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1.0 INTRODUCTION

AMEC Earth & Environmental, a division of AMEC Americas Limited (AMEC) has been retained by Nuna Burnside Engineering and Environmental Ltd. (NBEEL) to carry out the assessment of design soil parameters and undertake various engineering analyses (seepage, slope and geothermal) for a new sewage lagoon and solid waste disposal facility in the Kugluktuk community, Nunavut. The lagoon berm is proposed to be 3 m to 4 m high with side slopes of 3H:1V and 3 m wide crest. The solid waste disposal facility consists of the landfill perimeter berm, hazardous waste storage area, land farming area and bulky material burial pit. An initial geotechnical investigation, comprised of test pitting, was conducted by Alston Associates Inc. (May 2006) in the vicinity of the proposed lagoon and borrow source areas. The current phase of the geotechnical work is for support of the Nunavut Water Board License Application. Authorization to proceed with the current geotechnical analyses was received on December 7, 2006 via fax from Mr. G. Popowich, project manager with NBEEL.

The berm design drawings and geotechnical report of the NBEEL for the lagoon and solid waste disposal facility were provided to AMEC (Appendix D - geotechnical evaluation prepared by Alston Associates Inc.; test pit logs; results of laboratory tests and design drawings 1, 4 & G1).

2.0 SCOPE OF WORK

Based on a verbal discussion with NBEEL, the scope of work for geotechnical services should comprise 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:

- Kugluktuk Water System Modifications Water Intake Design River Engineering Study. AGRA Earth & Environmental Limited, Yellowknife, NWT (1997)
- Geotechnical Investigation Proposed Elementary School, Coppermine, Northwest Territories. HBT AGRA Limited (1993)
- Coppermine Sewage Lagoon Geotechnical Evaluation. Thurber Consultants Ltd. (1985)
- Coppermine Solid Waste Disposal Geotechnical Evaluation. Thurber Consultants Ltd. (1985a)
- Airphoto Interpretation Terrain Analysis Coppermine NWT. Thurber Consultants Ltd. (1979)

- Coppermine Water System Modifications Final Planning Report, Stanley Associates Engineering Ltd. (1995)

3.0 SUBSOIL CONDITIONS

Based on the provided test pit logs, the in-situ surficial material encountered at the proposed lagoon consists of up to 0.25 m thick black and damp organic material which is generally underlain by loose or loose to compact, light brown to brown sand with trace of silt. Permafrost was found to be located at 0.8 m to 1 m depth within impoundment area and north side of the lagoon.

On the south side of the proposed lagoon, occasional gravel or cobbles were observed within the sand layer. The permafrost in this area was identified to vary between 0.8 m to 1.8 m. Bedrock was encountered at shallow depth in some locations.

The results of grain size distribution in this area suggested that the in-situ sand is medium grained, poorly graded sand (SP) according to Unified Soil Classification System. The results of Standard Proctor tests for this material indicated that the maximum dry unit weight is approximately 16.2 kN/m^3 with optimum moisture content ranging between 9.5 % to 10.5 %.

Test pits were also carried out in the proposed borrow sources including northeast of Heart Lake, east of Heart Lake, west of road at end of runway and existing borrow pit. The surficial material in the northeast of the Heart Lake consisted of about 0.2 m peat/muskeg overlying dark brown, firm to stiff, medium to high plastic clay. The subsoil conditions for the rest areas generally consists of 0.2 m peat/muskeg overlying dark brown, fine grained clean sand or sand with trace silt. Permafrost was generally found to be at 0.8 m to 0.9 m depth. The exception was TP 625 (end of runway) where the clean sand, extending to 2.4 m depth, was underlain by stiff, high plastic clay to a 3.6 m depth below the ground surface. Permafrost at this location was not observed during site investigation within the test pit depth. The locations of the test pits within the borrow area were not provided.

4.0 SOIL PARAMETERS

Based on the correlation of dry unit weight with internal friction angle published in the Design Manual, NAVFAC DM 7.01, a range of the friction angle for compacted poorly graded sand (SP) may vary between 30° and 35°, depending on the sand moisture content and compaction energy on site. Due to the limit of test data completed at the site, the lower bound of 30 degrees is recommended for the design purposes.

The direct shear test was commonly used to estimate the friction angle of granular materials. However, the correlation between the internal friction angle and soil dry unit density have been investigated for many years and is well known. Therefore, the use of the published recommendation for the internal friction angle can be considered appropriate for the given project. Our experience with similar soils also allows to conclude that the use of the published values of the internal friction angle is appropriate, especially taking into account that the internal friction angle of 30 degrees is the low bound value for the compacted material.

In-situ soil within active layer is typically loose due to a disintegrating effect of freezing/thawing cycles. The typical friction angle (29 degrees) for the native loose granular material is recommended based on values provided in Bowles (1996). A typical range of the unit weight (14 kN/m^3 to 18 kN/m^3) may be considered in geotechnical analyses, representing loose granular material of various moisture content.

5.0 GEOTECHNICAL ASSESSMENT AND ANALYSES

5.1 Site Preparation

Prior to construction of the berms (including lagoon and landfill berm), the existing peat or organic mat, if any, should be removed from underneath of the proposed berm locations and the exposed native soil should be compacted with a minimum of 10 tons 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 intensify thawing of frozen soils under lagoon.

5.2 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 sand. In frozen conditions, such soils have compressive strength greater than 200 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 is generally higher than 125 kPa while the allowable bearing capacity of frozen sand, as was mentioned above, is not less than 200 KPa. Both values (125 kPa and 200 kPa) are greater than the vertical stress induced by the proposed berm. The creep settlement of the frozen sand under a minimal load will be negligible.

5.3 Thaw Settlement

A review of geological information for the existing Coppermine sewage lagoon and solid waste disposal (Thurber Consultants Ltd. 1985) indicated that the geological profile at the site likely consists of sand, approximately 2 m thick, over clay till over bedrock. Boreholes, drilled within the community, revealed that the thickness of overburden would be in a range of 7 m to 15 m. Moisture content of sand deposits ranged from 10 percent to 19 percent. 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 sand will be in an order of 0.01 and total settlement of the 2 m thick sand layer would be about 2 cm.

