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.

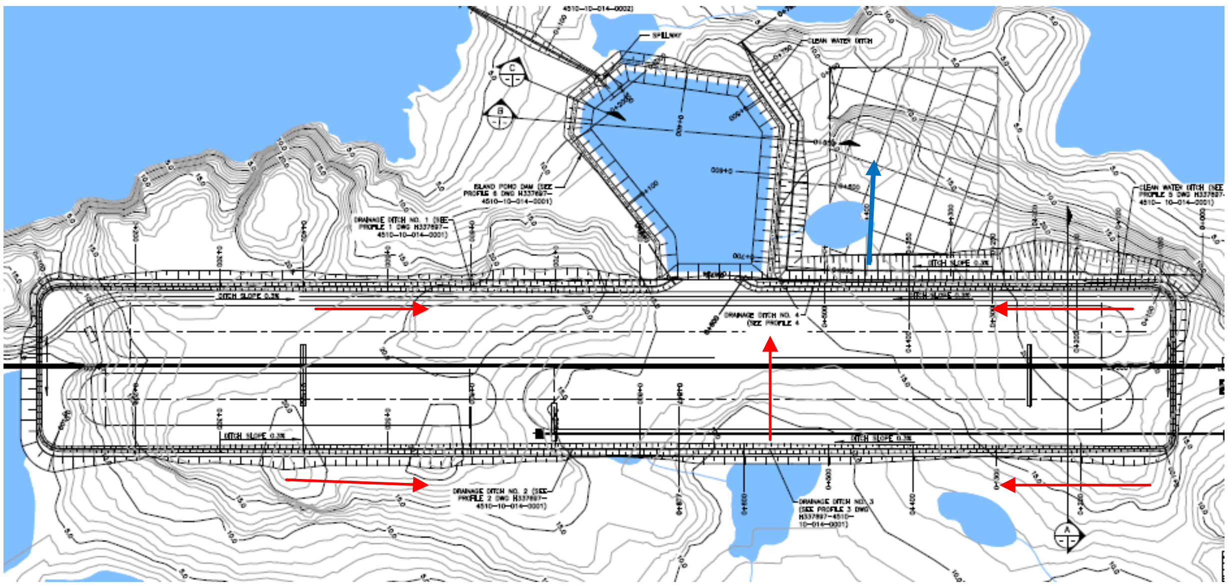

Figure 3-3: Ore Stockpile Platform Stormwater Management System

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:

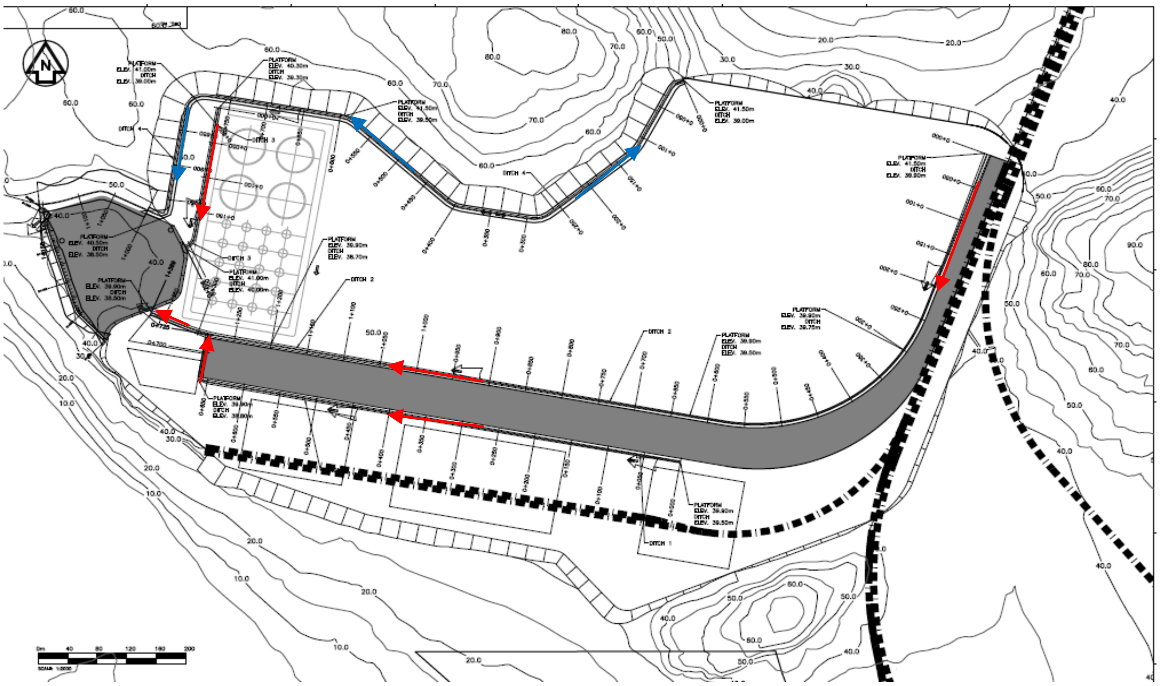

Figure 3-4: Stormwater Management and Drainage network

- 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.

