November 15, 2007

Dillon Consulting Limited
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Attn: Mr. Gary Strong, P. Eng., Project Officer

Dear Mr. Strong:

Re: Additional Stability and Seepage Analyses for P-Lake Sewage Lagoon, Cape Dorset, NU

1.0 INTRODUCTION AND BACKGROUND

Based on outcome request from the pre-hearing meeting/teleconference, that took place on October 1, 2007, in Iqaluit, NU, AMEC Earth & Environmental, a division of AMEC Americas Limited (AMEC) has carried out a supplementary geotechnical study that included seepage analyses and slope stability analyses for the Cape Dorset sewage lagoon berm.

AMEC submitted previously to Dillon Consulting Limited (Dillon) two geotechnical reports, dated October 13, 2005 and August 21, 2007. A field drilling program and slope stability analyses were not carried out during preparation of the 2005 geotechnical report. A seepage analysis was also not performed, assuming that water seepage cannot occur through the frozen core berm with installation of a liner.

The final design of the project was completed by Dillon in 2007 and berm construction was commenced in July 2007. AMEC was retained to undertake construction monitoring services during excavating the cut-off trench underneath the lagoon berms. The construction monitoring report was submitted to Dillon on August 20, 2007. Additional geotechnical analyses were also performed and summarized in a geotechnical report, submitted to Dillon on August 21, 2007. The geotechnical report contained a review of the geothermal conditions for the berm based on as-built information and in-situ subsurface conditions obtained during construction monitoring. The geothermal analysis was based on a revised schedule of the lagoon filling, and revised berm dimensions. Slope stability analyses were also undertaken for the berm and road sections. It was understood, based on the construction monitoring data, that a bentonite liner was installed in the berm material and embedded a minimum of 2 m into the native soil at the cut-off trench location as shown at the Dillon design drawings. Results of the geothermal analyses indicated that the native soil at the cut-off trench location will be frozen about one year completion of the berm construction. Hence, a seepage analysis was not considered to be necessary.

In August 2007, the Nunavut Water Board made a decision to hold a pre-hearing meeting/teleconference in accordance with Rule 14 of the Board's Rules of Practice and

Procedure for Public Hearings. As a result of the pre-hearing meeting/teleconference, a list of issues to be dealt prior to the public hearing was developed. It was understood that the geotechnical scope of work should comprise two tasks as indicated in the project memorandum prepared by BGC Engineering Inc. on October 2, 2007, as follows:

1. "Check stability of the upstream slope under rapid drawdown conditions based on maximum potential drawdown rate. Stability analysis should assess stability of the thin unfrozen zone of the upstream shell predicted from the geothermal analysis as well as conservatively assuming that the entire shell upstream of the liner is thawed, with a failure zone that incorporates the GCL liner as a potential lower strength element within the slope."

2. "Conduct a seepage analysis for the case assuming there are undetected defects/holes in the liner, with seepage taking place through the active zone in the downstream shell. Assess potential for this seepage to affect slope stability and integrity of frozen conditions in the berm and foundation."

This report presents the results of the work carried out in fulfilment of the request from the October 1, 2007 pre-hearing meeting.

2.0 SUBSOIL CONDITIONS & BERM DIMENSIONS

The soil profile, observed during excavation of the cut-off trenches, comprised mainly fill over silt over clay. Bedrock was encountered at about 35% of the cut-off trench length below fill or silt at shallow depth (less than 2 m). No bedrock was encountered along the remaining cut-off trench alignment (see report submitted to Dillon: "Cut-off Trench Excavation, Sewage Lagoon Berms Construction Monitoring, Cape Dorset, NU", AMEC, 2007). The base of the trench was in frozen silt, clay or bedrock over the entire cut-off trench length. It is expected that within such sections of the cut-off trench, the bedrock would be found at a depth of about 3 m below the ground surface. Thus, the soil profile and berm material is summarised as follows:

- berm material – sand and gravel;
- native soil from ground surface to 3 m depth - silt and clay;
- native soil below 3 m depth - bedrock.

Berm dimensions, applied in the slope stability and seepage analyses, were the following:

- Height of berm is 6 m.
- Width of crest is 4 m.
- Upstream and Downstream slope of berm are 2.5H:1V.
- The maximum water level at the upstream slope was assumed to be at El. 122.5 m.