The settlement strain in the clay till is slightly higher. It was assessed, based on published data, that moisture content of the clay till would be in a range from 20 percent to 25 percent, corresponding to the thaw settlement strain of about 0.02 to 0.04 (average 0.03). The geothermal analyses confirmed that the thaw depth under the interior slope of the berm in the clay till can reach about 4 m, corresponding to approximate thaw settlement 8 cm to 10 cm (usually thaw settlement is less than the thaw settlement strain, calculated or obtained in a laboratory). The settlement of the earth structure in a range 8 cm to 10 cm can be considered acceptable. However, a monitoring program should provide a regular topographic survey of the berm as outlined in Section 6. If the thaw settlement is greater than 15 cm, than some remediation measures should be taken, such as backfilling of settled areas.

5.4 Stability of Berm Slope (without seepage)

In order to determine the stability of the proposed berm slopes, the most critical section was considered for the analyses. The interior and exterior slope in this section are 3H:1V with the total slope height of approximately 4 m. The berm is supported by the unfrozen native granular material, underlain with frozen soil at 2 m below the original ground surface.

Three failure mechanisms were assumed in stability analyses:

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

Case 2: 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 that the berm slope is under construction or shortly after construction. The strength of the granular material after compaction governs the berm stability. The pore water pressure within soil particles will reduce the effective overburden stress of soil, and hence will decrease the shear strength of the soil at the failure slip. Assuming that granular material will be used for berm construction and short-term period of the unfrozen berm existence, the pore water pressure within the soil particles may be ignored. (Refer to XIII for stability of berm with consideration of pore water pressure.)

The limit equilibrium of the driving and resisting forces was predicted for assessment of the slope safety factor, using commercial computer software SLOPEW (Geostudio 2004). Due to various composition of the proposed berm fill, selection of the soil strength parameters is the major difficulty. For this reason, several sensitivity analyses were carried out with various soil strength parameters.

The potential failure slip with the low safety factor was initially determined, applying various radii of circle and various locations of the failure axis associated with the mean values of the soil strength parameters. The friction angle and the unit weight were fixed to their mean values, and the safety factors were computed by varying the cohesion. Similar calculations were performed by varying the friction angle or unit weight while other parameters were fixed to mean values. The estimated mean values, range of parameter values and results of slope stability 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
Compacted Sand	Apparent Cohesion (kPa)	5	2 to 8	1.95 to 2.35
	Friction angle °	30 °	28 ° to 33 °	2.05 to 2.2
	Unit Weight (kN/m^3)	18	17 to 19	2.15 to 2.1
Native Loose Sand	Apparent Cohesion (kPa)	0	0	----
	Friction angle °	29 °	28 ° to 30 °	2.1 to 2.15
	Unit Weight (kN/m^3)	16	14 to 18	2.1 to 2.15

Figure 1 illustrates the factor of safety for the 3H:1V slope and the potential failure slip for the mean values of the soil parameters. Figure 2 presents the sensitivity plot. The strength parameters and unit weight are normalized in a range from 0 to 1. The point where three sensitivity lines are crossing, characterizes the deterministic factor of safety at the mean values for the strength parameters and the soil unit weight. The results indicate that the factor of safety against the slope failure is higher than 1.5 that is acceptable in the geotechnical practice.

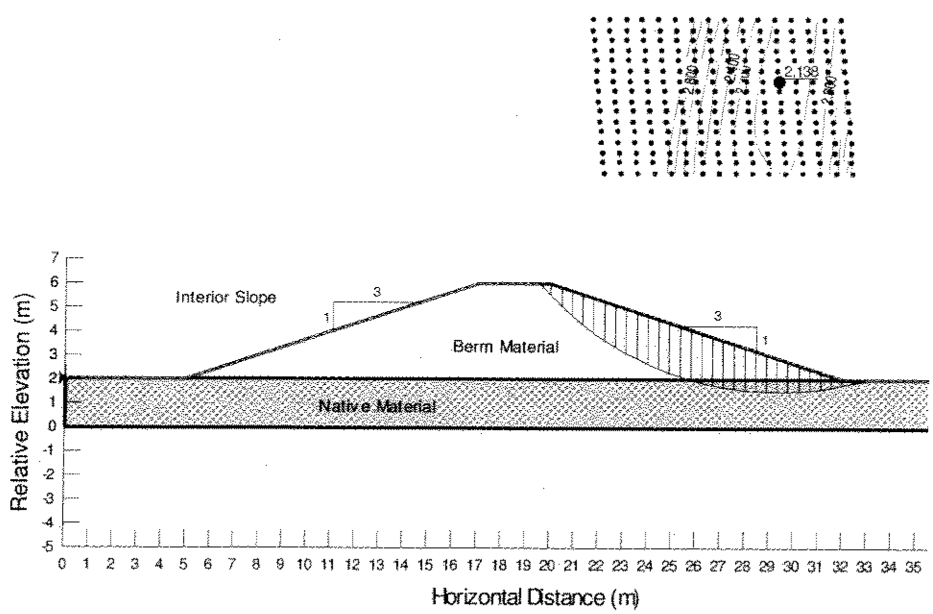


Figure 1 Factor of Safety for Lagoon Berm (Mean Value of Soil Parameters, No Pore Water Pressure)

