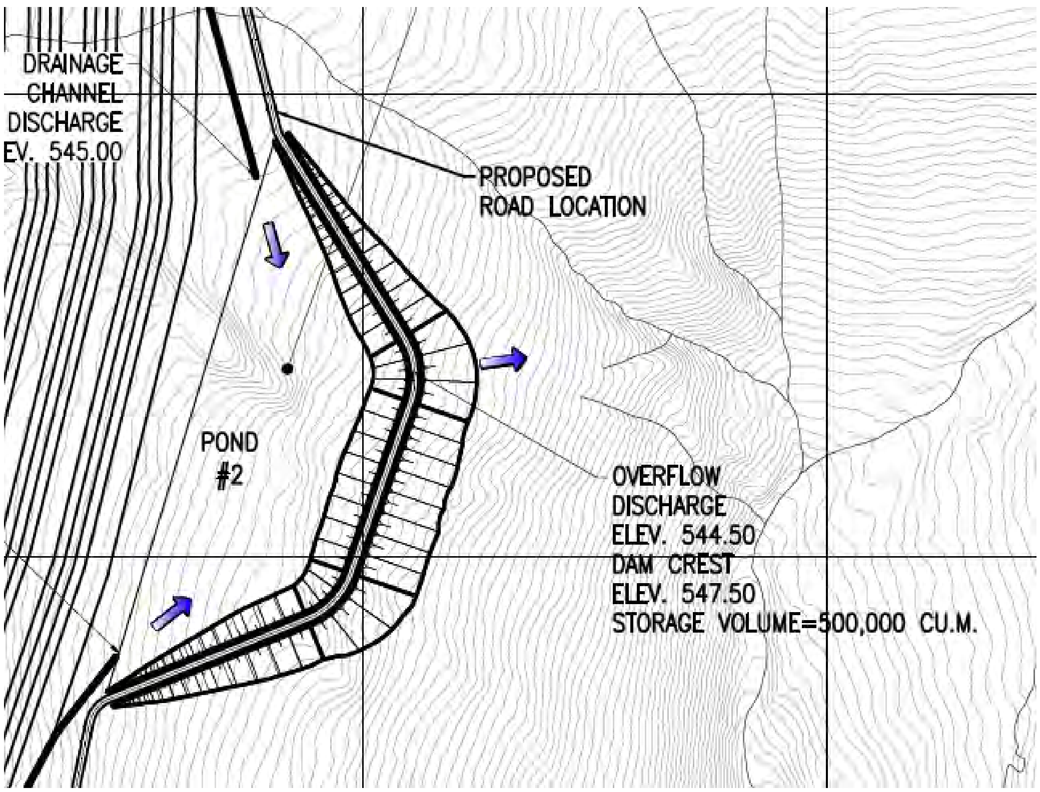

Figure 4-2: Mine site Pond 2 Configuration

4.1.1.3 *POND 3*

The location of pond 3 is shown in Figure 3-2. The dam to form pond 3 has a crest elevation of 204.5 m and an overflow weir at an invert elevation of 203.5 m. The overflow weir bottom width is 10 m with 2 (H):1 (V) side slopes. The storage is 0.15 MCM approximately. The surface area of the pond is about 3.6 ha. The dam has a SIGNIFICANT hazard classification and hence the IDF is the 1:200 year flood.

4.1.2 Steensby Inlet

The Steensby Inlet has two SWM ponds. Pond 1 is located on the island to treat the stormwater generated by the ore stockpile platform (Figure 3-3). The dam has a crest elevation of 13 m. An overflow weir has a bottom width of 10 m at invert elevation of 10.5 m. The dam has a SIGNIFICANT hazard rating and the IDF is the 1:200 year flood (Appendix A).

The SWM Pond 2 on the land is shown in Figure 3-4. The crest elevation of the dam is 40 m. The overflow weir invert elevation is 38 m. The width of the weir is 10 m. The side slope of the weir is 2 (H):1 (V). The dam has a storage of about 80,000 m^3. The hazard potential of this dam is SIGNIFICANT and hence the IDF is the 1:200 year flood event.

4.2 Peak Flow Estimation

The design of the drainage ditches requires the estimation of the peak flows for the design event. Flow estimation will be based on the following equations developed by Knight Piésold Consulting for drainage areas greater than or equal to 0.5 km^2:

$Q_2 = 1.1\ A^{0.79}$

$Q_5 = 1.7\ A^{0.77}$

$Q_{10} = 2.0\ A^{0.76}$

$Q_{25} = 2.6\ A^{0.75}$

$Q_{100} = 3.5\ A^{0.73}$

Where Q = peak flow instantaneous flow in m^3/s

A = drainage area in km^2 ($0.5\ km^2 \leq A \leq 1000\ km^2$)

When the drainage area is smaller than 0.5 km^2, the above equations cannot be used. In this case, the rational formulae will be applied for the estimation of peak design flows. The form of the equation is:

$Q = 0.28\ CIA$

Where, Q = peak instantaneous flow in m^3/s

A = drainage area in km^2

C = runoff coefficient = 0.9 (the runoff coefficient is high to reflect the high degree of saturation or freezing ground conditions during runoff flood event)

I = rainfall intensity corresponding to the time of concentration.

The time of concentration is calculated as: where T_c = time of concentration (hour), L = the main channel length (km) and S = the channel slope (m/m).

The rainfall intensity-duration-frequency (IDF) curves of design storms have been analyzed by Knight Piésold Consulting and the IDF curves are summarized in Table 4-1.

Table 4-1: Design Storm Intensity-Duration-Frequency (IDF) Curves (mm/hr)

Duration	2 yrs	5 yrs	10 yrs	15 yrs	20 yrs	25 yrs	50 yrs	100 yrs	200 yrs
5 min	9.5	12.0	14.0	15.1	15.9	16.5	18.3	20.1	22.0
10 min	7.2	9.0	10.5	11.3	11.9	12.4	13.7	15.1	16.5
15 min	6.0	7.5	8.7	9.4	9.9	10.3	11.4	12.6	13.7
30 min	5.0	6.3	7.3	7.9	8.3	8.6	9.5	10.5	11.4
1 hr	4.0	5.2	6.1	6.6	7.0	7.3	8.1	9.0	9.9
2 hr	3.0	3.9	4.6	5.0	5.2	5.5	6.1	6.8	7.4
6 hr	2.0	2.7	3.3	3.6	3.9	4.0	4.6	5.1	5.7
12 hr	1.3	1.8	2.2	2.4	2.6	2.7	3.1	3.4	3.8
24 hr	1.0	1.4	1.7	1.9	2.0	2.1	2.4	2.7	3.0

The determination of the peak flows for each of the ditches will be discussed in Section 5.

4.3 Flood Routing in Stormwater Management Ponds

To design the spillways for stormwater ponds, the equations described in Section 4.1 will not be sufficient since the storage routing effects cannot be evaluated by the simple peak flow estimation equations. The storages in the ponds play an important role in the determination of water levels and peak outflows from the spillway. In this case, a flood routing model was used to fully assess the impact of the storages and the required spillway dimensions to safely pass the design floods for each pond.

The US EPA SWMM model was used for the flood routing assessment. The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage / treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

A SWMM model was established for each SWM pond in the Mine site and Steensby Inlet areas. The SWMM model was used to:

- Determine the spillway dimensions required to pass the inflow design flood (IDF)
- Evaluate the water quality performance of the ponds with respect to TSS removal (Section 4.4).

To simulate the flood routing processes in the SWM ponds during IDF, the return period of the inflow design flood shall be determined. This IDF is associated with the dam classification based on CDA dam safety guidelines. This dam classification for each dam will be discussed in Section 6 (Dam Design Section). The following section describes the design storms used in the SWMM model.

4.3.1 *Design Storm*

Design storm has three components:

- Design frequency (return period)
- Storm volume (mm) and duration (hours)
- Temporal distribution

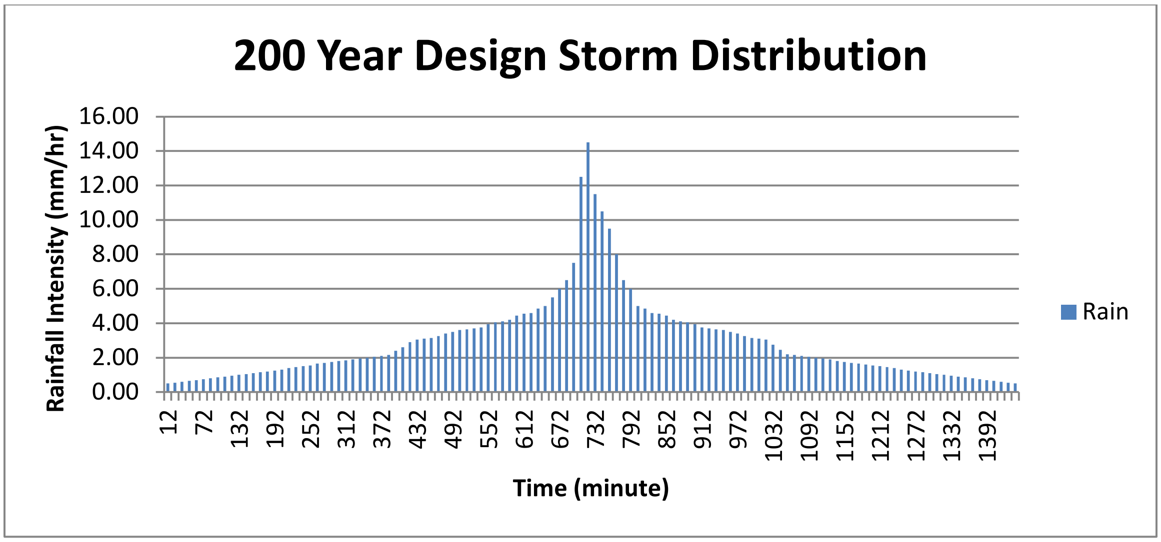

Figure 4-3: 200 Year Design Storm Distribution

The dam safety assessment results shown that the required IDF for all of the SWM pond embankment structure is the 1:200 year flood event. For the site, Knight Piésold Consulting determined that the 1:200 year design storm has 71 mm in 24 hour period.

The temporal distribution of the storm was developed based on the 'balanced storm' method. The 'balanced storm method' was described by D. H. Hoggan, 1996. The 24 hours 'balanced storm' temporal distribution of the 200 year storm is presented in Table 4-3. The total storm volume of this event is 71 mm. Figure 4-3 shows the intensity (mm/hr) for each rainfall block. The time interval is 12 minutes.

4.3.2 *Model Parameters*

The input to the model includes:

1. drainage areas of the sub-watershed
2. Surface roughness coefficient
3. Infiltration parameters
4. Sediment erosion parameters
5. Precipitation input
6. SWM pond configurations

The model will produce peak flows and flood hydrographs for each sub-watershed and will be able to calculate the combined flows at a confluence of sub-watersheds.

Table 4-3 summarizes the sub-watershed areas and the other basic parameters used in the model for Mine site.

Table 4-2: Mine Site SWMM model parameters

	Watershed Area (ha)	Percent Imperious %	Maximum Infiltration rate (mm/hr)
Pond 1	207.8	99	3
Pond 2	142.8	99	3
Pond 3	26.2	99	3

Note: 99% of imperious area is used for frozen ground conditions during spring runoff period which results in almost all precipitation becoming runoff.

Table 4-3: Steensby Inlet SWMM Model Parameters

	Watershed Area (ha)	Percent Imperious %	Maximum Infiltration rate (mm/hr)
Pond 1	23.3	99	3
Pond 2	61	99	3

4.3.3 *Spillway Rating Curves*

Spillway rating curves are calculated using standard weir equation:

Where Q = discharge (m^3/s)

C = weir coefficient = 1.70 (assuming broad crest weir)

B = Spillway bottom width (m)

H = head of water (m)

4.3.4 *Results*

The SWMM model is used to simulate flood routing processes in the stormwater ponds for the inflow design flood. The peak water levels in each of the ponds are obtained and summarized in Table 4-4 for the mine site and in Table 4-5 for the Steensby Inlet site.

Table 4-4: Peak flows and water levels in the ponds (Mine site)

	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak water level (m)	Crest Elevation (m)	Freeboard (m)
Pond 1	6.09	4.65	326.35	329	2.65
Pond 2	4.31	2.66	544.7	547.5	2.80
Pond 3	0.84	0.73	203.55	204.5	0.95

Table 4-5: Peak Flows and Water Levels in the ponds (Steensby Inlet)

	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak water level (m)	Crest Elevation (m)	Freeboard (m)
Pond 1	0.89	0.76	10.64	13	2.36
Pond 2	1.63	1.41	38.21	40	1.79

From Table 4-4 and Table 4-5, it is known that the spillway capacities are sufficient to safely pass the IDF. Also the freeboards meet the CDA dam safety requirement. The stormwater ponds reduced the peak flows 66% - 85% depending on the storage characteristics of the ponds.

Figure 4-4presents one example of the flood reduction function for Mine site Pond 2 IDF case. From the figure, it is evident that a significant peak flow reduction is achieved (2.66/4.31 =61.7%).

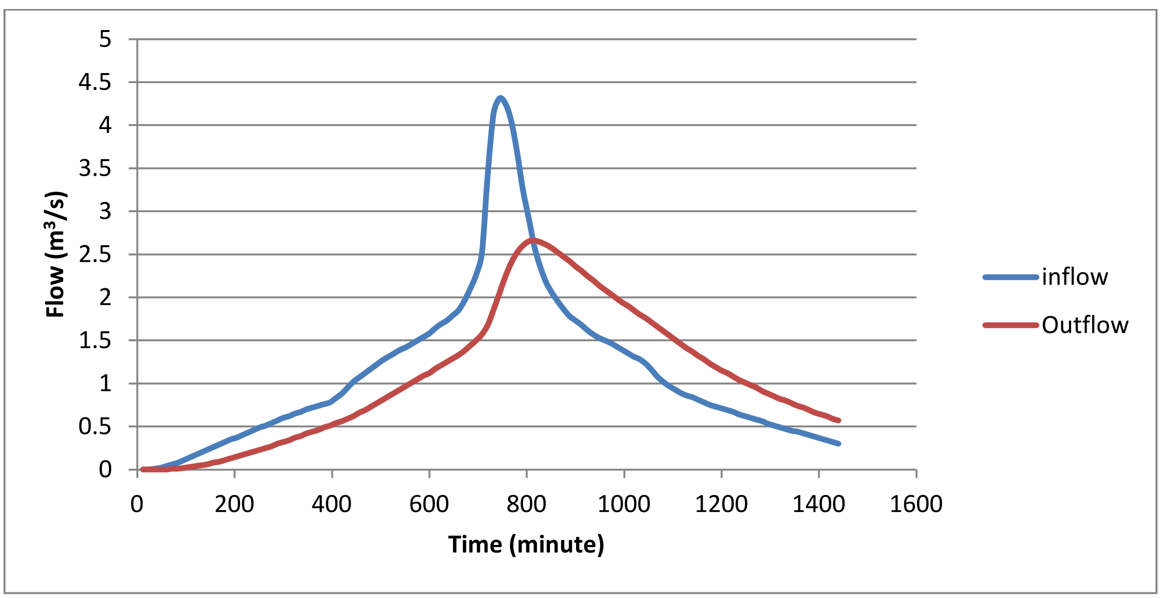

Figure 4-4 Inflow and outflow hydrographs, Mine Site Pond 2

In stormwater management, preventing peak flows from being higher than pre-development conditions is normally required. For this development, the flooding occurs normally on frozen ground and hence the pre- and post- development flood magnitudes does not change significantly and therefore the stormwater pond will improve the flood conditions of the site (compare to the pre-developed conditions). This is one benefit of the ponds.

4.4 Determination of Water Quality Capture Volume

4.4.1 WQCV Calculations

The water quality capture volume(WQCV) is an important design feature for stormwater quality control. The main pollutant to be controlled in the stormwater ponds is the sediment or total suspended solids (TSS) from the watershed. The target TSS concentration is 15 mg/L for all of the final discharge points. Many factors affect the TSS concentration including: A) amount of rainfall and runoff in the watershed, B) the sediment characteristics and the erosion potential, C) the pond storage and surface area, D) the outlet feature which determine the detention time, E) the TSS grain size distribution, and F) the size of the watershed and land use conditions, etc.

For the purpose of the stormwater pond design, the amount of rainfall and the detention time are the two key parameters that affect the performance of a stormwater pond. Current practice is to detent a 24 hours storm in the pond for 40 hours (Grizzard, 1986, Roesner, 1989) which will provide good TSS removal efficiency while the pond storage is still in manageable size. Longer detention time will lead to higher removal efficiency but requires a too large pond storage. Therefore, the detention time targeted for the water quality capture volume design is 40 hours.

The WQCV is the amount of storm to be treated in the detention storage. This amount varies from place to place. Typical values is to capture 25 mm storm (Ontario Ministry of Environment, 2003). For the Baffin land area, the 24 hours 25 mm storm is equivalent to a 1:2 year design storm approximately (Knight Piésold Consulting, 2010). This storm volume is used to estimate the WQCV storage requirement.

Table 4-6 summarize the WQCV for the ponds in the Mine Site and Table 4-7 presents the values for the ponds in the Steensby Inlet area.

Table 4-6: Pond WQCV Requirement (Mine Site)

	Drainage area (ha)	Design Storm (mm)	WQCV (M3)	Pond Surface (ha)	Depth between Core* and Spillway Invert m
Pond 1	207.8	25	51950	6.71	0.77
Pond 2	142.8	25	35700	10.9	0.33
Pond 3	26.2	25	6558	3.6	0.2

Table 4-7: Pond WQCV Requirement (Steensby Inlet)

	Drainage area (ha)	Design Storm (mm)	WQCV (M^{3})	Pond Surface (ha)	Depth between Core* and Spillway Invert m
Pond 1	23.3	25	5835	2.6	0.22
Pond 2	61.0	25	15250	2.85	0.54

Note: core elevation mean the top elevation of the seepage cut off materials inside the dam bodies

To make the required WQCV storage available, there is a need to maintain the water level lower than the spillway invert elevation so that the storm runoff will be stored in the pond and then slowly releases to a downstream water course. The slow release mechanism will be provided using a porous rock fill weir at the entrance of the spillway. The basic concept is illustrated in Figure 4-5.

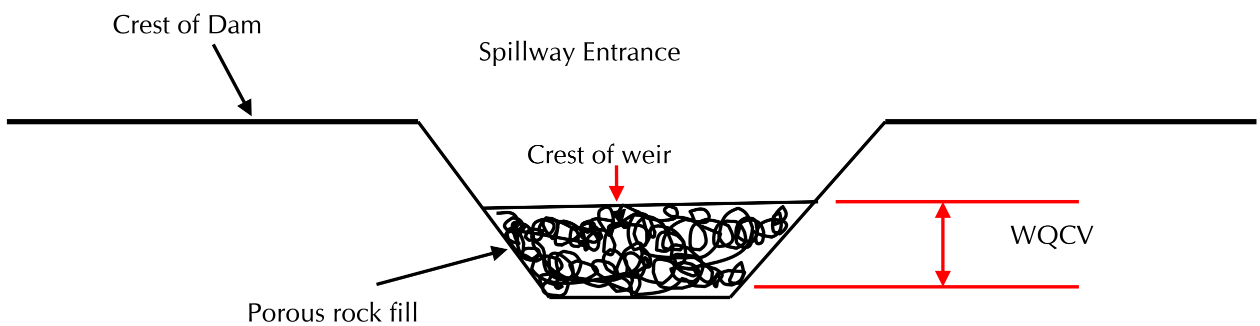

Figure 4-5: WQCV Concept Illustration

This slow flow release configuration is designed to work with dams where spillway can be constructed on natural ground. These dams include: Mine Site Pond 2 and 3 dams, the stormwater Pond 2 in Steensby Port of the laydown and storage area.

For dams for which the spillway cannot be located on natural ground (due to constrain of space), part of the embankment will have to be used as the spillway. In this case, the modified dam cross section option 2 (Figure 2.11 of Appendix B) will be used at the spillway location. This dam section allows a small amount of seepage flow into the porous rock fill area which acts as the slow flow release mechanism. This design will maintain the safety of the dam while providing the required slow flow release rate at the same time.

When rainfall occur, as long as the rainfall is smaller than or equal to 25 mm, all of the runoff will be stored in the WQCV zone (between normal water level and the invert of the pond spillway). The porous zone of the rock fill section will allow the runoff captured to slowly drain down to the normal water level. If the storm is 25 mm, then the time required for the water level to return to normal water level is 40 hours.

When the storm is higher than 25 mm, the WQCV will not be large enough to hold all of the runoff volume and spills will occur. The flow will directly run through the pond over the spillway and be discharged to the downstream river. In this case, the water quality standard may not be met (because there is no sufficient detention time to remove the TSS).

Based on the above discussions, it is evident that the provision of a porous zone above the spillway invert to allow the pond to drain slowly is a key design feature for water quality since without this discharge capacity the normal water level will be at the invert of the spillway and all runoff will be discharged directly to the downstream river. The TSS concentration may be too high.

For each pond, the depth between the porous weir and the invert elevation of the spillway is 1 m. (which is higher than the required values shown in Table 4-6 and Table 4-7, to provide higher TSS removal efficiencies).

4.4.2 SWMM Evaluation of the Pond Storage

A SWMM model was used to evaluate the performance of the WQCV in each pond. The input storm was the 1:2 year 24 hour design storm (25 mm of total rainfall volume). The most difficult parameter for this evaluation is the input TSS concentration since this value changes with many factors, such as the rainfall intensity and duration, the land surface conditions, the operation of the mining activities, etc. US EPA (1983) reported that typical stormwater TSS concentration is in the range of 180 mg/L - 548 mg/L depending on the land use. Therefore, a 300 mg/L and 550 mg/L was used in the model to simulate the performance of the SWM ponds. The value of 300 mg/L represents average concentration conditions and 550 mg/L represents the high concentration conditions. It is also noted that mining operation may result in much higher TSS load than Urban area. For this reason, the input TSS concentration five times higher than 550 mg/L (2750 mg/L) was also evaluated.

The equation for the evaluation of the TSS removal is based on the following treatment function of TSS in the SWM pond (SWMM Application Manual, 2009):

–

Where C = concentration of TSS (mg/L)

C* =TSS concentration that cannot be settled by gravity (mg/L) due to small grain size

K = model parameter related to detention time and pond representative depth

d = water depth in the pond

In this equation, it is known that the TSS concentration cannot settle in the pond by gravity is an important site specific parameter, depending on the sediment size distribution. This information, however, can only be available after the mining operation starts. Therefore, it is assumed that this value is less than 15 mg/L since if it is higher than 15 mg/L, no matter how big the sediment pond would be, the targeting TSS concentration will not be met.

