# Design and performance of a frozen core dam in Cape Dorset, Nunavut

Paul C. Cavanagh, M. Eng, P. Eng, and Alexandre Tchekhovski, Ph.D., P. Eng.
*AMEC Earth and Environmental, Calgary, Alberta, Canada*

ABSTRACT
Long-term fluid retention within the Cape Dorset sewage lagoon basin relies primarily on maintaining a frozen cross-section within the earth embankment. A geosynthetic clay liner (GCL) was also installed within the dam to limit fluid seepage through the embankment prior to freezing of the dam cross-section and to provide a contingency seepage barrier. The design concept was challenged to use readily available sand and gravel as the primary construction material, and to integrate the sewage lagoon operating requirements and activities with naturally occurring ground freezing mechanisms. The use of geothermal models during design and the comparison of predicted temperatures with actual data from operations is presented and discussed.

RÉSUMÉ
La rétention à long terme des fluides dans le bassin des eaux usées de Cap Dorset s'effectue principalement grâce au maintien d'un noyau en sol gelé dans la digue en terre. Un geotextile bentonitique (GCL) a été installé pour limiter le débit d'eau au travers de la digue pendant le gel de l'ouvrage et procurer une protection additionnelle dans l'éventualité d'un écoulement. Les principaux défis à la conception ont compris l'utilisation de sable et gravier, disponibles en abondance, comme matériaux primaires de construction, de même la conciliation des contraintes d'opérations normales du bassin avec le processus naturel du gel des sols. L'utilisation de modèles géothermiques lors de la conception et la comparaison des températures prédites avec celles relevées pendant l'opération sont présentées et discutées.

## 1 INTRODUCTION

The community of Cape Dorset is located on Dorset Island near Foxe Peninsula off the southern tip of Baffin Island in the Qikiqtaaluk Region of Nunavut. The sewage lagoon impoundment area occupies an inter-mountain depression about 800 m southeast of the community. The depression slopes gently to west discharging surface run-off water into P-Lake. The lagoon was needed to replace the existing sewage handling facilities, which were reaching the end of their useful life.

The sewage lagoon is bounded by two retention dams positioned on the west and east sides of the impoundment area (Figure 1) that were designed and constructed using a frozen core dam concept. The West (main) Dam is about 6 m high in the center and about 100 m long, while the East Dam is about 4 m high and considerably shorter. A geo-synthetic clay (GCL) liner was placed on upstream slopes of the dams and keyed in a cut-off trench beneath the embankments. Local granular material, consisting of sand and gravel, was used for construction of the embankments and backfilling of the cut-off trench.

Design of the lagoon hydraulics and civil works was carried out by Dillon Consulting on behalf of Public Works and Government Services Canada (PWGSC) in conjunction with the Government of Nunavut – Community and Government Services (CGS) and the Hamlet of Cape Dorset. AMEC Earth and Environmental (AMEC) provided geotechnical and geothermal design input, and quality assurance and quality control services during construction.

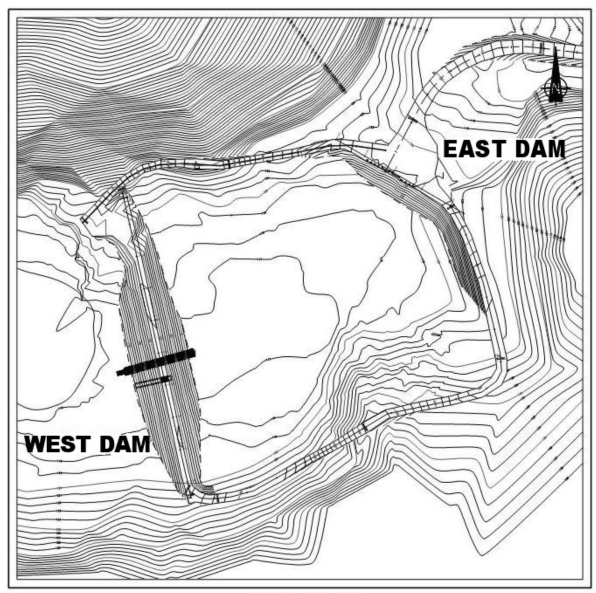


Figure 1. Locations of retention dams

## 2 SITE RECONNAISSANCE AND BACKGROUND INFORMATION

Cape Dorset is located at approximately 64°14' N latitude and 76°32' W longitude within the continuous permafrost zone and has a mean annual air temperature of about -9°C

.The thickness of the permafrost active layer varies from about 1.0 m to 2.0 m, depending on ground vegetative cover and surface disturbance, and the mean annual permafrost temperature within the study area ranges from about -5°C to -7°C at a depth of 10 m to 12 m below the ground surface.

