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SEWAGE LAGOON AND SOLID WASTE DISPOSAL FACILITY GEOTECHNICAL SERVICES QIKIQTARJUAQ, NUNAVUT

Submitted to:

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Orangeville, Ontario

Submitted by:

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Calgary, AB

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1.0 INTRODUCTION

AMEC Earth & Environmental, a division of AMEC Americas Limited (AMEC) was retained by Nuna Burnside Engineering and Environmental Ltd. (NBEEL) to provide geotechnical input and geothermal design work related to a project to replace the existing water reservoir, sewage lagoon and solid waste disposal facility for the Hamlet of Qikiqtarjuaq, Nunavut. A geotechnical assessment that included field reconnaissance, testpits and terrain mapping was conducted by NBEEL in 2005. The current phase of the geotechnical design work is in support of the Nunavut Water Board License Application. Authorization to proceed with the current geotechnical analyses was received on November 29, 2006 via fax by Mr. Matthew Paznar with NBEEL.

The berm design drawings, geotechnical report and construction specifications of the NBEEL for the reservoir, lagoon and solid waste disposal facility were provided to AMEC for information.

2.0 SCOPE OF WORK

Based on a verbal discussion with NBEEL, the scope of work for geotechnical services includes an assessment of design soil parameters and engineering analyses (seepage, slope and geothermal) comprising the following tasks:

1. Review of available subsoil information and provide basic soil parameters for the design;
2. Carry out slope stability analyses for the proposed berm slope with consideration of permafrost condition;
3. Perform seepage analyses for the proposed berm slope;
4. Conduct geothermal analyses to assess the ground temperatures in short term and long term considerations under lagoon operation;
5. Provide geotechnical recommendations regarding the design and construction of the lagoon and solid waste disposal berm.
6. Prepare a report that summarizes the work completed

In order to gain understanding of subsoil and permafrost conditions of the proposed sites, the following documents were additionally reviewed by AMEC:

- Foundation Soils Investigation – Existing Power Generation Facility, Broughton Island, NT. AGRA Earth & Environmental Limited, Yellowknife, NWT (1997)
- Geotechnical Investigation - Proposed School Building, Broughton Island, NT., GNWT Public Works and Highways. AGRA Earth & Environmental Limited (1988)
- Proposed Broughton Island Power House Addition, Geotechnical Investigation, Broughton Island, NT., NWTPC, Thurber Consultants Ltd. (1988)
- Geotechnical Investigation – Proposed Health Centre, Broughton Island. Hardy BBT Limited, Yellowknife, NWT (1990)

3.0 PROJECT DESCRIPTION

The new reservoir berm and lagoon berm are proposed to be between 5 m to 7.5 m high with side slopes at 3H:1V and 2H:1V. The width of berm crest is 4.5 m and 3 m. The solid waste disposal facility consists of perimeter and intermediate berms, hazardous waste storage area, and bulky material disposal area. The height of perimeter and intermediate berms are approximately 2 m. It is understood that the in-situ material, sand with gravel will be used for the construction of all berm slopes.

4.0 SUBSOIL CONDITIONS

Based on the provided test pit log at the existing water reservoir, the in-situ surficial material encountered up to 2 m of medium to coarse grained sand and gravel with various amount of cobbles and small boulders. Test pits excavated near the existing and the proposed wastewater lagoon sites revealed medium to coarse sand and gravel with cobbles and small boulders. The landfill site and the surrounding area consist of similar subsoil conditions, mainly medium to coarse sand and gravel with abundant cobbles and small cobbles. Permafrost was encountered at 2 m to 2.1 m below ground surface for each test pit location.

The results of grain size distribution in these areas suggested that the in-situ sand and gravel, with trace silt are poorly graded and classified as SP according to Unified Soil Classification System. The gravel content varied between 40 % and 43 %.

5.0 SOIL PARAMETERS

Depending on the composition of in-situ materials, the unit weight of well compacted sand-gravel is generally between 19 kN/m^3 and 21 kN/m^3. Based on the correlation of dry unit weight with internal friction angle published in the Design Manual, NAVFAC DM 7.01 and our geotechnical experience with similar soils, a range of the friction angle for compacted poorly graded sand (SP) with gravel typically varies between 30° and 36°, depending on moisture content, grain size characteristics and density. A mean value of 33 degrees was assigned for the design purposes.

6.0 GEOTECHNICAL ASSESSMENT AND ANALYSES

6.1 Site Preparation

Prior to construction of the berms (including reservoir, lagoon and landfill berm), the topsoil or vegetation, if any, should be removed from underneath of the proposed berm locations and the exposed native soil should be compacted with a vibratory roller to the satisfaction of a field engineer. Any soft material, observed in the prepared subgarde, should be removed and replaced with engineered fill, compacted to 95 percent of SPMDD.

However, AMEC does not recommend removal of the organic material from the sewage impoundment area. Permafrost disturbance within the impoundment area would promote thawing of frozen soils under lagoon.

6.2 Bearing Capacity and Settlement

Bearing capacity and settlement of frozen/thawed soils are important issues for the berm geotechnical design. The results of geothermal analyses (see Section 5.6) show that during the operation period of time, the berm will generally be founded on frozen medium to coarse sand and gravel with varying amounts of cobbles and boulders. In frozen conditions, such soils have compressive strength greater than 250 kPa. Minor thawing occurs only near the toe of the interior slope covered with water (see results of the geothermal analysis). The allowable bearing capacity of compacted unfrozen sand and gravel is generally higher than 175 kPa while the allowable bearing capacity of frozen sand and gravel, as was mentioned above, is not less than 250 KPa. Both values (175 kPa and 250 kPa) are greater than the vertical stress induced by the proposed berm. The long term settlement of the frozen sand under the proposed loads for reservoir/lagoon will be negligible.