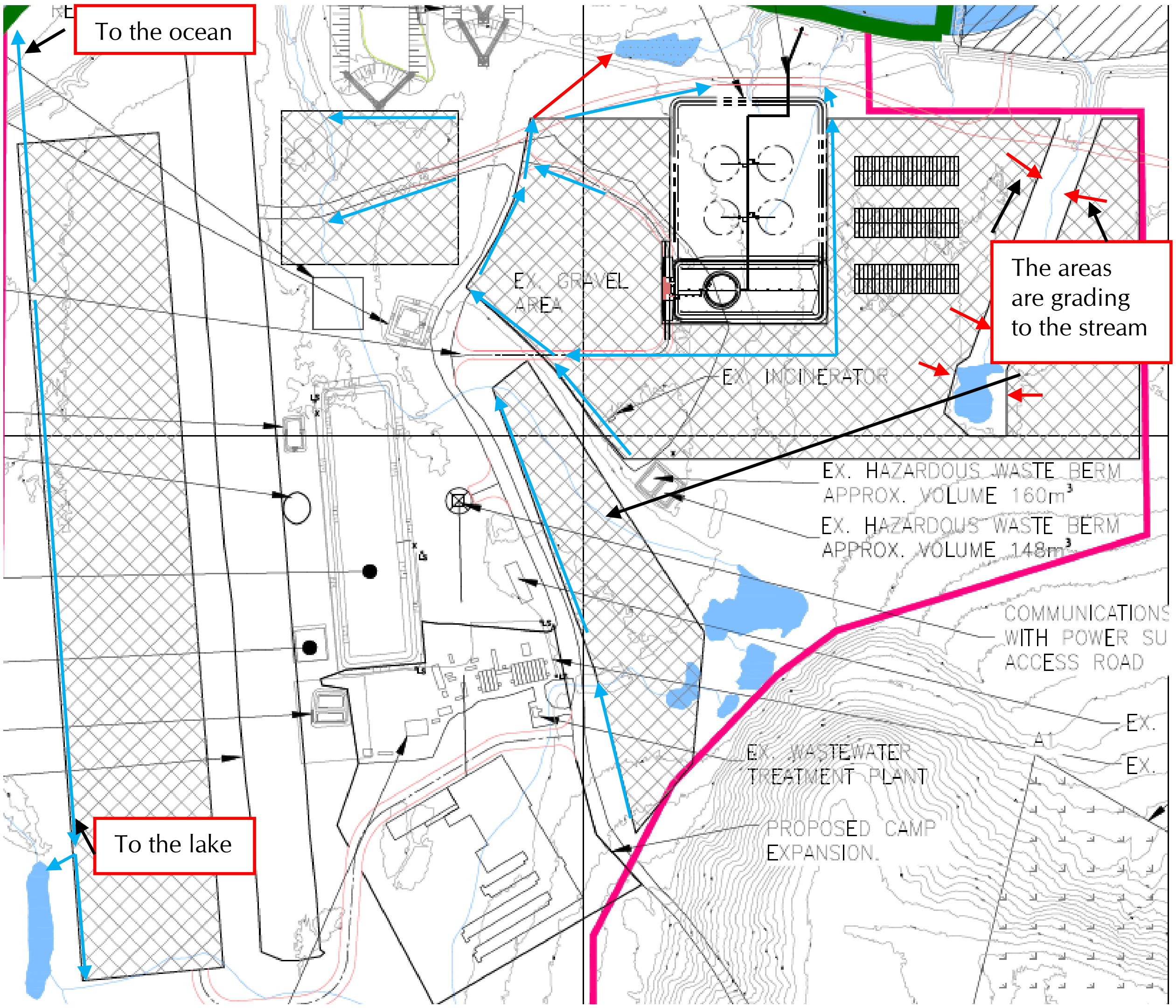

Figure 3-5: Milne Inlet Drainage Plan

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).

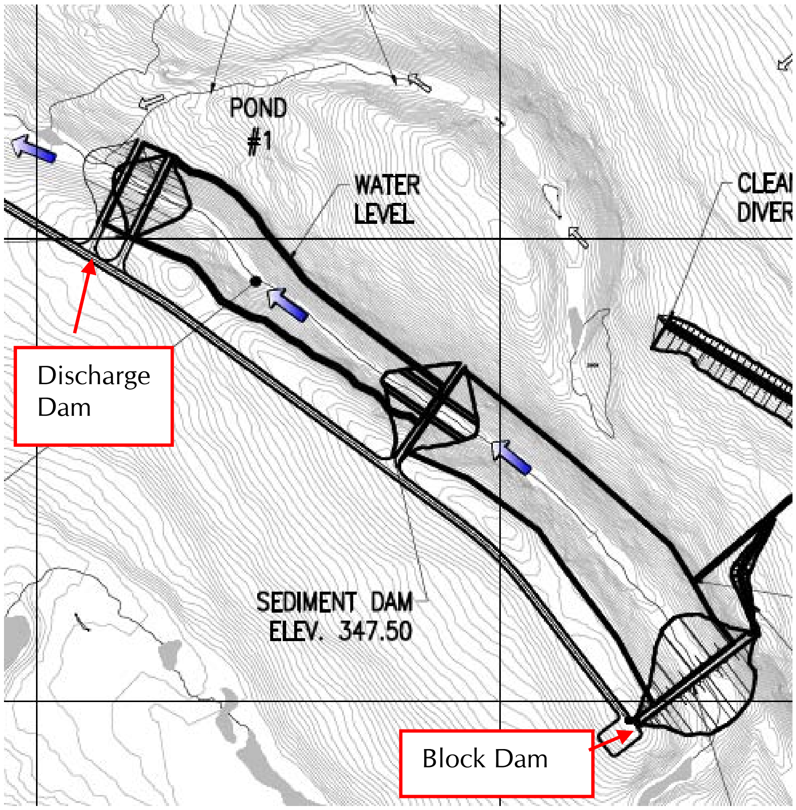

Figure 4-1: Mine site Pond 1 Configuration

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.

