Geotechnical Design of Thermopile Foundation for a Building in Inuvik

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ABSTRACT
A thermopile foundation has been designed for a new building in Inuvik. Challenging geotechnical ground conditions at the site include an up to 3 m thick unfrozen zone (closed talik) below the active layer, up to 10 m thick ice and ice-rich soils, and warm permafrost temperatures. A set of geothermal analyses were carried out to estimate the building foundation ground temperatures with depth and time under normal working conditions of the thermopiles and climatic conditions considering projected climate changes over the building design life of 50 years. Geotechnical evaluations were conducted to estimate the required pile embedment depths in consideration of specified pile long-term creep settlement criteria and design criteria against potential frost jacking. A total of three design groups of the thermopiles were recommended for the building foundation.

RÉSUMÉ
Une fondation sur pieux thermiques a été conçue pour un nouveau bâtiment à Inuvik. Les conditions géotechniques difficiles du terrain comprennent une zone dégelée allant jusqu'à 3 m d'épaisseur (talik fermé) en dessous de la couche active, des sols riches en glace, des couches de glace allant jusqu'à 10 m d'épaisseur, ainsi qu'un pergélisol chaud. Des analyses géothermiques ont été effectuées pour estimer les températures du sol sous la fondation du bâtiment selon la profondeur et le temps dans des conditions normales de fonctionnement des pieux thermiques, ainsi que dans des conditions climatiques qui prennent en considération le réchauffement climatique prévu au cours de la durée de vie du bâtiment, qui est de 50 ans. Des évaluations géotechniques ont été réalisées pour estimer les profondeurs d'enfouissement requises en considérant les critères de fluage à long-terme d'un pieu spécifique, ainsi que les critères de conception contre l'effet potentiel du soulèvement par le gel. Trois options de conception de pieux thermiques ont été recommandées pour les fondations du bâtiment.

1 INTRODUCTION

A new two-storey detachment building was proposed in Inuvik, Northwest Territories. During the early stage of the design process, the owner and design teams for the building evaluated several foundation options and adopted helix thermopiles as the foundation type for the building. The design life of the building is 50 years. The preliminary pile layout and pile axial service loads were provided by the structural design team of the building. A minimum clearance of 1.2 m between the pile cap below the building and the graded ground surface was specified and used in the design.

The structural design of the building proposed a total of 128 piles in five pile loading groups with variable pile spacings. The main scope of the geotechnical design of the thermopile foundation was to recommend the design geometries of the thermopiles, based on geothermal analyses and geotechnical evaluations.

This paper presents the design basis, methodology, and recommendations for the geotechnical design of the thermopile foundation.

2 SITE CONDITIONS

2.1 Site Location, Surface Conditions, and Climate

Inuvik is located at latitude 68° 18' N and longitude 133° 29' W and on a relatively flat, wooded plateau along the East Channel of the Mackenzie River Delta. The town is surrounded by the Caribou Hills. These hills rise about 100 m above the town site and have several steeply-sloped valleys that drain to the west. The proposed site location in Inuvik is shown in Figure 1.

Figure 1. Site Location Plan

The ground surface of the site is covered with paved parking areas, grass lawns, or exposed gravel fill near several existing buildings.

Environment Canada maintains a weather station in Inuvik and has records available since 1958. Over the last 30 years, the mean annual air temperature at Inuvik has averaged -8.0°C. The annual freezing and thawing indices for Inuvik have averaged 4172 and 1276 °C-days respectively, during this same period. Based on Canadian Climate Normals (1981 – 2010), the total annual precipitation at Inuvik has averaged about 241 mm.

The annual mean air temperatures in the region have increased over the duration of the record; however, this increase has occurred predominately over the last 40 years. There was no clear trend in annual mean air temperature between 1929 and 1969, but a warming of 0.8°C per decade thereafter (Burn and Kokelj 2009). The rates of warming are an order of magnitude higher for winter (1.1°C per decade) than they are for summer (0.1°C per decade) (Burn and Kokelj 2009).

2.2 Subsurface Conditions

Boreholes with depths from 5.3 m to 20.6 m were drilled in the general site area in 2004 (Kiggiak-EBA 2004), 2007 (AMEC 2007) and 2011 (Nehtruh-EBA 2012). The boreholes were drilled with a truck-mounted auger. The subsurface stratigraphy varied among the boreholes, and consisted of three layers in general. The top layer of silty sand and gravel fill was approximately 1 to 2 m thick, well-graded, damp, and had some organics. Beneath the fill layer was a silty sand and gravel with some cobbles. This layer was saturated and between 3 to 4.5 m thick. Beneath the sand and gravel were layers of silt and ice. The ice layers had less than 30% sediment and were up to 6.6 m thick. The silt layers contained 5 to 40% visible ice and were up to 7.9 m thick. The depth to bedrock was not confirmed during the site investigations, but there is a possibility that shale/claystone bedrock was encountered at a depth of about 20 m in one on the boreholes.