6.3 Thaw Settlement

A review of AMEC project files indicated that the thickness of the overburden soil (sand & gravel) in the Qikiqtarjuaq community is about 10 m to 15 m. Moisture content of predominantly sandy deposits ranges from 8 percent to 26 percent (average moisture content is about 15%). Such amount of water in soil is not enough to fill up pores (porosity 0.3) with ice, resulting in a minimum settlement of thawing soil (sand). It is estimated, based on our experience that the settlement strain of the overburden can be in an order of 0.01. It means that if thaw depth under the reservoir impoundment is greater than the overburden thickness (15 m), then the thaw settlement will be in an order of 15 cm maximum. The thaw settlement of the berm upstream slope would be considerably less (about 4 cm to 5 cm, see results of geothermal analysis).

6.4 Stability of Berm Slope (without seepage)

In order to determine the stability of the berm slopes for the proposed facilities, the most critical section is located at the downstream slope of the lagoon (Section B-B in design drawing No. 8). The upstream slope in this section is 3H:1V while the downstream slope is 2H:1V with a total slope height of approximately 8 m. The slope stability was assessed for the downstream slope of the lagoon berm because this slope is highest and steepest.

Three failure mechanisms were assumed in stability analyses:

<u>Case 1:</u> A circular failure within the berm material during first several months of the operation when the berm is still in an unfrozen state.

<u>Case 2:</u> A planar failure of the active layer on the exterior slope of the berm with a frozen core during summer periods of the berm operation.

Case 3: Berm stability against sliding along the interface of the berm and the native material.

Case 1

This case considers the berm slope under construction or shortly after construction. The strength of the berm fill after compaction governs the stability of berm slope. The pore water pressure within soil particles will reduce the effective overburden stress of soil and hence decrease the shear strength of soil at the failure slip. Considering that the materials for berm construction will be coarse granular soils, the pore water pressure within soil particles may be assumed to be zero.

Commercial computer software SLOPEW (Geostudio 2004) was applied for assessment of the slope stability. Due to a possible range of parameters within the berm fill, a series of sensitivity analyses was performed, applying various strength parameters of the fill.

The potential failure slip with the low factor of safety was firstly determined by varying with radius of the circle and location of the failure axis associated with the mean values of soil parameters. The friction angle and unit weight were then held in the mean values, and safety factors were computed by varying the cohesion. Such procedure was used for assessment of the model sensitivity to the friction angle and unit weight while keeping other parameters in the mean values. The estimated mean values and results of the analyses are summarized in Table 1.

Table 1 Input Parameters of Berm Fill and Results of Sensitivity Analyses

Input Parameters	Mean value	Range	Factor of Safety
Apparent Cohesion (kPa)	5	0 ~ 10	1.5 ~ 2.1
Friction angle°	33°	30° ~ 36°	1.65 ~ 2.0
Unit Weight (kN/m^3)	20	19 ~ 21	1.8 ~ 1.85

Figure 1 presents the sensitivity plots, where the strength and unit weight values are normalized within a value range between 0 and 1. Zero means the lowest value in the range and 1 means the highest value. The point where the three sensitivity lines cross, is the deterministic factor of safety, corresponding to mean values of the analysed soil. Figure 1 indicates that the safety factor against slope failure, is higher than 1.5 for any combination of soil parameters. The safety factor of 1.5 is generally considered to be acceptable in geotechnical practice. Figure 2 illustrates the factor of safety for the 2H:1V slope and the potential failure slip when all input parameters correspond to the mean values.

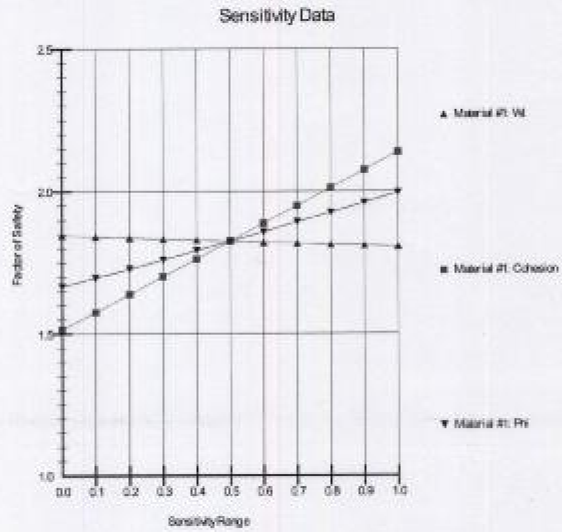

Figure 1 Sensitivity Plots for Slope Stability (No Pore Water Pressure)

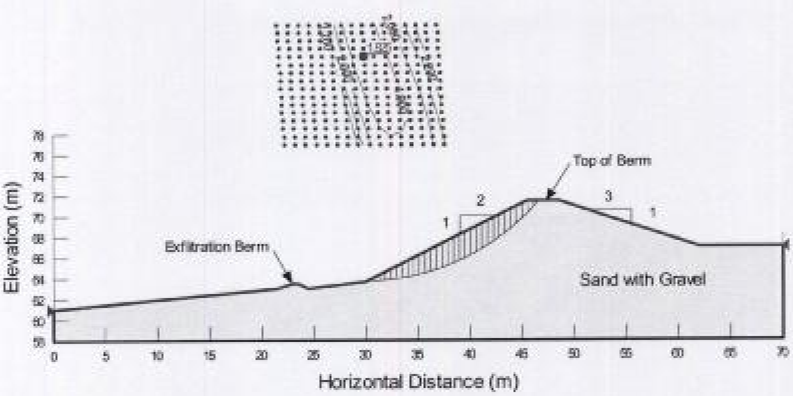

Figure 2 Factor of Safety for Lagoon Berm (Mean Value of Soil Parameters, No Pore Water Pressure)

Case 2

The stability of thawed material overlying an exterior frozen berm slope during summer periods was assessed. The thaw depth on the berm slope was obtained based on results of the geothermal analysis. The maximum thickness of the active layer, as predicted by geothermal analysis, can reach 1.5 m at the end of summer.