Table 4-8 and Table 4-9 summarize the simulation results for the Mine Site and Steensby Inlet respectively.

Table 4-8: SWM Pond Outflow TSS Concentration (Mine site)

	Input TSS= 300 mg/L		Input TSS = 550 mg/L		Input TSS= 2750 mg/L	
	Peak mg/L	**Mean mg/L**	**Peak mg/L**	**Mean Mg/L**	**Peak Mg/L**	**Mean Mg/L**
Pond 1	11.5	8.7	11.7	8.5	13.6	8.6
Pond 2	14.6	10.3	19.0	10.7	54	14.4
Pond 3	12.5	10.1	14.6	10.4	33.4	12.5

Table 4-9: SWM Pond Outflow TSS Concentration (Steensby Inlet)

	Input TSS = 300 mg/L		Input TSS = 550 mg/L		Input TSS = 2750 mg/L	
	Peak Mg/L	**Mean Mg/L**	**Peak Mg/L**	**Mean Mg/L**	**Peak Mg/L**	**Mean Mg/L**
Pond 1	13.3	10.4	16.2	10.8	41.2	14.4
Pond 2	16.7	10.8	22.5	11.5	73.5	17.4

From Table 4-8 and Table 4-9, it is know that when the 25 mm storm runoff is stored for 40 hours, the mean TSS concentrations of the outflows from the ponds will be less than 15 mg/L. The peak concentration could be higher but these high concentrations will last only for a hour or so. The basic requirement of concentration less than 15 mg/L is met. It is very difficult to reduce the peak concentration since this will need an extremely large pond and longer detention time.

It shall be noted that the 25 mm storm has a return period of 2 years. This means that, on average, all storms less than the 2-year event will be controlled to have TSS concentration less than 15 mg/L.

Figure 4-6 shows the TSS concentration variation during the 2-year storm event in Mine Site

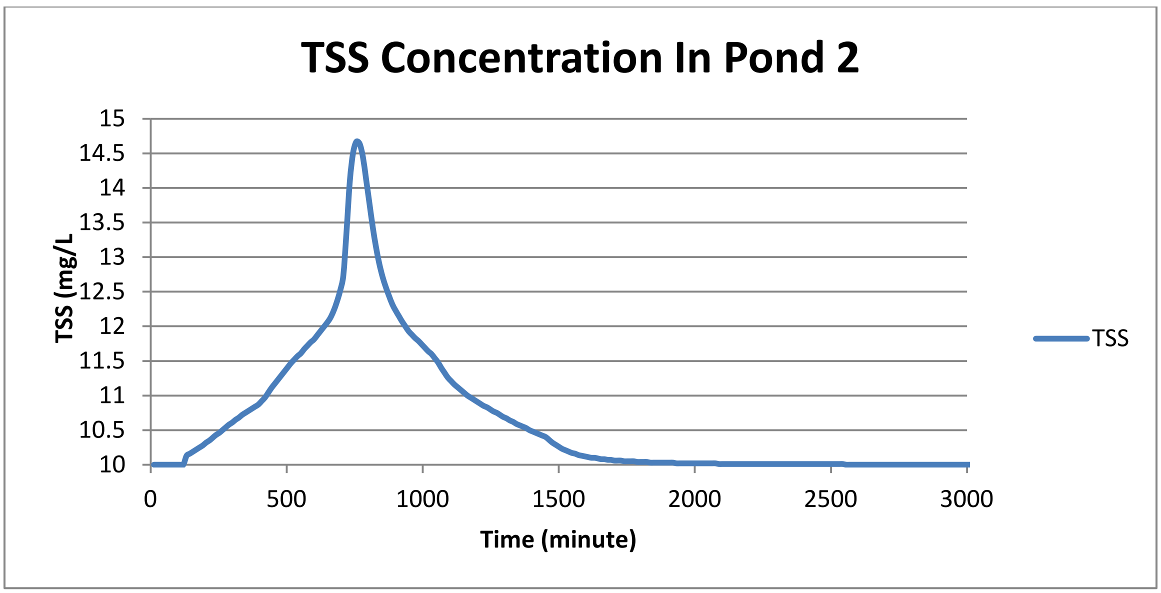

Figure 4-6: Pond TSS Removal Performance Example (Input TSS = 300 mg/L)

Pond 2 as an example of the TSS removal performance. This figure presents the out flow TSS concentration.

When the input TSS concentration is as high as 2,750 mg/L, the mean outflow TSS concentrations in most SWM ponds will still meet the requirement. The Pond 2 in Steensby Inlet will not have a higher mean TSS exceeding the 15 mg/L target.

It is concluded that the provided WQCV will meet the TSS concentration target for each of the ponds if the input concentrations are less than 2,750 mg/L and the TSS that cannot settle by gravity is below 15 mg/L. However, it is known that there are many factors affecting the TSS concentration of the site, uncertainties still exist. It is hence recommended that a monitoring system be established to measure the TSS concentration in runoff at various locations and if it is found that the TSS concentration exceeds the limit, additional treatment may be needed.

From Table 4-8, it is also interesting to note that the two cells arrangement in Mine site Pond 1 will improve the TSS removal performance due to additional detention time by the two-cell configuration.

5. Sizing of the Drainage Ditches

5.1 Mine Site Ditches

5.1.1 Waste Rock Stockpile

The drainage area for the waste rock stockpile was divided into four sub-areas. The four sub-areas were called NE, NW, SE, and SW and correspond with the channel alignments. The NW and SW channels combine to form an Outlet channel that leads to a sediment pond. The runoff was calculated using the equations given in Reference 1 as each sub-area was greater than 0.5 km^2. The 10 year design storm was used to size these channels. The runoff from each sub-area was calculated at the downstream end. Intermediate discharges along the proposed channel were calculated by prorating the discharge over the channel length.

The minimum channel bottom width listed in the Design Criteria is 1 m. This width was sufficient for all the channels except the Outlet channel at the waste rock stockpile. A 3 m channel bottom was used for its entire length.

The channel slopes ranged from 0.3 percent to 69 percent. The Outlet channel at the waste rock stockpile had the steepest slopes with a minimum slope of 14 percent and a maximum slope of 69 percent.

Reinforced concrete pipe (n = 0.013) was used for the closed drainage system in the Platform site. A minimum cover of 0.6 m was used over the top of the pipe. The minimum slope considered in the design was a slope that could achieve a pipe flow velocity of 1 m/s.

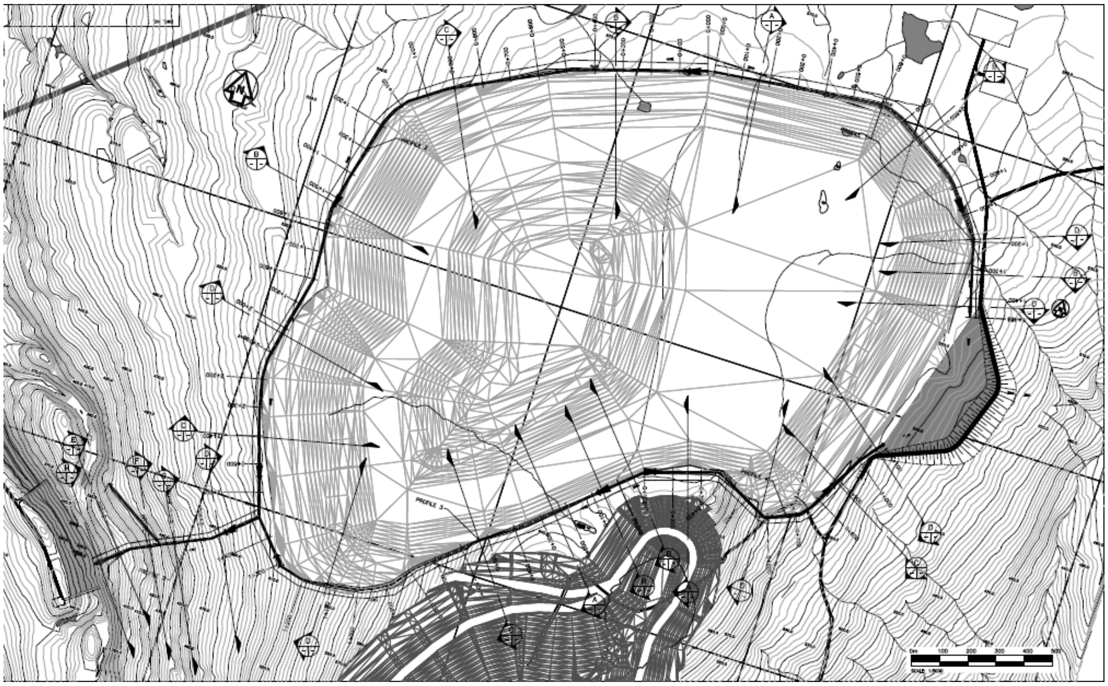

Figure 5-1: The Waste Rock Dump Ditches

Table 5-1: Ditch Size and Riprap Requirements (Waste Rock Stockpile Area)

Channel	Beginning Station (m)	End Station (m)	Channel Type	Discharge (cms)	Bottom Width (m)	d_{50} (mm)	d_{100} (mm)	Riprap thickness(mm)
Profile 1	0	120	A	0.1	1			
Profile 1	120	320	A	0.1	1			
Profile 1	320	370	B	0.4	1	80	100	100
Profile 1	370	655	B	0.5	1	80	100	100
Profile 1	655	890	B	0.8	1	80	100	100
Profile 1	890	1140	B	1.1	1	80	100	100
Profile 1	1140	1245	D	1.4	1	300	380	375
Profile 1	1245	1390	B	1.5	1	80	100	100
Profile 1	1390	1470	D	1.7	1	300	380	375
Profile 2	0	645	B	0.3	1	80	100	100
Profile 2	645	1160	C	0.6	1	160	200	200
Profile 2	1160	1885	B	1.1	1	80	100	100
Profile 2	1885	2160	D	1.8	1	300	380	375
Profile 2	2160	2470	C	2.0	1	160	200	200
Profile 2	2470	2680	C	2.3	1	160	200	200
Profile 2	2680	2795	D	3.2	3	300	380	375
Profile 2	2795	2960	D	3.2	3	300	380	375
Profile 2	2960	3035	G	3.2	3	650	820	813
Profile 2	3035	3110	F	3.2	3	540	680	675
Profile 2	3110	3130	F	3.2	3	540	680	675
Profile 2	3130	3255	E	3.2	3	480	600	600
Profile 2	3255	3290	H	3.2	3	SD	SmartDitch	
Profile 3	0	145	B	0.1	1	80	100	100
Profile 3	145	350	A	0.1	1			
Profile 3	350	565	C	0.3	1	160	200	200
Profile 3	565	705	C	0.5	1	160	200	200
Profile 3	705	910	D	0.6	1	300	380	375
Profile 3	910	1110	D	0.8	1	300	380	375
Profile 3	1110	1210	D	0.9	1	300	380	375
Profile 3	1210	1405	D	1.0	1	300	380	375
Profile 4	0	120	B	0.1	1	80	100	100
Profile 4	120	390	A	0.1	1			
Profile 4	390	500	D	0.4	1	300	380	375
Profile 4	500	560	D	0.5	1	300	380	375
Profile 4	560	615	D	0.6	1	300	380	375

Channel	Beginning Station (m)	End Station (m)	Channel Type	Discharge (cms)	Bottom Width (m)	d_{50} (mm)	d_{100} (mm)	Riprap thickness(mm)
Profile 4	615	740	D	0.7	1	300	380	375
Profile 4	740	1040	C	0.8	1	160	200	200
Profile 4	1040	1120	D	1.1	1	300	380	375
Clean Water	0	270	D	1.0	1	300	380	375

Figure 5-1 shows the ditches surrounding the waste rock dump area. The slope of the ditch in some area is steep and hence riprap protection is needed. Table 5-1 summarizes the ditch size and riprap requirement along the profiles. In Table 5-1, eight types of ditches are listed. Type A, B, C, D, E, F, G and H ditches have bottom width varying from 1 m. to 3 m. The different types of ditches are presented in Drawing H337696-4210-10-012-0001, Appendix C.

5.1.2 Ore Stockpile Platform

The offsite drainage area is about 0.2 km^2. The runoff is essentially undisturbed and is considered clean water. The design storm is the 100-year event. The runoff will be channelled into a North and a South channel. (See Figure 3-3) The outlet for these channels will be the existing drainage system.

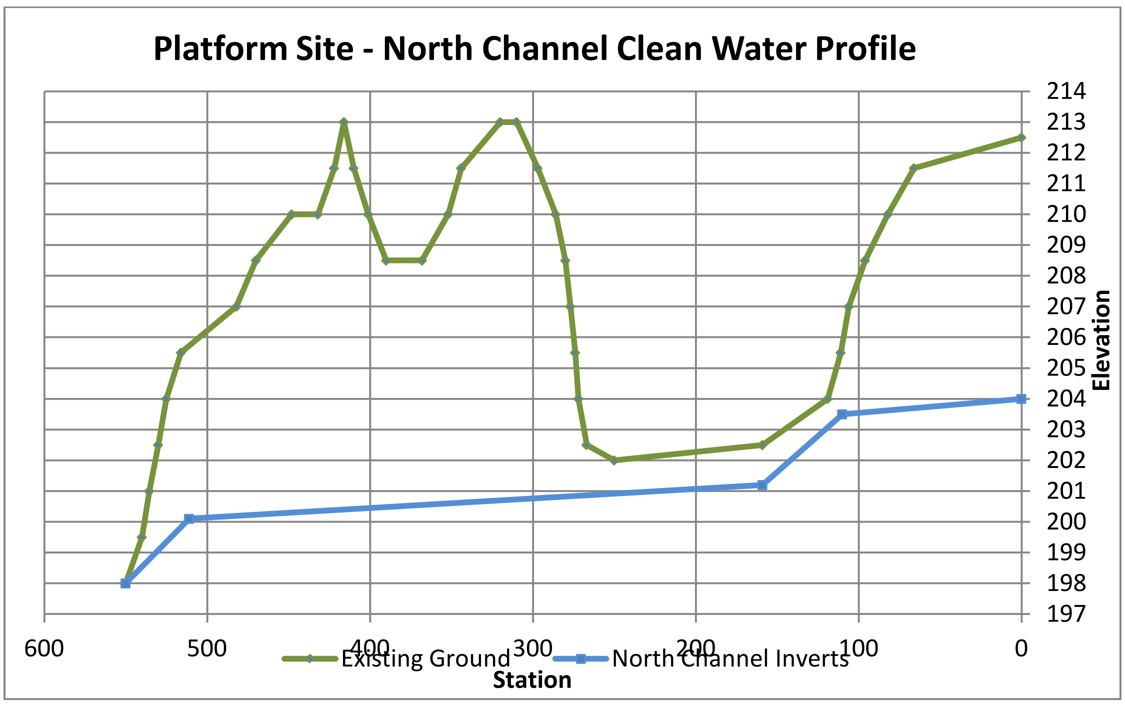

Figure 5-2: North Diversion Ditch (Ore Stockpile Platform

The channel profiles are shown in Figure 5-2 and

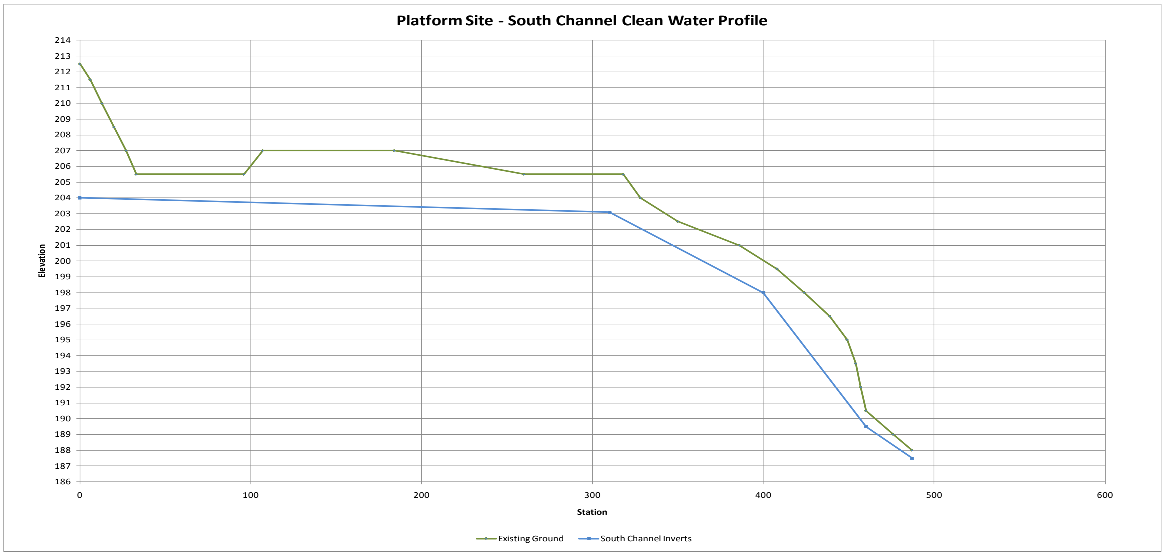

Figure 5-3. The channels range in slope from 0.3 percent to 14.2 percent. The discharges and D_{50} riprap for each section of the channel are shown in Table 5-2. The riprap for these channels was designed using References 6 and 7.

The interior of the Platform site will receive runoff from the stockpiles and will contain sediment. The design storm for the Platform site is the 10-year event. The drainage area for the Platform site is 0.23 km^2. The drainage of the area is served by grading the surface slope to flow to the surrounding ditches.

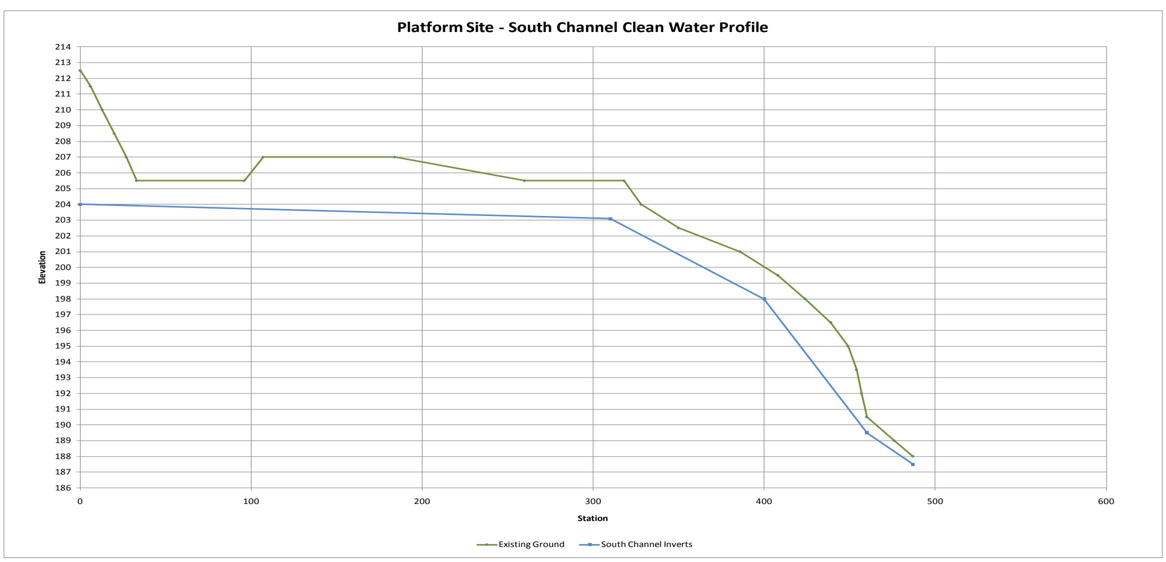

Figure 5-3: South Diversion Ditch (Ore Stockpile Platform)

Table 5-2: Ditch Size and Riprap Requirements (Clean Water Diversion Ditch)

Platform Clean Water Channels								
Channel	**Beginning Station (m)**	**End Station (m)**	**Channel Type**	**Discharge (cms)**	**Bottom Width (m)**	**d_{50} (mm)**	**d_{100} (mm)**	**Riprap thickness (mm)**
North	0	110	A	0.1	1			
North	110	159	B	0.1	1	80	100	100
North	159	511	A	0.45	1			
North	511	550	C	0.45	1	160	200	200
South	0	310	A	0.14	1			
South	310	400	C	0.14	1	160	200	200
South	400	460	D	0.14	1	300	380	375
South	460	487	C	0.14	1	160	200	200

5.2 Steensby Inlet

5.2.1 Ditch Surrounding Ore Stockpile Platform (Island)

This ditch collects flow from the ore stockpile platform and sends the runoff to the SWM pond for treatment. The total area is small and hence the minimum ditch with 1 m bottom width and 2:1 side slope will have sufficient flow capacity to carry the design flow of 0.32 m^3/s. The channel slope is 0.003 to 0.005

5.2.2 Ditch to the SWM Pond 2 (Fuel Farm and storage)

This ditch collects flow from the permanent laydown and storage area, and sends the runoff to the SWM pond for treatment. The total area is 61 ha. The ditch with 1 m bottom width and 2:1 side slope will have sufficient flow capacity to carry the design flow 1.56 m^3/s. The channel slope is 0.003 to 0.005. The ditches are shown in drawing number H337697-4510-10-014-0004 in Appendix C.

5.2.3 Clean Water Diversion Ditch

A clean water diversion ditch is required to allow the runoff from undisturbed area to bypass the system and to reduce the treatment requirement. Figure 3-3 shows the clean water ditches. From Figure 3-3 it is known that the ditch flows in two directions. The west part flows west and bypasses the SWM pond. The East ditch flows east and discharges to a existing water course north of the area. This ditch has a 1 m bottom width and 2 (H):1 (V) side slope with 0.005 channel slopes. The ditch has sufficient flow capacity to carry flows from the watershed. The ditch profiles and cross sections are presented in Drawing in Appendix C.

5.3 Milne Inlet

There are no permanent structures in Milne Inlet. The operations in this area will be short term activities. During operation, there is a need for drainage to avoid disturbance to works. For this reason, the drainage is aimed at draining stormwater into nearby streams without treatment (i.e. no stormwater ponds are required).

Figure 3-5 shows the overall drainage network for Milne Inlet site. In this area, if there is a stream nearby (about 150 m - 200 m), no ditches are planned. The land shall be graded to naturally drain to the existing stream. Where the distance to existing stream is longer than 150 m - 200 m, ditches are designed to collect the runoff and the ditches are then connected to the nearest existing stream.