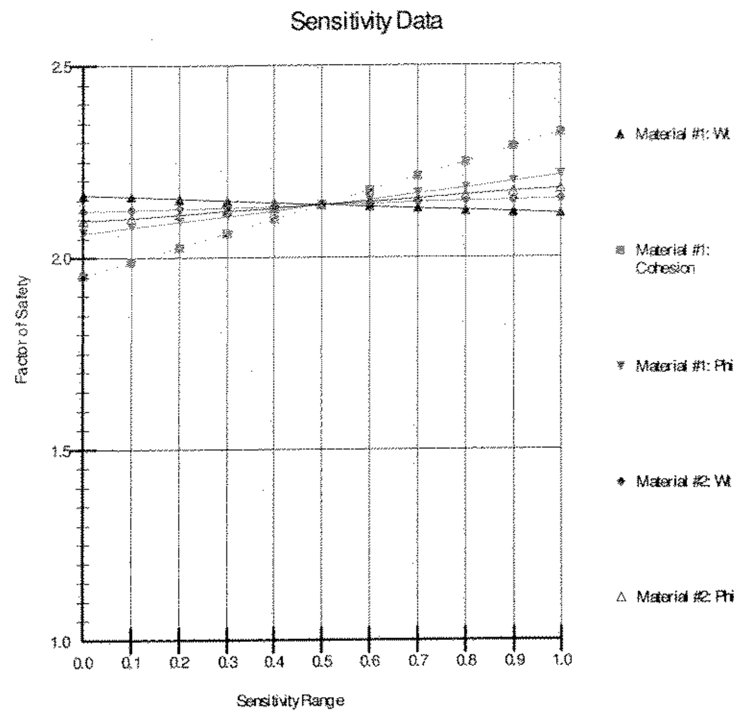


Figure 2 Sensitivity Plots for Slope Stability (No Pore Water Pressure)

Case 2

The active layer thickness on the berm slope was obtained based on results of the geothermal analysis. The maximum thickness of the active layer, as was predicted by geothermal analysis, can reach 2 m to 2.5 m at the end of summer.

The factor of safety was calculated using the limit equilibrium theory by comparing the total resistance force along the failure slip and the total driving force along the same failure slip. In addition to the resistance force along the slip, the side shear resistance of thaw material also was contributed to the total resistance, hence increasing stability of the slope.

The analyses were carried out for a 13 m long and 20 m wide berm slope. Similar to case 1, sensitivity analyses were carried out, using the same mean values and range of the soil parameters, outlined in Table 1. Results of the slope stability analyses for a thaw depth of 2 m and 2.5 m are presented in Figure 3. It can be seen at the figure that the safety factors against the slope failure within the active layer are well greater than the unity which suggests that the slopes will be in a stable condition.

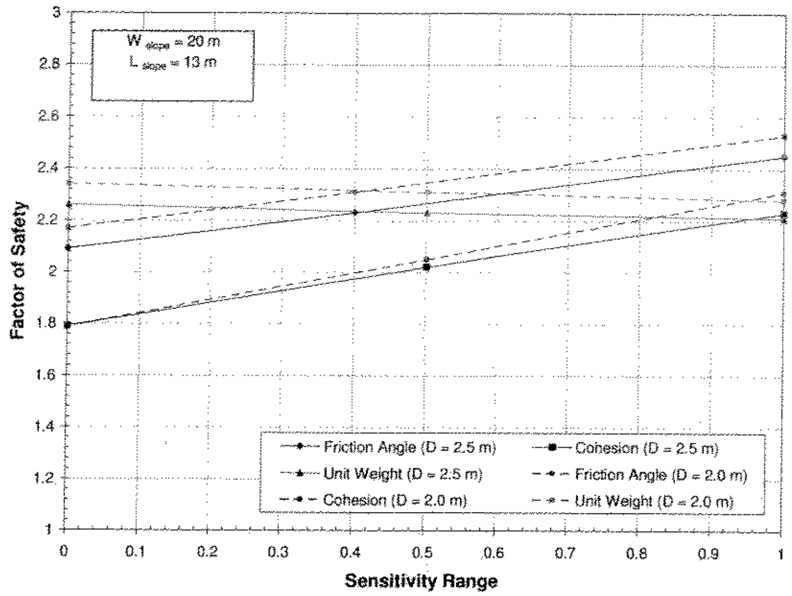


Figure 3 Sensitivity Plots for Stability of Active Zone

Figure 3 also indicates that the safety factor generally decreases by less than 5 percent with increasing of the active layer thickness from 2 m to 2.5 m. It was found also that the safety factor decreases by 3 percent to 5 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 0.45 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 greater than 4.

As mentioned before, the stability analyses were carried out in the most critical section. Hence, the discussed results may be applied to all other berm structures including landfill, hazardous waste and landfarm berms.

5.5 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 an assessment the exterior slope safety against the piping process shortly after completion of the berm construction, if no liner on the berm interior slope, a frozen core is not formed in the berm (one year after completion of berm construction) and the water level is raised instantaneously to the maximum design elevation. There is a concern that the water will infiltrate through the berm to exterior slope. The berm cross-section, used in the slope stability analysis, was also employed for the seepage analyses. The maximum water level at the berm interior slope was assumed to be 0.5 m below the berm crest. Two scenarios were analyzed:

- Berm supported by frozen native soil (impermeable layer).
- Berm supported by a 2 m active layer, overlying permafrost (impermeable layer).

The saturated hydraulic conductivity of the berm material (uniform sand) was assessed based on D. Swanson (1991) and the hydraulic conductivity functions for unsaturated conditions were estimated using the method, proposed by Green and Corey (1971). Figure 4 presents the total pressure head for Scenario 1 under steady flow conditions and Figure 5 shows the x-y hydraulic gradient within the proposed berm. It was found that the maximum hydraulic gradient near the toe of the exterior slope (i_{exit}) varies between 0.3 and 0.35.