The factor of safety of the slopes was calculated using the limit equilibrium theory by comparing the total resistance force along a failure slip to the total driving force along the same failure slip. The limit equilibrium theory, applicable to frozen slopes was developed by Dr. E. McRoberts. In addition to the base resistance along the slip, the side shear resistance of thaw materials also contributes to the total resistance, hence increasing stability of a slope. Resistances along both sides of failure zone were considered, applying a side factor of 0.7.

The analyses were carried out for an assumed failure slope 15 m long (based on berm section) and 20 m wide. Similar to case 1, sensitivity analyses were carried out using the same mean values and ranges of the soil parameters outlined in Table 1. The results of analyses for a thaw depth of 1 m and 1.5 m are presented in Figure 3. The factors of safety against failure within the active layer are greater than unity which suggests that the proposed slopes are in a stable condition. The factor of safety may decrease to about 1.3 when the apparent cohesion of active material is close to zero. It is our opinion that coarse material generally tends to dilate during shearing under a low confined pressure. It is reasonable to include a minimum apparent cohesion for dilating soil. Hence, the likelihood of zero cohesion is very low.

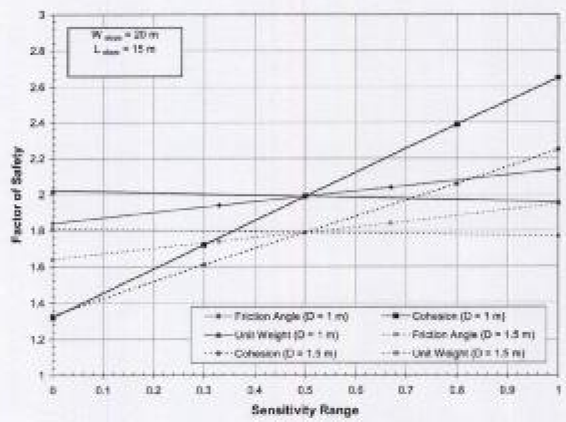

Figure 3 Sensitivity Plots for Stability of Active Zone

Figure 3 indicates also that the factor of safety of the active layer generally decreases by 10 to 15 percent with increasing of the active layer thickness from 1 m to 1.5 m. It was also found that the factor of safety decreases by 3 percent to 4 percent with increasing of the slope width from 20 m to 50 m.

Case 3

Stability of the berm against sliding due to the hydrostatic pressure applied on the interior slope was assessed by assuming that the berm is a rigid retaining structure. The friction coefficient of 0.5 was used at the interface between the berm material and the native ground, as recommended in Canadian Foundation Engineering Manual (1992, 3rd). The lateral pressure on the interior slope was estimated based on fluid pressure with a unit weight of 10 kN/m^3. It was found that the factor of safety against sliding along the interface is close to 4.

It should be noted that the above limit equilibrium analysis takes into account forces and moments equilibrium only. The deformation of the slope will undergo during the life service of the structures and re-distribution of internal forces due to berm deformation are not considered. Therefore, the results obtained from limit equilibrium analyses are generally considered to be slightly conservative.

6.5 Seepage Analyses

Seepage analyses for the proposed lagoon berm were carried out using the SEEP/W commercial finite element computer software. The purpose of the analyses was to assess the safety of downstream slope against piping shortly after construction if liner is not provided on the upstream slope, while frozen core has not formed yet in the berm material (within one year after completion of berm construction) and the water level on upstream slope was raised to the maximum elevation. In this case, water will infiltrate through berm materials to the downstream slope face. The critical section (west side of the proposed lagoon) used in slope stability analysis was employed for seepage analyses. The width of berm at the crest was 3 m. The maximum water level at the upstream slope was assumed to be at El. 71 m. Berm materials and native materials were assumed to have similar hydraulic properties.

The saturated hydraulic conductivity (K_{sat}) of in-situ materials were referenced to works performed by D. Swanson (1991) on poorly graded sand and the hydraulic conductivity functions were estimated using method proposed by Green and Corey (1971). Since coarse material was encountered on site, the hydraulic conductivity was increased (ie: made less conservative or more porous) by 50 %. It is understood that the K_{sat} has insignificant impact to the results of seepage gradient. AMEC considers that this parameter results in an adequate assessment of the seepage. Figure 4 presents the contour plot of total pressure head for the proposed berm under a steady flow condition. Figure 5 shows the contour plot of x-y gradient. It was found that the maximum gradient near the toe of downstream slope varies between 0.35 and 0.45.