The areas are small and the ditch having 1 m bottom width with 2 (H):1 (H) side slopes will be able to drain the stormwater generated from the areas. The ditches are shown in Drawing number H337697-7000-10-014-0001 in Appendix C.

6. Dams

The SWM ponds in the Mine Site and Steensby Inlet need embankment structures to create the storage required for stormwater treatment. This section describes the dam design aspect. First a dam safety assessment is performed to obtain the ICC rating of each dam structure and then important issues for the dam design are discussed.

6.1 Dam Safety Assessment

Due to the fact that the embankment structures for stormwater management meets the CDA definition of dams, according to the 2007 CDA guidelines, a dam safety assessment (DSA) was performed to evaluate the incremental consequence category (ICC) classification. This assessment is necessary since many of the design parameters must be consistent with the CDA dam safety requirements. If a dam is designed and constructed but it does not meet the dam safety requirements, it will have to do costly modifications to meet these requirements at a later stage. The design criteria are different for each ICC rating. The details of the dam safety assessment can be found in Appendix A. Here only the main conclusions are listed.

An ICC rating is based on an assessment of incremental impacts of dam failure on loss of life (LOL), social and economical losses and environmental impacts. If a dam causes hazard to the downstream area, this hazard is evaluated and rated based on the CDA guidelines.

Table 6-1: summary of Dam ICC ratings (Mine Site)

		Dam Height (m)	LOL	Social and Economic Loss	Environmental Damages	Overall
Pond 1	**Block Dam**	25	Low	Low	Significant	Significant
	Sediment Dam	25	Low	Low	Significant	Significant
	Discharge Dam	25	Low	Low	Significant	Significant
Pond 2 Dam		27	Low	Low	Significant	Significant
Pond 3 Dam		12	Low	Low	Significant	Significant

Table 6-2: S summary Dam ICC ratings (Steensby Inlet)

	Dam Height (m)	LOL	Social and Economic Loss	Environmental Damages	Overall
Pond 1 Dam	8	Low	Low	Significant	Significant
Pond 2 Dam	6	Low	Low	Significant	Significant

Based on CDA guidelines, the inflow design flood (IDF) and design earthquake (DE) for each structure are tabulated in Table 6-3 and Table 6-4.

Table 6-3: IDF and Design Earthquake Requirements (Mine Site)

		ICC	IDF	DE
Pond 1	**Block Dam**	Significant	1:200	1:1000
	Sediment Dam	Significant	1:200	1:1000
	Discharge Dam	Significant	1:200	1:1000
Pond 2 Dam		Significant	1:200	1:1000
Pond 3 Dam		Significant	1:200	1:1000

Table 6-4: IDF and Design Earthquake Requirements (Steensby Inlet)

	ICC	IDF	DE
Pond 1 Dam	Significant	1:200	1:1000
Pond 2 Dam	Significant	1:200	1:1000

6.2 Dam Section Design

6.2.1 Stability

Dam design is based on CDA guidelines for IDF, DE and stability. Table 6-5 summarizes the safety factors used for the Mary River Project dam design. Four load cases were checked. Table 6-5 summarizes the required Factor of Safety (FS) for the dam design based on CAD guideline corresponding to:

- steady state seepage corresponding to the normal water level (NWL)
- steady-state seepage at NWL in conjunction with earthquake loading
    - *Note:* The peak ground acceleration (PGA) for the site is 0.122 *g* based on data from the Canadian Geologic Society (CGS) corresponding to a 1:1000-yr return period. The detailed PGA for the site is shown in the Appendix B of the dam design report in Appendix B of this report.
- upstream slope stability subject to rapid drawdown
- slope stability of the upstream and downstream dam slopes at the end-of-construction before impounding water.

Table 6-5: Summary of the required Factor of Safety for Baffin land dam design based on CAD guideline

Load Combinations	Required Minimum FS	Type of Analysis
Steady Seepage corresponding to the NWL	1.5	Static analysis
Steady Seepage at NWL plus Earthquake Loads	1.0	Pseudo-static analysis
Upstream slope stability under rapid drawdown	1.2	Static analysis
Dam slope stability Just end of construction	1.3	Static analysis

6.2.2 *Thermal Conditions for Design*

The design basis thermal conditions are:

- The MAGT profiles at Baffinland Mary River is assumed to -10ºC (see Figure 6 of Appendix B)
- The reservoir-bottom mean water temperature is assumed to be 4ºC (see Figure 7 of Appendix B)
- The annual air temperatures was assumed to vary sinusoidally as follows:
  - max average air temperature is 7ºC in July
  - min. average air temperature is -25ºC in February.
- The natural active layer thickness is assumed to be 2 m (Wahl and Gharapetian, 2009)
- It is assumed that the foundation of the reservoir will thaw to the depth of 8 m in 50 years in the conceptual design stage.

6.2.3 *Additional Specific Requirement*

In addition to maintaining storm water retention requirements, the SWM ponds are required to have sufficient retention time to facilitate sedimentation of sediment within the reservoir (section 4.4.1). A small amount of seepage is required to help maintain the water level in control. The required seepage is assumed to be in the order of 10 L/s for the entire dam. This can be maintained by designing the dam to allow for controlled seepage to meet the flow requirements.

The anticipated type of service of the embankment is to retain water continuously.

6.3 Dam Section

Figure 6-1 presents a typical dam section for Mine site SWM Pond 2 dam. The dam has the following features:

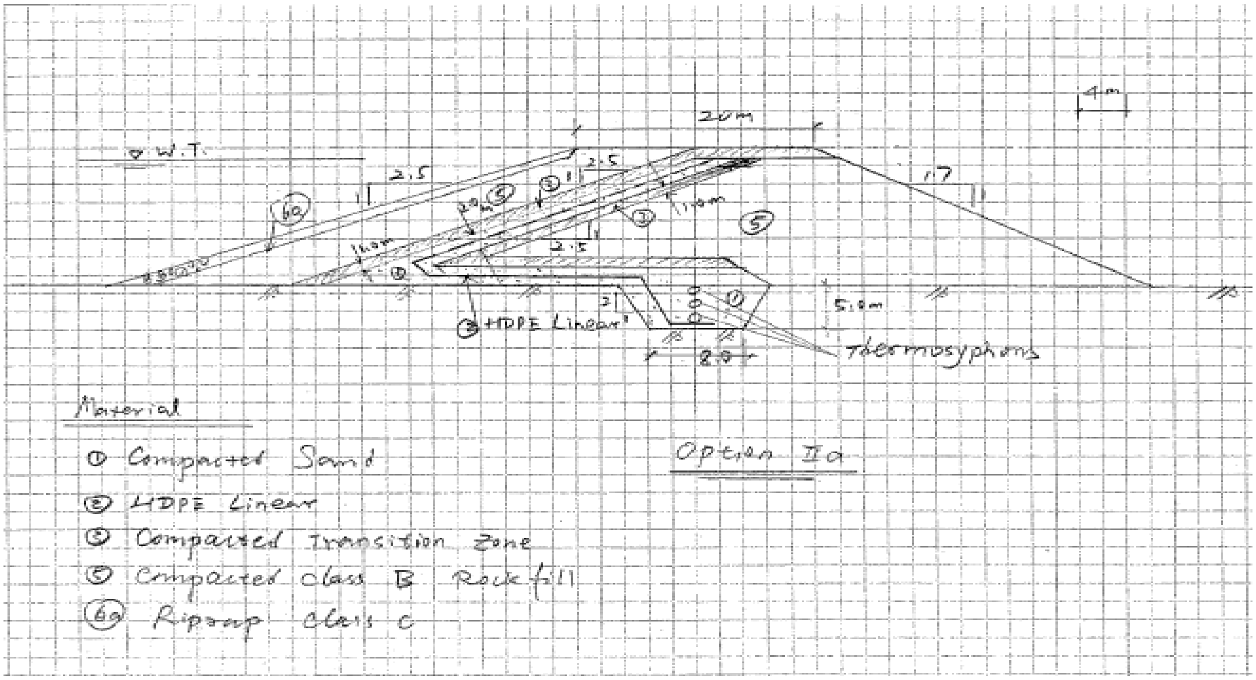

Figure 6-1: Typical Dam Cross Section

The dam consists of a rock fill dam with an HDPE liner as the primary seepage barrier. The main materials in this dam option consist of:

- Zone 1 – Bedding Material (Sand 0-13 mm or crusher fines)
- Zone 2 - Transient Zone
- Zone 3 – Compacted Rock fill
- Zone 5 - Riprap-Class C.

This dam section has been considered to permit a small amount of seepage through the upper part of the dam to control the reservoir during normal operating conditions. An additional liner is proposed to allow controlled seepage of water through the embankment without permitting it to enter the frozen key trench.

The estimated dam geometry consists of a 20 m wide crest (road traffic requirement) for this dam, 2.5H:1V U/S slope gradient, 1.7H:1V D/S slope gradient, a frozen key trench extending 5 m below ground surface and thermal siphons to maintain the thermal regime of the key trench. The crest of this dam is used for the access road and hence the crest width is design to be 20 m. For other dams, the crest width is set to be 5 m.

All of the dams use the same configuration with different crest elevation, spillway invert elevation and impermeable core elevation. These are summarized in Table 6-6 and Table 6-7.

Table 6-6: Dam Design Features (Mine site)

		Crest			Slope (H:V)		Spillway			Height m
		Elevation m	Width m	Length m	Up-stream	Down-stream	Width m	Side slope	Invert m	
Pond 1	Block Dam	355.0	5	150	2.5:1	1.7:1	-	-	-	25
	Sediment Dam	347.5	5	150	2.5:1	1.7:1	10	2:1	344.5	25
	Discharge Dam	329.0	5	150	2.5:1	1.7:1	10	2:1	326.0	25
Pond 2 Dam		547.5	20	800	2.5:1	1.7:1	10	2:1	544.5	27
Pond 3 Dam		204.5	5	400	2.5:1	1.7:1	10	2:1	203.5	12

Table 6-7: Dam Design Features (Steensby Inlet)

	Crest			Slope (H:V)		Spillway			Height m
	Elevation m	Width m	Length m	Up-stream	Down-stream	Width m	Side slope	Invert m	
Pond 1 Dam	13.0	5	600	2.5:1	1.7:1	10	2:1	10.5	8
Pond 2 Dam	40.0	5	500	2.5:1	1.7:1	10	2:1	38.0	6

7. Material Take Off Estimates

Material take off estimations were undertaken for the ditches and dams. These MTO estimations reflects the current design conditions. Some of the design may be modified and hence new MTO estimations will have to be undertaken when changes are made.

7.1 Ditches

A: Mine Site Waste Rock Dump ditches:

- Excavation volume: 234,000 m^3
  - Riprap volume and filter: 21,274 m^3
  - Fill material volume: not expected
  - Geo textile: 62,597 m^2

B: Mine site Ore Stockpile Clean Diversion Water Ditch:

- Excavation volume: 2,400 m^3
  - Riprap volume: not expected
  - Fill material volume: not expected

C: Mine site Ore Stockpile Drainage Ditch

- Excavation Volume (clean water Diversion): 147,600 m^3
  - Riprap volume: not expected
  - Fill material Volume: 183,545 m^3

D: Steensby Island Drainage Ditch

- Excavation Volume: 38,300 m^3
  - Fill material Volume: 729,760 m^3

E: Steensby Clean Water Diversion Ditch

- Excavation Volume: 103,700 m^3
  - Fill material Volume: Not expected

F: Steensby Drainage Ditch on the fuel farm and storage area

  - Excavation volume: 236,100 m^3
  - Fill material Volume: not expected

G: Milne Inlet Drainage Ditch

  - Excavation volume: 9,000 m^3
  - Fill volume: not expected

7.2 Dams

- Mine site Pond 1 Block Dam: 25,000 m^3
- Mine site Pond 1 Sediment Dam: 110,000 m^3
- Mine site Pond 1 Discharge Dam: 90,000 m^3
- Mine site Pond 2 Dam: 551,500 m^3
- Mine site Pond 3 Dam:
  - fill material: 152,837 m^3
  - excavation at spillway: 855 m^3
- Steensby Inlet Pond 1 Dam: 285,000 m^3
- Steensby Inlet Pond 2 Dam: 11,300 m^3
- Excavation at spillway 17,000 m^3

8. Remaining Works

The current design deals with the overall stormwater management and drainage system for the Mine site, Steensby Inlet and Milne Inlet. The major structures have been designed. There are still details to be completed in the next phase of the design. The detailed design will include:

- Hydraulic design of spillway structures, energy dissipater (if required) and erosion control measures. At this stage, the spillway dimensions were determined to make sure that the dams can pass the inflow design flood.
- Dam sections were designed to be stable under different load conditions. The section is not intended to allow overtopping of the dam body since the dam is an embankment structure and overtopping of the dam body shall be avoided. However, it is known that Pond 1 in Mine site and Pond 2 in Steensby may have to allow overtopping of the dam body due to various constrains. For these dams, special design will be required to allow overtopping.
- Culverts at several locations where ditches cross roads and / or other structures. At these locations, culverts are needed.
- It has to be realized that the design is a dynamic process. Some of the design features may need to be adjusted to meet the requirements of other disciplines. A few iterations between different requirements may be needed to make the entire system work. Therefore, some additional works will be required to make adjustments in the next phase of the design.

9. References

1. Canadian Dam Association, 2007, Canadian Dam Safety Guidelines
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Attachment A

Dam Safety Assessment Memo

Project Memo

August 23, 2011

TO: John Binns FROM: Ross Zhou

Baffinland Iron Mines Corporation
Mary River Project

Dam Safety Assessment

1. Introduction

The Mary River Project is a proposed iron ore mine and associated facilities ocated in northern Baffin Island, in the Qikiqtani Region of Nunavut. The Project involves the construction, operation, closure, and reclamation of a 18 million tonne-per-annum open pit mine that will operate for 21 years. The high-grade iron ore to be mined is suitable for international shipment after only crushing and screening with no chemical processing facilities. A railway system will transport an additional 18 Mt/a of ore from the mine area to an all-season deep-water port and ship loading facility at Steensby Port where to ore will be loaded into ore carriers for overseas shipment through Foxe Basin.

In the drainage system for stormwater management at the Milne Port, the Steensby Port and the mine site, dykes will be constructed for establishing stormwater management ponds. Based on the definition of Canadian Dam Association's Dam Safety Guidelines (CDA, 2007), a water retaining structure with storage over 30,000 m^3 and height exceeding 2.5 meters is defined as a dam and hence must meet the dam safety requirements. The dam safety requirements consistent of many aspects including risk management system, meeting the design standards and having proper operation, maintenance and surveillance procedures (OMS). And if the dam is classified as HIGH incremental hazard potential (IHP), a proper emergency preparedness and response plan (EPRP) is required.

Due to the fact that the embankment structures for stormwater management meets the CDA definition of dams, according to the 2007 CDA guidelines, a dam safety assessment (DSA) was performed to evaluate the incremental consequence category (ICC) classification. This assessment is necessary since many of the design parameters must be consistent with the CDA dam safety requirements. If a dam is designed and constructed but it does not meet the dam safety requirements, it will have to do costly modification to meet these requirements at later stage. The design criteria are different for each ICC rating. Therefore, a ICC classification must be assessed before any actual work starts.

This dam safety assessment (DSA) is not a full scaled DSA and hence it only addresses the main issues to allow the selection of proper inflow design flood and design earthquake. Many other aspects required by the CDA guidelines will have to be addressed later (for example, if a dam is classified as HIGH ICC structure, EPRP document must be prepared. If required, the EPRP will be done in later stage).

2. CDA Dam Classification and IDF Requirements

Dam classification forms the basis of dam design criteria. Every dam must first be classified based on consequences or risk of dam failure. The CDA dam classification system is presented in Table 2- 1. In the table, a classification of consequences is based on three aspects: incremental loss for loss of life (LOL), Environmental and cultural values (EC), and infrastructure and economics (IE). Based on the degree of damages, each dam will be assigned a incremental consequence category (ICC). The inflow design flood (IDF) will be determined according to the ICC classification.

Table 2- 1: Dam Classification

Dam class	Population at risk [note 1]	Incremental losses		
		Loss of life [note 2]	Environmental and cultural values	Infrastructure and economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area contains limited infrastructure 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 kind highly possible	High economic losses affecting infrastructure, public transportation, and 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 affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of *critical* fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

Note 1. Definitions for population at risk:

None—There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.

Temporary—People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).

Permanent—The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).

Note 2. Implications for loss of life:

Unspecified—The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

Table 2- 2: Inflow Design Flood Requirement (CDA, 2007)

Consequence Class	IDF
Low	1/100-year
Significant	Between 1/100 and 1/1000-year (Note 1)
High	1/3 between 1/1000-year and PMF (Note 2)
Very High	2/3 between 1/1000-year and PMF Note 2)
Extreme	PMF

Note 1. Selected on basis of incremental flood analysis, exposure and consequence of failure.
Note 2. Extrapolation of flood statistics beyond 1/1000-year flood (10^{-3} AEP) is generally discouraged. The PMF has no asssociated AEP. The flood defined as "1/3 between 1/1000-year and PMF" or "2/3 between 1/1000 year and PMF" has no defined AEP.

Table 2- 2 presents the IDF requirement corresponding to each of the ICC classification.

According to the CDA 2007 Dam Safety Guidelines, each dam has to be evaluated separately. This memo describes the results of the assessment for each structure in the mine site, the Milne port and Steensby Port. At this stage, there is no dam safety guidelines in Nunavut and hence the assessment will use the CDA guidelines as the basis of the evaluation.

3. ICC Classification

3.1 Minesite Stormwater Pond 1 Discharge Dam

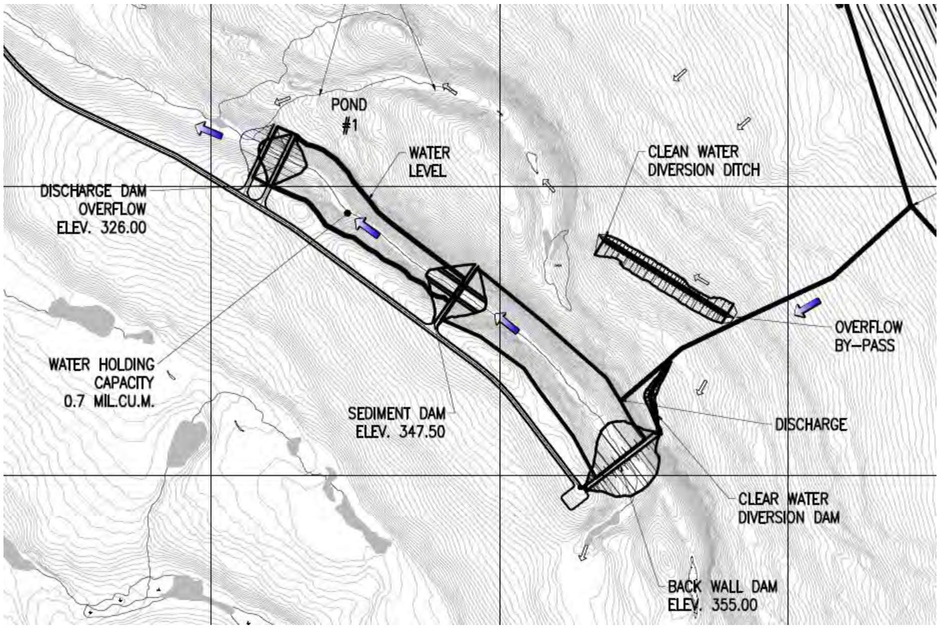

Figure 3-1: Dam Locations of Pond 1

This dam is the downstream most structure to retain stormwater in Pond 1. The dam shown on Figure 3-1 has an overflow weir at elevation 326 m. Te length of the dam at the crest is about 150 m. The height of the dam is about 25 m. The dam retains 0.7 million m^3 of water at the normal water level. If the dam fails, the released water will be discharged to the downstream area and eventually be stored in Camp Lake.

There is an access road which may have some erosion damages. In the downstream area, there is no permanent residents and hence no loss of life (LOL) will be resulted. The sediment in the pond will be released to the downstream area and may reach Camp Lake. The sediment will settle in Camp Lake leading to some environmental damages to the lake water quality. The water in Camp lake is used for water supply and hence this high concentration of sediment may have some impacts to the water quality. According to this description, the dam will have zero (0) LOL. There will be no third party economic losses. Therefore, this dam is classified as LOW incremental consequence category (ICC) for LOL and Economics. With respect to the environmental losses, the ICC is classified as SIGNIFICANT due to the impacts to water quality in the downstream area.

The overall ICC category is then SIGNIFICANT.

Based on the CDA guidelines, the inflow design flood shall be between 1:100 year and the 1:1,000 year flood. Due to the relatively low impacts to the downstream area from LOL and economic aspects, and is significant for environmental impact, a 1:200 year design flood is appropriate.

For earthquake, the design level will be the 1:1,000 event based on the CDA guidelines.

3.2 Minesite Stormwater Pond 1 Sediment Dam

This dam is located upstream of the discharge dam (Figure 3-1) and downstream of the back wall dam. This dam is acting as sediment barrier for the stormwater pond. The dam is approximately 25 m high and crest length is about 150 m. The crest elevation is at 347.5 m. If the dam fails, the water will be retained in the downstream pond between the discharge dam and the sediment dam. Then if the discharge dam fail because of the failure of the sediment dam, the ICC is SIGNIFICANT. Therefore, the sediment dam will have the same ICC classification as the discharge dam. The design flood shall therefore be the 1:200 year event. The design earthquake will be the 1:1,000 year event.

3.3 Minesite Stormwater Pond 1 Back Wall Dam

The back wall dam is located on the upstream end of the stormwater Pond 1 to form the upstream cell of the pond. The dam is 25 m high and about 150 m long at the crest. If the dam fails, there will be no LOL and no third party economical damages. The environmental impact would be significant because the released water contains high concentration of sediment from the waste rock stockpile. The overall ICC category assigned to this dam is SIGNIFICANT.

The inflow design flood for this dam shall be the 1:200 year flood and the design earthquake is the 1:1,000 year event.

3.4 Minesite Stormwater Pond 2 Dam

Figure 3-2 shows the location of the Pond 2 dam. This dam is approximately 15 m high and 800 m long at the crest. The volume of water stored is in the order of 500,000 m^3. The dam crest is an access road. The dam discharges to the Mary River.

If the dam fails, the outflow will enter Mary River and be discharged to the downstream water course. There will be no LOL since there are no residents in the downstream area. The economical damage will be the road operation which is a short term and internal damages. There is no third party damages. Therefore the ICC for LOL and economical damages are LOW.