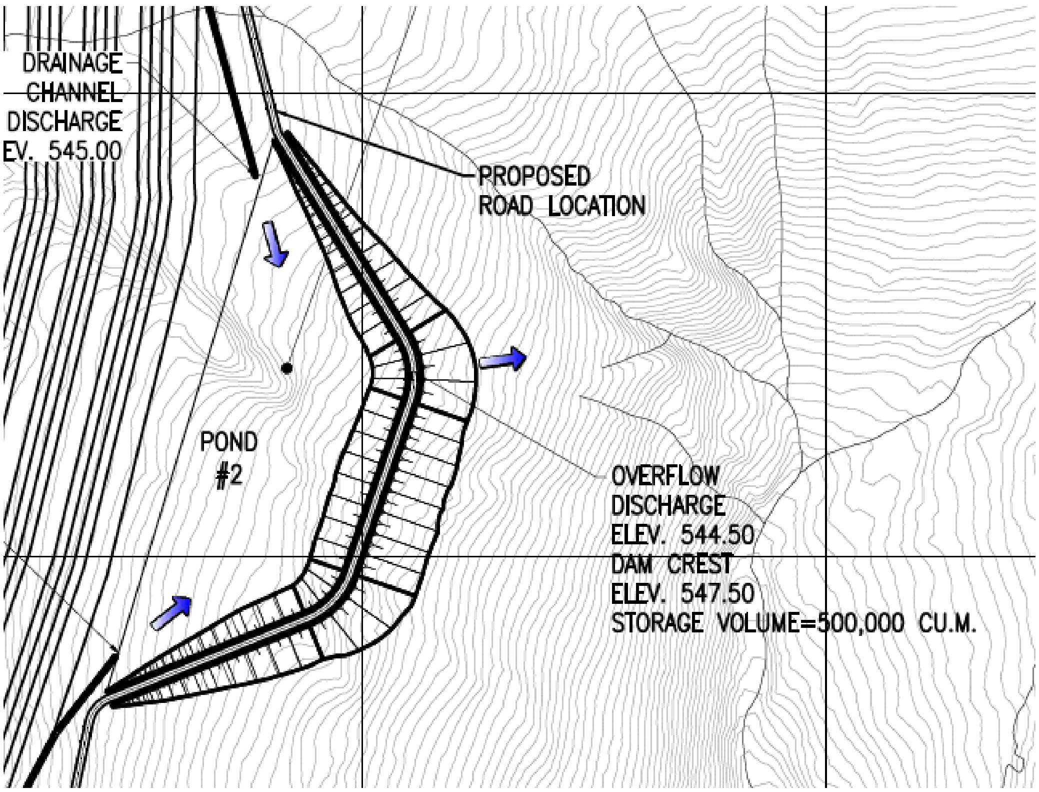

Figure 4-2: Mine site Pond 2 Configuration

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

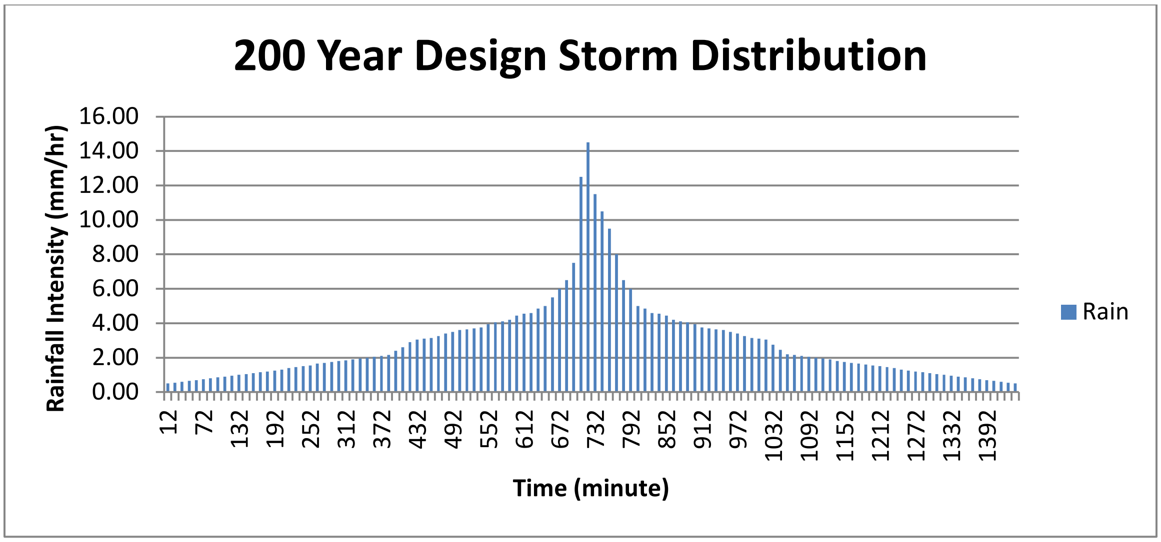

Figure 4-3: 200 Year Design Storm Distribution

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%).

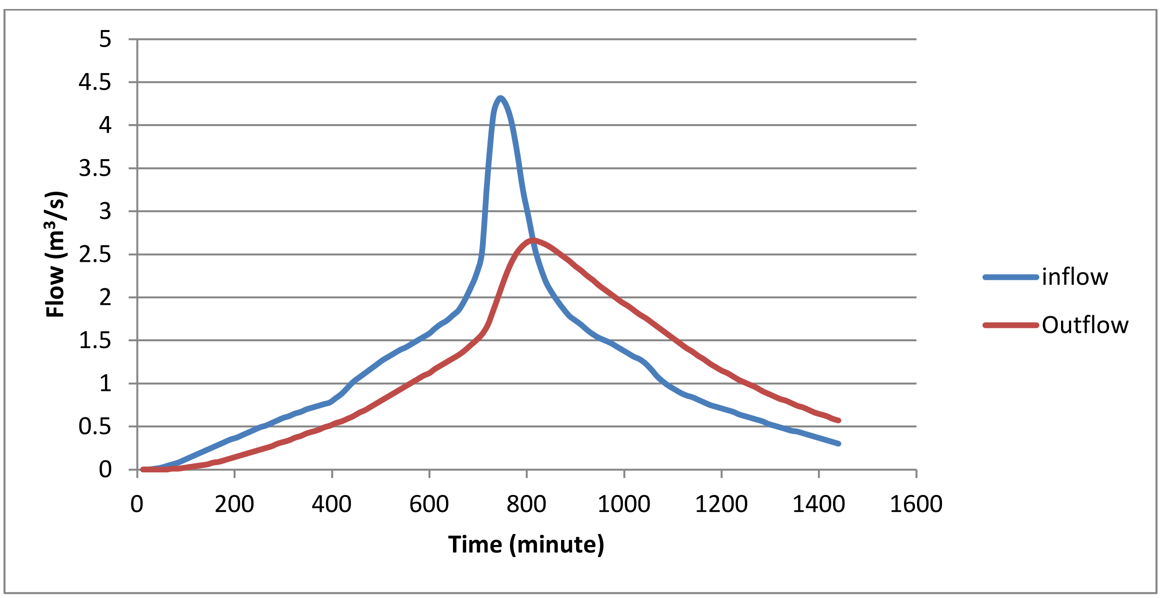

Figure 4-4 Inflow and outflow hydrographs, Mine Site Pond 2

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.