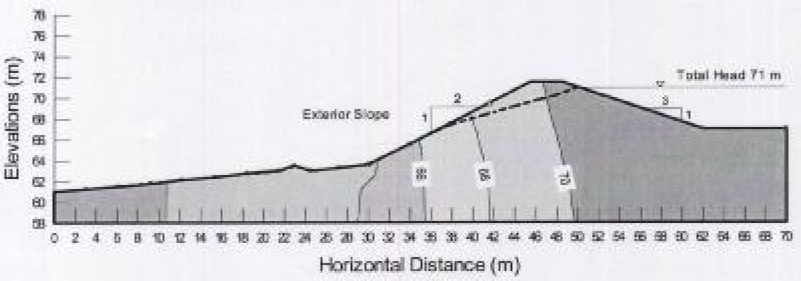

Figure 4 Total Pressure Head within Lagoon Berm due to Seepage

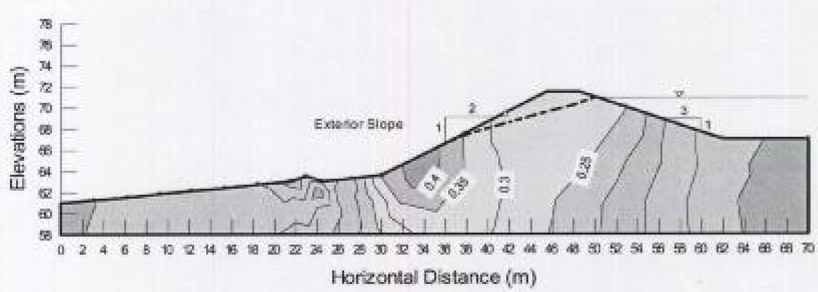

Figure 5 X-Y Gradient within Lagoon Berm due to Seepage

The factor of safety against piping 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. The i_{cr} may be evaluated using void ratio (*e*) and specific gravity (G_s) of soil and generally varies within a range of about 0.85 and 1.1 (Das, 1983). The factor of safety against piping was calculated to be approximately 2 which is acceptable for short term period prior to the berm material being frozen.

Seepage analysis for the lagoon berm has also confirmed that there is a potential of seepage through the berm during the first year of the berm operation, assuming that the lagoon is filled to the top immediately after construction. Since the lagoon will be placed into service at the end of August, seepage is not expected during the first winter following to completion of the berm construction. It was further understood, that during each summer, the sewage water is discharged from the lagoon beginning around July 1. If water level in the lagoon is lower than the annual thaw depth, then a liner is not required to reduce seepage of fluid through the upper portion of the berm. Recommended maximum fluid levels that are required to maintain this condition are provided below.

- May – any water level.
- June - 0.5 m below berm crest.
- July – 1.2 m below berm crest.
- August – 1.7 m below berm crest.
- September – 2 m below berm crest.

If water levels will be above the maximum levels, then a liner should be installed to prevent the seepage. These levels can be adjusted depending on actual thaw depths observed on an annual basis.

6.6 Stability of Berm (with seepage)

As mentioned before, the pore water pressure within soil particles will reduce the effective overburden stress of soil, and hence decrease the shear strength of soil at the failure slip. The stability of berm slope was also assessed by considering the pore water pressure within the berm due to seepage. The results of the analyses, using mean value of soil parameters and the sensitivity plots are given in Figure 6 to Figure 7, respectively. The results indicate that the stability of the berm slope is more sensitive to the cohesion of the compacted soil than other parameters. If the lower bound of this parameter was assumed, the factor of safety may decrease to less than unity. However, it is our experience that a mean value of cohesion shown in Table 1 should be warranted if the granular material for berm fill is compacted to the specified density. Monitoring compaction during construction is required to ensure that the specified density is attained. Alternatively, if full time monitoring is not to be provided, it is recommended that a liner be installed on the inside slope of the downstream berm for the lagoon.

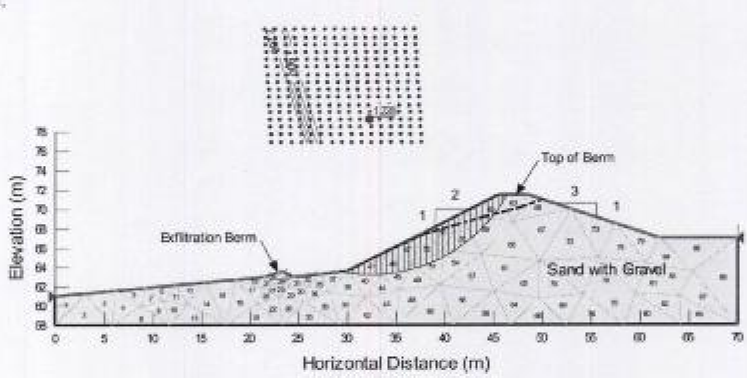

Figure 6 Factor of Safety for Lagoon Berm (Mean Value of Soil Parameters)

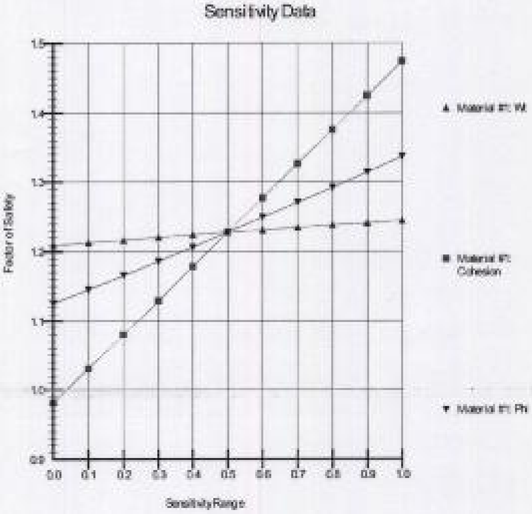

Figure 7 Sensitivity Plots for Factor of Safety (With Seepage)

6.7 Geothermal Analyses

The geothermal modeling program SIMPTEMP, 2D version (developed in-house by AMEC) was used to analyze the geothermal regimes for the lagoon berm. The geothermal simulator uses the finite element method to compute a numerical solution of the heat transfer problem. Physical/mathematical algorithms used in the SIMPTEMP model have been published, and the simulation process has been verified both against well-known analytical solutions of the heat transfer problem, and as compared with numerical solutions produced by other commercial/non-commercial geothermal software. AMEC has successfully used the SIMPTEMP program for a variety of geothermal applications over a ten years period.