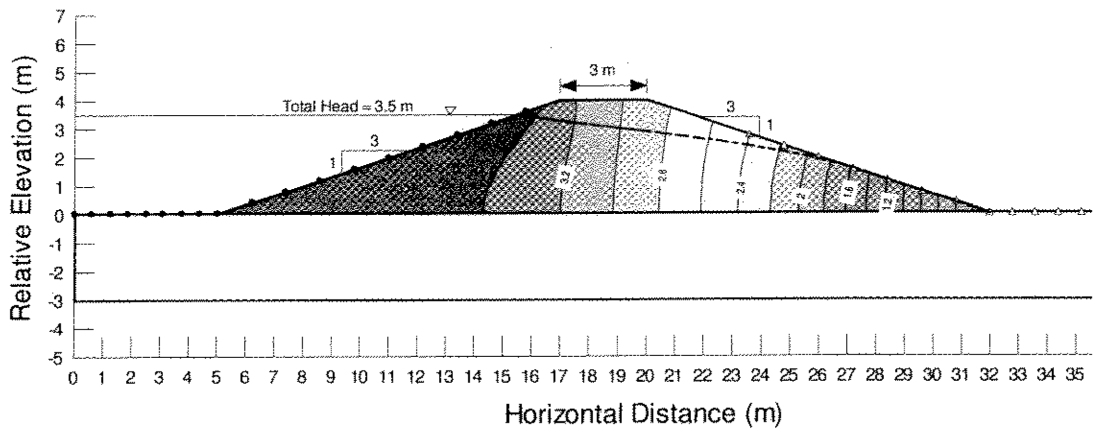


Figure 4 Total Pressure Head within Berm Fill due to Seepage (Scenario 1)

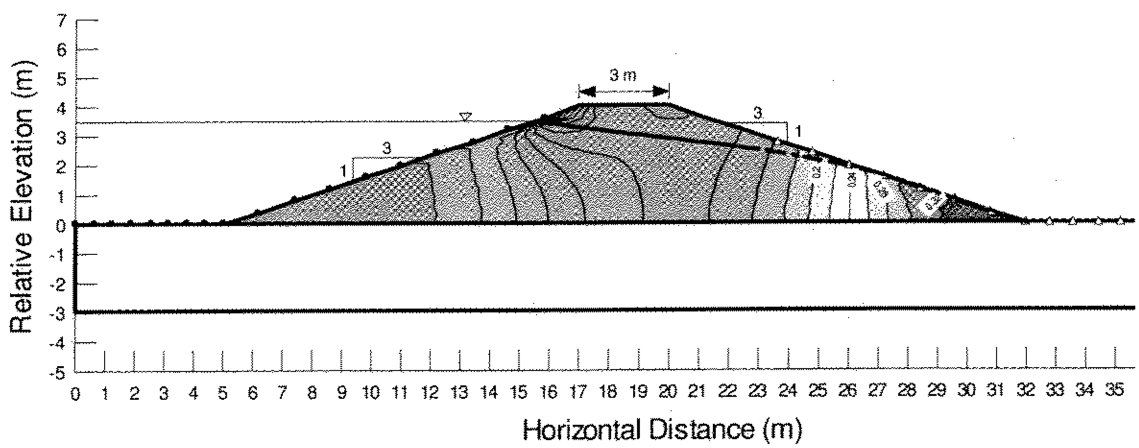


Figure 5 X-Y Gradient within Berm Fill due to Seepage (Scenario 1)

The safety factor against the piping can be defined as the ratio of i_{cr} to i_{exit} where i_{cr} is the critical hydraulic gradient of berm material. The i_{cr} may be evaluated using void ratio and specific gravity of soil and generally varies from about 0.85 to 1.1 (Das 1983). The safety factor against piping was calculated to be greater than 2 which is acceptable for a short term period prior to freezing of the berm material.

Figure 6 illustrates the estimated XY gradient for Scenario 2. It was found that a 2 m thick permeable layer has an insignificant impact to the XY gradient values.

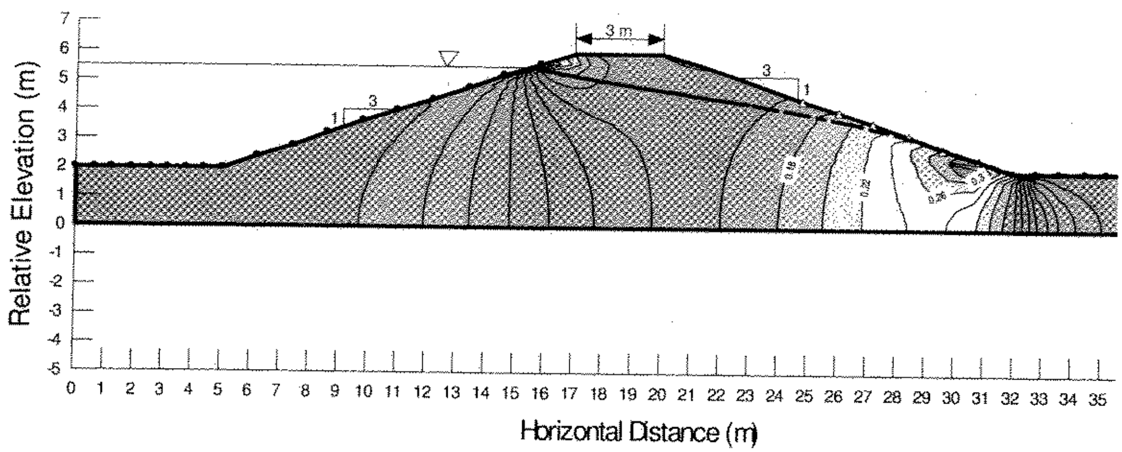


Figure 6 X-Y Gradient within Berm Fill due to Seepage (Scenario 2)

In order to assess a potential impact of soil hydraulic properties on the piping, the hydraulic conductivity function was changed by increasing the saturated hydraulic conductivity by 100 %. The estimated XY gradient is presented in Figure 7. Comparisons of Figure 7 and Figure 5 indicated that the stability against piping is not sensitive within a typical range of the soil hydraulic conductivity.