For environmental damages, there will be high concentration of sediment released to the Mary River and this will lead to water quality problem. But the impact shall be short term water quality problem. The ICC classification for this dam is therefore SIGNIFICANT.

Based on this classification, the inflow design flood shall be the 1:200 year flood and the design earthquake will be the 1:1,000 event.

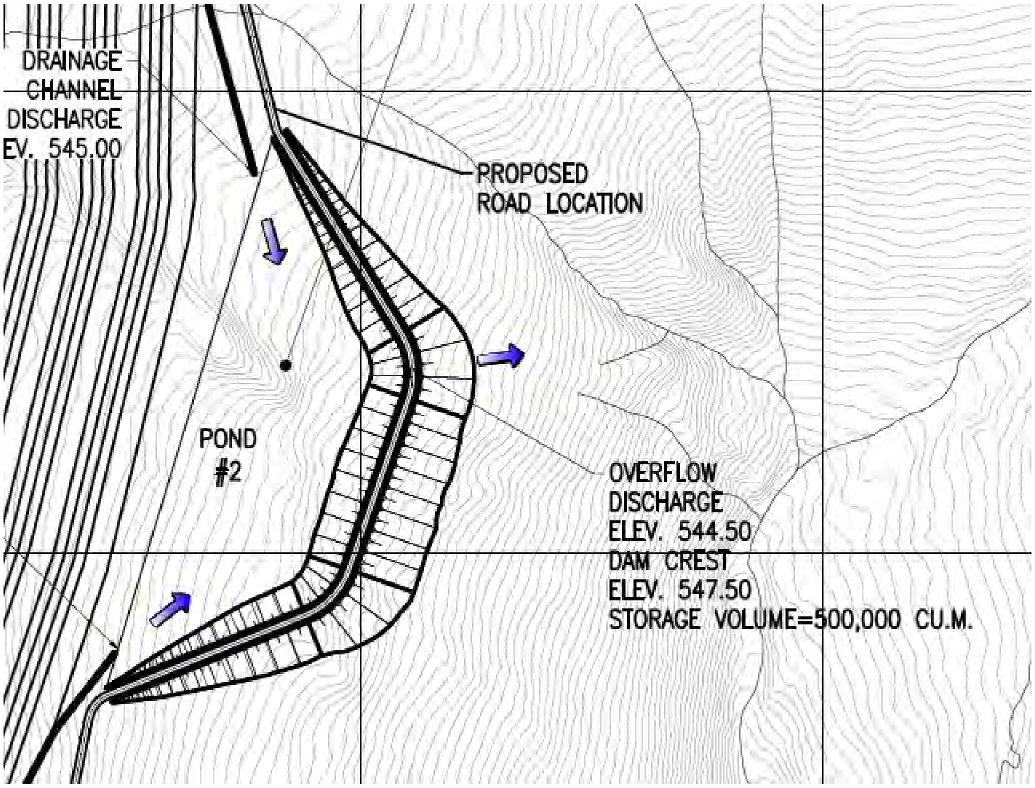

Figure 3-2: Minesite Pond 2 Dam

3.5 Minesite Stormwater Pond 3 Dam

This dam is located downstream of ROM Stockpile to form the stormwater management pond. The dam is shown on Figure 3-3.

The dam crest elevation is 264.3 m. The dam is 9.3 m high and about 150 m long. The storage is 35,000 m^3.

The failure of this dam will lead to no LOL and third party economical damages and hence the ICC for LOL and Economic damages are LOW. The failure of the dam will lead to high concentration of sediment be released to Mary River which will have short term water quality impacts to the river. The ICC assigned to the dam for Environmental aspect is SIGNIFICANT. And the overall ICC classification is SIGNIFICANT.

The inflow design flood shall therefore be the 1:200 year flood and the design earthquake is the 1:1,000 year event.

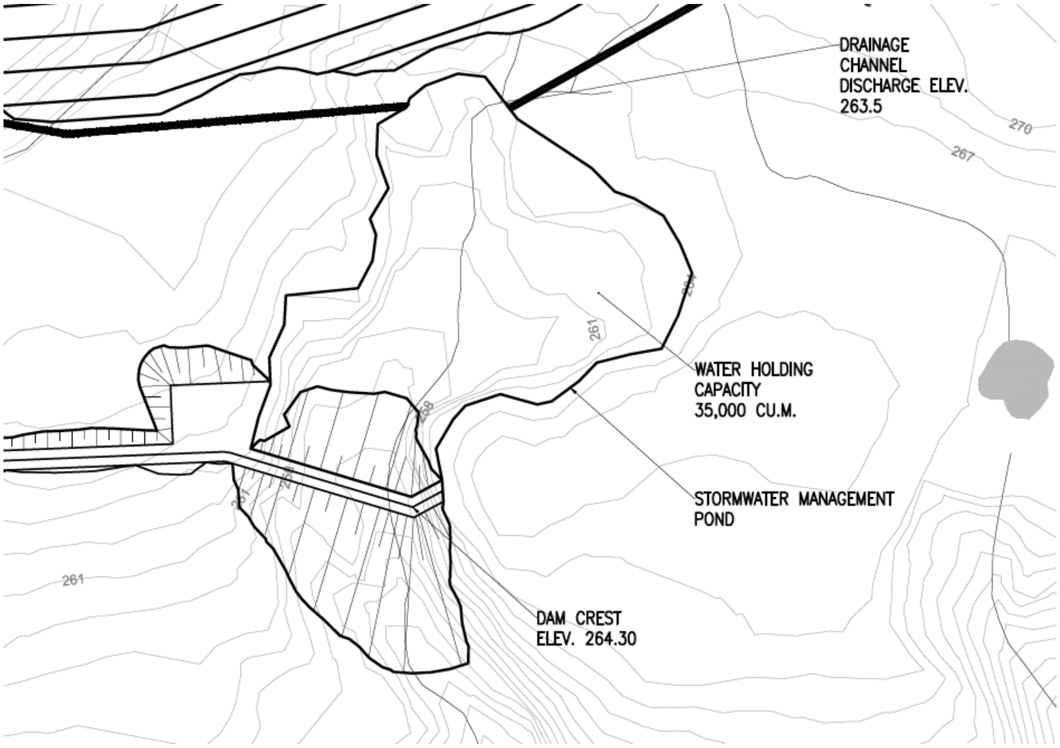

Figure 3-3: Stormwater Management Pond 3 Dam

3.6 Minesite Stormwater Pond Dam

This dam is located just upstream of the waste water clarification pond. The dam is about 12 m high and more than 400 m long. The storage capacity of the pond is 150,000 m^3. If the dam fails, there will be no LOL and third party economical damages. Therefore, the ICC for LOL and economical losses are LOW. The released water will lead to water quality problem in Sheardown Lake. The ICC classification for environmental impact is SIGNIFICANT. The overall ICC for this dam is then SIGNIFICANT.

The inflow design flood shall be the 1:200 year flood and the design earthquake level is the 1:1,000 year event.

The dam is shown on Figure 3-4.

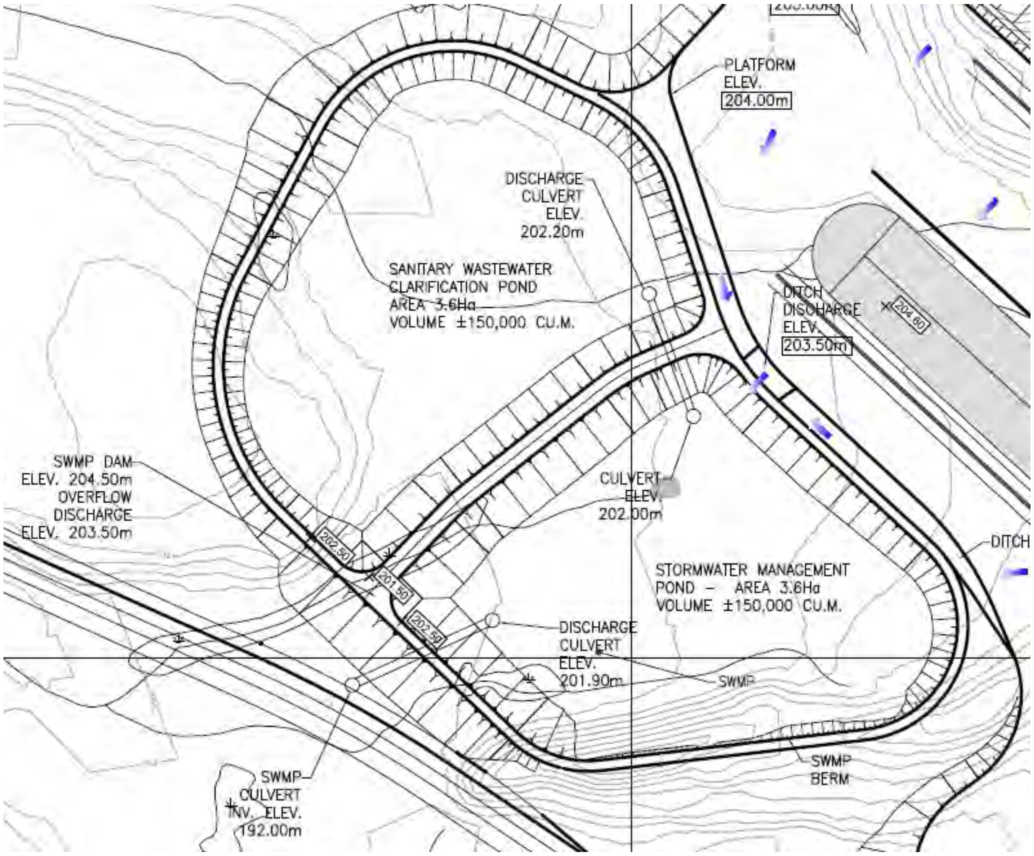

Figure 3-4: Stormwater Management Pond 4 Dam

3.7 Milne Port Stormwater Pond 10 Dam

This dam is shown on Figure 3-5. This dam is located on the west side of the proposed ore stockpiles in the port operating area. The pond collecting runoff from the stockpile and then the runoff will be pumped to Pond 9. The storage capacity of the pond is 40,000 m^3, the dam height is about 6 m. and the crest length is about 250 m.

If the dam fails, the storage will be discharged to Phillips Creek. The downstream 1,200 m runway will be flooded. There will be no LOL and no third party economic losses. The ICC for LOL and economical losses are LOW. The released sediment will lead to environmental damages to the downstream Phillips Creek. The environmental loss is classified as SIGNIFICANT. The overall ICC is SIGNIFICANT.

The IDF for this dam shall be the 1:200 year flood and the design earthquake is the 1:1,000 year event.

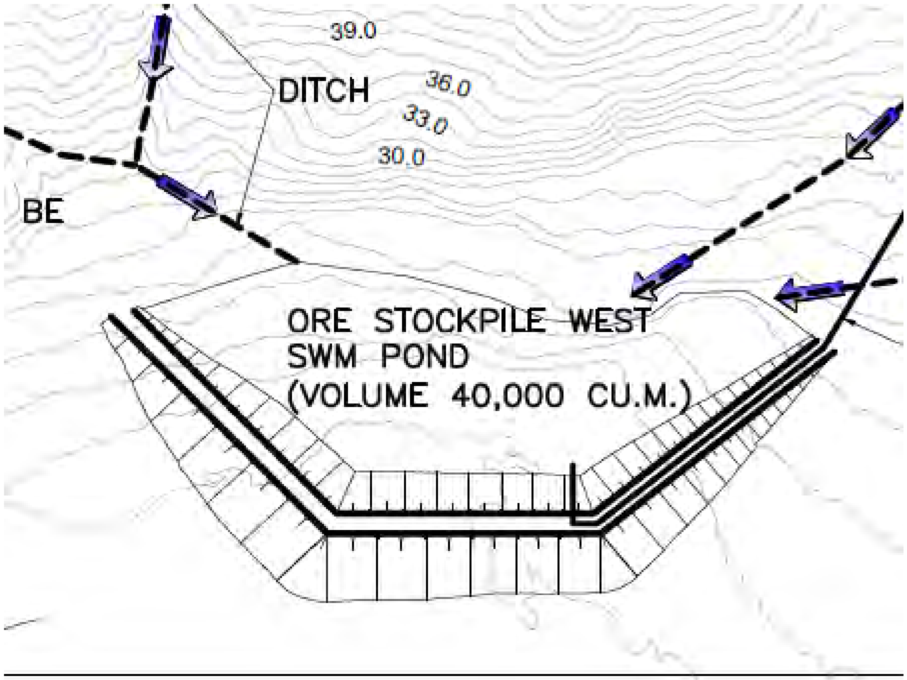

Figure 3-5: Pond 1 Dam (Milne Port)

3.8 Milne Port Stormwater Pond 9 Dam

This dam is the ore stockpile east pond within the platform. The dam crest is 52 m and the depth of the dam is about 6 m. The total storage of pond has is 200,000 m^3. There will be no LOL and third party economical damages if the dam fails since the pond is located just upstream of the ocean and hence the failure of the dam will lead flows be discharged into the ocean. Therefore, the ICC for LOL and economical damages are LOW. The released water contains high concentration of sediment which will lead to some environmental damages to the downstream water body. The ICC for environmental damages is SIGNIFICANT.

To properly design the dam, a 1:200 year flood shall be used for inflow design flood and the design earthquake is the 1:1,000 year event.

Figure 3-6 shows the general layout of the proposed stormwater management pond.

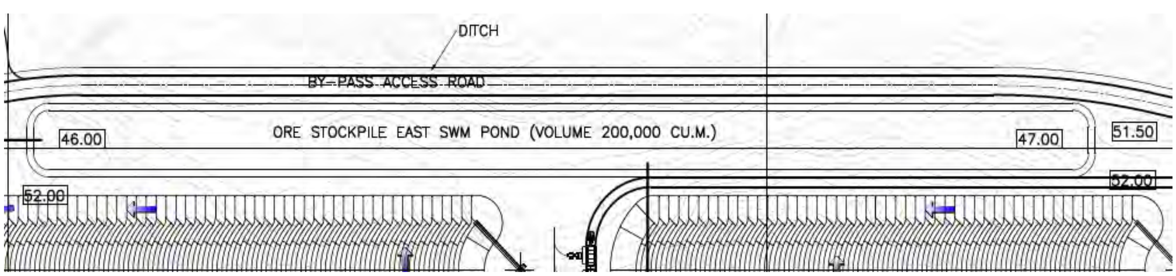

Figure 3-6: SWM Pond no. 9, Milne Port

3.9 Steensby Port, Ore Stockpiles Stormwater Management Pond Dam

This dam is shown on Figure 3-7. The dam is about 8 m high and 600 m long. The pond has a storage capacity of 125,000 m^3. There will be no LOL and third party economical damages if the dam fails since the pond is located just upstream of the ocean and hence the failure of the dam will lead flows be discharged into the ocean. Therefore, the ICC for LOL and economical damages are LOW. The released water contains high concentration of sediment which will lead to some environmental damages to the downstream water body. The ICC for environmental damages is SIGNIFICANT.

To properly design the dam, a 1:200 year flood shall be used for inflow design flood and the design earthquake is the 1:1,000 year event.

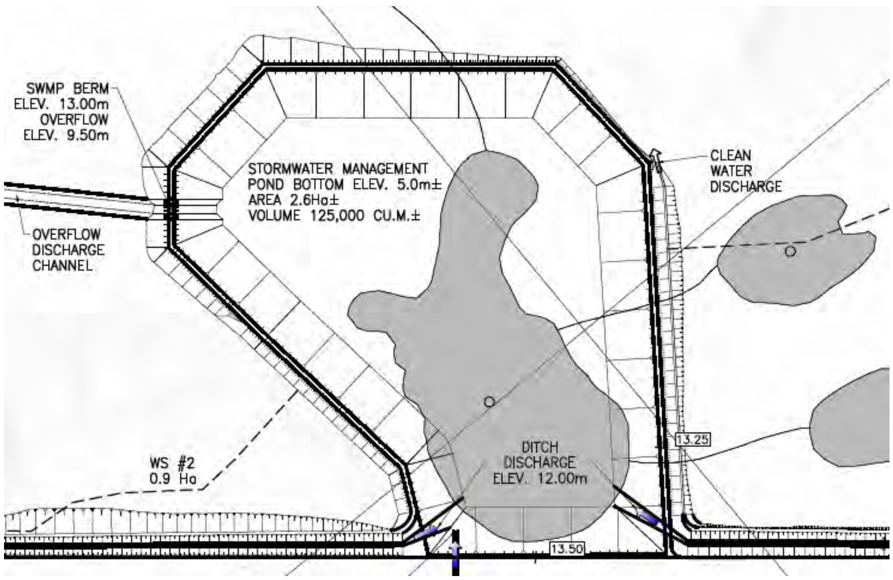

Figure 3-7: Steensby Port Ore Stockpile Stormwater Management Pond

3.10 Steensby Port, Platform Stormwater Management Pond Dam

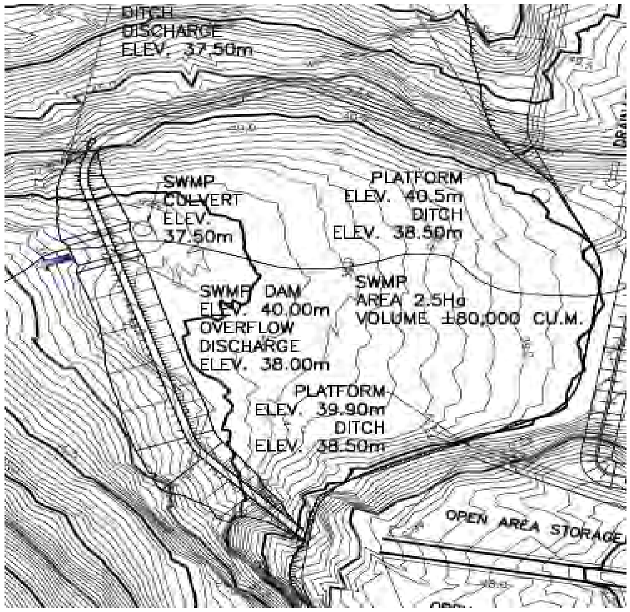

Figure 3-8: Platform Stormwater Management Pond Dam

This dam is located at the west side of the platform for collecting stormwater from the platform area. The dam height is about 4 m and the length of the crest is about 500 m. The total storage is about 80,000 m^3.

If the dam fails, there will be no LOL and third party economical damages. The ICC for LOL and economical losses are LOW. for environmental damages, there will be short term water quality problem to the ocean. The ICC for environment perspective is therefore SIGNIFICANT. The overall ICC is SIGNIFICANT.

Therefore, the inflow design flood for this dam shall be the 1:200 year flood and the design earthquake is the 1:1.000 year event.

4. Freeboard Requirement

For preventing overtopping of the crest during significant wind event, a minimum of 0.9 m freeboard is required during the passage of the inflow design flood (USBR, 1987).

5. Conclusions

It is concluded based on this assessment that:

1. The stormwater management dams have a SIGNIFICANT ICC classification based mainly on environmental damages to the water quality. The LOL and economical damages are LOW. Due to this ICC rating, the inflow design floods and design earthquake are determined
2. The inflow design flood corresponding to the SIGNIFICANT ICC rating shall be the 1:200 year flood. The IDF will be used for designing the spillways for each of the dams.
3. The design earthquake level will be the 1:1,000 year event. The design earthquake will be used for determining the embankment stability during dynamic conditions.

6. References

- Baffinland Iron Mines Corporation, 2010, Mary River Project, Environmental Impact Statement
- CDA, 2007, Dam Safety Guidelines, Canadian Dam Association
- USBR, 1987, Design of Small Dams, A Water Resource Technical Publication, US Bureau of Reclamation

RZ:rz

Attachment B

Dam Design Report

Project Report

Baffinland Mary River Project

Mary River Project

Conceptual Design for Dam

2011-11-09	A	Approved for Use - Environmental Permit	G. Liang	S. Hinchberger	J. Binns	
DATE	REV	STATUS	PREPARED BY	CHECKED BY	APPROVED BY	APPROVED BY
YYYY-MM-DD			HATCH			CLIENT

Table of Contents

Safety • Quality • Sustainability • Innovation

1. Introduction

The Mary River Project is located on the northern half of Baffin Island at Latitude 71° and Longitude 79° approximately 1000-km northwest of Iqaluit, the capital of the Nunavut Territory The mineral properties of BIM consist of three mining leases covering a total area of 1593.4 ha. The Project involves the construction, operation, closure, and reclamation of a 21 million tonne-per-annum open pit mine that will operate for 21 years. In addition to developing the mine site, two ports (Steensby and Milne) will be developed to transport ore from Baffin Island to processing facilities elsewhere. Figure 1-1 shows the location of the Mary River Project.

As part of the storm water management system, Baffinland has identified the need for a series of storm water management (SWM) ponds at the Milne Port, the Steensby Port and the Mine Site, respectively. The SWM ponds will require construction of embankment dykes or dams on permafrost. The current report focuses on embankment dam options at the Mine Site only. Two conceptual dam designs are presented along with a recommendation of the preferred option. It is also felt that the options can also be considered for the Milne and Steensby Ports. The following sections summarize: i) Scope of the work; ii) General background; iii) Conceptual design of the embankment; iv) discussion and v) conclusion and recommendation.