The analysis considered the following geometry for lagoon:

- Height of dyke is 5 m (upstream).
- Width of crest is 3 m.
- Upstream slope is at 3H:1V and downstream slope is at 2H:1V
- Local soil (sand and gravel) is proposed for the dyke core construction.

Table 2 below provides surface temperatures that were applied at the berm, downstream terrain beyond the berm, ice surface and also reservoir water temperatures in summertime.

Table 2 Surface Temperatures and water temperatures Applied in Geothermal Model

Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Berm	-27.6	-27.6	-24.5	-11.9	-7.1	0.8	6.4	6.2	2.3	-4.8	-11.1	-17.6
Downstream Terrain	-21.5	-21.5	-19.1	-9.2	-5.5	0.8	6.4	6.2	2.3	-3.7	-8.6	-13.6
Ice Surface	-21.5	-21.5	-19.1	-9.2	-5.5	---	---	---	---	-3.7	-8.6	-13.6
Water	---	---	---	---	---	0.6	4.9	4.8	1.8	---	---	---

The provided temperatures were derived by application of various n-factors to the mean monthly air temperatures at Qikiqtarjuaq weather station for period from 1971 to 2000.

The initial temperature of the berm material and active layer was taken to be 2 °C, while the frozen soil below the active layer was appointed to be -5 °C. The soil profile consisted of 15 m thick overburden (sand and gravel at moisture content of 12%), overlying bedrock (moisture content of bedrock 2%). It was assumed in the analysis that the berm material properties are the same as properties of the overburden. The water level in the reservoir was at the maximum elevation, beginning from October 1. The model ran for 30 years.

Figure 8 shows that after the first year of the berm operation, the majority of the berm core has a temperature in a range above 0 °C. Also due to the warming effect of the reservoir water a portion of the berm at the upstream slope has a positive temperature about 1 °C to 2 °C. Figure 9 to Figure 12 present the estimated temperatures from the fifth to thirtieth year of the berm operation.

The numerical simulation suggests that a frozen core within lagoon will be located at 2.0 m to 2.5 m below the crest of berm after 1 year of operation. The potential of percolation of water/effluent through the berm will depend on the water level inside the lagoon during operation. If water levels will be higher than the maximum levels (see Section 5.5), then a liner will be required. If the operation scenario with high water level (higher than maximum) will be used, AMEC recommends installation of the geomembrane liner in the lagoon berm for seepage protection. A typical section for the liner layout is provided in Figure 13. The cut –off trench, at least 2 m deep into the native soil, should be excavated at the position of the upstream slope crest. The liner should be placed vertically in the cut-off trench and then backfilled with compacted clayey material or grouted. The liner should then follow the ground surface to the toe of the upstream slope and then cover the upstream slope to elevation higher than the expected maximum water level, as shown in Figure 13. The liner should be covered with 0.5 m thick armour layer (riprap). An alternative liner option is shown in Figure 14. It is understood that the constructability of the alternative option is more complex however the amount of required liner is almost half as much as the option shown in Figure 13.