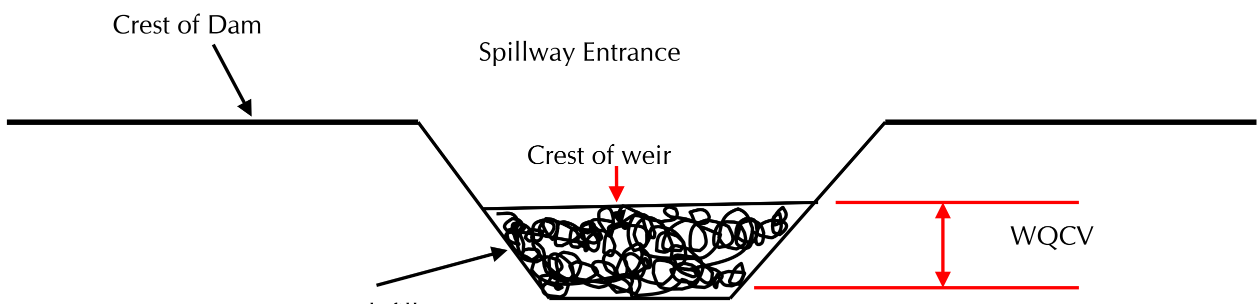

Figure 4-5: WQCV Concept Illustration

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

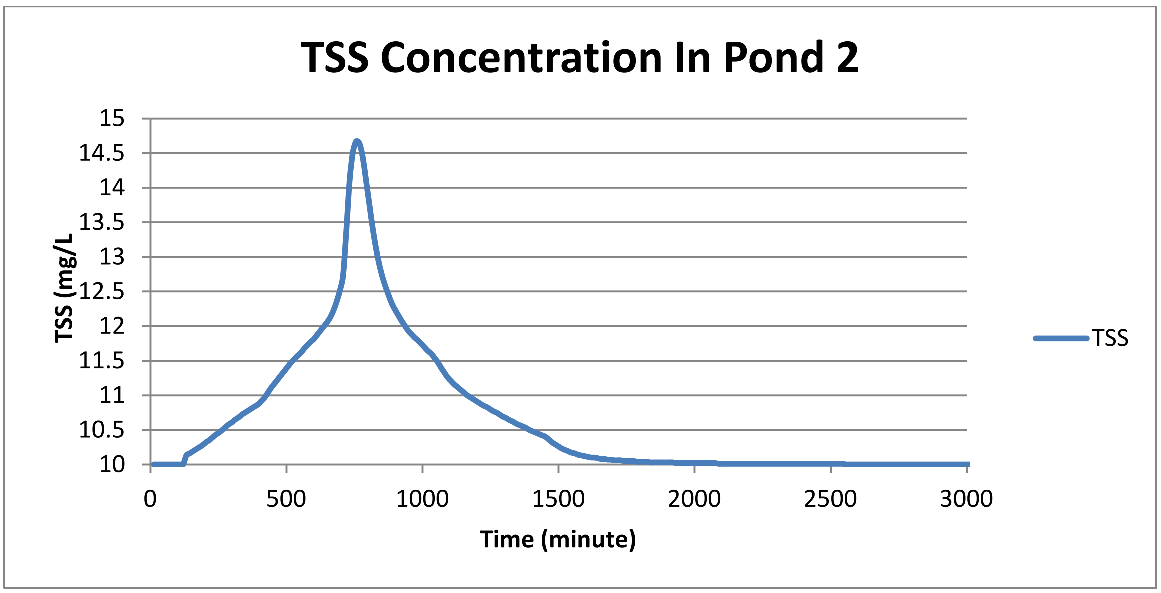

Figure 4-6: Pond TSS Removal Performance Example (Input TSS = 300 mg/L)

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.

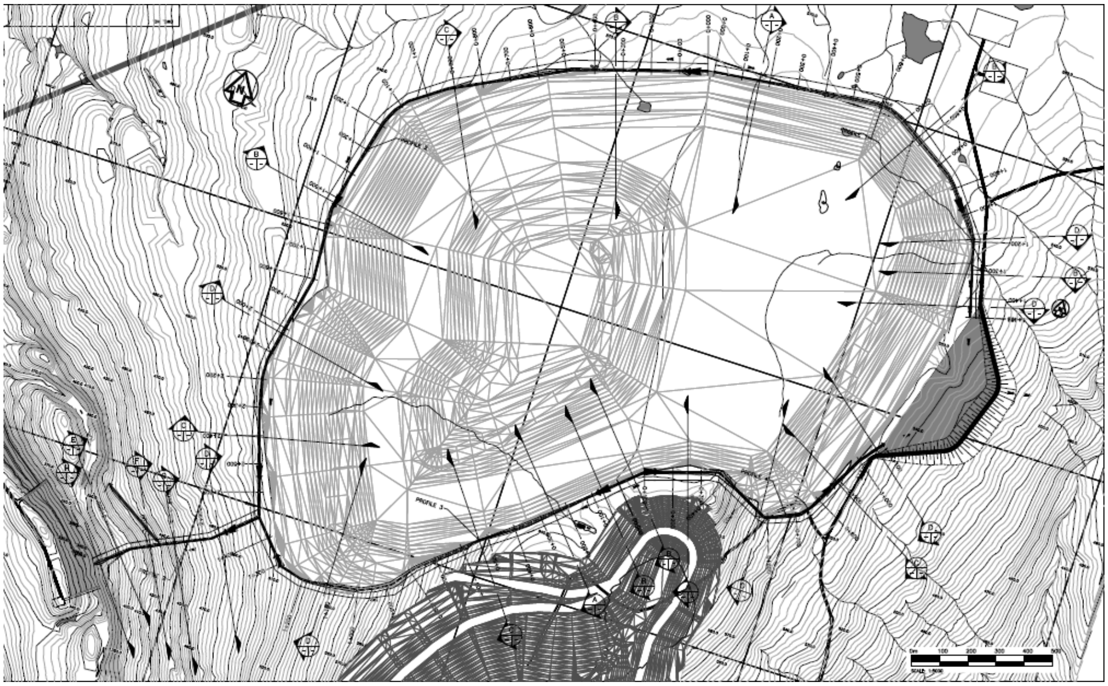

Figure 5-1: The Waste Rock Dump Ditches

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 km². 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.