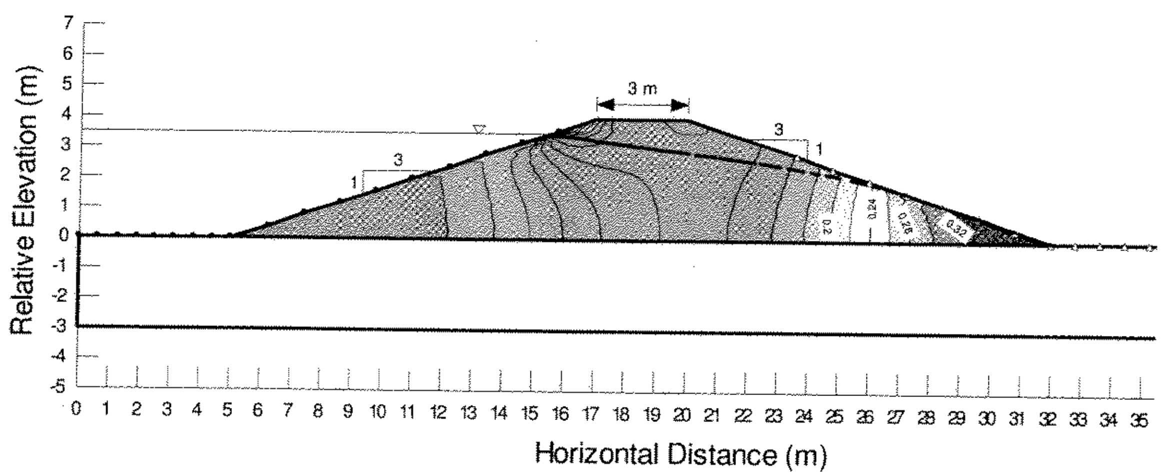


Figure 7 X-Y Gradient within Berm Fill due to Seepage (Case 1 & High Conductivity)

5.6 Stability of Berm (with seepage)

As mentioned earlier, the pore water pressure within soil particles will reduce the effective overburden stress of soil and decrease the shear strength of soil at the failure slip. The stability of the berm slope was re-assessed by incorporating the pore water pressure (see Figure 4) within the berm in the slope stability analysis. The results of the analyses, using the mean values of the soil parameters and the sensitivity plots for both scenarios are illustrated in Figure 8 through Figure 11. It was noted that Scenario 2 (2 m of the unfrozen sand below the berm) yields a lower factor of safety. The results also indicate that stability of the berm slope is more sensitive to the cohesion and unit weight of the soil than other parameters. If lower bounds of the soil parameters were assumed, the safety factor is 1.1 which is close to a marginal value. Based on this result, AMEC recommends installation of a liner over the interior slope of the lagoon berm.

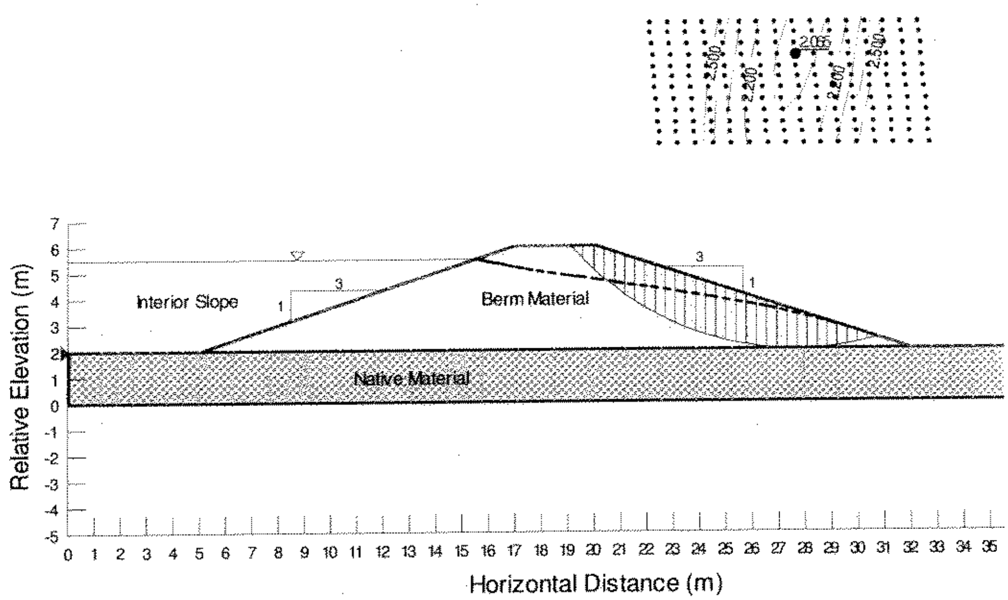


Figure 8 Factor of Safety for Lagoon Berm (Mean Value of Soil Parameters – Case 1)

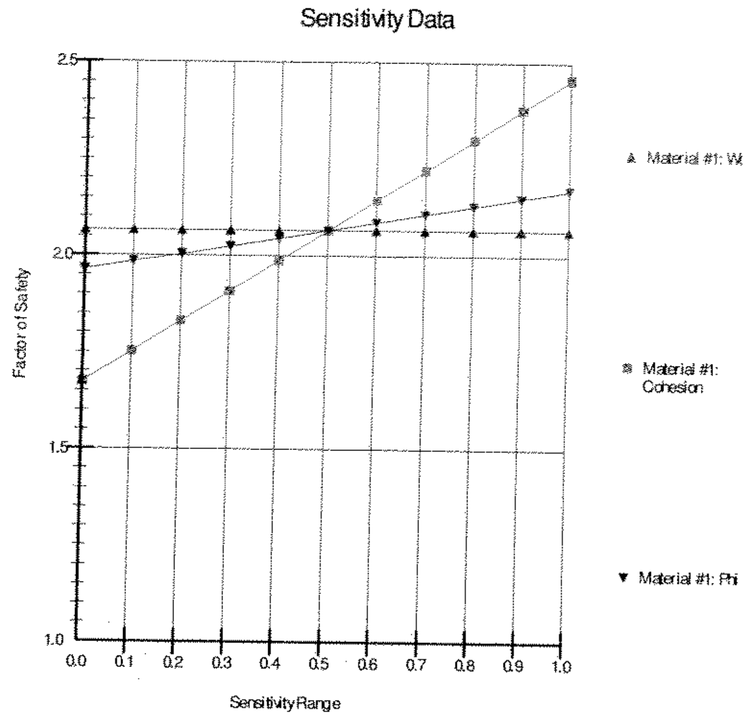
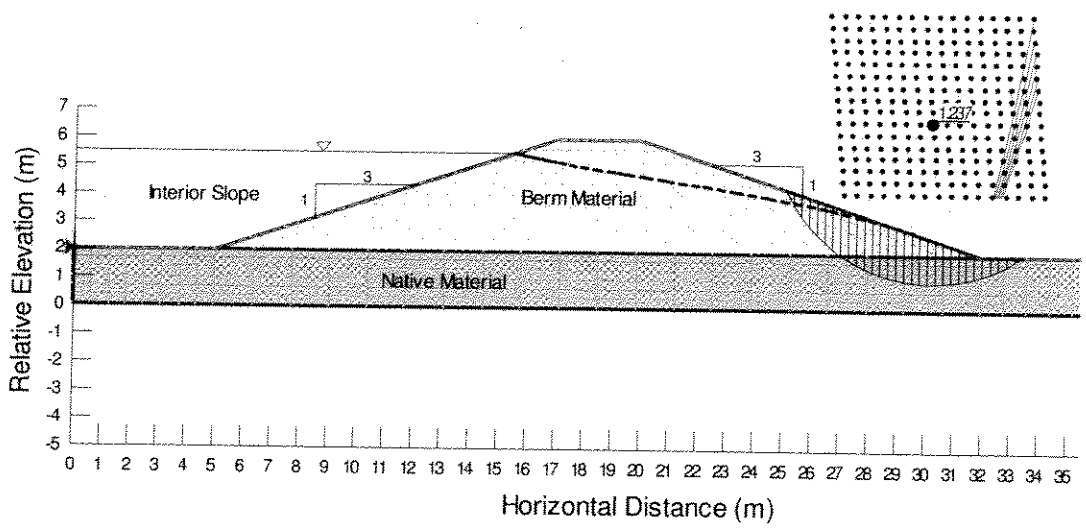