The bedrock in the Cape Dorset area generally consists of Precambrian gneiss and marble with granite intrusions of the Aphebian era and is closely jointed with numerous small faults. The surficial materials include glacial till, talus and marine beach deposits. Isolated deposits of glacial silty sand and gravel (till) overlie bedrock in the uplands. Talus is the most common surficial material in the area with material composition that varies from silt to gravel sizes and the deposits are typically several meters thick (Laymon, 1992).

Field reconnaissance and aerial photograph interpretation of the proposed sewage lagoon site was undertaken in August 2005. The proposed lagoon site comprised a low lying depression (wetlands) covered with a thin organic layer. Rounded and subrounded boulders and rock fragments were strewn over the ground surface (Photograph 1) suggesting of the presence of glacial deposits (till), including silt, sand, gravel and boulders. The overburden material (till) was about 1 m to 2 m thick.

Photograph 1: Lagoon impoundment, looking west toward main dam location

## 3 DAM DESIGN AND CONSTRUCTION

The upstream and downstream slopes of the dyke were designed and constructed at 2.5 H:1V. The main dyke is 6 m high and 4 m wide at the crest. Silty sand, sand and gravel was used for the dyke construction. The material was placed in unfrozen, compacted lifts that were about 250 mm thick to 98 percent of Standard Proctor Maximum Dry Density (SPMDD). A layer of cobbles (up to 200 mm in size) was placed on the dyke slopes in a layer about 0.5 m thick for protection against water and wind erosion. Construction took place in the summer of 2008, and the first filling with sewage began in the summer of 2010.

A GCL was installed within the upstream slope of the dam and extended from the base of a 2 m deep cut-off trench below the dam shell to near the upstream crest. The cut-off trench was backfilled with compacted silty sand material, similar to that was used for the embankment construction. The synthetic liner system was intended to limit possible seepage through the dam in the early years of operation until a frozen core was formed within the embankment.

Due to expected uncertainties in performance of the frozen core dam, the observational approach was incorporated into the design and construction, which included a risk assessment of dam stability with an unfrozen core or during rapid fluid drawdown in the impoundment area. The observational approach also included development of a temperature monitoring program and contingency plans.

## 4 GEOTHERMAL MODELING

The geothermal modeling software SIMPTEMP, developed in-house by AMEC, was used to analyze the temperature regime of the Cape Dorset dam.

SIMTEMP is a finite element 1-2-3 dimensional software designed for the prediction of temperature, thaw/frost depth and frost heave/settlement in various materials due to environmental changes or due to construction activities. The comprehensive formulation provides the capability to analyze both simple and complex geothermal problems. The software can be used for a simulation with conductive or convective heat transfer, or both. The model has been used for various geothermal applications such as:

- Freezing and thawing around buried pipelines.
- Prediction of soil temperature under buildings.
- Prediction of dam shell/core temperatures.
- Prediction of soil temperature due to various changes of environmental conditions such as removal of vegetation, change of snow cover or solar radiation parameters.
- Prediction of permafrost processes such as thermokarst and frost heave.

The algorithms used in the SIMPTEMP model were published by Tchekhovski and Zenova (1989), and the results of calculations were verified with well-known analytical solutions of heat transfer, and 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 in Canada and internationally for more than ten years.

### 4.1 Boundary Conditions

The air temperature data and snow depth used for the present analysis were based on the Climate Normals from the Cape Dorset weather station for the period from 1971 to 2000. The mean monthly air temperatures and snow thicknesses used for the SIMPTEMP model are presented in Table 1.

Table 1. Mean monthly air temperatures and snow thicknesses.

| Month | Temperature (°C) | Snow Thickness (m) |
|---|---|---|
| Jan | -25.0 | 0.48 |
| Feb | -26.0 | 0.47 |
| Mar | -21.6 | 0.55 |
| Apr | -14.1 | 0.58 |
| May | -5.5 | 0.41 |
| Jun | 2.3 | --- |
| Jul | 7.4 | --- |
| Aug | 5.7 | --- |
| Sep | 1.5 | --- |
| Oct | -3.9 | 0.13 |
| Nov | -11.7 | 0.25 |
| Dec | -20.2 | 0.40 |

Mean monthly surface temperatures were applied over the exposed dam surface, on the ground surface adjacent to the downstream slope of the dam and over the water surfaces within the impoundment area upstream of the dam using various n-factor coefficients for the surface being considered.