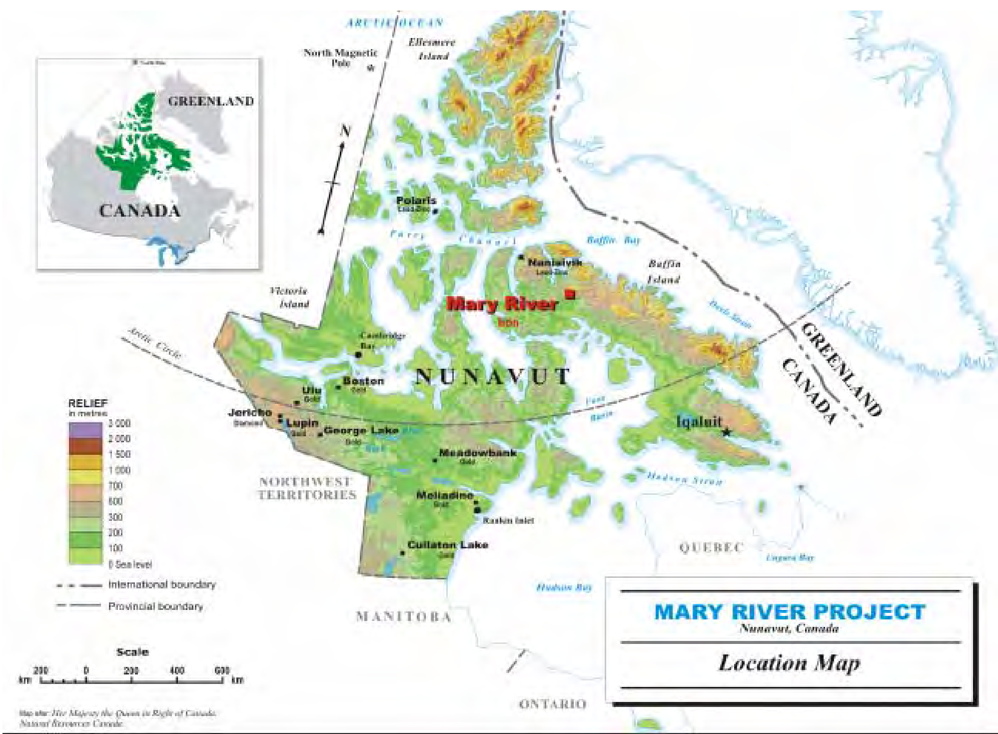

Figure 1-1: Location of Baffinland Mary River Project

1.1 Scope of work

Earth and rock fill embankment dams with various heights are required to provide containment for SWM ponds at the Mary River sites (Mine Site). Hatch was retained to develop a conceptual design for the proposed dams. The scope of work at this stage included:

- A literature review of the design and construction of embankment dykes and dams on permafrost.
- A review of the site specific geothermal data for embankment/dam design.
- Assessment of the main factors to be considered for design.
- Establishment of a preliminary design basis for the embankment/dams.
- Develop feasible embankment / dam options.
- Identify the preferred option for the next design stage.

In this report, the conceptual design of the dams at Mine site SWM Pond 2, 3 and the Milne Port SWP 10 are considered. The maximum dam height is 15 m; However, the recommended dam option could be used at the Mary River sites.

2. Literature Review-Embankment Dams Built on Permafrost

The design, construction and operation of embankment dams and dykes in the Arctic and Subarctic present unique problems related to freezing temperatures and the behaviour of frozen soil and rock materials exposed to the influence of unfrozen water. Challenges of major concern typically experienced in these conditions include:

- Thawing of structures foundered on permafrost.
- Earth movement due to freeze-thaw cycles.
- Placing of frozen soils and fill.
- Scheduling of construction in remote areas having a harsh climate.

A literature review was conducted to investigate the performance of previously constructed embankment dams and dykes on permafrost. Table 2-1 summarizes the typical problems experienced (Francis, 1987), which can be categorized as either (a) Thawing erosion due to seepage (see cases No. 1-5) and failure to maintain the thermal regime of the embankment and its foundation (see cases No. 6-15). Table 2-2 provides a summary of successful cases of embankment dam construction on permafrost. The typical dam sections in Appendix A correspond to the cases shown in Table 2-1and Table 2-2.

The design of water-retaining embankments on permafrost can be divided into two general types: i) frozen and ii) thawed. Frozen embankments are design so that the embankment and its foundation are frozen through the entire service life of the structure. In contrast, thawed embankments are designed to thaw during the service life and design provisions are made to accommodate and tolerate the effect of thawing. The following summarizes the main considerations for designing embankments in cold regions and for selecting the type of embankment for a particular site.

- the anticipated type of service of the embankment (i.e. retain water continuously or only intermittently)
- the width, depth, temperature and chemical composition of the body of water to be retained by the embankment
- regional and local climate conditions
- the temperature of the existing permafrost
- the extent and depth of permafrost in the area
- the availability and type of earth materials available for construction
- the accessibility of the construction site for logistics involving man-made construction materials (i.e. geomembranes and geosynthetics)
- the effects of construction and operation of the embankment and reservoir on the environment
- frost action on dry slopes and crest of the embankment
- the consequences to life, property, and environment in the event of embankment failure
- the orientation of the downstream slope (dry slope) of the embankment with respect to solar radiation
- the economics of constructing a selected design in the code region.

Table 2-1: Summary of Embankments built on Permafrost– Embankments with Problems (Francis, 1987)

No.	River Name	Location	Embank. Type	Height (m)	Length (m)	Built Year	Problem Description	Reference
1	Unknown	Northern Former USSR	Compacted earth	21.4	230	1967	Municipal water supply dam was completed in 1970. A breach occurred through embankment at supply intake pipes due to thermal erosion and seepage.	Anisimov & Sorokin (1975)
2	Hess Creek	Livengood, AK	Hydraulic & compacted earth fill	24	488	1946	The dam was for mine water supply. In 1962, embankment breached at interface with spillway due to thermal erosion and seepage	Rice & Simoni (1963)
3	Myla River	Zarechnyy Region, Former USSR	Compacted frozen sand	-	-	1954	Constructed of un-compacted frozen sand during winter. Seepage through earth dam and joints in wooden spillway caused thawing and failure of dam in 1954	Lyskaniv (1964)
4	Vilyuy River (Dam I)	Former USSR	Sand and silt Dyke w/ crib cut off	12	300	1960	During initial operation in 1960, large seepage occurred and spillway was completed destroyed in the first flood. After reconstruction in 1969, leakage was observed from reservoir through caverns in the foundation and at contact points with the spillway. Causes of problems: i) spillway too small for flood; 2) ice-retaining structures not located far enough U/S from the dam; 3) fissures in the foundation not sealed; 4) poorly compacted cutoff	Biyanov (1966)
5	Vilyuy River (Dam II)	Former USSR	Embank. with clay-ice core	3	-	1960	Clay-ice core constructed in winter with frozen clay and water. Seepage along spillway-embankment contact resulted in degradation of frozen core and loss of water-retaining function.	Biyanov (1966)
6	Dolgaia River	Noril'sk, Former USSR	Refrigerated earth	10	130	1942	A "Clay-concrete core" with two rows of freezing pipes parallel to dam axis. Thermal region of the core was not maintained causing thawing of the embankment	Tsvetkova(1960), Borisov & Shamshura (1959)
7	Srednity El'gen River	Kolyma River Basin, Former USSR	Earth	-	-	-	Large deformation and cracks occurred along the dam due to seepage and thawing. Seepage developed where timber piling was used as a cutoff.	Tsvetkova (1960)

No.	River Name	Location	Embank. Type	Height (m)	Length (m)	Built Year	Problem Description	Reference
8	Myaun-dzha River	Kolyma River Basin, USSR	Earth fill w/core	8	860	1952	The abutment of the dam was not protected by freeze pipes and thawing occurred at this location in the summer. The ensured seepage caused failure of the dam	Tsvetkova (1960)
9	Amozer River	Near Mogocha on the Amer Railroad, USSR	Grib-core Rock-earth fill	4	-	1910-1916	Failed due to seepage and thawing through body of the embankment	Tsvetkova (1960)
10	Kvadrat-nyy River	Noril'sk, USSR	Compacted earth-fill	6	-	-	Dam used for cooling water supply for electric power station. Failed within one year after construction by thawing of foundations and abutment soils	Biyanov & Shamshura (1959)
11	Stake 89 (Picket Creek)	Noril'sk, USSR	Compacted earth-fill	5.5	-	-	Failed two years after construction when seepage through the unfrozen soil thawed the frozen soil.	Tsvetkova (1960)
12	Mykyrt River	City of Petrovsk-Zabaykalskiv, USSR	Earth	9.5	-	1792	In attempting to repair the wooden spillway of the 137-yr-old dam, proper measures were not taken to preserve the frozen embankment and it failed. The dam had to be completely rebuilt in 1945.	Tsvetkova (1960)
13	Pravaya Magda-gacha River	Northern Previous USSR	Compact earth with concrete diaphragm	7.3	-	-	Failed after two years of operation. Large deformation of dam resulted in cracks in the diaphragm all along the embankment dam and at the junction of the weir. Final failure occurred during heavy thunder storm when leakage appeared at the crest. Failure occurred over a 65m length.	Tsvetkova (1960), Saverenskii (1950)
14	Bol'shoy Never River	Skovorodino, USSR	Earth silt and gravel with clay core	9.6	530	1932	The clay-ice core became semi-liquid and the stability of the dam was threatened. In 1934 ballast was applied to the slopes and wooden piling was driven, soil behind the piling was replaced by more impervious materials and a wood gallery was constructed to catch the seepage. Deep thawing of the foundation soil and bedrock in 1936 did not cause serious problems.	Tsvetkova (1960)
15	Vilyuy River (Dam V)	USSR	Random earth fill w/ timber	16.8	332	-	Constructed on ice-saturated clayey silt and disintegrated rock overlying fissured clay-limestone. In the spring of 1965 and 1966 boils appeared downstream of the dam. Seepage was caused by thawing of ice in rock joints during construction.	Biyanov(1966)

Table 2-2: Summary of literature review of successful embankments built on Permafrost

No.	Name	Permafrost/ Location	Foundation Material	Type	Function	Height (m)	Reservoir depth (m)	Impervious barrier	Reference
1	Ekati Diamond Mine (2002)	North West Territories (Continuous Permafrost)	N/A	Rock fill with central frozen key trench, geomembrane core and GCL on U/S slope.	Surface water management	15	13.3	-Frozen key trench of min. embedment 2.0m, and Thermosyphons; -Polypropylene (UPP) geomembrane (used in core)-GCL on U/S side of dam-Non woven LP geotextile used as an upstream cushion)	Gräpel et. al (2005)
2	Diavik Diamond Mine (2001)	North West Territories (Continuous Permafrost)	Varies: Frozen Silty Sand Till (ice rich upper zone), over Bedrock	Rock fill with central frozen key trench and HDPE liner	Dredged sediment control	9 - 14	10	-HDPE liner-Frozen cut-off trench to ice-poor soil or bedrock (min 1m)	Holubec et. al (2003)
3	Snap Lake Dam 1 (2000)	North West Territories (Continuous Permafrost)	Intact bedrock	Rock fill with HDPE liner and frozen cut-off trench	Residual processed kimberlite storage	7	5.5 (total storage)	-Frozen cut-off trench to intact bedrock-Textured HDPE liner	J. Cassie (2003)
4	Kettle Dykes (1971)	Manitoba (Discontinuous Permafrost)	Varies: Frozen Silts and cemented Sands, bedrock, sandy clay	-Semi pervious (sand fill) homogenous fill with U/S and D/S filters-thaw-consolidation design	Hydroelectric	8	~4	-Wide structure and low gradients allowed for controlled seepage.	N. J. Smith (1983)
5	Kelsey Dykes (1971)	Manitoba (Discontinuous Permafrost)	Bedrock	-Earth fill (clay core with gravel shell)-thaw-consolidation design	Hydroelectric	6	N/A	-Wide structure and low gradients allowed for controlled seepage	N. J. Smith (1983)

Note: the available typical cross sections of the dam design are summarized in Appendix A

3. Site Specific Background

3.1 Summary of Mary River Dams

Nine SWM Ponds and embankment dams are proposed for the Mary River Project. Table 3-1 summarizes the characteristics of the dams; Figure 3-1 shows the general layout of the dam locations. As required, only the conceptual design of the dams at Mine site SWM Pond No. 2 and No. 3 and the Milne Port SWM Pond No. 10 are considered. The maximum dam height is 15m. In general, however, the recommended dam option could be considered for SWM ponds at the other Mary River Project sites.   Geotechnical data is currently unavailable for dam design. However, based on boreholes drilled in adjacent areas (MWD 003 and 004) and site visits between 25-29 July, 2011, the foundation conditions likely consists of a 0-13.5m of glacial till deposits underlain by bedrock. The bedrock is assumed to be fractured. The ground water table was not reported in the borehole logs of MWD003 and 004.

Table 3-1 : Summary of the proposed dams for Baffinland Mary River Project (Hatch Memo, 2011)

No	Name1	Function	Max. Height (m)	Dam Crest el. (m)	Length (m)	Reservoir Capacity (m3)	ICC classification
1	Minesite SWM Pond No. 1 Discharge Dam	This dam is the downstream most structure to retain stormwater in pond #1	25	327	160	7E6	Significant
2	Minesite Stormwater Pond No. 1 Sediment Dam	This dam is acting as sediment barrier for the stormwater pond	25	347.5	150	-	Significant
3	Minesite Stormwater Pond No. 1 Back Wall Dam	to form the upstream cell of the pond	25	355	150	-	Significant
4	Minesite Stormwater Pond No. 2 Dam	Storm water management	27	547.5	800	5E6	Significant
5	Minesite Stormwater Pond No. 3 Dam	Storm water management	12	204	400	1.5E+5	Significant

No	Name1	Function	Max. Height (m)	Dam Crest el. (m)	Length (m)	Reservoir Capacity (m3)	ICC classification
6	Steensby Port, Ore Stockpiles Stormwater Manageme nt Pond Dam	Storm water management	8	13	600	1.3E+5	Significant
7	Steensby Port, SWM pond Dam	Storm water management	12	40	500	80,000	Significant

Note: only highlighted dams are considered in this stage and the maximum dam height is 27 m

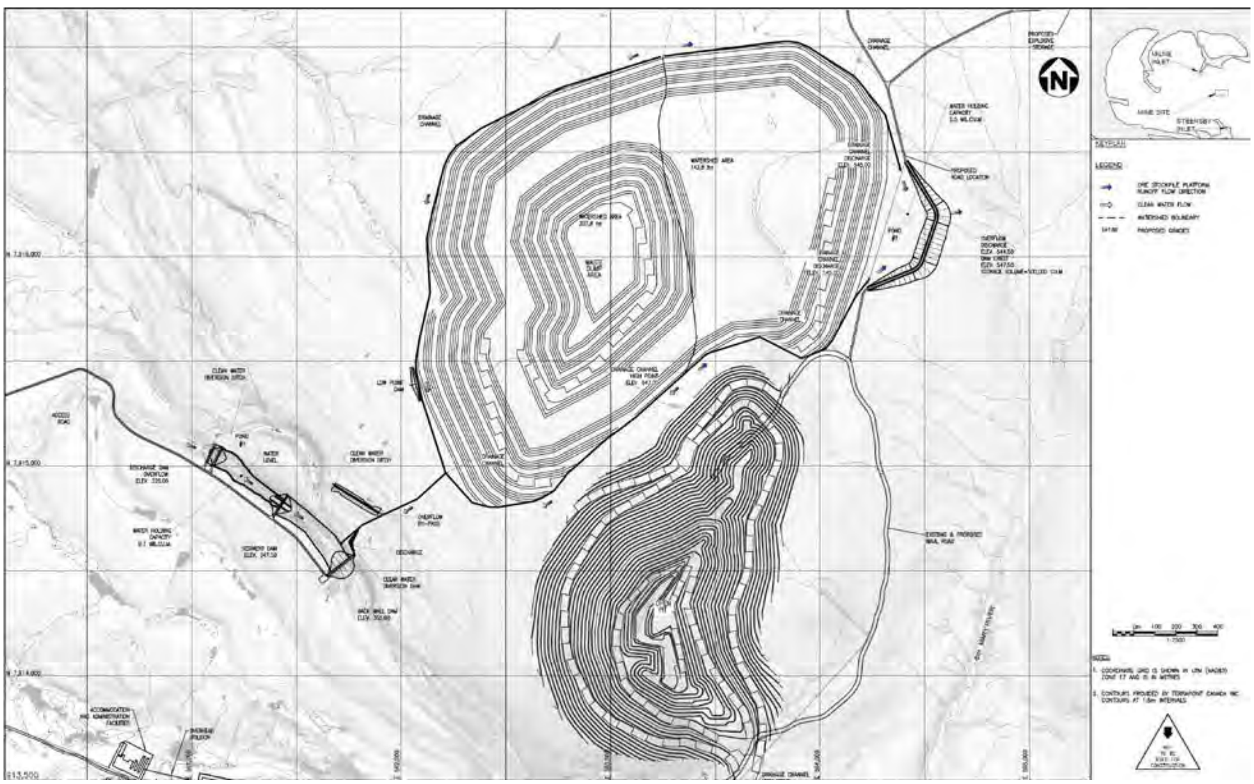

Figure 3-1: - General layout of Baffinland dams in Mine Site

3.2 Climate

The North Baffin region is located within the Northern Arctic Ecozone, as delineated in the National Ecological Framework for Canada (Agriculture and Agri-Food Canada, 2000). Typical Arctic environments exists at the Baffinland Mary River Project sites. The area experiences average annual temperature of -15 C. North Baffin Island has a semiarid climate with relatively little precipitation. The region experiences 24-h darkness with less than 2 hours of twilight from approximately November 12 to January 29. Frost-free conditions are short and are from late June to late August. There is continuous daylight from approximately May 5 to August 7. The months of July and August bring maritime influences and are usually the wettest (snow may still occur). During September to November, temperature and the number of daylight hours start to decrease, and by mid-October the mean daily temperature is well below 0°C. The highest amount of snowfall typically occurs during this period. A condition called "Arctic white out" often occurs during this time, where diffuse white clouds blend into the white snow covered landscape, reducing visibility and increasing the likeliness of disorientation. This condition can also occur in April and May.

3.3 Regional Geology

The local bedrock is of the Mary River Group, which is part of the Committee Belt. This belt comprises an assemblage of granite-greenstone terrains, rift basin sediments and volcanic rocks which lie within the northern Churchill Province and extend from south-west of Baker Lake for over 2000 km to north-western Greenland (Jackson and Berman, 2000). The Committee Belt is joined to the south by the Baffin Orogen. Figure 3-2 shows the Regional Geology Map of Baffin Island (Jackson et. al, 2000). The Committee Belt has been divided into major assemblages, which include the following:

- Archean-age banded granite migmatites and three or more phases of gneissic granitic intrusions, traversed by deformed amphibolite dikes. Ages 3.7 to 2.85 Ga, are unconformably overlain by the Mary River Group. The units are strongly metamorphosed.
- Late-Archean Mary River Group; a diverse assemblage of metasedimentary and metavolcanic rocks, preserved in narrow, folded greenstone belts. Ages 2.76 to 2.72 Ga. Belts generally show a lower sequence of varied metavolcanics, overlain by metasedmentary-metavolcanic sequences including iron formation, succeeded by an upper group of metavolcanic and metapelitic clastic sedimentary units with high-level metamorphism.
- Paleoprotorozoic Piling Group; metasedimentary/metavolcanic sequence including quartzites, marble, sulphidic iron formation, black schists, mafic metavolcanics. Ages 1.9 to 1.8 Ga with medium-level metamorphism.
- Mesoproterozoic Bylot Supergroup, in the Borden Rift Basin; siliciclastic and carbonate sedimentary rocks, some mafic volcanic units. Age 1.27 Ga with low-level metamorphic facies.
- Early Paleozoic Cambro-Ordovician (Turner Cliffs-Ship Point Formation); unmetamorphosed clastic and carbonate sedimentary rocks, locally preserved in northwesterly-trending grabens. Age 400 to 500 million years.

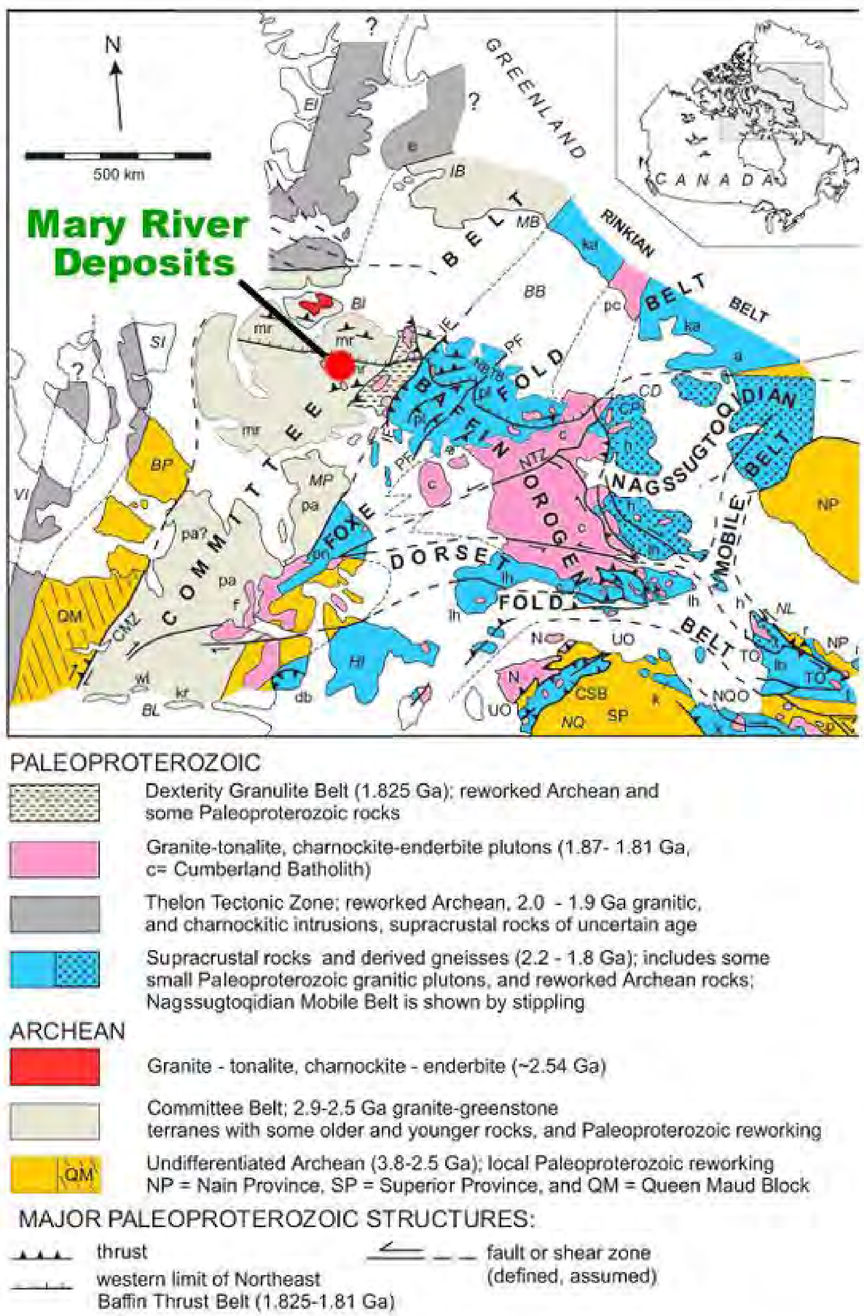

Figure 3-2: Regional Geology Map of Baffin Island (Jackson et. al, 2000)

3.4 Geothermal Conditions

The thermal state of permafrost in the Baffinland Mary River area is based on the literature review related to the thermal state of permafrost in North America. The Mary River Project is located on the northern half of Baffin Island at Latitude 71° and Longitude 79°. Permafrost monitoring is currently conducted at 350 sites throughout the permafrost regions of North America (Smith et. al 2010). Figure 3-3 shows the permafrost distribution map of North America based on Brown et al. (1997). Figure 3-4 summarized Mean annual ground temperature (MAGT) during the International Polar Year period where data were available ((Smith et. al 2010). The source summary data are given in IPA (2010).