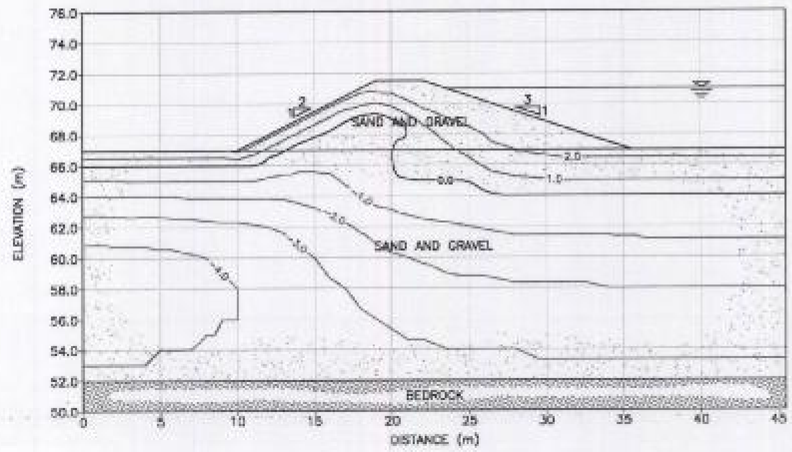

Figure 8 Berm Temperatures after 1 year of Operation

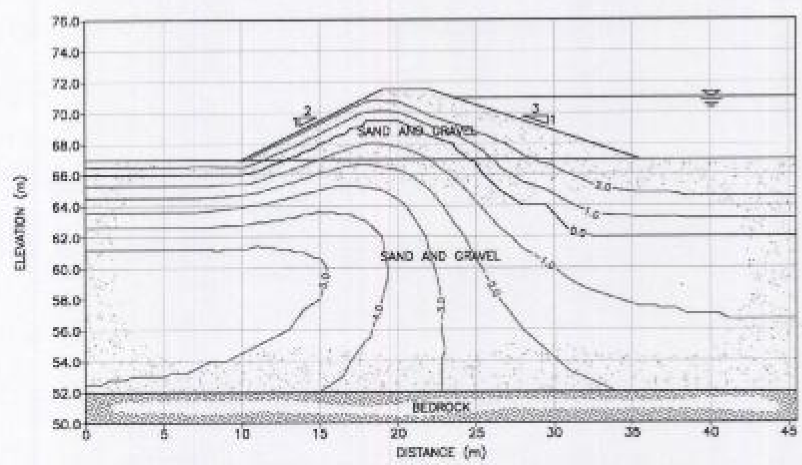

Figure 9 Berm Temperatures after 5 years of Operation

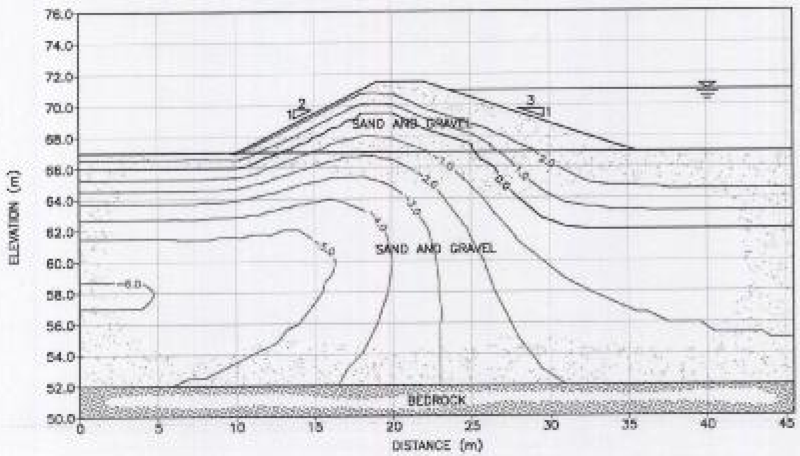

Figure 10 Berm Temperatures after 10 years of Operation

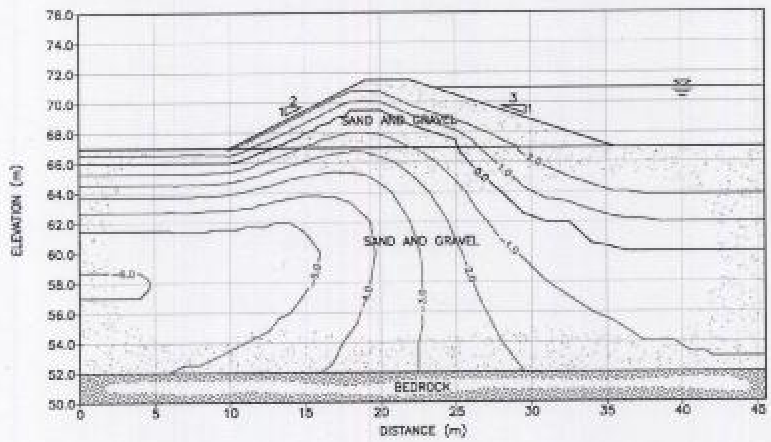

Figure 11 Berm Temperatures after 20 years of Operation

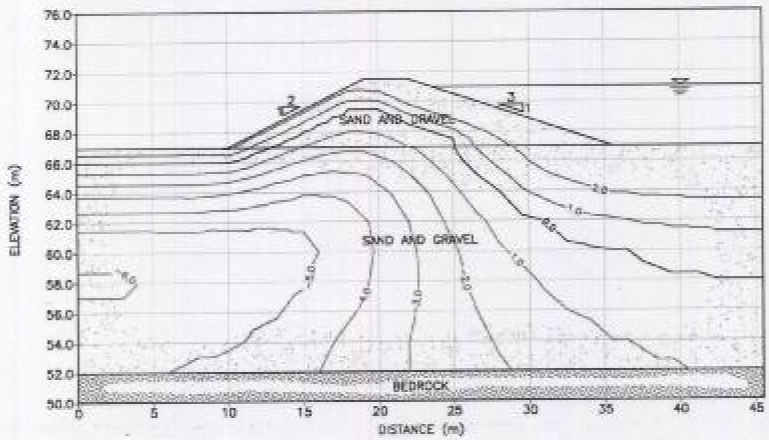

Figure 12 Berm Temperatures after 30 years of Operation

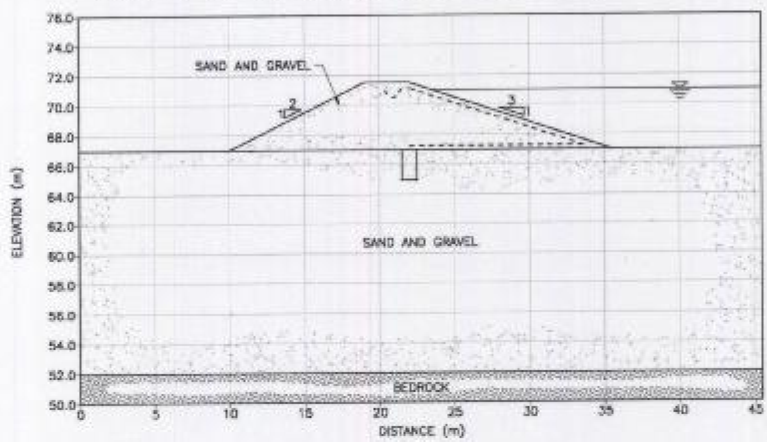

Figure 13 Proposed layout of HDPE Liner (Option 1)

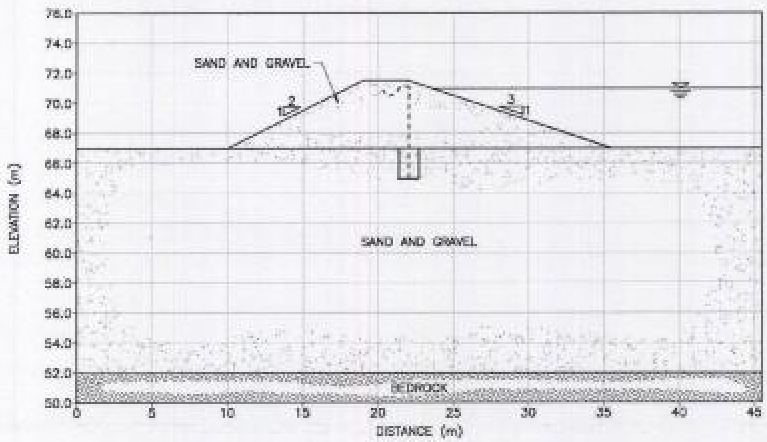

Figure 14 Proposed layout of HDPE Liner (Option 2)