3.0 SOIL PARAMETERS

The mean soil parameters used in previous analyses (see report submitted to Dillon: "Additional Geotechnical Analyses for P-Lake Sewage Lagoon, Cape Dorset, AMEC, August 21, 2007) were used in the current analyses. These values are presented in Table 1.

Table 1: Input Parameters of Berm Fill and Native Soil

Input Parameters		Mean Value
Compacted Sand & Gravel	Apparent Cohesion (kPa)	2
	Friction angle°	33°
	Unit Weight (kN/m^3)	19
Native Silt & Clay	Apparent Cohesion (kPa)	25
	Friction angle°	0°
	Unit Weight (kN/m^3)	18.5

4.0 SLOPE STABILITY AND SEEPAGE ANALYSES

Based on request from the pre-hearing meeting, two main scenarios were considered:

- Long term stability for downstream saturated slope as a result of seepage through damaged liner (berm with frozen core).
- Short term stability for upstream saturated slope under a rapid drawdown

An additional scenario was considered in long term stability for downstream saturated slope assuming a berm with no frozen core and liner. Modeling of this scenario was not listed in the pre-hearing meeting/teleconference scope of work or in the BGC memorandum.

It is understood that the pore water pressure within saturated soil will reduce the effective overburden stress of soil, and hence decrease the shear strength of soil at the failure slip. For all scenarios, SEEPW (Geostudio 2004) finite element computer software was used to evaluate the phreatic surface within the berm slopes under assumed boundary conditions. Based on results of the seepage analyses, steady state slope stability analyses were then undertaken.

The saturated hydraulic conductivity of sand and gravel (berm material) was estimated to be in a range of $1x10^{-3}$ m/s to $1x10^{-5}$ m/s (B. M. Das, 1983). For the transient boundary conditions, suitable for rapid drawdown of the water level in the impoundment area, the hydraulic conductivity function for unsaturated material and volumetric water content function were estimated using method proposed by Fredlund et al. (1994). The native silt and clay and frozen soil were assumed to be impermeable material, having low values of the hydraulic conductivity.

Commercial computer software SLOPEW (Geostudio 2004) was applied for assessment of the slope stability. During slope stability analysis, the GCL liner was considered to be a weak material compared to the berm fill such that the strength of liner was ignored. Circular failure mechanism within active layer was adopted to calculate the minimum factor of safety against slope failure.

The following sections provide descriptions on the above mentioned three scenarios.

4.1 Long Term Stability for Downstream Slope with Frozen Core and a Damaged Liner

Based on the results of geothermal analyses, frozen core will occur within one year after completion of the berm construction. The thickness of the active layer measured from the top of the berm surface to frozen material at the end of the first summer varies from about 2 m to 3 m. It is understood that a GCL liner was installed near the central portion of the berm as shown at the Dillon design drawings. It was assumed that there are undetected defects/holes in the liner. Since a two-dimensional seepage analysis was carried out, meaning that the liner damage (opening) extends along the entire length of the berm. It was assumed that the opening is 25 mm wide in the berm cross section and located near the top of the frozen core. In the seepage model, elements representing the opening were assigned as a material with high permeability.

The worst upstream hydrological condition was assigned when the water level in the impoundment area is located at the upper bound elevation, El. 122.5 m. Seepage analyses were carried out for the upper and lower bound values of the saturated hydraulic conductivity, 1×10^{-3} m/s and 1×10^{-5} m/s, respectively. Based on results of the seepage and slope stability analyses, there are three issues for consideration:

<u>Slope stability</u>

Figure 1 presents the potential failure slip which yields the lower factor of safety of 1.6. Insignificant changes in the present location of the failure slip would be expected if the liner is modeled as a “low strength element.” The phreatic surface for the upper bound value of the saturated hydraulic conductivity (i.e. 1×10^{-3} m/s) is also shown on Figure 1.