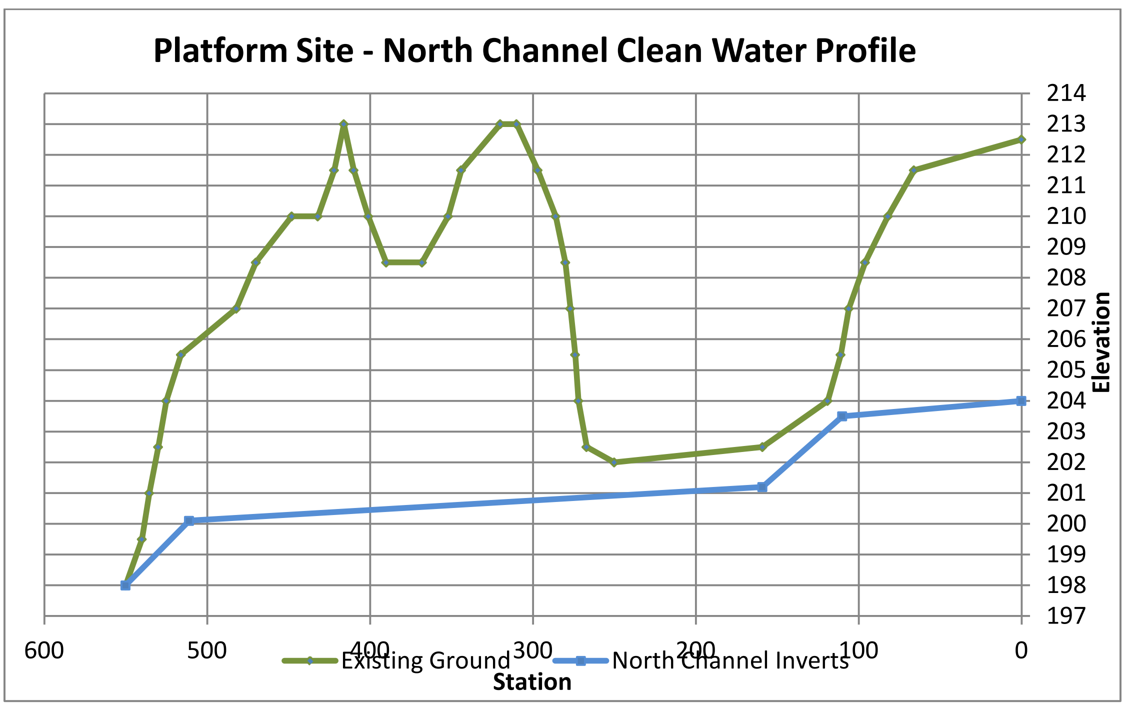

Figure 5-2: North Diversion Ditch (Ore Stockpile Platform

The channel profiles are shown in Figure 5-2 and

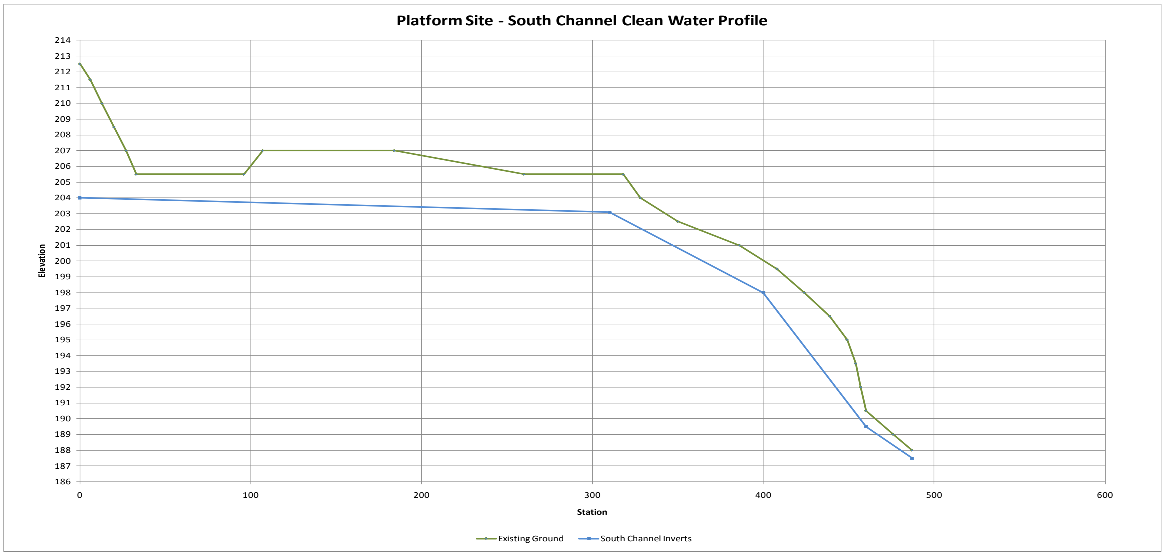

Figure 5-3. The channels range in slope from 0.3 percent to 14.2 percent. The discharges and D_{50} riprap for each section of the channel are shown in Table 5-2. The riprap for these channels was designed using References 6 and 7.

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.

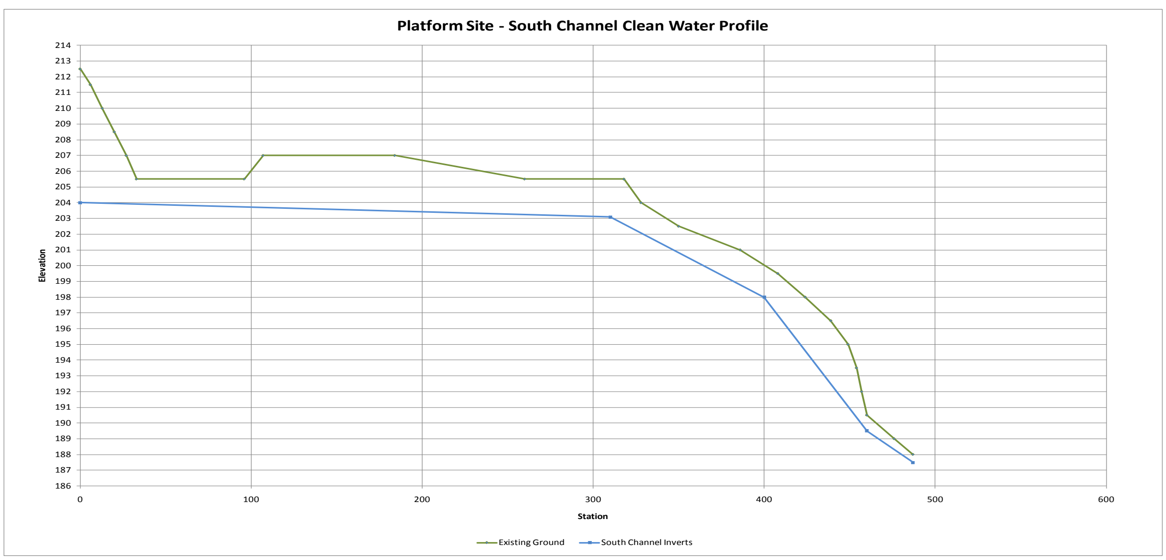

Figure 5-3: South Diversion Ditch (Ore Stockpile Platform)

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:

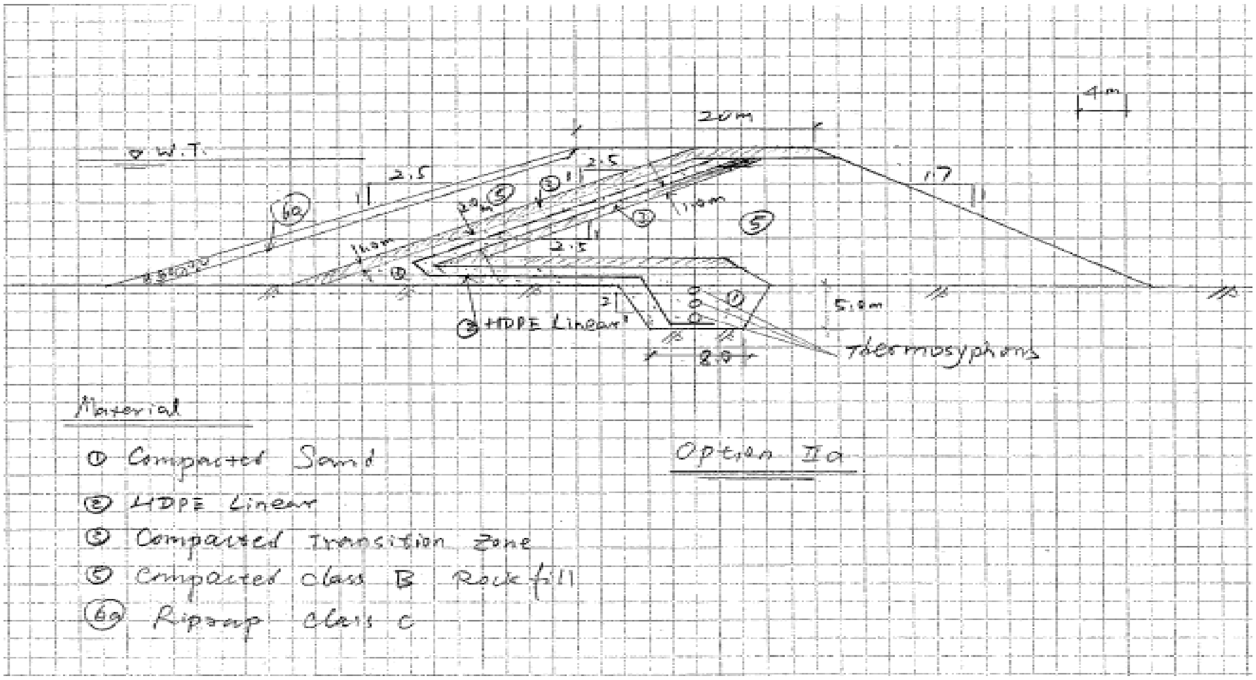

Figure 6-1: Typical Dam Cross Section

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
2. Grizzard, T.J., Randall, C.W., Weand, B.L. and Ellis, K.L., "Effectiveness of Extended Detention Ponds," Urban Runoff Quality - Impacts and Quality Enhancement Technology, Edited by B. Urbonas and L. Roesner, ASCE, New York, NY 1986.Hoggan Daniel H., 1996, Computer assisted floodplain hydrology and hydraulics, second edition, MaGraw-Hill, New York, San Francisco, and Washington, D. C.
3. Ontario Ministry of Environment, 2003, Stormwater Management Planning and Design Manual
4. Roesner, L. A., Urbonas, B., Sonnen, M.A., Editors of Current Practices in Design of Urban Runoff Quality Facilities, Proceedings of an Engineering Foundation Conference in July 1988 in Potosi, MO, Published by ASCE, New York, NY, 1989.6. Soil Conservation Service, Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, PB-281-278
5. U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels, EM1110-2-1601, July 1991
6. US EPA, 1983, Results of the National Urban Runoff Program, Volume I Final Report. NTIS PN84-185552, US EPA, Washington, DC.
7. US EPA, 2009, Storm Water Management Model Application Manual, National Risk Management Research Laboratory Office of Research and Development
8. US EPA, 2009, Storm Water Management Model User's Manual, National Risk Management Research Laboratory Office of Research and Development

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

Safety • Quality • Sustainability • Innovation

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

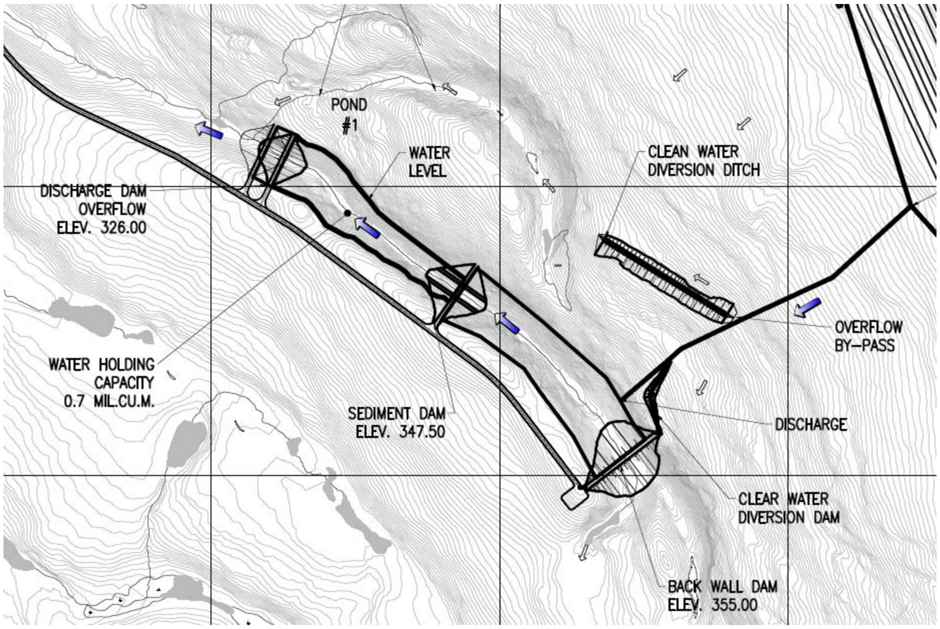

Figure 3-1: Dam Locations of Pond 1

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.

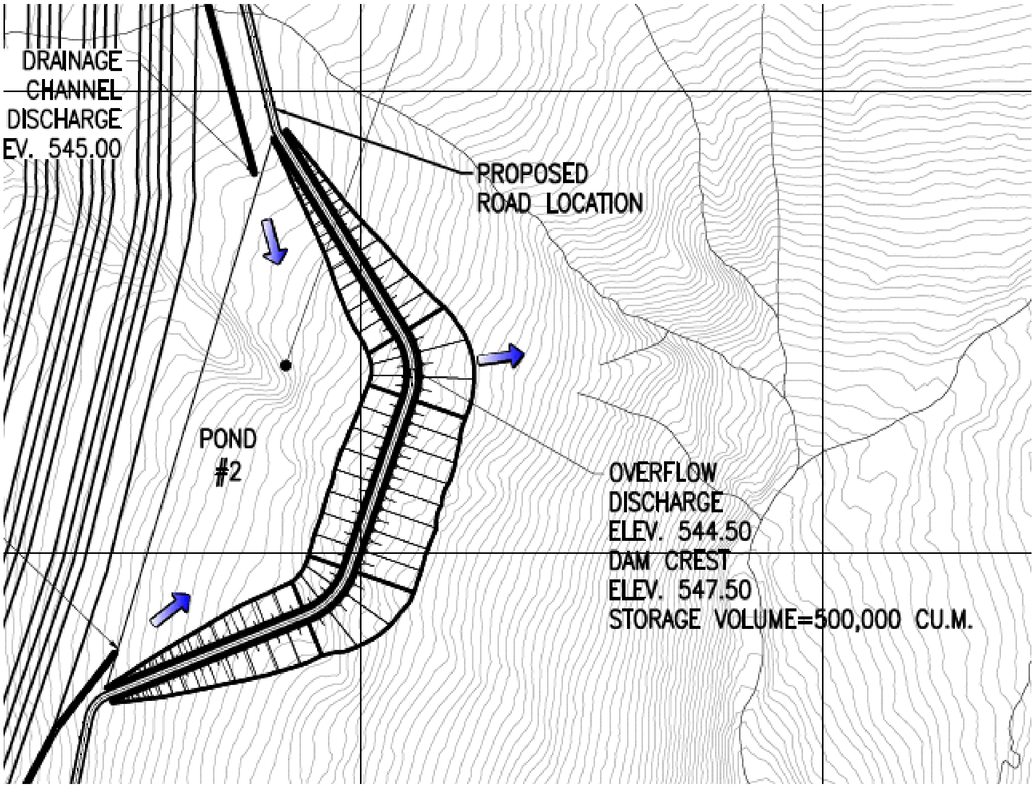

Figure 3-2: Minesite Pond 2 Dam

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.