It can be seen that the thermal states of permafrost of Baffinland Mary River area are:

- Continuous permafrost area;
- The MAGT is -5 º to -10ºC

Long-term monitoring sites operating in the eastern Arctic are located on Ellesmere Island (Smith et. al 2010). Several new boreholes were drilled and instrumented in the Baffin Region of Nunavut during 2008 to provide baseline permafrost data for community climate change adaptation plans (Figure 3-3). Figure 3-5 shows MAGT profiles based on the monitoring. It can be seen that the MAGT ranges between -5 and -10ºC at the five communities in the Baffin region. It is considered acceptable to assume the MAGT profiles at the Mary River Project are similar to those for the Baffin region (i.e. -5 ~ -10 ºC). The active layer thickness (i.e. the zone of freeze-thaw) is up to 2 m thick. The total permafrost depth is about 500 m (Wahl and Gharaptian 2009).

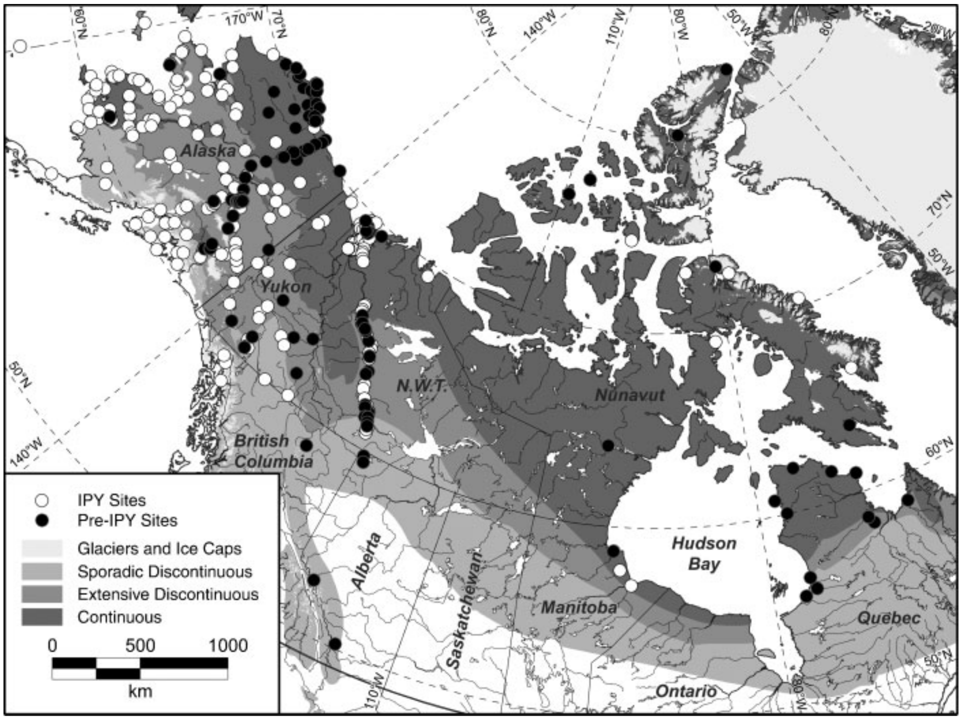

Figure 3-3: The permafrost distribution map of North America based on that of Brown et al. (1997)

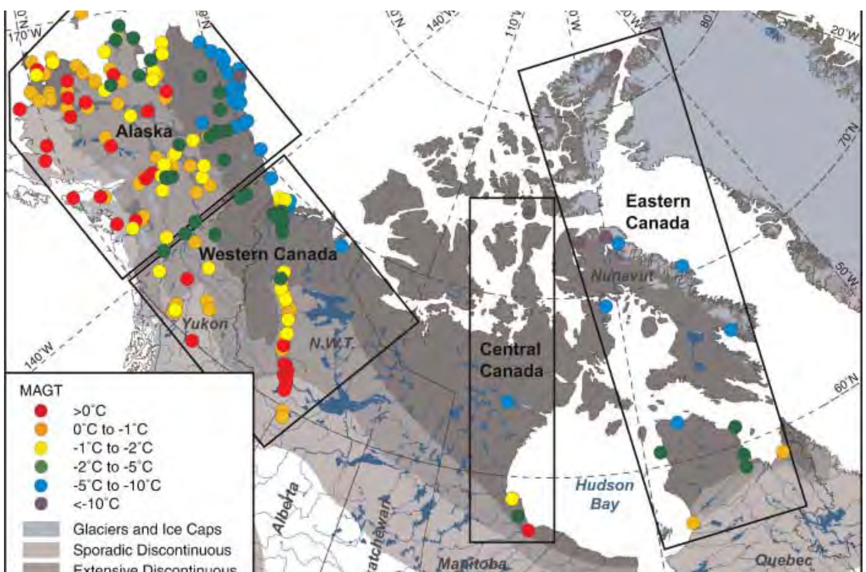

Figure 3-4: Mean annual ground temperature (MAGT) during the International Polar Year period where data were available

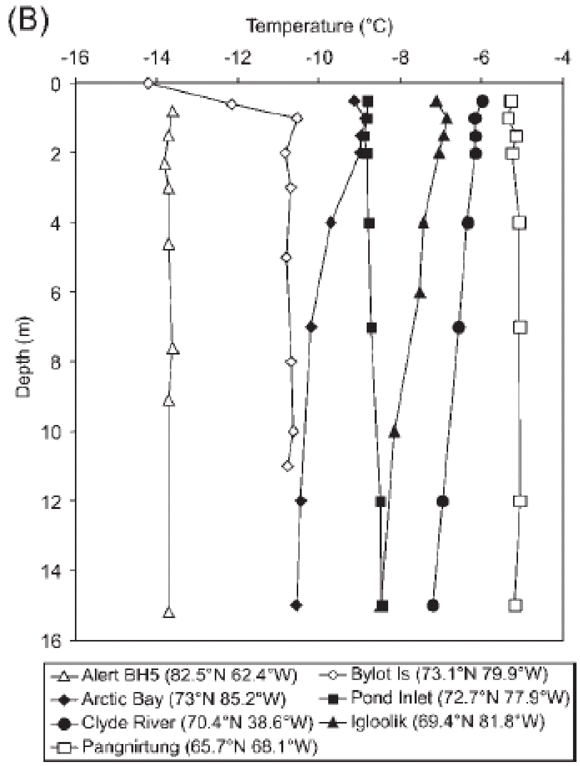

Figure 3-5: Mean annual ground temperature profiles during the International Polar Year for selected sites in the eastern and high Canadian Arctic (Smith et. al. 2010)

Literature data on the thermal regime of reservoirs in Arctic climates could not be found. However, Figure 3-6 shows the temperature profile with depth for a fresh water lake in the high Canadian Arctic. The temperature readings were taken during the spring and summer months, from April to August between 1985 to 2008. It can be noted that the water temperature increases with depth and the lake bottom temperature does not exceed 4 degrees. For a smaller water body such as a reservoir, the temperature variation with depth can be expected to be far less. For the purpose of thermal analysis and design, assuming a mean reservoir-bottom temperature of 4 °C should be adequate.

Table 3-2 presents the monthly average air temperature for Pond Inlet, NU from Environment Canada from 1976 to 2005. It can be seen that the highest average air temperature is 6.2 °C in July and the lowest is -32 °C in February. The average annual temperature is about -15 °C.

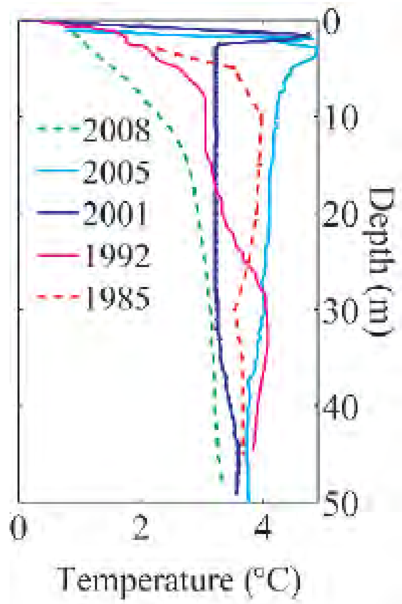

Figure 3-6: Temperature profile for Lake C3 in the high Canadian Arctic (Derek R. Mueller et. al. 2009)

Table 3-2: Monthly Average Air Temperatures for Pond Inlet, NU

Month	Average Temp (C)
January	-32.2
February	-34.0
March	-30.0
April	-21.4
May	-9.1
June	2.0
July	6.2
August	4.4
September	-1.3
October	-10.5
November	-21.9
December	-28.0

3.5 2011 Site Visit

A site visit to the Mary River area was conducted from July 18 to 22, 2011. The objectives were to:

- inspect the proposed dam site to assess the topography, geology, construction materials and thermal conditions
- assess requirements of geotechnical site investigations at the dam sites.

Some shallow test pits were excavated using a shovel to investigate the near surface soils and depth to frozen ground. In general, the surface at the dam site comprises glacial till and frozen ground was encountered 0.5 m below the ground surface. Figure 3-7 and Figure 3-8 show the pictures taken at the dam site for the proposed Mine site SWM Pond No. 2.

Figure 3-7: Photographs taken during the 2011 site visit - Dam site of for the proposed Mine site pond No. 2

Figure 3-8: Photographs taken during the 2011 site visit - Test pit in the dam site of for the proposed Mine site pond No. 2

4. Dam Design Criteria

4.1 CDA Guidelines for Dam Slope Stability

The dam design criteria is based on 2007 CDA Dam Safety Guidelines. Table 4-1 summarizes the safety factors recommended for the Mary River Project dam design. Four load cases are proposed. Table 5 summarizes the required Factor of Safety (FS) for the dam design based on CAD guideline corresponding to:

- Steady state seepage corresponding to the normal water level (NWL)
- Steady-state seepage at NWL in conjunction with earthquake loading. Note: The peak ground acceleration (PGA) for the site is 0.122 *g* based on data from the Canadian Geologic Society (CGS) corresponding to a 1:1000-yr return period. The detailed PGA for the site is shown in the Appendix B
- Upstream slope stability subject to rapid drawdown
- Slope stability of the dam slopes at the end-of-construction before impounding water.

Table 4-1: Summary of the required Factor of Safety for Baffinland dam design based on CAD guideline

Load Combinations	Required Minimum FS	Type of Analysis
Steady Seepage corresponding to the NWL	1.5	Static analysis
Steady Seepage at NWL plus Earthquake Loads	1.0	Pseudo-static analysis
Upstream slope stability under rapid drawdown	1.2	Static analysis
Dam slope stability Just end of construction	1.3	Static analysis

4.2 Thermal Conditions for Design

The design basis thermal conditions are:

- The MAGT profiles Baffinland Mary river is assumed to -10 ºC (see Figure 3-5).
- The reservoir-bottom mean water temperature is assumed to be 4 ºC (see Figure 3-6).
- The design basis for the local air temperatures as follows:
  - The air temperature for the warmest condition is 7ºC.
  - The air temperature for the coldest condition is -25 ºC.
- The natural active layer thickness is assumed to be 2 m (Wahl and Gharapetian, 2009).
- It is assumed that the foundation of the reservoir will thaw to the depth of 8 m in 50 years in the conceptual design stage.

4.3 Additional Specific Requirement

In addition to maintaining storm water retention requirements, the SWM ponds are required to have sufficient retention time to facilitate sedimentation of sediment within the reservoir. A small amount seepage is required to help maintain the water level in control. The required seepage is assumed to be in the order of 10 L/s for the entire dam. This can be maintained by designing the dam to allow for controlled seepage to meet the flow requirements.

The anticipated type of service of the embankment is to retain water continuously.

5. Conceptual Dam Options

Mine site SWM Pond No. 2 dam has a height of 15m and it is the highest among three dams. Consequently, conceptual designs have been prepared for Mine site SWM Pond No. 2 dam. The two design options evaluated are:

- Option 1 – Rock fill dam with central plastic cut-off wall;
- Option 2 – Rock fill dam with High Density polyethylene (HDPE) Liner and central cut-off trench.

The design options are discussed in the following sections.

5.1 Option 1- Rockfill Dam with Central Plastic Concrete Cut-off Wall

Figure 5-1 shows the typical cross section for Option 1. The option consists of a rock fill dam with an inner core of compacted ¾ inch minus rock fill, a central plastic cut-off wall and a compacted transition zone. The following zones are envisioned:

- Zone 1 – Compacted ¾ inch minus rock fill
- Zone 2 - Plastic cut-off wall
- Zone 3 - Compacted Transition zone
- Zone 5 - Compacted Class B rockfill
- Zone 6a - Riprap-class C
- Zone 7- Bentonite enriched soil.

The estimated dam geometry consists of a 20 m wide dam crest (transportation requirement), 2.5H:1V U/S slope gradient, 1.7H:1V D/S slope gradient, 0.9 m wide central plastic concrete cut-off wall extending 5.5 m below the ground surface, and thermosyphons installed in the lower key trench to maintain the frozen permafrost foundation.

5.2 Option 2 - Rockfill Dam with High Density Polyethylene (HDPE) Liner and Frozen Key Trench

Figure 5-2 shows the typical cross section for Option 2. The option consists of a rock fill dam with an HDPE liner as the primary seepage barrier. The main materials in this dam option consist of:

- Zone 1 – Bedding Material (Sand 0-13mm or crusher fines)
- Zone 2 - Transient Zone
- Zone 3 – Compacted Rockfill
- Zone 5 - Riprap-Class C.

Figure 5-3 shows a modified typical dam cross section corresponding to Option 2. This dam section has been considered to permit a small amount of seepage through the upper part of the dam to control the reservoir during normal operating condition. An additional liner is proposed to allow controlled seepage of water through the embankment without permitting it to enter the frozen key trench.

The estimated dam geometry consists of a 20 m wide crest (transportation requirement), 2.5H:1V U/S slope gradient, 1.7H:1V D/S slope gradient, a frozen key trench extending 5m below ground surface and thermal siphons to maintain the thermal regime of the key trench.