It was assumed that practically no snow would accumulate on the dam slopes and crest but would accumulate beyond the toe of the dyke. Therefore, an n-factor of 0.9 was applied to the mean monthly air temperatures to obtain the mean monthly winter temperatures on the dyke surface. An n-factor of 1.2 was applied to the mean monthly air temperatures to obtain the dyke surface temperature in summertime. Based on 1D geothermal analysis, it was estimated that n-factors for the terrain type shown in Photograph 1 would be 0.65 and 0.83 for the winter and summer air temperatures, respectively. These n-factors represent the insulating/ warming effect of snow cover in the winter, and the cooling effect of the moss/lichen vegetation in summer.

To account for minimal snow cover on the dam surface and a reasonable rate of global warming over an operating life of about 25 years, it was considered that the mean annual air temperature in Cape Dorset in 2035 would be 1°C to 1.5°C warmer than the present air temperature. Such warming was not considered to have a significant affect on the ground temperature in the embankment or surrounding ground area. However, if warming occurs at rates higher than that assumed above, then contingency plans could be developed to address the specific issues at the time.

Filling the lagoon with sewage was estimated to occur at a rate of about 0.45 m per month. At this rate, the water would achieve full depth against the upstream slope of the dam (about 5.4 m) after 12 months. It was further understood that the fluid will be discharged, or decanted, from the lagoon annually during each September leaving about 0.45 m of fluid on the ground surface at the start of each winter freezing season. This schedule was modeled over 30 years of lagoon operation. Snow accumulation was included in the model on the lagoon surface in the winter similar to the downstream terrain area, by using an n-factor of 0.65 applied to the mean monthly air temperatures for the winter months (October through May). From June through September, it was assumed that the water temperature over the entire depth of the water column was the same as the mean monthly air temperatures. Table 2 provides data on the mean monthly surface temperatures that were applied over the upper boundary of the geothermal models.

Table 2. Mean monthly surface temperatures on model mesh, deg. Celsius.

| Month | Dyke Crest and Slopes | Downstream Surface | Water |
|---|---|---|---|
| Jan | -22.5 | -16.2 | -16.2 |
| Feb | -23.4 | -16.9 | -16.9 |
| Mar | -19.4 | -14.0 | -14.0 |
| Apr | -12.7 | -9.2 | -9.2 |
| May | -5.0 | -3.6 | -3.6 |
| Jun | 2.8 | 1.9 | * |
| Jul | 8.9 | 6.1 | * |
| Aug | 6.8 | 4.7 | * |
| Sep | 1.8 | 1.2 | * |
| Oct | -3.5 | -2.5 | -2.5 |
| Nov | -10.5 | -7.6 | -7.6 |
| Dec | -18.2 | -13.1 | -13.1 |

* Water temperature equals air temperature.

The initial dam surface and active layer temperatures were taken as 2°C, corresponding to the dam material temperature and active layer temperature at the end of summer. The initial soil temperature from the base of the active layer and to a depth of 12 m was taken to decrease linearly from 0°C to -5°C. The soil temperature then was warmed gradually down to the bottom of the mesh with the geothermal gradient of 0.02°C/m. The initial temperature for discharged sewage in wintertime was assumed to be 5°C.

Zero heat flux was applied at the lateral boundaries of the grid, while the heat flux at the mesh bottom corresponded to the ground geothermal gradient of 0.02°C/m.

### 4.2 Materials Properties

Physical and thermal properties within the dam and dam foundation materials were selected based on available published data (Farouki, 2004) and previous experience with similar materials. Saturated and unsaturated dam materials were considered in the upstream and downstream portions of the dam section, respectively. Table 3 summarizes the material physical and thermal properties applied for the geothermal analyses.

Table 3. Physical and thermal soil properties.

| **Soil Type** | **Dry Density ($kN/m^3$)** | **Moisture Content (%)** |
|---|---|---|
| Bedrock | 28 | 2 |
| Unsaturated dam - sand and gravel | 20 | 7 |
| Saturated dam - sand and gravel | 19.6 | 15 |
| Silt & Clay | 17 | 20 |
| Water | 10 | --- |
| **Soil Type** | **Thermal Conductivity ($W/m/^oC$)** | **Heat Capacity ( $MJ/m^3/^oC$)** |
| | **Frozen/Unfrozen** | **Frozen/Unfrozen** |
| Bedrock | 2.90/2.90 | 2.58/2.58 |
| Unsaturated dam - sand and gravel | 2.90/2.73 | 2.26/2.68 |
| Saturated dam - sand and gravel | 2.61/2.26 | 2.26/2.51 |
| Silt & Clay | 1.69/1.57 | 2.14/2.49 |
| Water | 2.20/0.58 | 1.95/4.19 |

4.3 Results

Predicted temperature contours at the end of summer for various years after operation are presented in Figures 2 through 5. Figure 2 shows that at the end of the first winter, the lower portion of the active layer under the lagoon would be about 0°C. Due to summer dam construction, the active layer under the dam crest will be in an unfrozen state as well. Further downstream, the dam and active layer are frozen with temperatures in the range of -2°C to -4°C. At the end of the first summer after construction, the dam temperature is only marginally below 0°C, including the active layer under the dam crest which was unfrozen at the end of the winter (Figure 3).