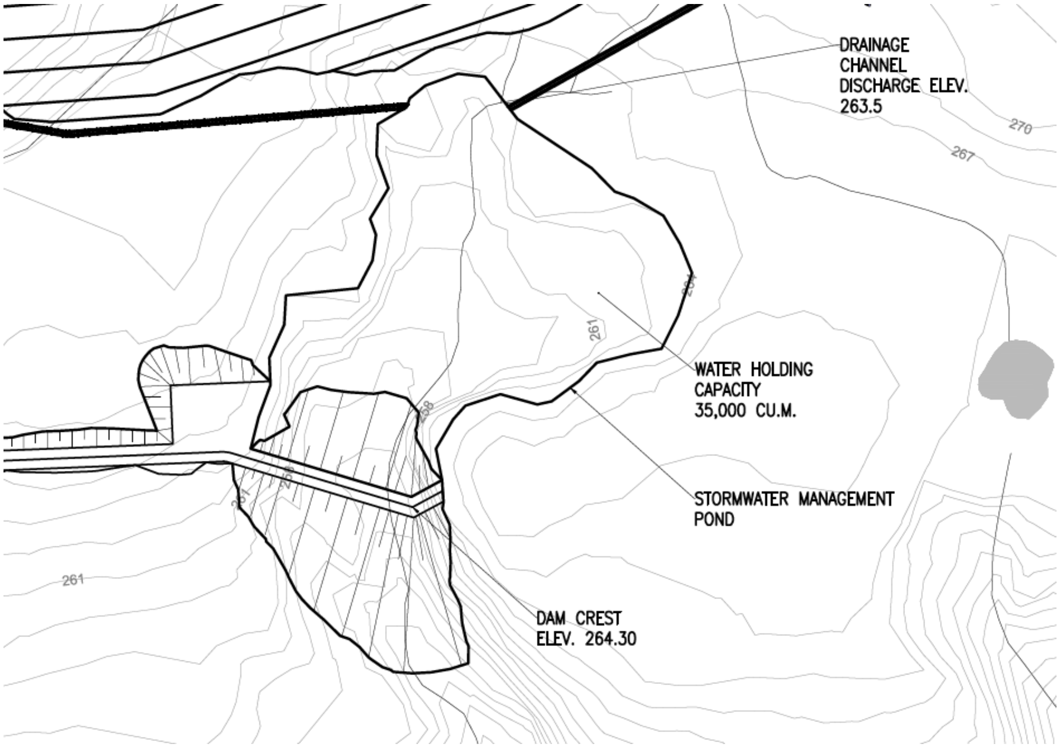

Figure 3-3: Stormwater Management Pond 3 Dam

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.

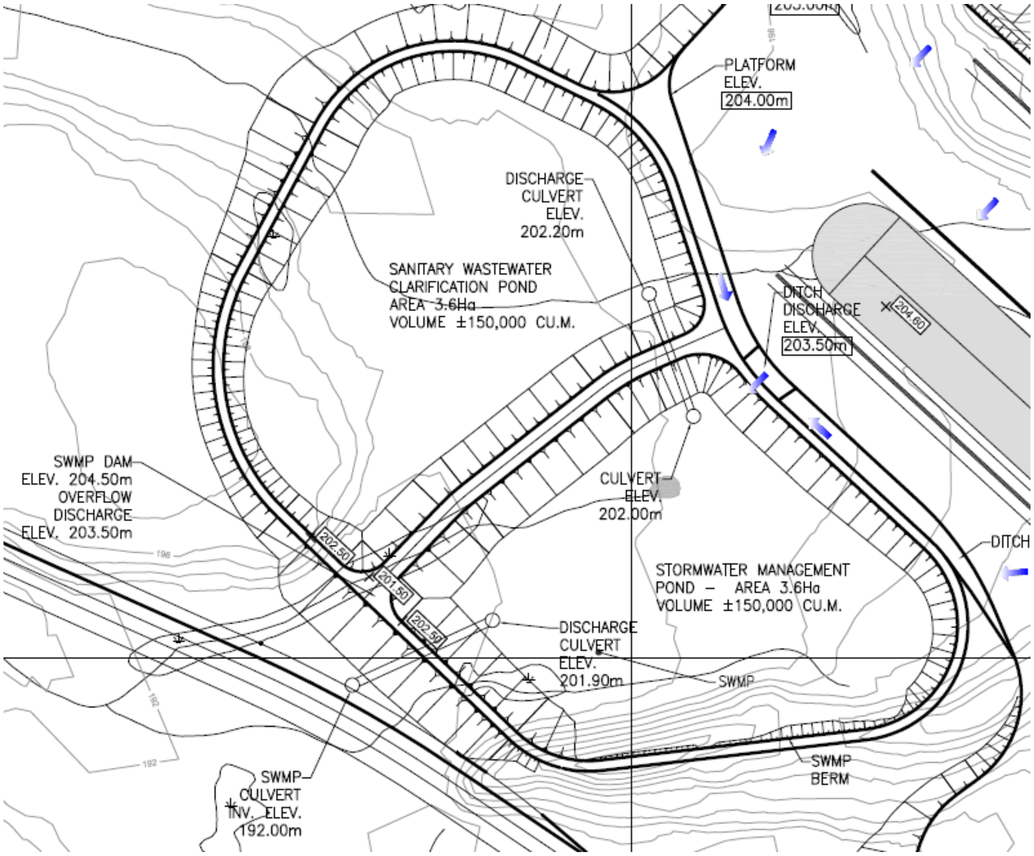

Figure 3-4: Stormwater Management Pond 4 Dam

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.

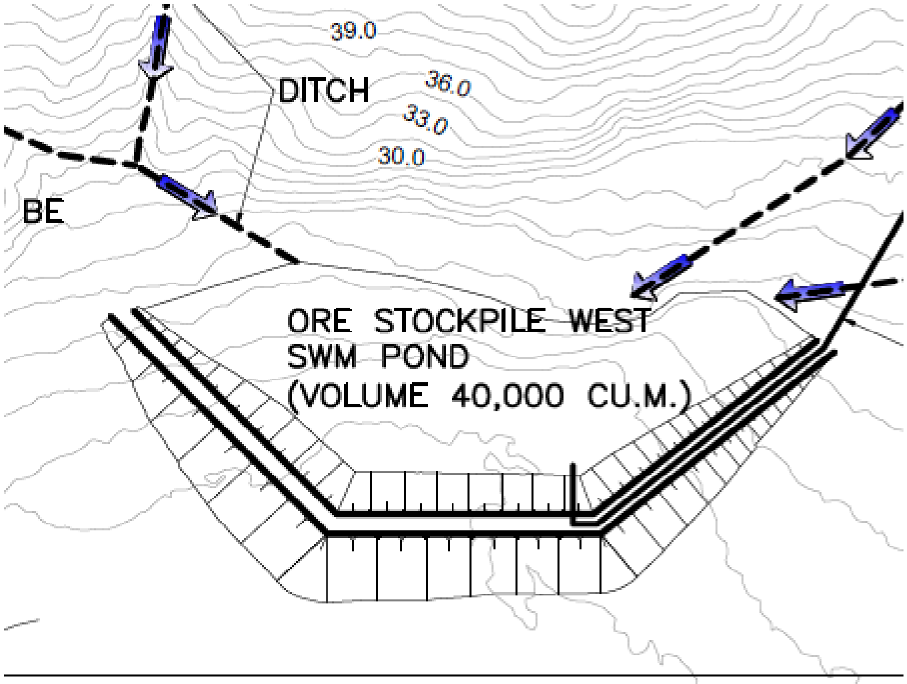

Figure 3-5: Pond 1 Dam (Milne Port)

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.