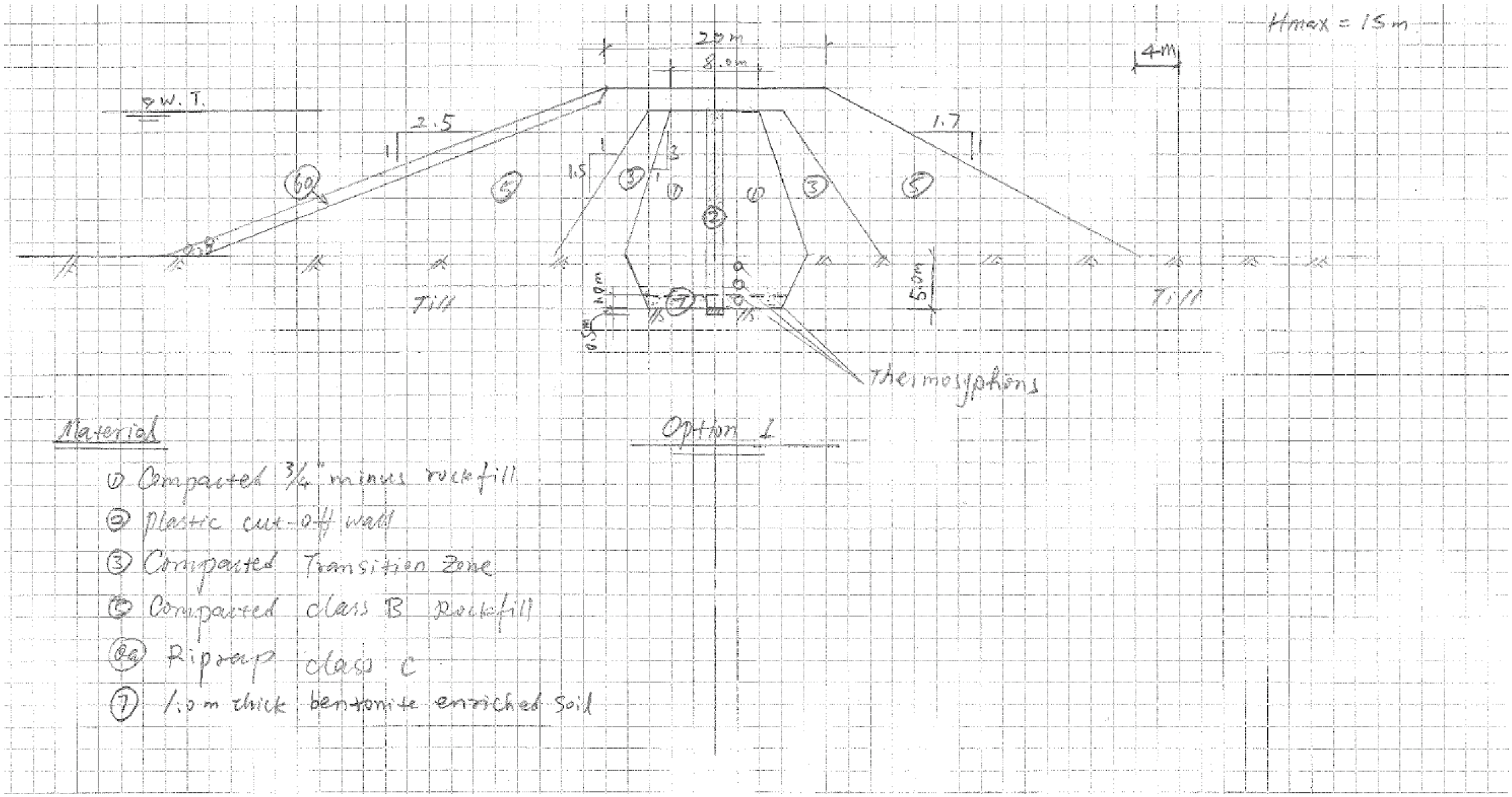

Figure 5-1: Dam Option

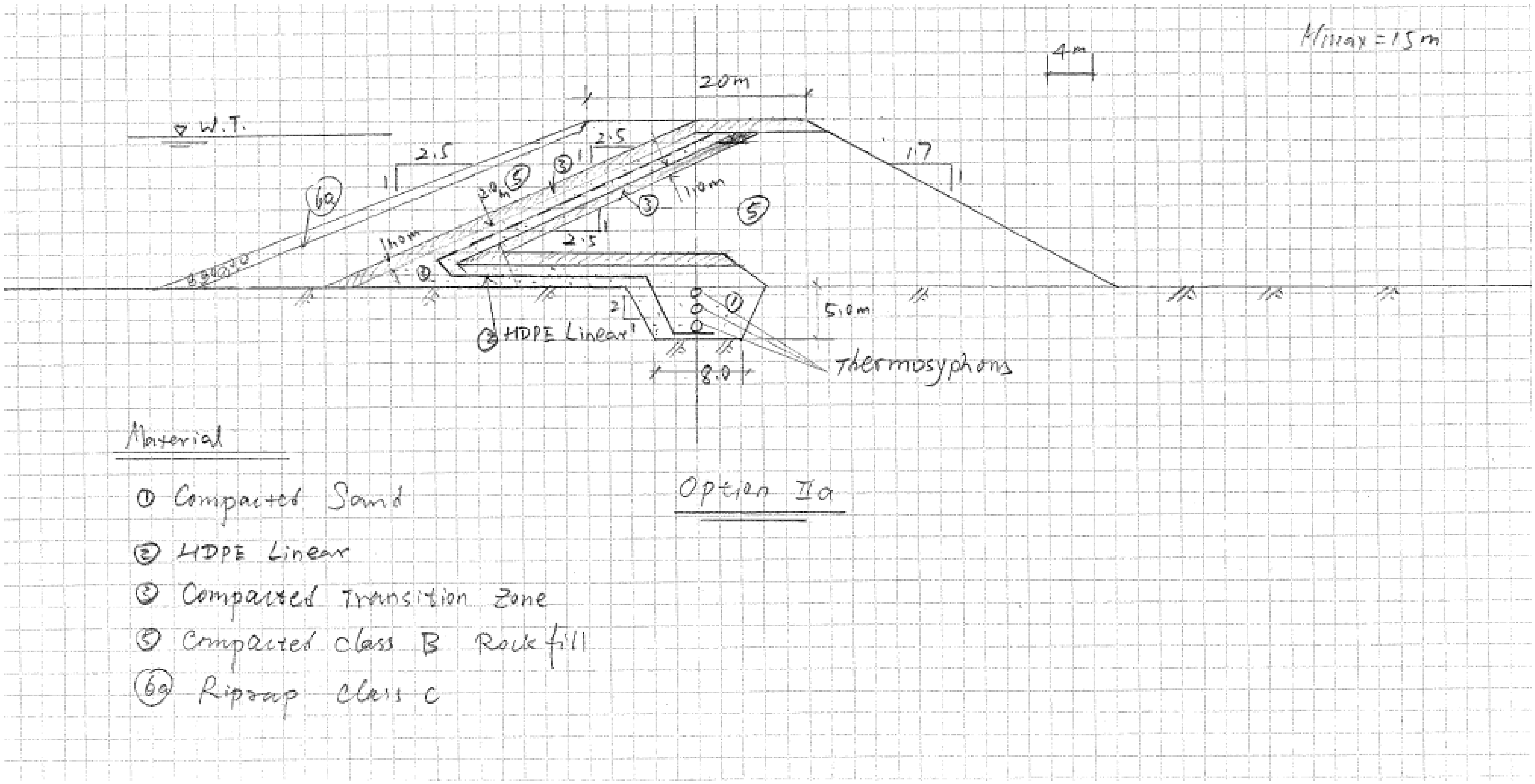

Figure 5-2: Dam Option 2

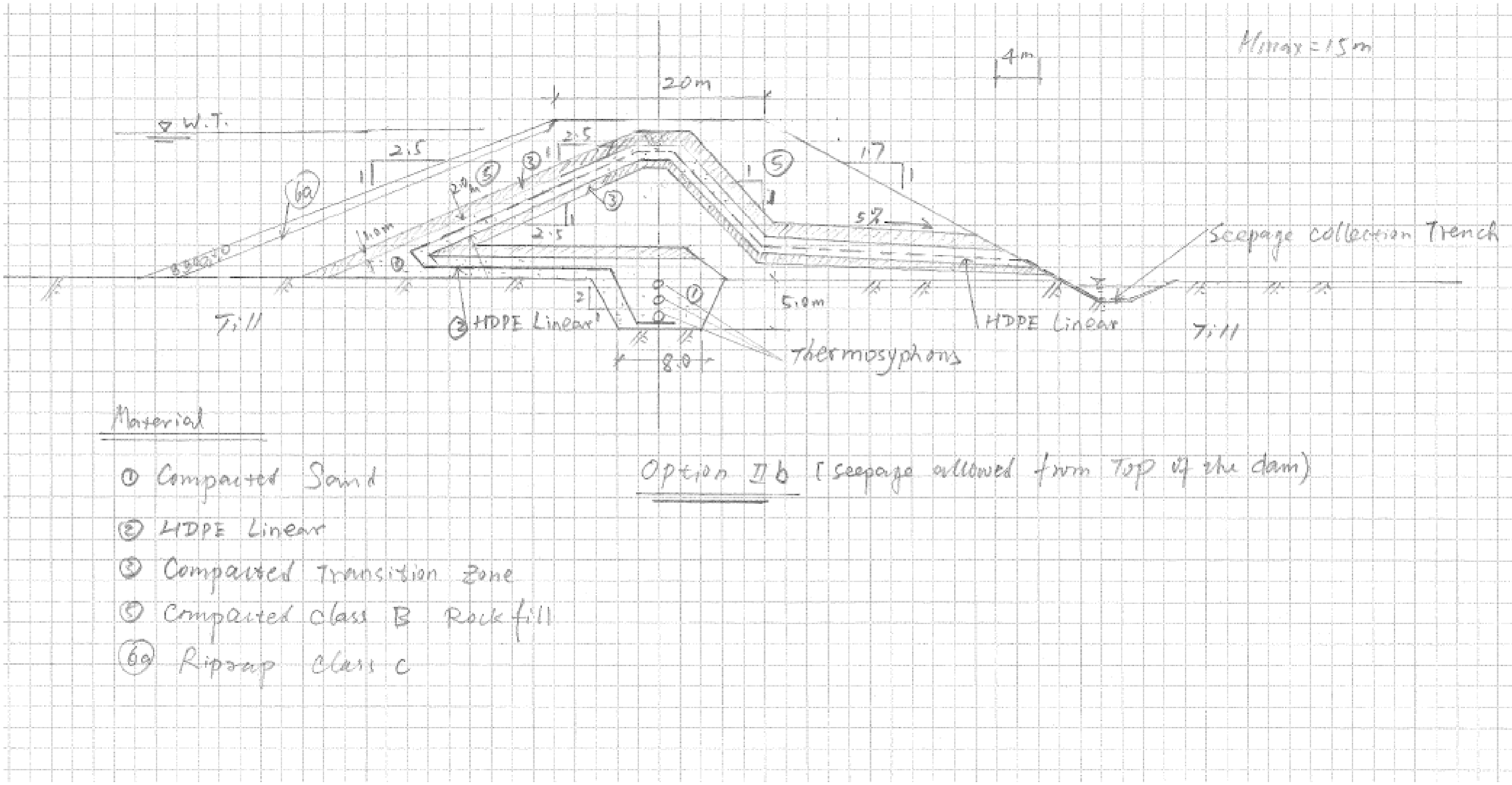

Figure 5-3: Mortified Dam Option 2

6. Discussion

6.1 Feasible Seepage Defence

Previous boreholes at the Mary River Project site indicate that the area is generally underlain by 0-13.5 m of Till followed by bedrock. The till is generally granular in nature, implying that impervious fill materials (e.g. sitly calys) are not readily available on site. In addition, the till is typically frozen and excavation of the material will be difficult and borrow development unecomonmic. Due to lack of the natural unfrozen imperious materials, the only feasible means of controlling seepage in the embankment area:

- Plastic cut-off wall (e.g. Diavik Dam A154) or
- Geosynthetic liner (e.g. Diavik dredged sediment control dam: HDPE liner, Ekati surface water management dam: polypropylene liner).

In the foundation, a frozen key trench (e.g. Diavik dredged sediment control dam, Ekati surface water management dam: Frozen key trench with thermosyphons) is most likely required. For conceptual design purposes, a 5 m depth has been assumed but the actual depth will likely be in the order of 2 m to 3 m supported by adequate thermal modeling and with thermal siphons to ensure maintenance of the frozen conditions.

6.2 Dam Slopes and Crest Width

Although steeper upstream slopes are typically feasible for rock fill embankments, the U/S of Options 1 and 2 have been flatted to accommodate possible settlement of the upstream section due to thawing of the foundation soils when the reservoir is filled. It is assumed that the foundation of the reservoir will thaw to the depth of 10 m in 50 years in the conceptual design stage.

In general, the 2.5H:1V and 1.7H:1V are designed for the upstream and downstream slope of the dam, respectively. Finite element analysis should be done to develop settlement criteria for the upstream slope and to ensure acceptable strains on impervious elements in the dam (i.e. cut-off wall or geomembrane). Geosynthetic reinforcement could be considered to control internal strains in the embankment.

The width of the dam crest could be varied based on the requirement of the transportation on the dam crest. For the dams with transportation requirement, the crest width is 20 m; For the dams without transportation requirement, the minimum width of crest is 5 m which just meet the access requirement for construction.

6.3 Requirements for Thermal Analysis

The reservoir stores unfrozen water and associated heat energy, which will invariably result in the thawing of foundation soil and rock. The jointed rock will be more susceptible to thawing due to the low porosity and water stored in the rock mass compared to soil.

Although the Mary River Project area is located in the continuous permafrost region of Canada and the natural MAGT is in the range of -5 o to -10°C (assumed -10°C for design in this stage), the heating effect of the reservoir water which will cause growth of the thawed zone and may cause complete thawing of the dam foundation. To avoid this, design must be undertaken using thermal analysis as a basis for design to ensure the thermal regime (i.e. frozen ground) is maintained.

6.4 Geotechnical Investigation

The July 2011 site visit indicated that the ground surface around the Mine Site SWM Pond No. 2 is covered by glacial till material. Although the depth of the till is unclear, the frozen ground was found just 0.5 m below the ground surface when excavations were made by hand using a shovel. Due to the high drilling cost in this area, one borehole drilling to the bedrock for each dam site is considered to be adequate. Reasonable assumptions can be made regarding the soil foundations in the central portions of the dam sites and the probable bedrock foundations at the higher ground comprising the abutments. Suitable details can be designed for each condition.

6.5 Construction Material

There are abundant natural till / sand and granular material deposit in the Mary River Project area. However, many of these deposits are frozen and development of borrow areas to produce significant quantise of granular and till material for dam construction is unlikely.

Rock fill will be abundant and for dam construction it can be crushed and processed to provide the required materials.

Due to lack of the natural thawed imperious materials, a Plastic cut-off wall or Geosynthetic liner with frozen key trenches (thermosyphons) appear to the most appropriate for dam construction. Comparing Plastic cut-off wall and Geosynthetic liner, construction of a plastic concrete cut-off wall will require special equipment and technique for cold region construction, which is likely to be expensive compared to using geosynthetics. In addition, there is more precedent for the use of Geosynthetic liners; Consequently Option 2 is preferred.

6.6 Seepage Requirements

The project hydrologist has requested that a design be developed, which will permit some seepage for water levels above the internal core of the dam. This has been done and is presented in Figure 11. There is no precedent for permitting seepage over the top of cores for dams or dyke in cold regions. The possible effect could be loss of thermal regime in the frozen key trench and loss of seepage control. To mitigate this effect, the geomembrane core has been extended downstream to direct seepage water away from the key trench.

Although this should be adequate, it would be preferable if a small 2m high broad crested rock fill weir could be constructed in the spillway entrance to provide similar function. The weir could be designed to permit the 10l/s seepage and with zoning to filter the leakage thereby retaining any sediment.

7. Recommendations

Considering design, construction and cost for the Baffinland SWM Pond dams, the following dam section is recommended:

- Rock fill dam with HDPE Liner and central cut-off trench (Option 2) is recommended for design;
- The dam could be designed as shown in Figure 5-3 to permit seepage over top of the core as required by hydrologists. However, considering risk to satisfactory performance and cost, it is recommended that the spillway be built with a rock fill broad crested weir at the entrance, which will serve similar function.

For final design, the preliminary design basis described herein should be improved and finalized for the next phase, and geotechnical site investigations done at each dam site to characterize the foundation conditions. In addition, the dams should be designed using thermal analysis to ensure integrity of the thermal regime. Access roads should be designed for each dam.

8. References

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Appendix A

Typical Cross Section Used for Previous Embankment on Permafrost Foundation

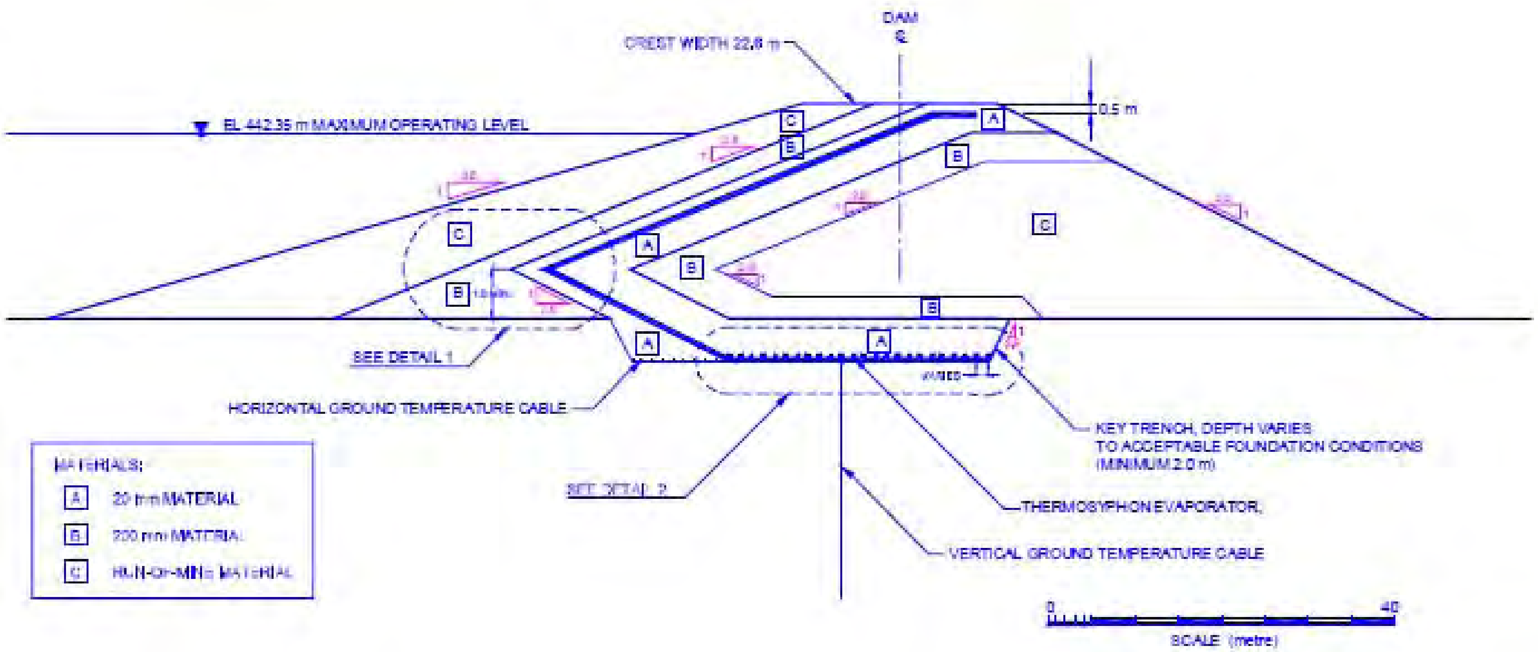

Figure A1- 1: Typical Section for EKATI Diamond Mine Dam (Gräpel et. al 2005)

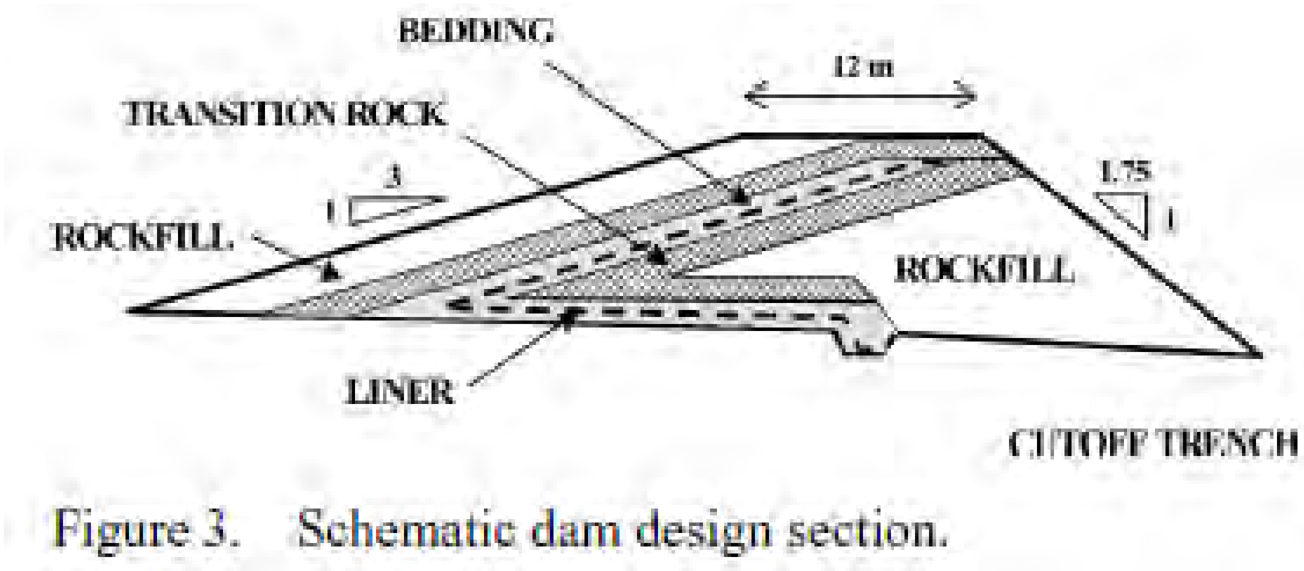

Figure A1- 2: Diavik Diamond Mine (West Dam) (Holubec et. al 2003)

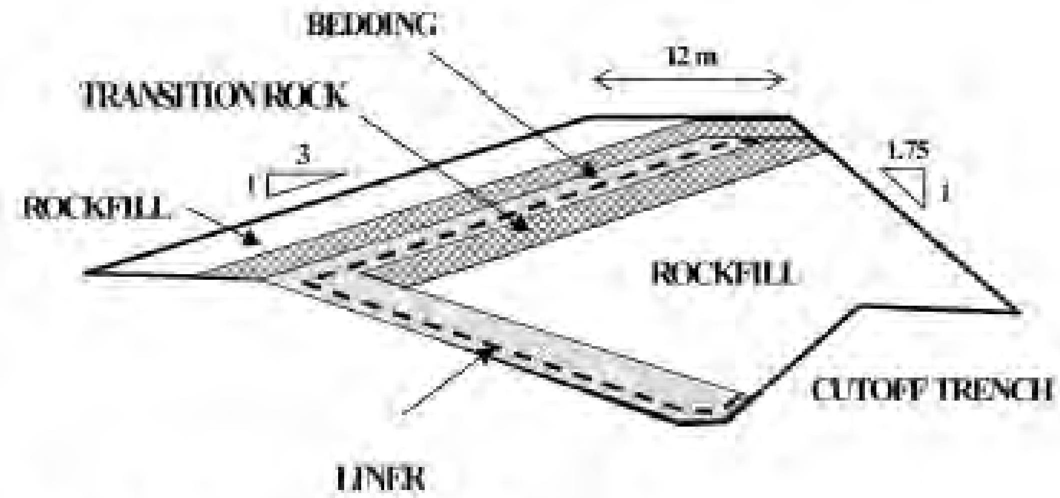

Figure 4. Dam section at the talik.

Figure A1- 3: Diavik Diamond Mine (West Dam) (Holubec et. al 2003)

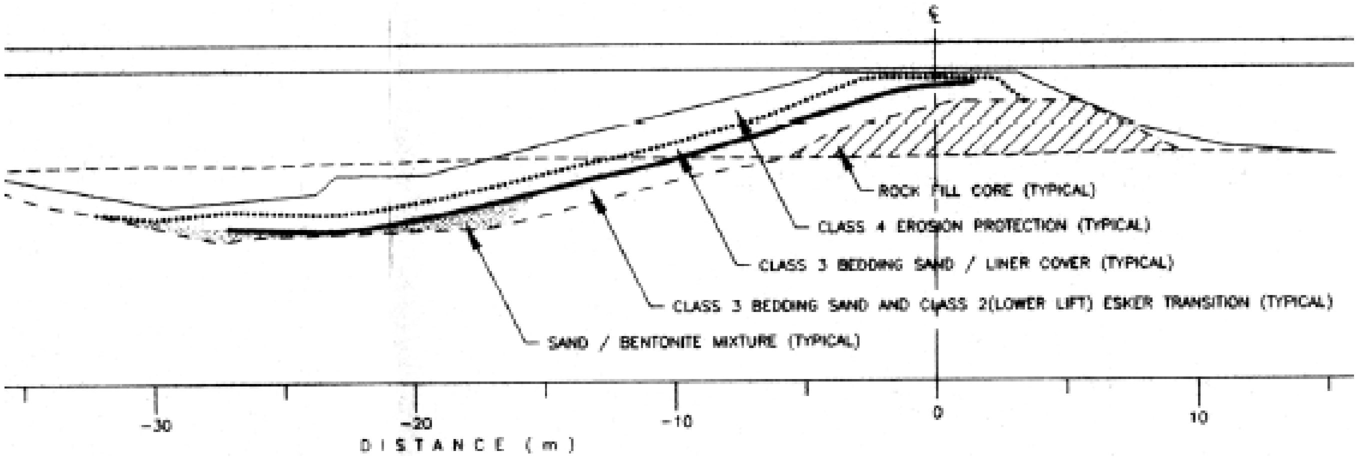

Figure A1- 4: Snap Lake Dam 1 (J. Cassie 2003)

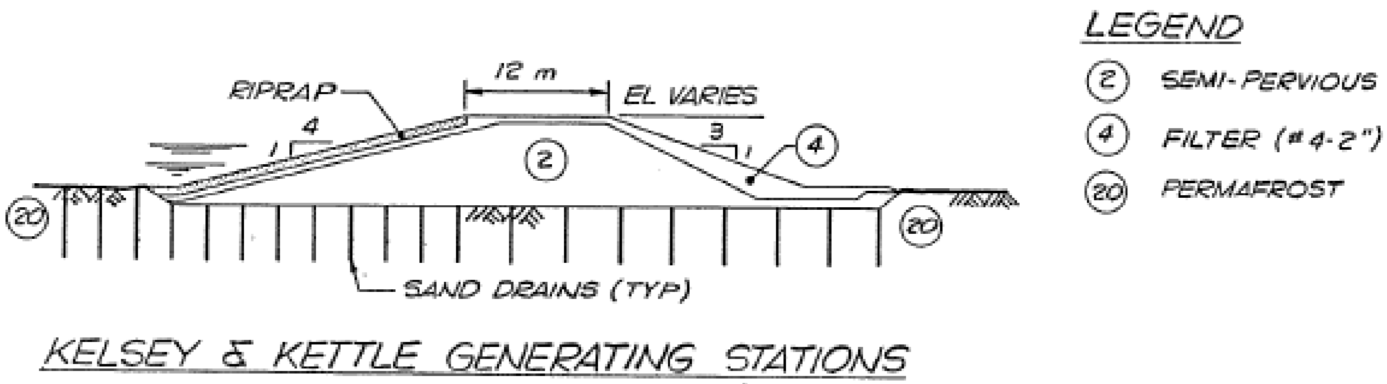

Figure A1- 5: Kelsey Dyke (N. J. Smith 1983)

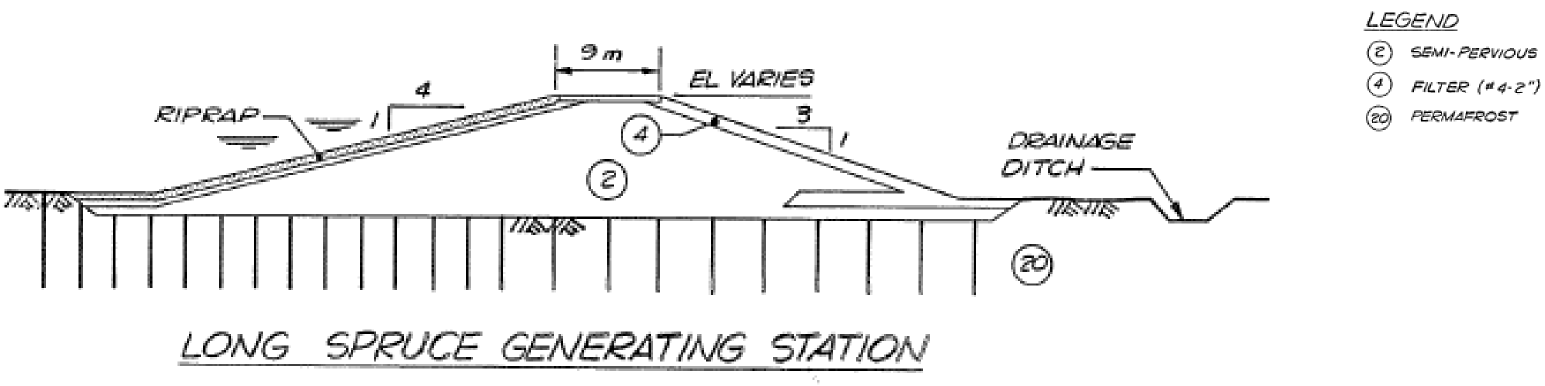

Figure A1- 6: Long Spruce Dykes (N. J. Smith, 1983)

Appendix B

Seismic Data

HATCH™ Calculations	By S. Shaif	Date July 18/11	Project No. H337697
	Checked	Date	Calculation No.
Subject Mary River, Baffin Island, NU			Page 1 of 1
Estimate of PGA for 1/10,000 year return period - Firm Ground and Hard Rock			