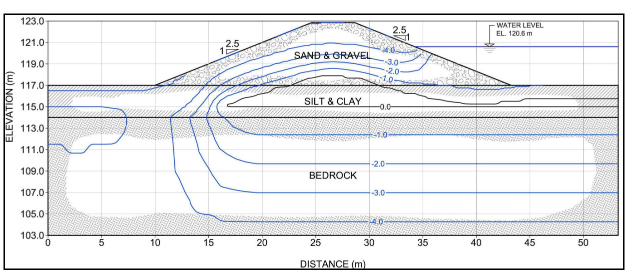

Figure 2. Dam temperature at end of first winter

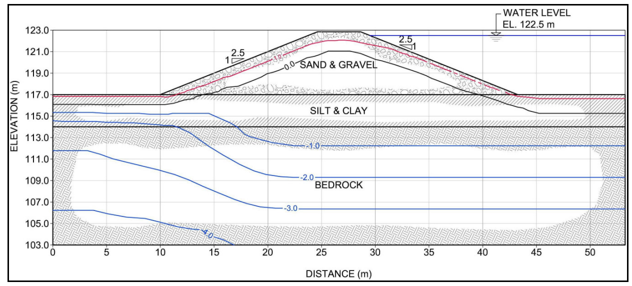

Figure 3. Dam temperature at end of first summer

The dam temperature decreases rapidly at the start of the second year of lagoon operation. Figure 4 demonstrates that the dam base temperature will be between -2°C and -3°C at the end of the third year. It is important to note that due to the gradual filling of the lagoon a talik does not form within the lagoon containment area. Gradual decreases in the embankment temperature are noted during the following years of the lagoon operation. It was predicted that after 30 years, the dam base temperature would be in a range from -3°C to -5°C (Figure 5).

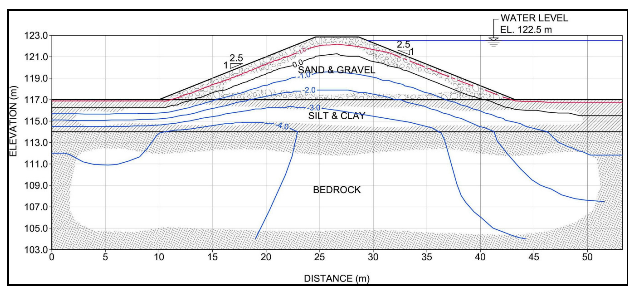

Figure 4. Dam temperature at end of third year

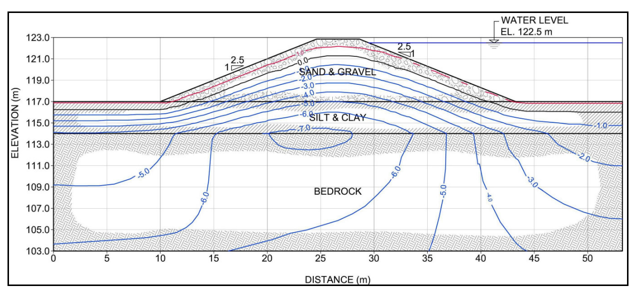

Figure 5. Dam temperature after 30 years of operation

The main conclusion from the geothermal modeling results is that relatively small dams can be constructed in summer in unfrozen conditions but will eventually develop a frozen core. However, a delay in lagoon filling for 1 to 2 winter seasons should be considered to allow for complete freezing of the embankment. Also, because of regular (annual) decanting of the fluid within the lagoon containment area, the dam will develop and maintain a frozen core over the intended operational life of the structure.

## 5 SLOPE STABILITY ASSESSMENT

Three potential failure mechanisms were considered in the slope stability analyses:

- a circular failure within the dam material during first several months of the operation when the dam was still unfrozen and during a rapid drawdown of sewage in September;
- a planar failure of the active layer on the exterior slope of the dam with a frozen core during summer periods of dam operation;
- dam stability against sliding along the interface of the dam and the native material.