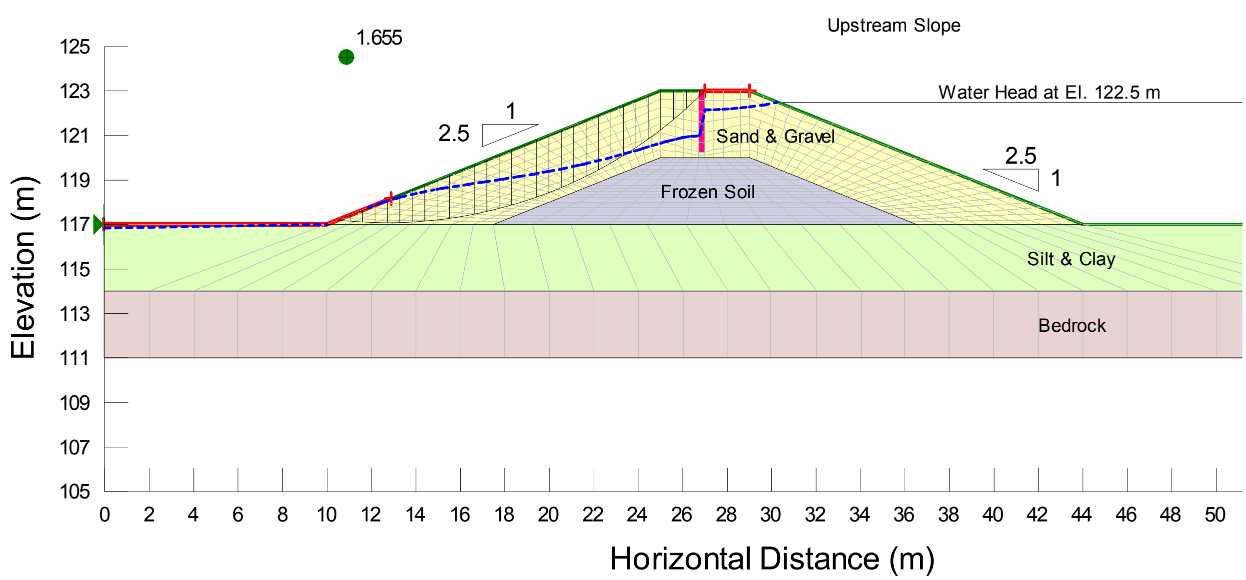

Figure 1: Factor of Safety and Phreatic Surface for Downstream Slope with Opening along Liner

Amount of seepage near toe of downstream slope

The amount of seepage per linear meter of the berm near the downstream slope toe was obtained based on the results of the seepage analyses (Table 2).

Table 2: Seepage at Downstream Slope Toe

Hydraulic Conductivity of Berm Material (m/s)	Amount of Seepage near the Toe of Downstream Slope (m^3/s) – 3 m Active Zone	Amount of Seepage near the Toe of Downstream Slope (m^3/s) – 2 m Active Zone
1×10^{-3}	3.3×10^{-4}	2×10^{-4}
1×10^{-5}	3.3×10^{-6}	2×10^{-6}

Piping potential assessment

The factor of safety against piping near the toe of the downstream slope can be defined as the ratio of i_{cr} to i_{exit} where i_{cr} is the critical hydraulic gradient of berm material and i_{exit} is the estimated maximum gradient based on results of the seepage analyses. The i_{cr} of soil may be evaluated using void ratio (*e*) and specific gravity (G_s) of soil and generally varies within a range from 0.85 to 1.1 (Das, 1983).

Figure 2 presents the contour plot of the estimated maximum gradient for the lagoon berm with a damaged liner above the frozen soil. The hydraulic gradient near the toe of berm was estimated to be 0.2 to 0.4 and the factor of safety against piping was calculated to be greater than 2.