Figure 9 Sensitivity Plots for Case 1



Factor 10 Factor of Safety for Lagoon Berm (Mean Value of Soil Parameters – Case 2)

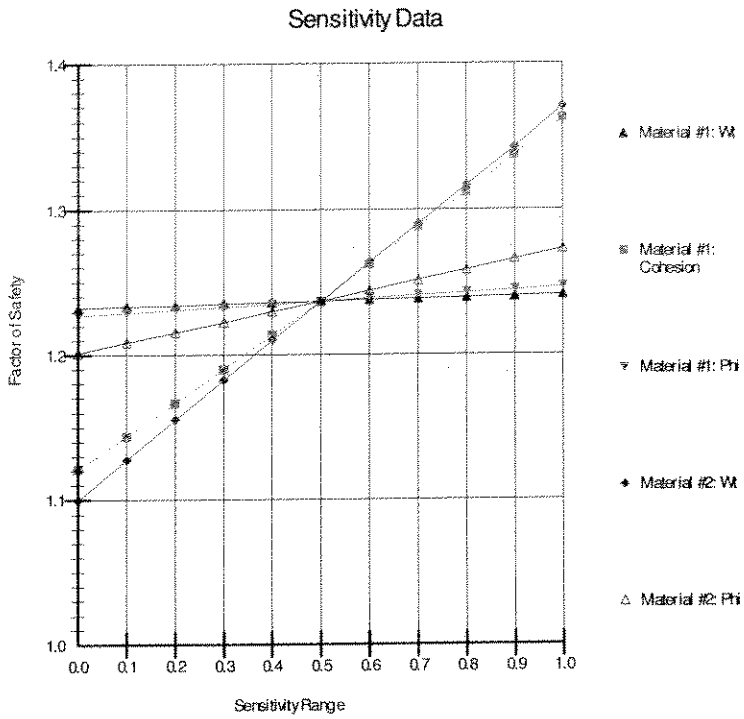


Figure 11 Sensitivity Plots for Case 2

The discharge schedule indicates that the water depth at the interior slope of the lagoon may not reach 3.5 m within the first few years of the lagoon operation. For comparison purpose, similar analyses were carried out assuming that the water level to be located 1.5 m below the berm crest. An estimated phreatic line within the berm and the safety factor for such scenario, applying mean values of the soil parameters are illustrated in Figure 12 and the results of sensitivity analyses are presented in Figure 13. It can be concluded that the berm will be stable if the water level in the lagoon will be maintained 1.5 m below the berm crest.