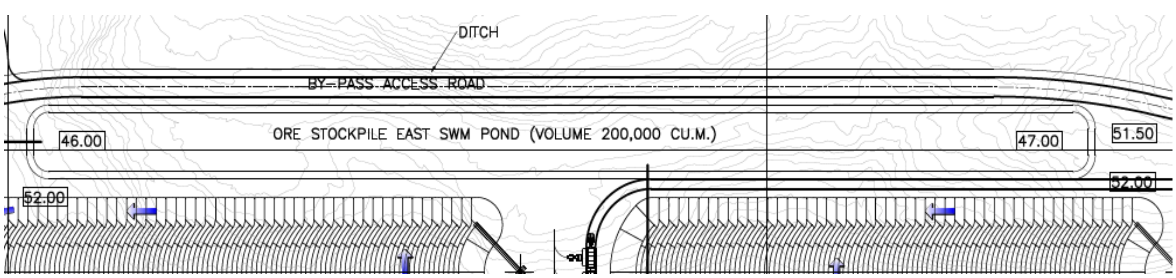

Figure 3-6: SWM Pond no. 9, Milne Port

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.

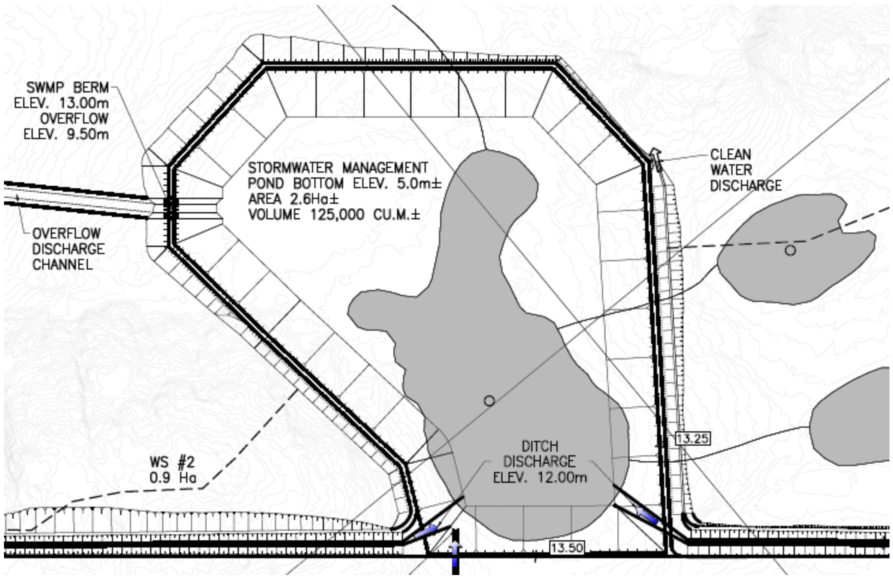

Figure 3-7: Steensby Port Ore Stockpile Stormwater Management Pond

3.10 Steensby Port, Platform Stormwater Management Pond Dam

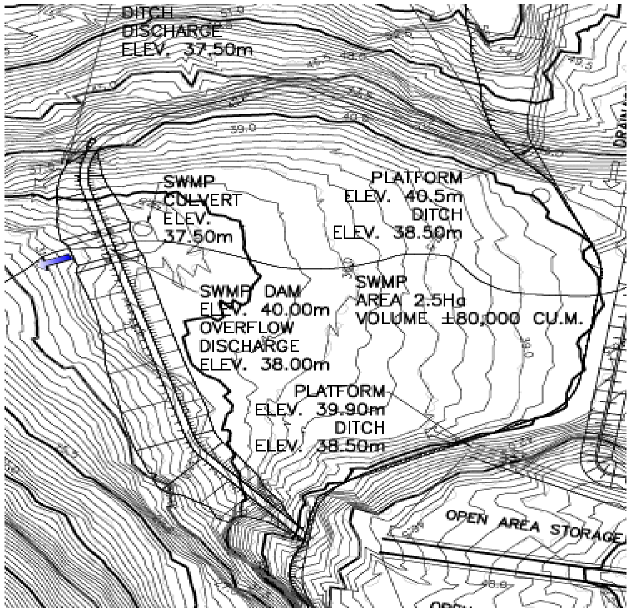

Figure 3-8: Platform Stormwater Management Pond Dam

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

Baffinland Iron Mines Corporation
Conceptual Design for Dam

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

Safety • Quality • Sustainability • Innovation

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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.

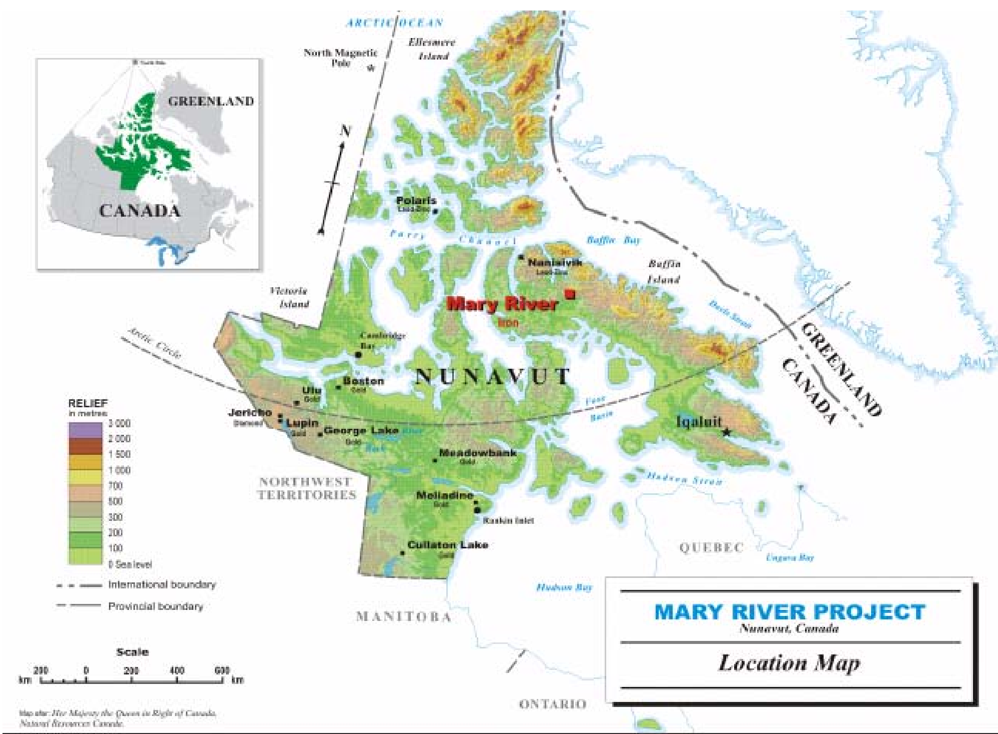

Figure 1-1: Location of Baffinland Mary River Project

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