Ground ice was found in the Inuvik area as pore-ice, ice wedges, massive tabular ice, and segregated ice (Burn and Kokelj 2009). The structures of the ground ice encountered in the boreholes could not be described because the samples were disturbed. However, the ice at the base of the active layer is probably segregated ice, and the thick ice layers at depth could be buried glacial ice or segregated ice (Nehtruh-EBA 2012).

Porewater salinity was reported in both Kiggiak-EBA (2004) and Nehtruh-EBA (2012). The values ranged from 1 to 7 parts-per-thousand (ppt) and averaged approximately 5 ppt. This corresponds to a freezing point depression of approximately -0.3°C.

Groundwater was encountered in the unfrozen layer above permafrost in two of the five boreholes drilled in 2004 (Kiggiak-EBA 2004). Groundwater was also encountered in the unfrozen layer above permafrost in all the boreholes except one during the October 2011 site investigation. The groundwater was accompanied by severe sloughing which was challenging for drilling and sampling. In a borehole that was drilled to 20.6 m depth, there was 14.5 m of saturated slough at the bottom of the hole, and more than 2 m of water above the slough (Nehtruh-EBA 2012).

2.3 Ground Temperature and Permafrost Condition

Inuvik lies in the continuous permafrost zone and ground ice is common in the region (Heginbottom et al. 1995). When development of the town began in the mid-1950s, the mean annual ground temperature was -3.4°C at 14.3 m in depth and active layer depths were 2.0 to 2.3 m (Pihlainen 1962). Ground temperatures in the region have increased since then (Hoeve 2012).

Single-bead thermistor was installed at 12.8 m depth in one of the boreholes during Nehtruh-EBA's 2011 site investigation, and ground temperatures have been measured periodically over the following year. The ground temperature was -1.5°C in both February and July 2012 at the 12.8 m depth in the borehole. Ground temperatures were measured at 1-m intervals in all seven boreholes on the last day of the January 2007 site investigation, and ranged from -0.6 to -2.0°C at 8 m depth (AMEC 2007), averaging approximately -1.3°C.

The thicknesses of the active layer at the site have not been confirmed. The ground was unfrozen from 2 to 3 m depths in two of the five boreholes drilled at the end of January 2004 (Kiggiak-EBA 2004). This unfrozen zone was interpreted as a still freezing active layer. Unfrozen ground was not reported for AMEC's January 2007 site investigation, but the ground temperature, at a couple of the boreholes, was -0.3°C at depths of 2 to 4 m. These warm temperatures were interpreted in the report as a still freezing active layer, which is probable given the depressed freezing point caused by the porewater salinity. Nehtruh-EBA's 2011 site investigation occurred in early October close to when the maximum thaw depth is reached. At this time, unfrozen soil was encountered to depths between 1.8 and 5.5 m below grade, with saturated sand and gravel above the permafrost. This was considered to be deeper than what would occur during seasonal thaw, so it was suspected that a closed talik (unfrozen zone beneath the active layer) was present beneath the site.

3 THERMOPILE AND DESIGN APPROACH

3.1 Thermopile

A thermopile is a combination of a thermosyphon and an adfreeze pile that carries a structural load. Similar to a thermosyphon, a thermopile has an evaporator section in the ground that extracts heat from the surrounding soil, and a radiator section above ground that dissipates the heat in the air. The centre of the sealed section of the thermopile is normally charged with pressurized carbon dioxide that passively cools the ground around the pile when the ground surrounding the evaporator section is warmer than the ambient air. Each year, there is a summer period when heat is not transferred out of the ground and the ground surrounding the thermopiles has potential to warm up and even thaw.

The structural load is normally transferred from the pile to the permafrost soils by means of adfreeze bonding. The strength of this bond is temperature-dependent. There is a steep rise in the adfreeze strength as the soil temperature decreases. Similarly the compressive and shear strengths of the soil also increase with a decrease in temperature. Ice-rich soils tend to creep under shear stresses. The creep rate is also temperature dependent and decreases with a drop in temperature. Thermopiles reduce the surrounding soil temperatures by passively transferring heat from the soil to the atmosphere, thereby increasing the adjacent soil strength and decreasing the pile creep rate.

A helix thermopile incorporates thin-bladed helixes on the pile surface to increase pile load capacity, not only by adfreeze, but also by compression. A helix thermopile is usually installed in a pre-drilled hole and the annulus is filled with a saturated sand or gravel backfill. The load is transferred from the pile by adfreeze and compression to the backfill and, provided that the load does not cause the sand or gravel backfill to shear, through the backfill or the backfill / native material interface.