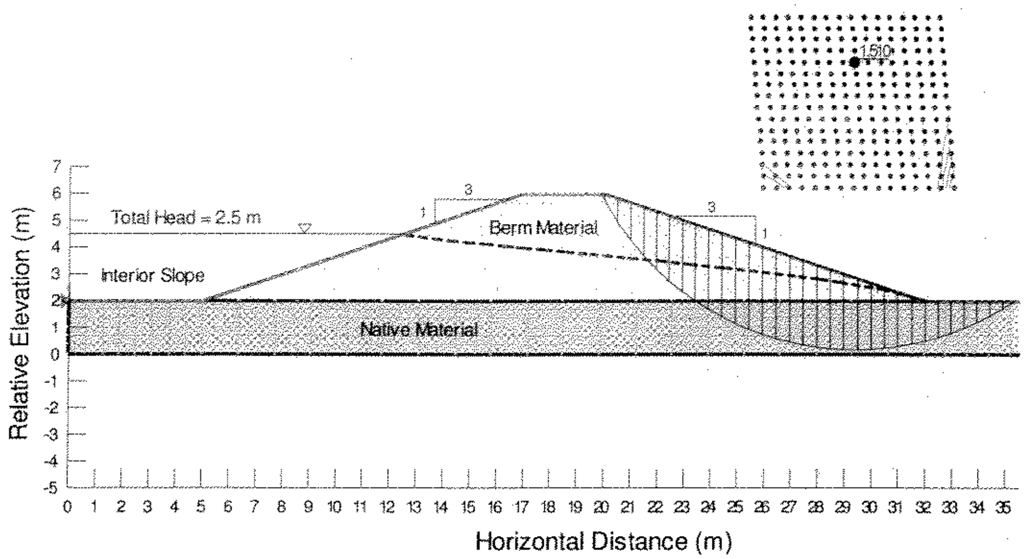


Figure 12 Factor of Safety for Lagoon Berm for Water Level at 2.5 m Depth

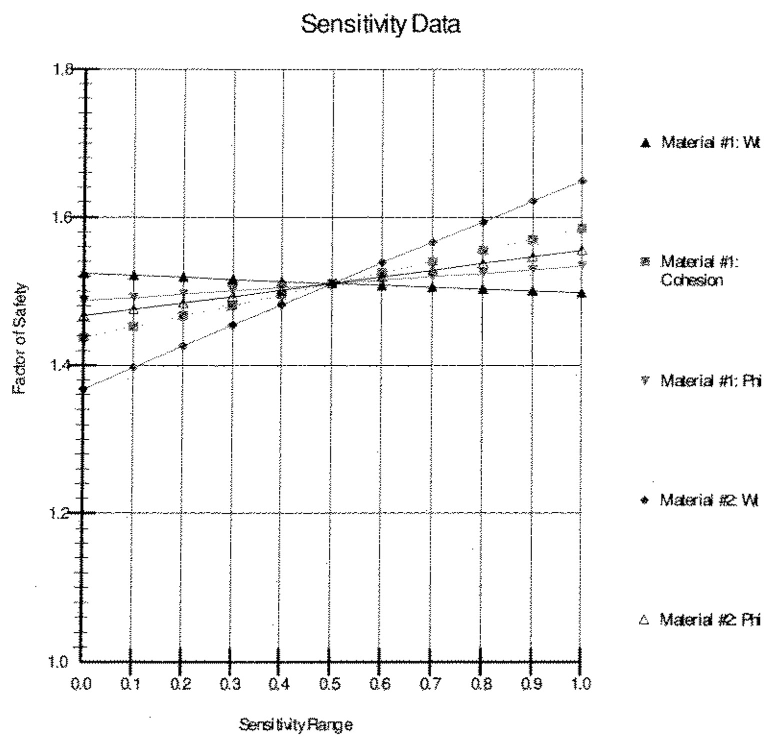


Figure 13 Sensitivity Plots for Water Level at 2.5 m Depth

5.7 Geothermal Analyses

The geothermal modeling program SIMPTEMP, 2D version (developed in-house by AMEC) was used to analyze the berm geothermal regime. The geothermal program uses the finite element method to compute a numerical solution of the heat transfer problem. Physical/mathematical algorithms used in the SIMTEMP 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 the ten years period.

The analysis considered the following geometry for lagoon:

- Height of berm is 4 m.
- Width of crest is 3 m.
- Depth of water is 3.5 m.
- Interior and exterior slopes of berm are 3H:1V.
- Local soil (sand, trace silt) 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	-25.0	-24.7	-22.8	-15.3	-4.8	6.8	13.9	11.4	3.6	-6.5	-17.6	-23.0
Downstream Terrain	-19.5	-19.2	-17.7	-11.9	-3.7	6.8	13.9	11.4	3.6	-6.5	-17.6	-23.0
Ice Surface	-19.5	-19.2	-17.7	-11.9	-3.7	---	---	---	---	-6.5	-17.6	-23.0
Water	---	---	---	---	---	5.2	10.7	8.8	2.8	---	---	---

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

The initial temperature of the berm material and the active layer was taken to be 4 ^{o}C, while the frozen soil below the active layer was assigned at -4 ^{o}C. The soil profile consisted of 2 m thick sand layer (moisture content 10 percent) and a 13 m thick clay till layer (moisture content 20 percent to 25 percent) overlying bedrock (moisture content 2%). It was assumed in the analysis that the berm material properties are the same as properties of the native sand layer. Liner was not included in the analyses since it would have a negligence impact on the results of geothermal modeling. The water level in the lagoon was instantaneously raised at the maximum elevation (0.5 m below the berm crest), beginning from October 1. The model ran for 30 years.

Figure 12 shows that after the first year of berm operation, the active layer at the berm crest is about 3 m. The majority of the berm core has a temperature in a range from 0.5°C to 3°C while the ground temperature near the berm core and under the dyke is about -0.5°C. Due to the warming effect of the lagoon water, the ground temperature beyond the interior slope of the berm (impoundment area) is about one degree warmer than the ground temperature beyond the exterior slope of the berm.

Figures 13 through 16 show that no significant temperature changes were observed within and underneath the berm from the fifth to thirtieth year of the berm operation. However, it can be seen that the thickness of the unfrozen zone under the lagoon impoundment increases up to 10 m. The thickness of the active layer in the berm was estimated to be 2 m to 2.5 m.

The numerical simulation showed that a frozen core within the lagoon berm will be formed only near the berm base. Thus, a potential for the percolation of water/effluent through the berm will depend on elevation of the water level in the lagoon. If the water level will be at the elevation, shown in Figures 14 through 18, then the percolation of the water would occur. AMEC recommends that a geomembrane be installed in the lagoon berm for seepage protection. A typical section for layout of the liner is provided in Figure 19. The cut–off trench, at least 2.5 m deep, should be excavated at a position of the interior slope crest. The liner should be placed vertically in the cut-off trench and then backfilled with compacted clayey material or grouted. The liner curtain should then follow the ground surface to the toe of the interior slope and then cover the interior slope to elevation higher than the expected maximum water level, as shown in Figure 19. The liner should be covered with a 0.5 m thick riprap layer. An alternative liner option is shown in Figure 20. The constructability of the alternative option is more complex however the liner is nearly half as wide. The alternative option suggests covering the interior berm slope with a 0.5 m thick riprap layer to protect the slope against thermal and wave erosion.

It is understood that seepage was observed near the interface of active zone and permafrost during site investigation, therefore a drainage system (such as sumps or/and pumps) may be required for dewatering of the cut-off trench and installation of the liner. Thus, a liner, installed over the entire lagoon area, may also be considered to minimize the potential risk of not being able to successfully dewater the cut-off trench during construction.