### 5.1 Circular Failure

A simplified geological profile was developed for the slope stability analyses using available geotechnical information. The strength of the dam fill after compaction governs the stability of dam slopes. Considering that materials for the dam construction are granular soils, the pore water pressure within soil particles within the first several months has been assumed to be zero.

Commercial computer software SLOPEW (Geostudio 2004) was used to undertake the slope stability analyses. Due to various compositions of the granular material that may be used for construction, and some uncertainties in properties of the native silt and clay, the applicable soil input parameters (unit weight, cohesion and internal friction angle) may vary over a wide range of values. In order to evaluate the impact of the soil input parameters on the slope stability, sensitivity analyses were performed by varying these parameters. The potential failure surface with the lowest safety factor was initially determined using a circular search associated with the mean values of the strength parameters. The friction angle and cohesion were then kept at the mean values, and factors of safety were computed by varying the unit weight. Similar calculations were performed by varying the friction angle and cohesion. The estimated mean values, range of the input parameters and results of the analyses are summarized in Table 4.

Table 4. Input parameters of dam fill and results of sensitivity analyses.

| **Input Parameters** | | **Mean Value** | **Range** | **Factor of Safety** |
|---|---|---|---|---|
| Sand and Gravel | Apparent Cohesion (kPa) | 2 | 0 to 4 | 1.45 to 1.5 |
| | Internal Friction Angle□° | 33° | 31° to 35° | 1.45 to 1.5 |
| | Unit Weight (kN/m$^3$) | 19 | 18 to 20 | 1.4 to 1.55 |
| Native Silt and Clay | Apparent Cohesion (kPa) | 25 | 20 to 30 | 1.25 to 1.7 |
| | Friction angle□° | 0° | ---- | ---- |
| | Unit Weight (kN/m$^3$) | 18.5 | ---- | ---- |

Figure 6 presents a sensitivity plot for the dam slope. The strength parameters and unit weight are normalized to values ranging from 0 to 1. Zero indicates the lowest value in the parameter range, and 1 indicates the highest value. The point where the sensitivity lines cross is known as the deterministic factor of safety at the mean values of the strength parameters. The potential failure surface and safety factor (near 1.5) using the mean values is shown in Figure 7.

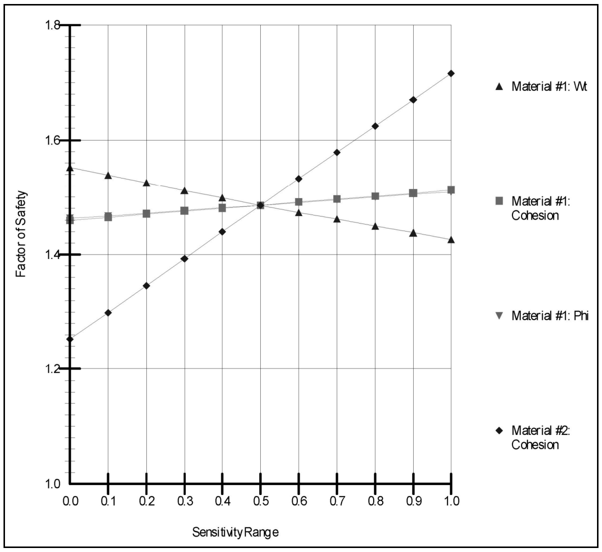

Figure 6. Sensitivity plot for dam slope stability.

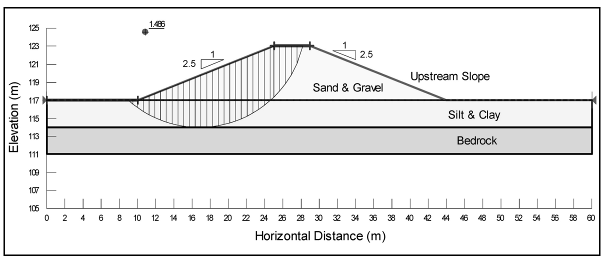

Figure 7. Factor of safety for lagoon dam (mean value of soil parameters).

The safety factor against slope failure is close to 1.5 also for various soil parameters provided in Table 4. However, if the undrained shear strength of the overburden soil was at the lower bound value (20 kPa), the safety factor will decrease to about 1.3. It was considered that this low strength represented a relatively soft material that did not exist at the site. This was confirmed through field observations and quality control procedures during construction. The results of the geothermal analysis also suggest that the active layer thickness beyond the downstream slope is less than 1 m, i.e. majority of the overburden will be frozen during the lagoon operation.