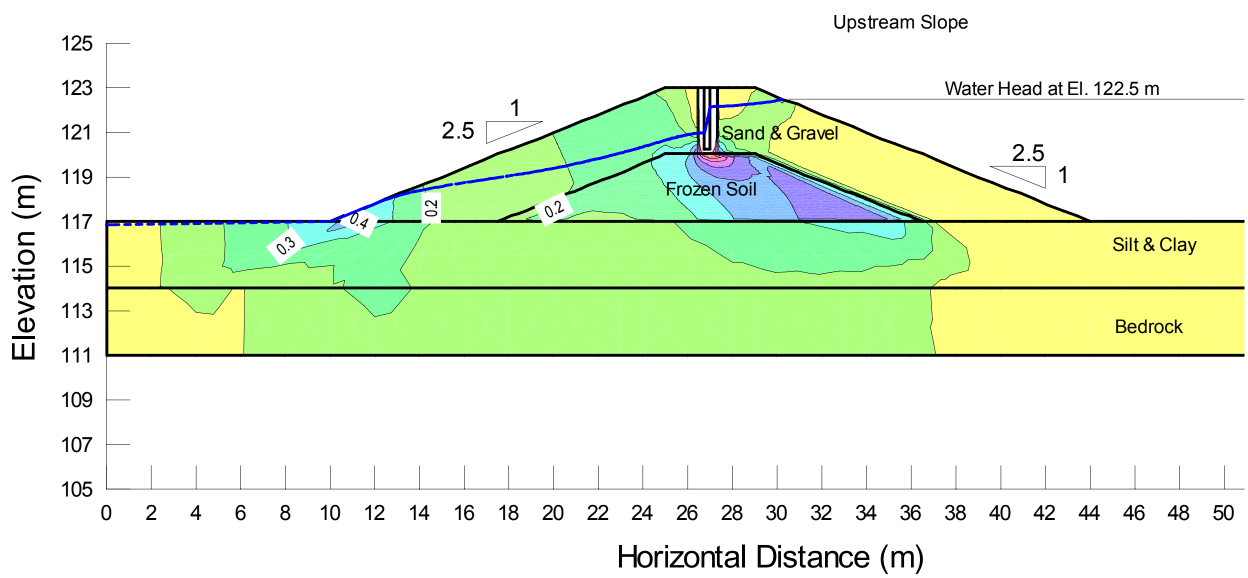

Figure 2: Estimated Maximum Gradient within Lagoon Berm (Ksat = 1×10^{-3} m/s)

4.2 Short Term Stability for Upstream Slope Under Rapid Drawdown

According to the current operational schedule, the water level in the impoundment area will increase at a rate of approximately 0.45 m per month, up to elevation 122.5 m, and the water then will be discharged in 2 weeks, until the water level will be dropped down to an elevation of 0.45 m above the ground surface.

During drawdown of water level on the upstream slope, excess pore water pressure may develop within the berm fill material. The rate of the pore pressure dissipation will depend on the hydraulic conductivity of the berm material. For assessment of the rapid water drawdown on the slope stability, the lower bound value of the saturated hydraulic conductivity (1 x 10^{-5} m/s) was used in the seepage analysis.

Figure 3 illustrates the estimated phreatic surface within upstream slope as the water level in the impoundment area is lowered according to the provided schedule. Number of days, shown on contour lines, presents the phreatic line corresponding to the days after commencement of the drawdown. The results indicated that the phreatic surface response almost simultaneously with the change of the water level in the impoundment. It means that the risk of building up of the excessive pore water pressure within the upstream slope is very low.

Figure 4 presents a similar scenario by assuming that the GCL liner was installed improperly and malfunctions. Locations for the phreatic surface at the berm upstream slope under this scenario are similar to that provided in Figure 3.