No geothermal analysis was carried out on landfill section where the height of berm fill is approximately 2 m. Based on results of the geothermal analysis for the lagoon berm, it is our opinion that the majority of the berm will be unfrozen at the end of summer and seepage can occur through the berm if considerable amount of precipitation will be accumulated in the waste material. A hydrological balance should be calculated for the landfill area. If amount of precipitation (rain and snow) is less than evaporation potential, then it can be concluded that no significant amount of water will remain within the landfill area at the end of summer and a liner will not be required for the landfill berm from seepage concern. However, the final decision should be made with consideration of environmental concern. If the evaporation potential is less than precipitation, then a geomembrane liner is recommended for seepage control. Details of the liner layout may refer to Figure 13 and Figure 14.

Since no measurement of ground temperature was performed during the site investigation, the permafrost temperature (initial soil temperature) in the foregoing geothermal model was predicted using a 1D geothermal analysis. The analysis includes parameters which determine the soil temperature: soil thermal conductivity, heat capacity and latent heat, snow cover, snow density and air/water temperature. The results suggest that the mean annual permafrost temperature at the site can be in a range from -4 °C to -6 °C (mainly depending on thickness of snow cover) and thickness of the active layer can be in a range from 0.5 m (mossy ground vegetation) to about 1.5 m (bared ground surface).

It is understood that the initial soil temperature is an important parameter to predict the berm and impoundment temperature during the first years of the berm operation. However, after that time, the temperature conditions of the impoundment and berm are changed dramatically due to a warming effect of water in the impoundment and the berm configuration. The geothermal analysis has shown that it takes more than 30 years when soil temperatures within the impoundment and berm will be stabilized. At that time, the soil temperatures will differ considerably of the initial soil temperatures.

Moreover, AMEC experience in geothermal modeling shows that variations of soil thermal properties within a reasonable range of values, provide insignificant changes to long-term soil temperature. The boundary conditions have a greater impact on the berm temperature throughout the years of the berm/lagoon operation. Thus, AMEC considers that the estimated soil thermal properties and applied boundary conditions, used in the foregoing analysis, resulted in an adequate assessment of the berm temperature regime for long term consideration.

Potential of climate warming impact was not incorporated in the geothermal model. AMEC was involved in prediction of climate warming rate for several other projects, including Diavik Diamond Mine project. Our experience shows, that within range of the predicted warming rate, no significant change of the berm temperature occurs during the 5 to 10 years immediately after construction. There is a potential after this period of time that implementation of the contingency plan might be required for the berm, based on results of monitoring program.

AMEC recommends implementation of the temperature monitoring program within the berm as described in Section 6 below. If actual berm/ground temperatures differ considerably (i.e., are warmer than predicted) or warming trends are noted, then a contingency plan should be implemented. Currently, it is considered that an appropriate contingency plan would include installation of thermosyphons to reduce berm temperatures.

AMEC does not expect a noticeable change of the thawing depth under the lagoon impoundment due to climate warming because it mainly occurs by increasing of winter air temperatures. However, the thawing depth under the impoundment depends mainly on the water temperature which is a function of the summer air temperature.

6.8 Aggregate Assessment

Characterization and analysis of materials available on site and suitable for berm construction was carried out for three borrow sources, located near the existing sewage lagoon, proposed waste water lagoon and proposed water reservoir. Three samples were derived from test pits, excavated at the borrow sources sites. Methodology used for analysis of the granular materials from the test pits is provided below.

Sieve analyses were performed on selected samples by Alston Associates Inc. The results of all sieve analyses are presented in Appendix C – soil testing results. The grain size distribution curves of these materials were then compared to upper and lower bound gradation envelopes for 25 mm minus gravel material typically used for foundation subgrade, embankment and road construction. The results of comparisons are presented in Figure 15. The solid lines, shown in Figure 15 demonstrate the upper and lower bound of gradation envelopes for the specified gravel material, and the dashed lines represent the gradations of the granular materials obtained in the test pits.

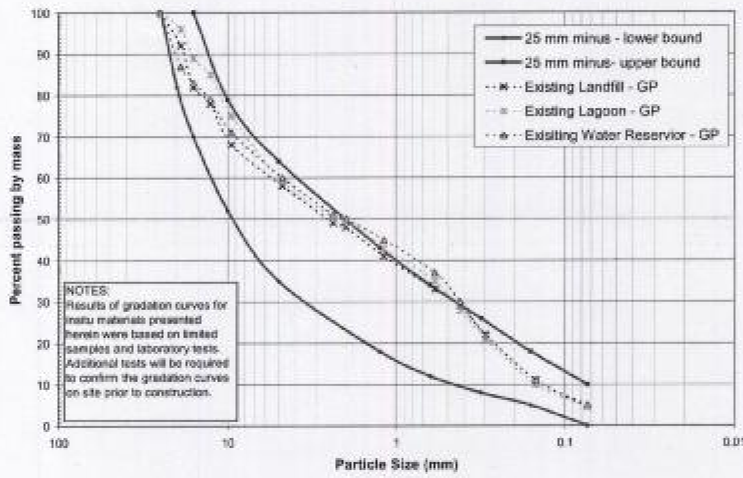

Figure 15 Comparison of Gradations of In-situ Material to 25 mm minus Crushed Gravel

It was found that the particle size distribution for the sampled materials practically fall within the specification envelope for 25 mm minus gravel. The minor discrepancy of the grain size distribution within the particle size from 0.4 mm to 2 mm should not have a noticeable impact on performance of these materials for berm construction if the gravel is properly compacted. Therefore, materials obtained from these sites are considered to be acceptable as a 25 mm minus gravel resource.