The active layer thickness in the impoundment area may extend 2 m below the ground surface in the first year of the lagoon operation. The slope stability analysis for the upstream slope has confirmed that the safety factor in these conditions is above 1.4. The active layer thickness will decrease during the following 3 to 5 years, resulting in an increase in the safety factor.

During drawdown of sewage on the upstream slope, excess pore pressure may develop within the dam fill material. The rate of the pore pressure dissipation will depend on the hydraulic conductivity of the dam material. For assessment of the rapid water drawdown on the slope stability, the lower bound value of the saturated hydraulic conductivity (10-5 m/s) was used in the seepage analysis. The results indicated that the phreatic surface responds almost simultaneously with the change of the sewage level in the impoundment. It means that the risk of building up of the excessive pore pressure within the upstream slope is very low.

## 5.2 Planar Failure

The stability of the active layer overlying a frozen slope was also assessed. The maximum thaw depth (thickness of the active layer) on the dam slope can be in an order of 2 m to 2.5 m at the end of summertime.

The safety factor of the slopes was calculated using the limit equilibrium theory (Hanna A. and McRoberts E. 1988) by comparing the total resistance force along a failure slip (interface between active layer and frozen soil) to the total driving force along the same failure slip. In addition to the base resistance along the slip, the side shear resistance of the active layer also contributed to the total resistance, increasing stability of the slope.

The analyses were carried out for an assumed failure slope of 17 m long (along the dam slope) and 20 m wide. Similar to the circular failure, sensitivity analyses were carried out using the same mean values and ranges of the soil parameters as it was outlined in Table 4. The results of analyses for the active layer, 2 m and 2.5 m thick, are presented in Figure 8. The factors of safety against the failure within the active layer are greater than 1.5 which suggests that the dam slopes will be in stable conditions. Figure 8 also indicates that the factor of safety decreases by 3 percent with increasing the active layer thickness from 2 m to 2.5 m, and decreases by 5 percent with increasing the slope width from 20 m to 190 m (the maximum length of the dam.

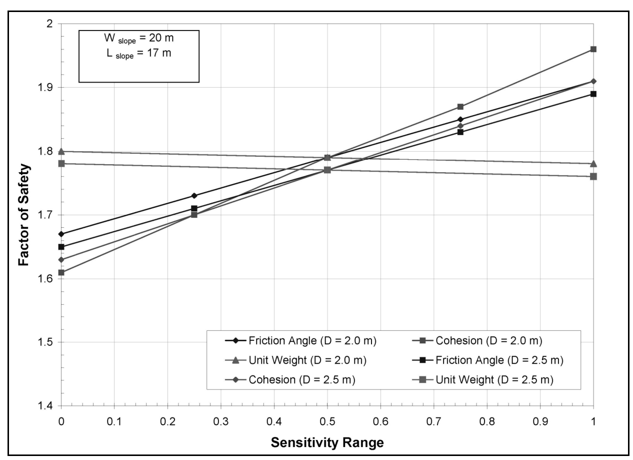

Figure 8. Sensitivity plots for stability of active zone within lagoon dam.

## 5.3 Sliding

Stability of the dam against sliding due to the hydrostatic pressure applied on the interior slope was assessed by assuming that the dam is a rigid retaining structure. The friction coefficient of 0.25 was used at the interface between the dam material and the native ground, as recommended by the Canadian Foundation Engineering Manual, 4th Edition, 2006). It was found that the factor of safety along the selected interface is greater than 3.

## 6 CONSTRUCTION MONITORING

Construction monitoring was carried out in the summer of 2007 and commenced with inspections of the cutoff trench excavation and backfill. The design required that the trench base be keyed into frozen material or was at least 2 m deep. The observed soil profile along the cut-off trench walls consisted predominately of silty/clayey soil, with bedrock encountered along approximately 35 percent of the total length. The overburden material was typically frozen from a depth of 1.0 m to 1.5, and with no visible ice. The soil conditions assumed in the design and geothermal modeling was generally confirmed, however a few differences were noted. It was determined that shallow bedrock occupied considerably less area of the impoundment and the main dam alignment and that fine-grained soils were more prevalent on the ground surface of the impoundment area and dyke alignment than was assumed. This had relatively minor implications for the dam design since the fundamental concept of a frozen core dam was still viable and appropriate with these observed conditions.