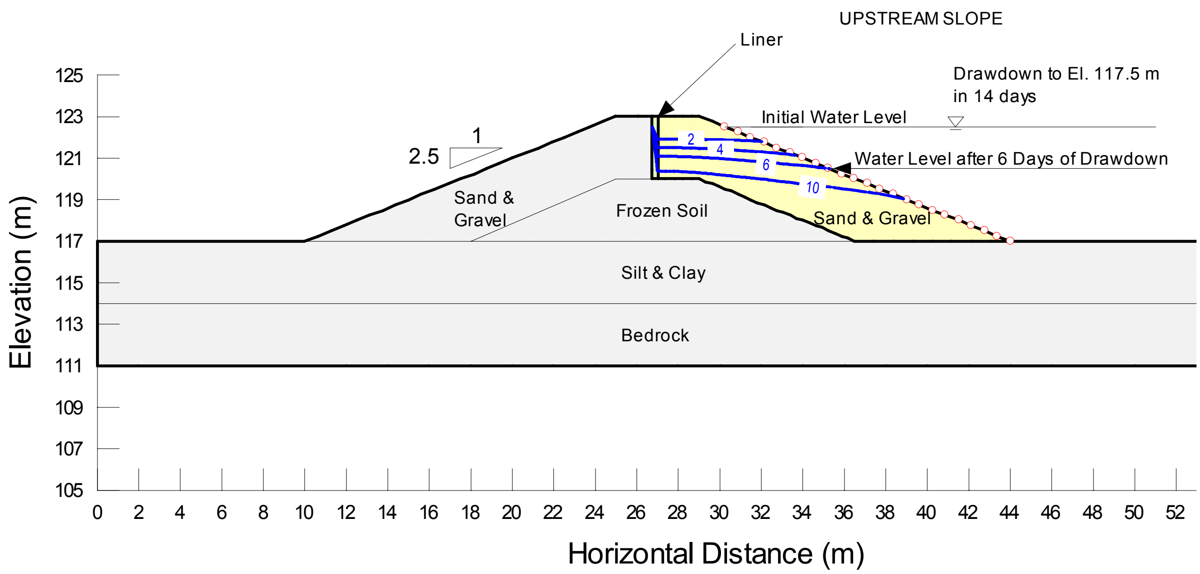

Figure 3: Phreatic Surface at Upstream Slope During Drawdown

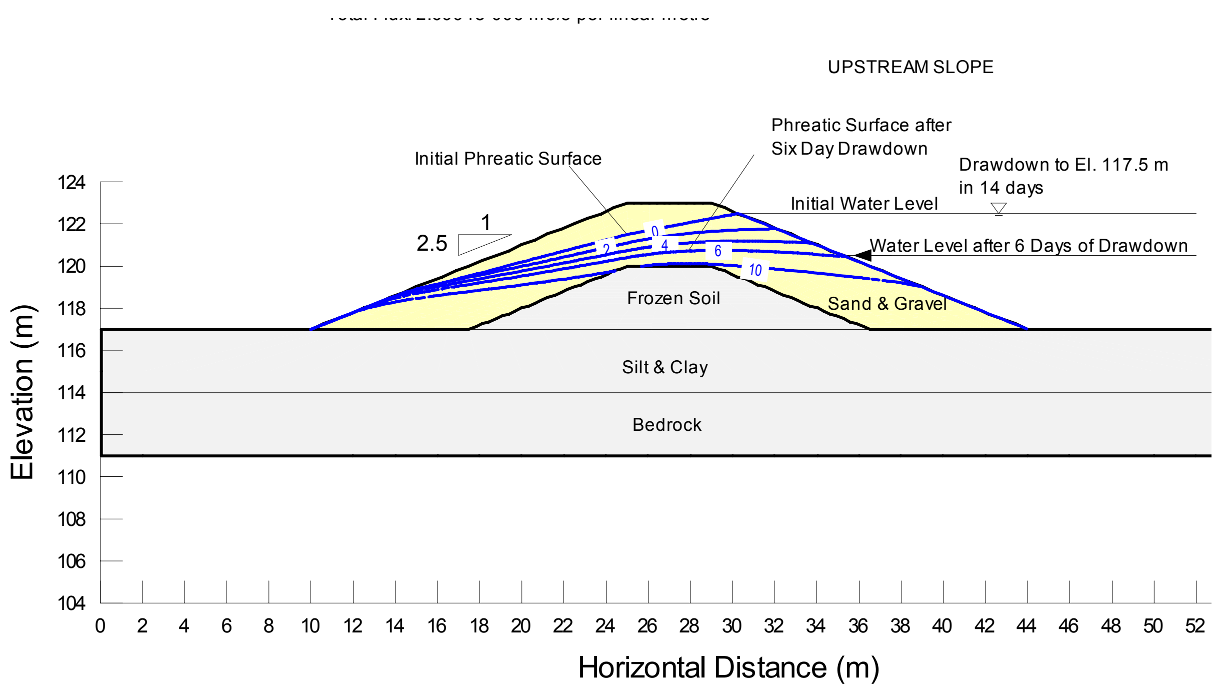

Figure 4: Phreatic Surface at Upstream Slope During Drawdown (berm with no liner)

For stability analysis, a conservative phreatic surface was assigned at the upstream slope, i.e. the phreatic surface was modeled at 1 m or 2 m higher than the water level in the impoundment during the drawdown. The berm material at the downstream slope (behind the liner) was assumed to be unsaturated. The factor of safety against slope failure is calculated to be 1.5 and 1.3 for 1 m and 2 m water head difference respectively, which meets the minimum requirement outlined in the Dam Safety Guidelines (1999) for short term condition. Figure 5 and 6 present the potential failure slip and factor of safety.