Estimate of PGA for 1/10,000 year return periods
Based on Firm Ground

Return Period (years)	100	475	1000	2475	10000
2005 NBCC, Firm Ground	0.025	0.075	0.131	0.239	0.570*
2005 NBCC, Hard Rock	0.018	0.054	0.094	0.172	0.410*

* Estimated Value

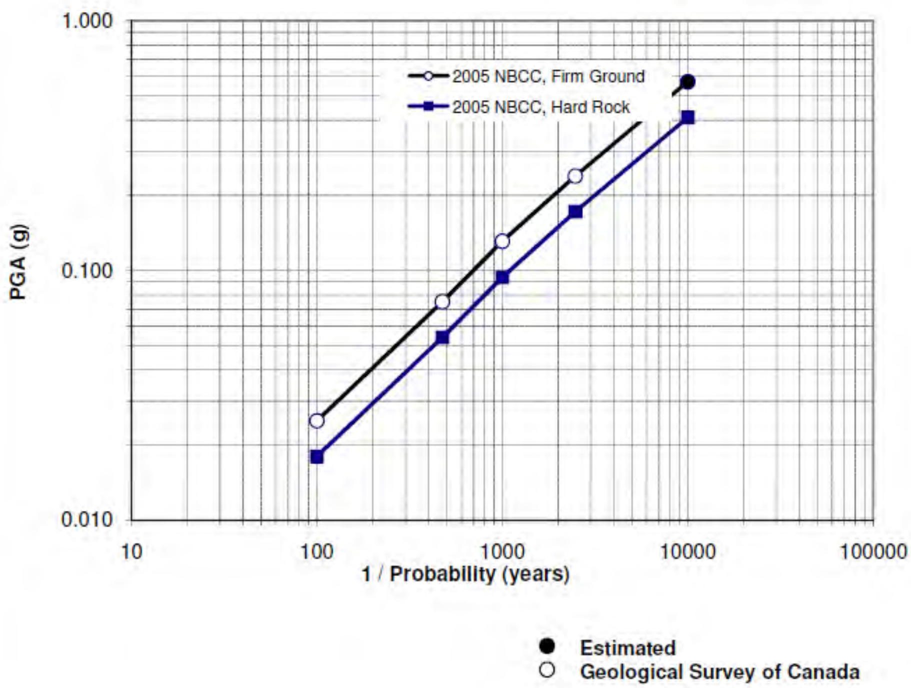

Figure B1- 1

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: Shathli Shaif, Hatch — July 18, 2011

Site Coordinates: 71.9215 North 79.3612 West

User File Reference: Mary River, Baffin LAnd

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.425	0.205	0.117	0.040	0.239

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.067	0.165	0.249
Sa(0.5)	0.043	0.102	0.142
Sa(1.0)	0.027	0.063	0.085
Sa(2.0)	0.008	0.020	0.028
PGA	0.025	0.075	0.131

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. XXXXX (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français

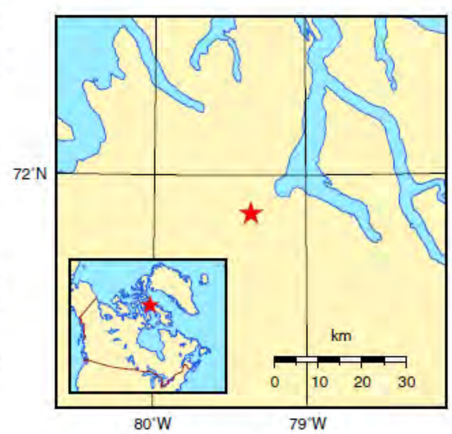

Figure B1- 2

Appendix 2: Development of Permafrost in Waste Rock Dumps – Preliminary Geotechnical Evaluation

THURBER ENGINEERING LTD.

TECHNICAL MEMORANDUM

To:	John Binns	**Date:**	23 Nov 2011
cc	Harry Charalambu ,Ramli Halim	**Rev:**	1
From:	Bruce Smith, Steve Sather, Sabia Remtulla	**File:**	19-1605-126

MARY RIVER PROJECT
DEVELOPMENT OF PERMAFROST IN WASTE ROCK DUMPS
PRELIMINARY GEOTECHNICAL EVALUATION

1. PURPOSE

This memorandum presents the results of a preliminary literature review to assess the factors that are expected to control the development of permafrost within the proposed waste rock dump at the Mary River Mine, specifically with respect to the development of permafrost within the waste rock dump as it reaches its final configuration with a volume of about 600 Mt.

A plan map showing the layout of the Mary River Iron Mine, including the open pit mine and waste rock dump, is attached as Figure A.1 in Appendix A. As shown on the map, the waste rock will be placed on the north side of the open pit mine.

This technical memorandum presents a summary of the information that has been gathered to date with respect to the development of permafrost in waste rock dumps in Northern Canada and describes a number of options that can be considered to ensure permafrost will develop in the dump at the Mary River Mine.

2. HEAT TRANSFER PROCESSES

For the purpose of this discussion, permafrost may be defined as any soil or rock which remains below freezing ($0^{o}C$) during the thaw season each year. There are three heat transfer processes, as discussed in more detail in the following sections, which will have a major influence on the development of permafrost within the proposed waste dump at the Mary River Mine:

1. Convective air flows;
2. Heat conduction; and
3. Exothermic chemical reactions.

Solar radiation is an important source of heat, of course, but it will only influence the surface temperature of the waste rock dump and will not be discussed further.

Mass transport of heat, due to the flow of water in drainage courses though embankments, is also an important heat transfer mechanism in permafrost regions, however it is not considered further in this memorandum since it is understood that measures will be taken to prevent the flow of large volumes of water through the waste rock dump.

3. CONVECTIVE AIR FLOWS

Theoretical methods of modelling convective air flows in waste rock dumps have been developed and, together with temperature measurements in a test waste rock pile at the Diavik Mine, have demonstrated that air convection can have a major influence on ground temperatures and the development of permafrost within waste rock dumps (Arenson et al, 2007; and Pham et al, 2008).

These investigations have shown that air convection dominates the thermal regime in dry, porous rock dumps that have a high permeability to air. In contrast, the influence of convective air flow is negligible in well graded waste rock which has a low air permeability, particularly if the void spaces are partially or completely filled with water or ice.

If the waste rock dump is porous, such that it has a high permeability to air flows, then during the winter months, cold dense air flows into the waste dump, displacing warm air so that the interior temperatures in the dump can fall to minus $20^{o}C$ or lower. If the rock dump is very permeable to air, then the temperatures within the dump can fluctuate in response to daily changes in the ambient air temperature.

During the thaw season, when ambient air temperatures are warmer, the cold air within the waste dump may remain within the rock dump because the cold air is denser than the ambient air. The degree to which the cold air is contained within the dump depends on the physical configuration of the waste dump, the variability and distribution of porous zones within the dump and wind speeds and directions.

For example, if the waste dump is raised above the surrounding ground surface and the air permeability is high, the denser, cold air will flow out of the dump and be replaced by warmer ambient air during the thaw season.

Similarly, if a strong wind blows for several days from one direction, then the air pressure on the windward side of the dump will be higher than on the lee side and the cold air can be blown out of the dump and be replaced with warm air. This situation is exacerbated, of course, if the waste dump is higher than the surrounding area and particularly if the height of the dump is high relative to its width.

Finally, the orientation and continuity of porous layers within the waste dump can have a major influence on interior temperatures throughout the year.

Figure A.2 in Appendix C illustrates the ground thermal regime that could develop in a porous waste dump at the Mary River Mine, late in the thaw season. The depth of thaw due to convective air flow during the thaw season is difficult to predict since it will depend on the distribution and continuity of porous layers within the dump. However, under unfavourable conditions, the depth of annual thaw could range up to tens of meters or more. Therefore, during the early stages of mine operations, when the waste dump is relatively small, the entire mass of waste rock could thaw during each thaw season.

4. HEAT CONDUCTION

In most natural soil deposits in Northern Canada, where the void spaces are filled with water or ice, the soil is effectively impervious to air flows, convection has a negligible effect on the temperature regime and heat conduction dominates heat transfer processes.

Heat conduction in permafrost is a process that has been studied extensively over several decades and is well understood. Numerical models have been developed and calibrated against field measurements, such that it is possible to make reasonably accurate predictions of ground temperatures and the development of permafrost, provided an adequate set of input data is available.

Ground temperatures have been measured as a function of depth in the vicinity of the Mary River Mine for several years. A typical distribution of ground temperature with depth, which illustrates the ground temperatures during late March (when ground temperatures are lowest) and in late August (when ground temperatures are a maximum), is presented on the upper portion of Figure A.3. As shown, ground temperatures near the ground surface vary significantly throughout the year, being very cold in the winter and rising well above freezing during the thaw season.

As illustrated, annual ground temperatures attenuate with depth and become more or less constant at about minus 10^{o}C below a depth of about 10 metres in the vicinity of the Mary River Mine. This temperature, which is referred to as the average annual ground temperature, is a function of the average annual air temperature at each particular location and generally increases at lower elevations and latitudes. For example, the average annual ground temperature in the vicinity of the Ekati Diamond Mine has been found to be about minus 6^{o}C (Arenson et al, 2007).

Ground temperatures reach a maximum in the late fall when the maximum depth of thawing occurs. In the example shown in the upper portion of Figure A.3, the maximum depth of thaw occurs at a depth of about 1.5 metres, which is the point at which the temperature curve crosses the freezing point (0^{o}C). The zone above 1.5 metres is not considered to be permafrost (since it thaws every year) and is referred to as the active layer, while the zone below the active layer is permafrost, since it never thaws.

Heat conduction in moist soils is dominated by the latent heat of fusion of ice to water (and vice versa) and therefore by the water content of the soil. The curve shown on the lower portion of Figure A.3 illustrates the significant influence that water content has on the depth of thaw in a well graded gravel.

As illustrated on the figure, at high water contents, the maximum annual depth of thaw at a location near the Mary River Mine will be about 2 metres, depending on the average water content in the near surface soil. In contrast, the depth of maximum thaw can range up to about 5 metres, due to heat conduction alone, if the near surface soil is well drained and contains no moisture. Convective air flows, which begin to dominate in dry, porous material, could increase the depth of thaw even further, depending on the air permeability in the dry gravel; however this effect has not been included in the curve shown on the lower portion of Figure A.3.

If, over a very long period of time, the soil in the upper layers of the dump were to dry out completely, (for example if there were many years with no precipitation), then the depth of annual thawing could increase, depending on the extent to which convective air flows begin to affect the heat transfer process. Under very dry climate conditions however, ARD caused by surface water infiltrating into the dump would not be of concern.

5. EXOTHERMIC CHEMICAL REACTIONS

It is understood that exothermic chemical reactions, such as the oxidation of pyrite and other minerals, may have a significant influence on the temperatures within a waste rock dump, depending on the concentration of the reactive chemicals, the ground temperatures within the dump (warmer ground temperatures can accelerate the chemical reaction and the rate at which heat is generated) and other factors which have not been investigated as part of this review (Morin, 2003).

The heat from such reactions would be transferred by conduction to the surrounding ground and could degrade the permafrost. It is recommended that the potential for exothermic reactions occurring in the waste rock dump be investigated by a qualified geochemist, since such reactions could have a significant influence on the development of permafrost within the dump.

If necessary, a number of methods for dealing with this potential source of heat can be considered, including segregation of the highly reactive material and submerging it in water; segregating the highly reactive material within the waste rock dump and cooling it with ventilation ducts; or distributing the reactive material throughout the waste dump so that the critical mass of reactive material required to generate high temperatures cannot occur.

6. DISCUSSION

General

If convective air flow and exothermic reactions in the waste rock dump can be limited, such that only conduction dominates the process of heat transfer and the water content within the upper 3 or 4 metres can be increased to about 5 percent, then, as illustrated on Figure A.4, it can be expected that the annual depth of thaw would be less than 2 metres and the internal temperature within the dump at depth would stabilize after a few years to converge to the average annual ground temperature in this area (minus 10^{o}C).

That is, it should be possible to develop permafrost within a major portion of the waste rock dump at the Mary River Mine, particularly in view of the relatively low average annual ground temperatures that have been recorded in this area.

Minimizing Convective Air Flows

If the waste rock is dry and porous, convective air flows will dominate the heat transfer process and may prevent the development of permafrost within a significant portion of the waste dump. Heat flows due to air convection appear to be difficult to predict and control and therefore it is believed that the most effective approach will be to implement methods to limit or prevent convective air flow. Limiting air flow is also an advantage because it will reduce the availability of oxygen to potential acid generating (PAG) rocks within the dump.

A number of methods for minimizing convective air flows in the waste rock dump can be considered as follows:

1) Waste rock is normally placed by end dumping the rock, which causes a porous layer of cobbles and boulders to form near the base of each bench that can be highly permeable to air. Methods of end dumping the waste rock in cells that will break up the continuity of these porous layers have been used successfully to reduce air flow into rock dumps at other mines and should be considered during detailed design of the waste rock dump at the Mary River Mine (Chamber of Mines of South Africa, 1996).

2) A second option which can be considered is to locate the waste rock dump in a closed depression, (or surround the dump with containment dikes) so that surface water flows into, and remains within, the dump. The water will fill the void spaces in the waste rock and freeze, reducing the permeability to air and minimizing the depth of thaw that occurs each thaw season.

   In non-permafrost regions, this approach would be unacceptable because surface water which flowed into the waste rock dump would infiltrate into the ground below the dump and contaminate the ground water. In this case, however, the site is underlain by permafrost to depths of several hundred metres, so that water from the dump cannot infiltrate into the ground below the dump.

3) It may be possible to develop cost effective methods of blasting the waste rock in the mine such that most of the material is well graded, so that it will limit convective air flows within the dump. This approach might be feasible at the Mary River Mine; however it will depend on the mechanical properties of the waste rock formations, the blasting pattern and other factors. The feasibility of controlling the gradation of the waste rock cannot be determined until the mine begins operations.

4) Another approach would be to reduce the air permeability of the waste rock dump by placing one or more layers of well graded material (such as the local sandy till) to a depth of 2 or 3 metres over the surface of the waste rock dump during mining operations. The effectiveness of this approach could be enhanced significantly by watering the sandy till as it is being placed, to reduce the annual depth of thaw within the dump.

   It is understood that implementation of this method is being considered on a trial basis at the Diavik Mine; however it has not been possible to confirm this. If the use of a sandy till cover is considered for the Mary River Mine, it would be important to undertake trials during the early stages of mine operations to confirm its effectiveness and refine the placement procedures.

5) Finally, consideration can be given to covering the final surface of the waste dump with an impermeable capping layer of natural material or a synthetic membrane liner, which would prevent convective air flow into the dump and in addition, if the layer were properly designed and installed, would prevent snow melt and rainfall from infiltrating into the dump.

   The use of a membrane liner may be a cost effective solution, however if the waste rock in the dump remains dry, then the annual depth of thaw due to heat conduction could still range up to about 5 meters. Field observations have demonstrated that membrane liners such as high density polyethylene (HDPE) can be expected to last for at least 50 years without deteriorating, provided they are covered to protect them from sunlight and ground settlements are limited. While there is no conclusive evidence available at this time, it is possible that such liners will deteriorate over periods as long as several hundred years.

Whatever option is chosen to promote the development of permafrost within the dump and minimize the depth of annual thaw, it will be essential to install thermistor strings and other instrumentation in strategic locations throughout the dump to measure actual ground temperatures and verify that permafrost is developing within the dump as expected.

Climate Change

While there is considerable debate about the causes of climate warming, most scientists agree that average air temperatures in the northern hemisphere have been increasing since the end of the Little Ice Age about 200 years ago. The predicted rate of temperature increase over the next few centuries is also the subject of much debate however, recent estimates (Natural Resources

Canada, 2011) indicate that mean annual air temperatures on northern Baffin Island will increase between 1995 and 2060 by 4 to 5 degrees Celsius, which corresponds to a rate of increase of between 6.2 and 7.7 degrees per century.

Average annual ground temperatures in Northern Canada are generally proportional to the average annual air temperature at each location and will therefore increase at about the same rate that the average annual air temperature increases. As mentioned, the average annual ground temperature at the Mary River Mine Site is about minus 10^{o}C and therefore if the average annual air temperature were to increase at the predicted rates, then the mean annual ground temperature will be close to 0^{o}C, 130 to 170 years from the present. It may take an additional 50 or 100 years before permafrost at the Mary River Mine site degrades completely, due to latent heat effects.

As described on the Natural Resources Canada website, future climate predictions must be treated with caution, since they are subject to change based on the acquisition of additional climate data and refinements to the predictive models.

7. IMPLICATIONS OF PERMAFROST FOR PREVENTING ARD FORMATION

It is understood that geochemical tests on rock samples from the open pit mine indicate that only a small portion of the waste rock is likely to generate acid rock drainage (ARD), however further testing is required to confirm this.

It is understood that consideration is being given to minimizing ARD from the waste rock dump by segregating the potential acid generating (PAG) waste rock within the dump and encapsulating it in non-PAG material to minimise the infiltration of air and water. In addition, the formation of permafrost within the dump would inhibit ARD. The technique of minimizing ARD by freezing waste rock has been attempted at a number of other hard rock mines in Northern Canada and Alaska and a number of organizations have and are continuing research into methods for long term containment of ARD in permafrost regions. Thurber Engineering has completed a preliminary review of relevant published literature concerning this issue; however it has not been possible to interview mine operators in Northern Canada and Alaska

An overview of the difficulties of predicting and containing ARD from the waste rock dumps at the Ekati Diamond Mine, which is located about 400 km north of Yellowknife, Northwest Territories, has been published by Morin (Morin, 2003). During initial mine planning, it had been expected that permafrost would quickly aggrade into the waste rock dumps at the Ekati Mine and therefore it was expected that seepage and ARD from the dump would be negligible.

However, within a year after the start of mine operations in 1998, ARD was observed at several monitoring stations, which led to further investigations and the development of mitigation measures; which are reported to have been successful. (Hayley, 2011). It is understood that ground temperature measurements in the waste rock dumps have established that the annual depth of thaw in the dumps ranges to a maximum of about 5 metres.

The waste rock dumps at the lead-zinc mine at Nanisivik in northern Baffin Island were decommissioned at least 5 years ago, when they were covered with about 2 metres of well graded material to prevent convective air flows and minimized the infiltration of surface water. It is understood (Cassie, 2011) that the waste rock dumps have remained frozen and annual site inspections have found no water seepage (and therefore no ARD) emanating from the dumps.

The experience at these northern mines has prompted further research into the factors affecting the development of permafrost in waste rock dumps, including the development of convective thermal models and the construction and monitoring of a small waste rock pile at the Diavik Diamond Mine, which is located about 30 km south of the Ekati Mine (Arenson et al., 2007).

8. RECOMMENDATIONS

The results of this preliminary review have identified at least one (Nanisivik) cost effective method to ensure that permafrost will develop in the waste rock dump and be effective at preventing ARD from the dump. It is recommended however, that as currently proposed, seepage water from the Mary River waste rock dump be controlled and contained in holding ponds, where it can be monitored and treated as necessary during mine operations.

Thermal analyses of the waste rock dump to predict the long-term distribution of permafrost within the dump will be required. The heat transfer processes can be simulated with available computer models, however, none of the models can confirm that permafrost will develop within the dump, without more reliable input data regarding the method of placing the waste rock and the resulting properties of the waste rock in the dump. In addition, it will be important to calibrate the thermal analyses against existing case histories including, in particular, the Nanisivik mine. Therefore it is recommended that detailed thermal analyses be postponed until detailed information from the monitoring program of the waste rock dump at Nanisivik can be obtained and more information becomes available with respect to the properties of the proposed waste rock dump at Mary River.

During the first few years of mine operations, once the properties of the waste rock are better defined, including the grain size distribution and the chemistry and distribution of the PAG rocks, methods of containing and treating the PAG rock in the waste rock dump can be further developed, tested and refined, with the objective of establishing those procedures that will ensure that ARD can be minimized in the long term.

Monitoring of ground temperatures and the development of permafrost within the waste rock dump and measuring the properties and volume of seepage water from the dump should continue during mine operations and after decommissioning of the dump. This work should include periodic updating of the thermal analyses, which should be calibrated to actual measured ground temperatures and incorporate any changes to the climate change predictions produced by Natural Resources Canada.

It is recommended that a qualified geochemist review the potential for exothermic chemical reactions occurring in the Mary River waste rock dump, so that the most effective methods for mitigating this potential source of heat can be established.

The scope of this review has been limited by time constraints and it is recommended that a more comprehensive study and review of the literature and other sources of information be undertaken since there may be additional published information concerning the control of ARD using permafrost in northern regions.

It is recommended that environmental scientists at some of the other mines in permafrost regions be contacted, including the Ekati and Diavik Mines in the Northwest Territories, the decommissioned Nanisivik Mine at Arctic Bay on Baffin Island and the Red Dog Mine in Alaska. These mine operators will have practical experience in the control of ARD from waste dumps in permafrost that should be of benefit to the design of the waste rock dump at Mary River.

It is recommended that Baffinland Iron Mines Corporation consider becoming involved in some of the studies that other agencies are undertaking into various methods for controlling ARD from waste rock dumps in Northern Canada. Full access to the various research programs will provide Baffinland with the most recent information concerning this issue and will, in turn, allow Baffinland to influence the direction of some of the research and provide researchers with practical experience from the mining operations at Mary River.

9. CLOSURE

We trust that the foregoing evaluation and recommendations meet your current requirements. Please contact us at any time if you have questions or require additional information.

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Review Principal

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Attachements:

- References
- Statement of General Conditions
- Appendix A - Figures

PERMIT TO PRACTICE
THURBER ENGINEERING LTD.
Signature
Date 23 Nov 2011
PERMIT NUMBER: P0176
The Association of Professional Engineers, Geologists and Geophysicists of the NWT / NU

THURBER

References:

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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this Report expressly addresses proposed development, design objectives and purposes, and then only to the extent there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation or to consider such representations, information and instructions.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS WE MAY EXPRESSLY APPROVE. The contents of the Report remain our copyright property. The Client may not give, lend or, sell the Report, or otherwise make the Report, or any portion thereof, available to any person without our prior written permission. Any use which a third party makes of the Report, are the sole responsibility of such third parties. Unless expressly permitted by us, no person other than the Client is entitled to rely on this Report. We accept no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without our express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and this report is delivered on the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

(see over . . . .)

c) Design Services: The Report may form part of the design and construction documents for information purposes even though it may have been issued prior to the final design being completed. We should be retained to review the final design, project plans and documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the report recommendations and the final design detailed in the contract documents should be reported to us immediately so that we can address potential conflicts.

d) Construction Services: During construction we must be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Client's benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

THURBER

APPENDIX A

Figures