The results and conclusions are based on limited laboratory tests. It is recommended that additional sampling and testing be performed to confirm that suitable gravel is available in quantities required to complete construction.

6.9 Drainage

All existing ditches, if any, should be covered with a minimum 200 mm thick riprap (Class 1M, nominal size 175 mm) and re-directed the drainage paths around the site rather than over it. Where necessary, the existing ditches will be re-graded to provide better hydraulic gradient.

Table 3 Rock Gradation of Riprap (Class 1M)

Class of Riprap	Rock Gradation			
	None greater than	20 % to 50 %	50 % to 80 %	100 % greater than
1M	300 mm	200 mm	175 mm	125 mm

The proposed riprap placed on existing ditches reduces velocity of water flow and hence reduces potential erosion effects. A geo-textile layer (Amoco 4506 non-woven or equivalent), placed on a ditch bottom prior placement of the riprap may also be considered to reduce potential erosion.

The access area to the bulky metals disposal site and capped disposal area showed signs of erosion due to runoff cutting channels across the site. These channels should be filled in as part of the ongoing work at the site. Future erosion should also be controlled by improving the drainage around the site so that this doesn't occur again in the future.

6.10 Erosion Protection

Based on field performance of the existing facilities over the last 17 years, there was no evidence of erosion caused by wave, ice or wind on the existing slopes. However, the upstream slopes should be provided with adequate protection to guard against erosion and possible breaching due to wave and ice action, as outlined in Dam Safety Guidelines prepared by Canadian Dam Association (1999). It is recommended that a layer of 500 mm thick riprap (class 1) be placed on the upstream slope of water reservoir and lagoon for erosion protection.

7.0 CONSTRUCTION MONITORING

It is recommended that a field geotechnical engineer or technologist be assigned to be onsite during the entire time of the berm construction to provide proper quality control and quality assurance. His responsibility will include the following:

- Inspection of engineered fill, including such fill parameters as gradation, moisture content, unfrozen state, inclusions of boulders;
- Coordination with the contractor requirements of the inspection during various stages of the berm construction;
- Estimation of lift thickness corresponding to capacity of available compaction equipment;
- Inspection of compaction level with the use of sand cone method, proof rolling or equivalent for each lift;

- Record all activities on the site and direct these activities if they contradict to the earth work specifications;
- Review of design drawing and specifications prior the construction is commenced;
- Provide recommendations if unforeseen site conditions will be encountered, including specify drainage conditions and permafrost conditions.

7.1 Fill Placement and Compaction

The percent of Standard Proctor Maximum Dry Density (SPMDD) is defined as the ratio between the field dry density of the compacted material and the laboratory maximum dry density compacted with standarized percedures at a known energy level. In general, the material directly below a structural foundation requires neglected deformation due to compaction of soil particles after concentrated load is applied. Therefore, 100 percent of the SPMDD is generally specified. However, minor deformation on a slope, such as berm slope for lagoon and reservoir, is considered to be acceptable and practical. Therefore, 95 percent of SPMDD is recommended in this design.

All material used for berm construction should be unfrozen at the time of placement and placed in lift thickness compatible with the compaction equipment being used, to a maximum thickness of 300 mm and uniformly compacted to specified density at a moisture contents within three percent of optimum. Any cobbles and boulders within berm fill should be removed. The Proctor tests should be carried out on stockpiled materials prior to construction. The achieved density of berm materials should be assessed on a regular basis. It is recommended that at least three tests be carried out for each lift. Additional tests may be performed where necessary, as determined on site by the field engineer. Sand cone method or equivalent method may be used together with proof rolling of the compacted lifts by a heavy loaded truck.

7.2 Post Construction Monitoring

It is also recommended that the following programs be implemented following completion of construction:

- Thermal monitoring: Thermistors should be installed to monitor ground temperatures. Two thermistors strings are recommended to be installed in drill holes through the crest of each berm structure, extending a minimum of 5 m into the native materials. The temperature readings should be taken twice per year for the first 5 years of operation, after which the monitoring frequency should be reviewed.

- Movement monitoring: Survey monuments should be installed along the interior and exterior crest of the berm. Design and depth of the monuments should provide that they are not subjected to frost heave forces. The monuments should be surveyed for vertical and horizontal movements twice during the first year of operation, after which the monitoring frequency should be reviewed.

- Seepage monitoring: If any seepage is detected downstream of the berm, remediation program should be developed. Water quality samples should also be taken weekly and analysed for critical constituents.

All field observations, recommendations and monitoring data including field testing results obtained in conjunction with any approved monitoring programs would be documented and submitted to the NWB.

8.0 CLOSURE

The geotechnical analyses and recommendations presented herein are based on data provided to AMEC by Nuna Burnside Engineering and Environmental Ltd., review of the published reports and AMEC design experience for similar structures in permafrost areas. AMEC did not undertake any geotechnical investigations at the proposed reservoir, lagoon and landfill sites. If additional geotechnical/permafrost investigations will be carried out and encountered soil conditions appear to be considerably different than those assumed in the present report, AMEC should be advised immediately and the recommendations contained herein should be revised, if necessary.

This report has been prepared for the exclusive use of Nuna Burnside Engineering and Environmental Ltd. and its agents for the specific application described in this report. Any use that a third party makes of this report, or any reliance or decisions based on this report are the sole responsibility of those parties. It has been prepared in accordance with generally accepted permafrost and foundation engineering practices. No other warranty, expressed or implied, is made.

Respectfully submitted,

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Date January 31, 2007
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The Association of Professional Engineers, Geologists and Geophysicists of the NWT / NU

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