## 7 TEMPERATURE MONITORING

Three thermistor strings were installed in the main dam, and one thermistor string was installed in the minor dam in the fall of 2008. Locations of the thermistor strings are shown in Figure 1. Thermistor strings were installed in boreholes using an air rotary drill rig to depth of about 18.8 m. An electrical metallic tubing (EMT), 12 mm inside diameter, was installed in each borehole to house the thermistor string at each location. EMT couplings and caps were used to assemble the tubes into a single conduit. The thermistor string was then installed in the assembled conduit, and the conduit was placed into the drilled borehole. The annulus between the conduit and the borehole wall was filled with sand.

Dyke and soil temperatures have been monitored from November 20, 2008 to December 4, 2009 using automatic data collections systems (data loggers). The top thermistor sensor at each location was installed about 1 m above the ground surface, and the remaining thermistor sensors were installed below the ground surface at the following depths: 1.6 m, 4.2 m, 6.8 m, 9.4 m, 11 m, 14.6 m, and 18.8 m. A brief analysis of the measured temperatures in the main dam is provided below.

### 7.1 Logger 1

The embankment and native soil temperatures were observed to be less than 0°C in November 2008 (Figure 9). The maximum temperature (about -0.2°C) was measured at a depth of 4.2 m, and the minimum temperature was observed at a depth of 18.8 m (-3.6°C). In May, 2009, the temperature had gradually dropped down at all of the thermistor locations. The coldest temperature (about -7°C to -8°C) was measured at 1.6 m below ground surface due to the impact of the cold air temperature. Data on September 19, 2009 demonstrates that the dyke and soil temperatures to 9.4 m were above 0°C. However, on November 1, 2009, the temperatures down to a depth of 9.4 m were already below 0°C and it was concluded that the positive temperature readings in September were due to the ingress of melt water and runoff water into the EMT conduit.

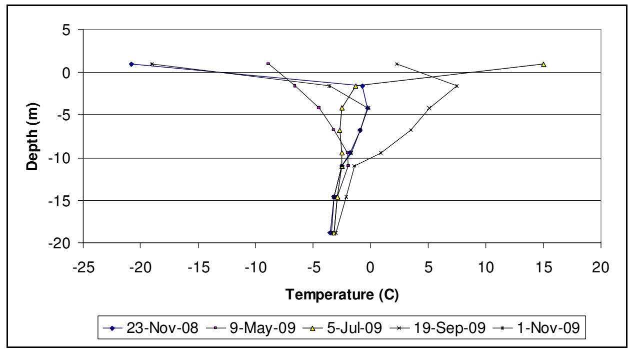


Figure 9. Logger 1

### 7.2 Logger 2

In May 15, 2009, the coldest temperature was observed at a 1.6 m depth (-11°C), and the warmest temperatures, ranging from -4.1°C to -4.4°C were measured at depths deeper than 9.4 m (Figure 10). At the end of July, 2009, the dyke temperature at a depth of 4.2 m was -4°C, and the soil temperature at a depth of 6.8 m was even colder (-5.5°C). The thickness of the active layer at the end of July was about 2.5 m. The active layer commenced rapid freezing in the fall, and the temperature at 1.6 m depth on November 1, 2009 was already -0.6°C. Below 1.6 m the dyke/soil temperatures in October 2009 remained steady, ranging from about -1°C (4.2 m) to about -4.5°C (below 9.4 m).

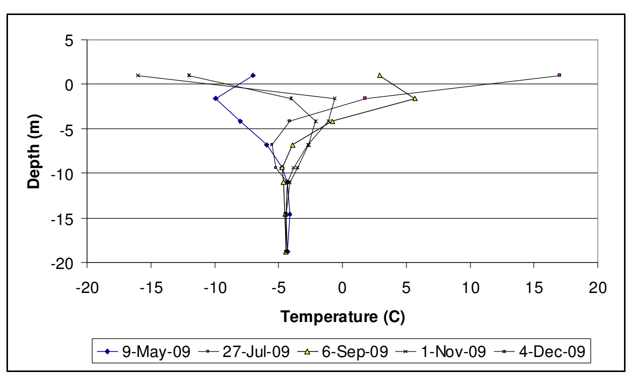

Figure 10. Logger 2 – middle of main dam.

## 7.3 Logger 3

In January 2009 the warmest temperature -0.9°C was measured at 4.2 m depth (Figure 11). Above this depth, the dyke temperature was considerably colder (-5°C at 1.6 m). The soil temperature below the dam base gradually decreased down to about -4°C, at 18.8 m. In the summer of 2009 the measured dyke/soil temperatures below about 2 m remained below 0°C, where the temperature at 1.6 m remained above 0°C until the end of October 2009.