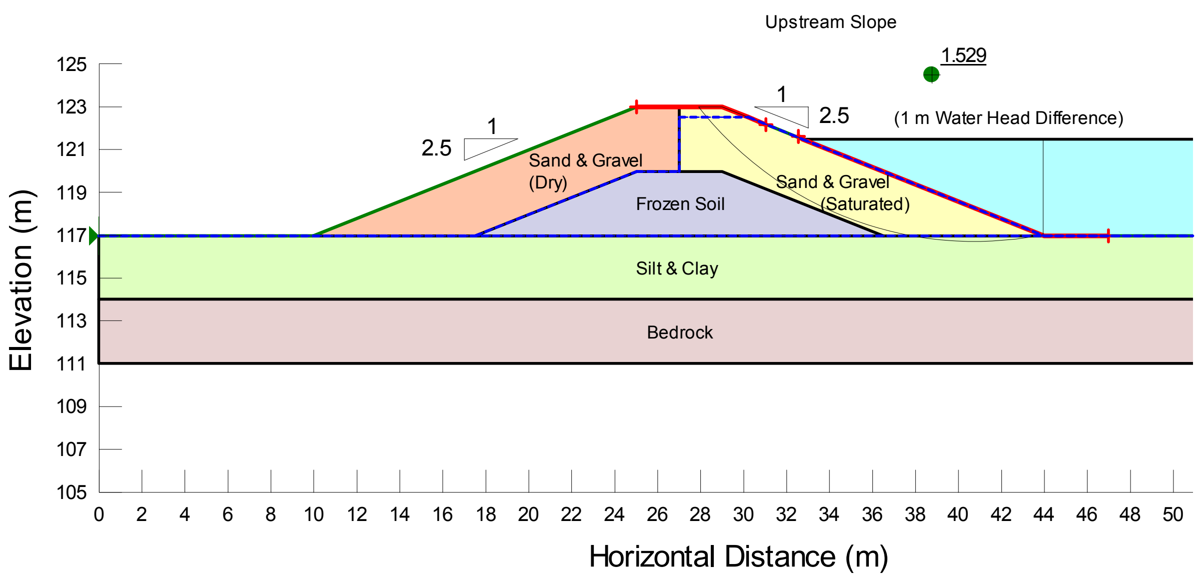

Figure 5: Factor of Safety for Upstream Slope with 1 m Water Head Difference

Similar to the long term slope stability assessment (Section 4.1), AMEC does not expect significant changes in the present location of the failure slip, if the liner be modeled as a low strength element.

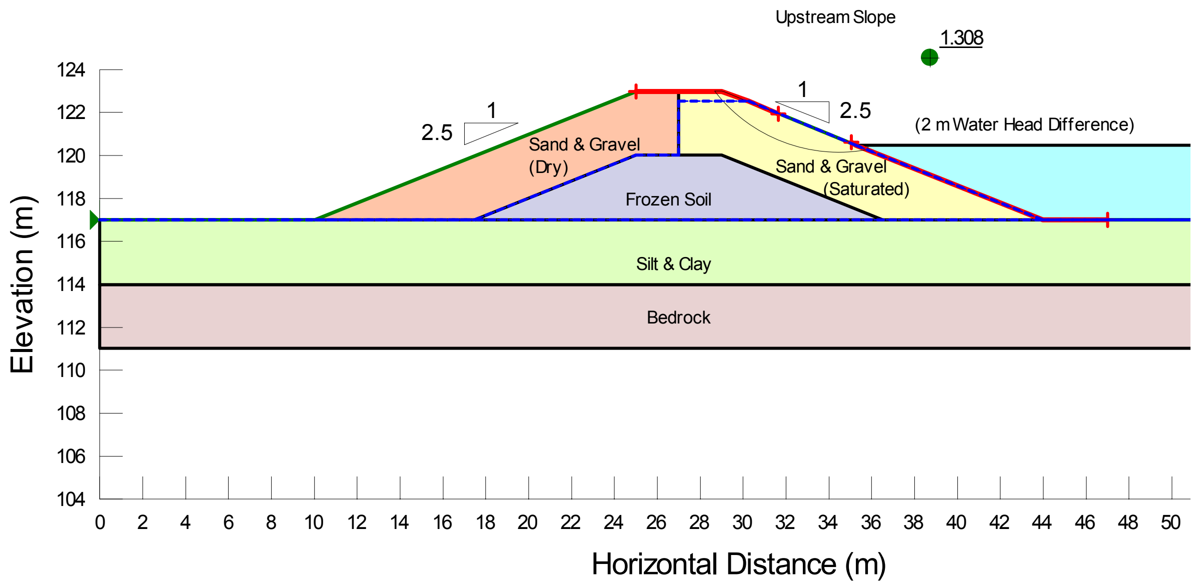

Figure 6: Factor of Safety for Upstream Slope with 2 m Water Head Difference

For this scenario, no noticeable change of the failure slip location is expected, if the liner is modeled as a low strength element.