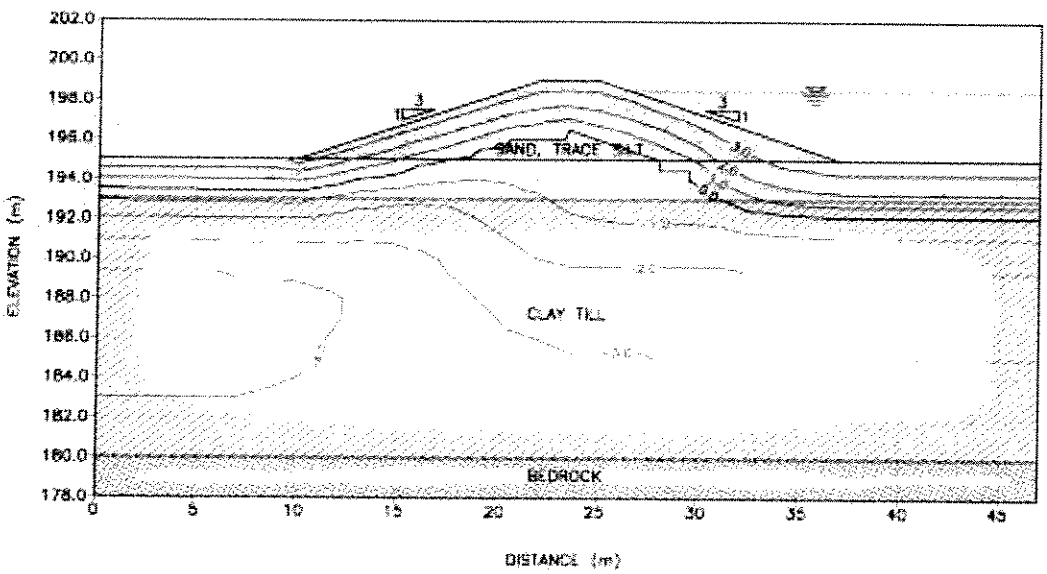


Figure 14 Temperatures in Lagoon Berm after 1 Year of Operation

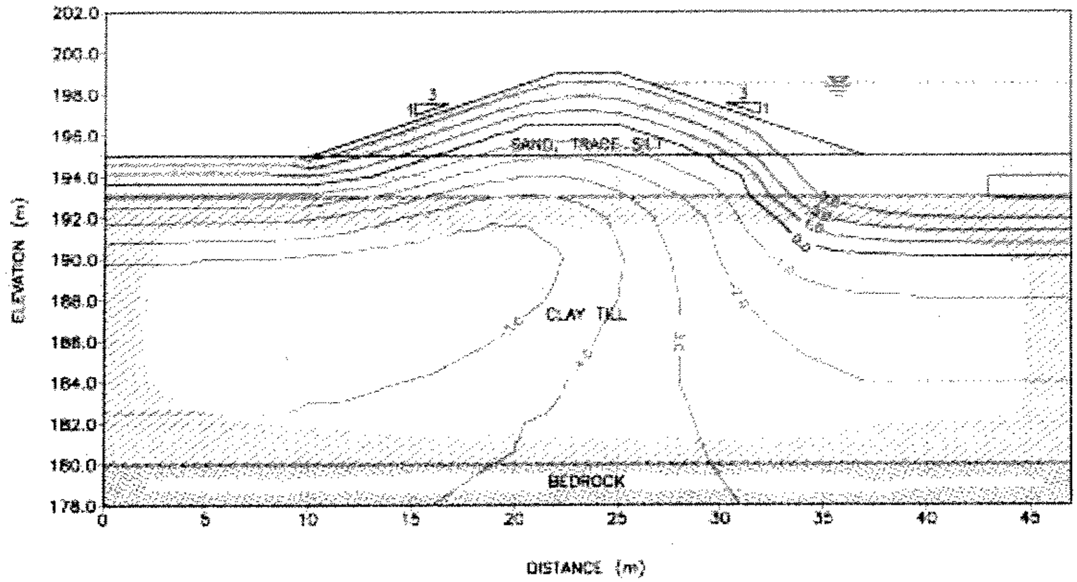


Figure 15 Temperatures in Lagoon Berm after 5 Years of Operation

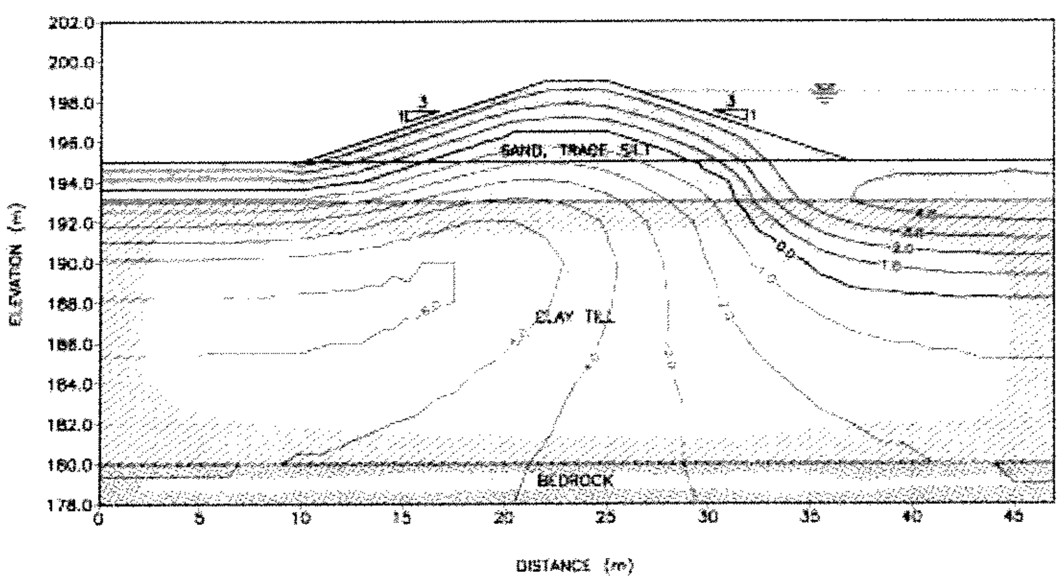


Figure 16 Temperatures in Lagoon Berm after 10 Years of Operation