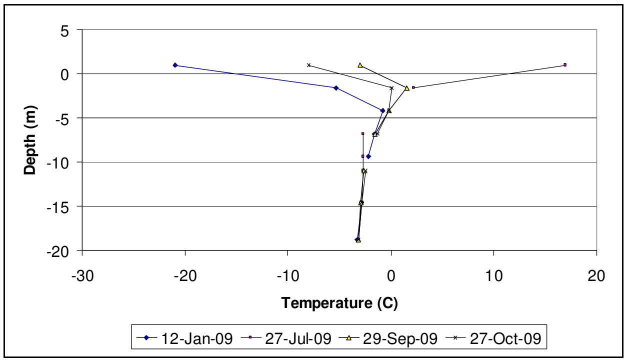

Figure 11. Logger 3 – south section of main dam.

## 7.4 Measured and predicted temperatures

Temperature data from Logger 2 and 3 were used to compare measured dyke/soil temperatures with those estimated during the modeling and design phases. The temperatures from the model, following 1 year of operation, are presented on Figure 3 and correspond to the end of summer (about September 30). The measured temperatures for the same date have been selected from Loggers 2 and 3 for comparison in Table 5.

Table 5. Comparison of predicted and measured dyke temperatures.

| Depth, m | Predicted Temp., °C | Logger 2 Temp, °C | Logger 3 Temp., °C |
|---|---|---|---|
| 1.6 | +0.2 | +1.3 | +1.4 |
| 4.2 | -0.3 | -0.9 | -0.2 |
| 6.8 | -0.6 | -3.2 | -1.5 |
| 9.4 | -1.0 | -4.3 | No data |
| 11.0 | -1.3 | -4.6 | -2.6 |
| 14.6 | -2.5 | -4.5 | -2.8 |
| 18.8 | -3.6 | -4.4 | -3.1 |

The predicted and measured temperatures show agreement between the data sets. However, measured temperatures at 1.6 m are warmer than was predicted in the model for the same time period. This discrepancy in the temperatures is likely explained by a difference in the mean summer air temperatures (used in the model) and actual temperatures experienced during the summer of 2009. For instance, the mean summer air temperature in 2009 in Cape Dorset was estimated at +7.3°C, while the mean summer air temperature in the model was assumed to be +4.2°C based on Climate Normals for 1970 to 2000.

The measured temperatures, below 4.2 m, are in general slightly colder than those predicted in the model. These differences in temperatures are likely due to the warming effect of sewage water, which was included in the geothermal modeling conducted during the project design phase. The measured temperatures represent conditions with little or no sewage water within the impoundment area.

## 8 CONCLUSION

Summer construction with non-saturated granular material can be used for small frozen core dams if impoundment filling is scheduled to allow for sufficient time for the embankment to freeze. Periodic decanting also helped to maintain the frozen conditions within the dam soils. Climate conditions and the schedule for impoundment filling are important factors to consider when determining the dimensions and operating parameters for the dam and sewage impoundment area. The thickness of the active layer becomes an important factor in design and operation of the dam and in dettermining the depth and position of the cut-off trench. Geothermal modeling was a valuable tool in determining the design and operating characteristics of the structure and was shown to adequately predict development of the frozen embankment concept based on permafrost, soil, and lagoon conditions.

The observational approach is an important component for the design, construction and operation of frozen embankments. The need to have periodic reviews and to monitor actual site conditions and changes along with incorporating contingency plans are important considerations for the overall approach to be viable. The integration of geothermal modeling, slope stability and seepage analyses with operational characteristics allowed for an optimized design and cost effective approach to the design and construction of a sewage containment facility.

## 9 ACKNOWLEDGEMENTS

The paper was published with the approval of the Department of Community and Government Services, Government of Nunavut. The authors would like to acknowledge Dillon Consulting Limited for providing design information.

## 10 REFERENCES

*Canadian Foundation Engineering Manual, 4th Edition*. 2006. 488.

Chekhovskyi, A. and Zenova, O. 1989. Nomographs Of Soil Annual Average Temperature, *Frost in Geotechnical Engineering: International Symposium*, Saariselka, Finland, 1. 86-94.

Farouki, O. 2004. Ground Thermal Properties, *In Thermal Analysis, Construction, and Monitoring Methods for Frozen Ground*, ASCE. 259-277.

Hanna, A. and McRoberts, E. 1988. Permafrost Slope Design for a Buried Oil Pipeline, *5th International Permafrost Conference*, Trondheim, Norway, 2. 1247-1252.

Laymon, C. 1992. Glacial Geology of Western Hudson Strait, Canada, with Reference to Laurentide Ice Sheet Dynamics, *GSA Bulletin*, 104. 1169-1177.