4.3 Long Term Stability for Downstream Slope with no Frozen Core and Liner

As mentioned above, modeling of this scenario was not listed in the pre-hearing meeting/teleconference scope of work or in the BGC memorandum. This case is considered to be the worst assumption when the fill material of the lagoon berm remains unfrozen and the installed liner is malfunction during operation of the facilities. It was assumed that the lagoon water will infiltrate through the berm and emerge on the downstream slope. The estimated phreatic surface and potential failure slip are summarized at Figure 7 below.

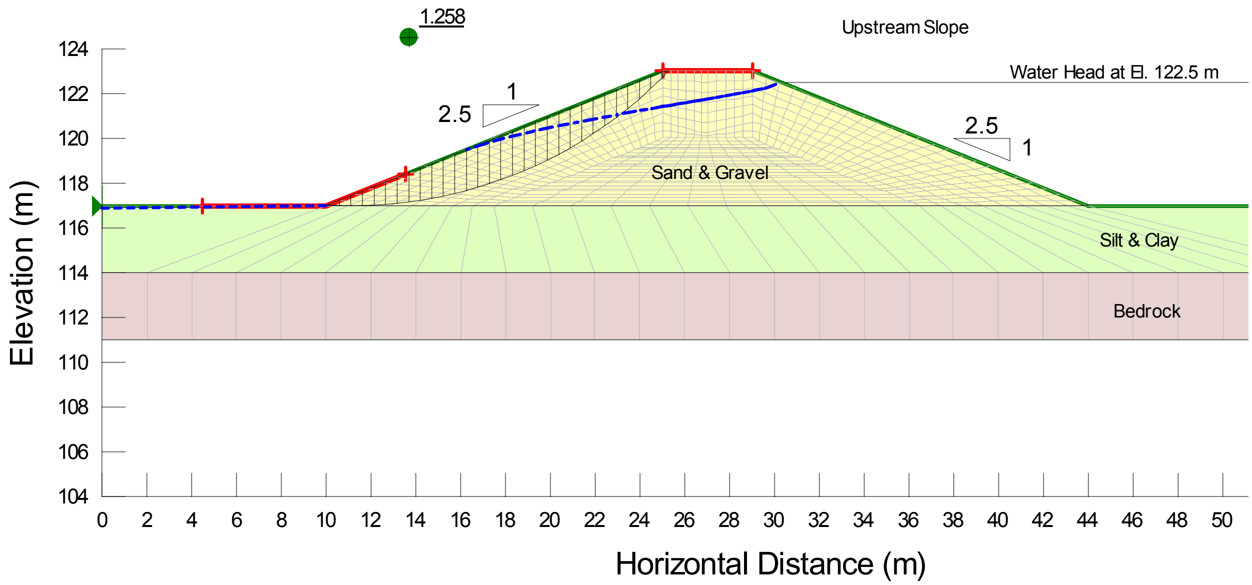

Figure 7: Factor of Safety for Downstream Slope (berm with no frozen core and liner)

The factor of safety is estimated to be 1.3 which is less than the minimum requirement of 1.5 recommended in the Dam Safety Guidelines (1999) for long term consideration. However, this scenario is very unlikely and the above factor of safety should be sufficient. It is considered that should this condition occur, mitigation strategies such as:

- additional drainage provision,
- support berms, or
- modified operational procedures

could be implemented to address the potential for reduced stability under this unlikely scenario. Periodic monitoring of the berm performance and conditions is recommended.

5.0 CLOSURE

The geotechnical analyses presented herein are based on data provided to AMEC by Dillon Consulting Limited, field observation during trench excavation, limited shallow drilling during access road construction, review of the published reports and AMEC design experience for similar structures in permafrost areas.

The results of the slope stability analyses demonstrate that the berm will be stable (safety factor equal to 1.3) even if the frozen core cannot be created and the liner were not installed. However, this scenario is considered to be very unlikely. The safety factor for realistic scenario is 1.6 (Section 4.1), exceeding the commonly used safety factor of 1.5 for an assessment of stable slopes. The safety factor against potential of piping is 2, which can be considered as a amec

low limit of the accepted safety factor. However, AMEC considers that the piping process cannot be developed in the sandy/gravely material used for the berm construction.

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Respectfully submitted,

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Date November 15, 2007
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The Association of Professional Engineers, Geologists and Geophysicists of the NWT / NU

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