3.2 General Thermopile Design Approach

The thermopile design presented in this paper generally consists of the following main steps:

- Understand the site climatic and geotechnical conditions;
- Assume preliminary design geometrical parameters of each group of thermopiles;
- Conduct geothermal analyses to predict ground temperatures with time and depth in the area around the thermopiles;
- Estimate the minimum pile embedment length to meet long-term pile creep settlement design criteria;
- Estimate the minimum pile embedment length against potential frost heave;
- Estimate the minimum pile embedment length to withstand the pile design axial load without shear failure;
- If required, update the geothermal analyses and geotechnical evaluations based on revised thermopile parameters; and
- Finalize the thermopile designs.

4 GEOTHERMAL ANALYSIS

4.1 Geothermal Analysis Model

Geothermal analyses for this study were carried out using our proprietary finite element computer model, GEOTHERM. The model simulates transient, two-dimensional or axisymmetrical, heat conduction with change of phase and a variety of boundary conditions, including heat flux, convective heat flux, temperature, and ground-air boundaries. The heat exchange at the ground surface is modeled with an energy balance equation considering air temperatures, wind velocity, snow depth, and solar radiation. The model facilitates the inclusion of temperature phase change relationships for soils.

The model has been verified by comparing its results with closed-form analytical solutions and many different field observations. The model has successfully formed the basis for geothermal evaluations and designs of water and tailings dykes/dams, foundations, pipelines, utilidor systems, landfills, and ground freezing systems in both arctic and sub-arctic regions since 1980's.

4.2 Climatic Conditions

Climate data required for the geothermal analyses include air temperature, wind speed, solar radiation, and snow cover. Mean air temperature, wind speed, and snow cover data used in the geothermal analyses were obtained from Environment Canada's weather station at the Inuvik Airport. Mean daily solar radiation data are Climate Normals (1951 - 1980) at Inuvik. The mean annual air temperature during the period of 1981 to 2010 at Inuvik was -8.0°C.

The historical air temperature data at Inuvik indicate that the climate has been warming since the 1970s. Based on the observed warming trend in the historical air temperatures, the geothermal evaluations for this project considered the long-term effects of climate change on air temperatures.

The Canadian Standards Association (CSA 2010) adopted the climate change projections from Environment Canada (2009). Inuvik is located in the W2 Arctic Zone (Environment Canada 2009, CSA 2010). The projected mean temperature changes from the 1971 to 2000 baseline under the moderate greenhouse gas emission scenario (A12) were used in the geothermal analyses.

4.3 Soil Profile and Properties

The soil profile used in the geothermal analyses was developed primarily based on the soil profile in BH-02 in the October 2011 site investigation. A deep unfrozen layer of 5.5 m above the permafrost with massive ice layers was observed in the borehole. This soil profile represents a reasonably conservative profile for the geothermal and geotechnical design of the thermopiles. The soil profile consists of 1.8 m of moist silty sand/gravel fill, 3.7 m saturated sand/gravel, 0.9 m ice, 2 m silty sand till, 3 m ice, 1.6 m silt and gravel till, 2.7 m ice, and silt and gravel/sand 4.3 m till over clay till over shale bedrock. The till layers had various excess ice of up to 25%.

Geothermal properties of the materials were determined indirectly from well-established correlations with soil index properties (Farouki 1981, Johnston 1981) or past experience. Soil index properties were estimated from available geotechnical information or based on past experience.

4.4 Geothermal Model Calibration

A one-dimensional geothermal analysis was conducted to calibrate the geothermal model against the observed ground geothermal conditions at the project site. The analysis was conducted by applying the measured air temperatures and other climatic data. The analysis simulated the mean climatic conditions with the measured air temperatures of 1961 to1990 over a 30-year period, followed with the air temperatures of 1990 to 2010 over a 20-year period, and then the air temperatures from November 2010 to October 2011 for a year. All other climatic inputs were assumed to be mean values.

The calibration geothermal analysis indicated that the estimated ground temperatures were consistent with the measured ground temperatures in one of the ground temperature cables. This suggests that the geothermal model and associated input parameters are reasonable and can be used for the geothermal design of the building foundation.

4.5 Geothermal Analysis Approach and Cases

Several trial geothermal analyses were initially carried out to evaluate the overall thermal performance of thermopiles with various key pile parameters such as pile embedment depths and thermopile radiator section areas. Based on the trial analysis results and the pile layout plan provided, four cases of final geothermal analyses were conducted for the thermopiles, as summarized in Table 1.

Table 1. Cases simulated in geothermal analyses

Case	Simulated Thermopile Plan Layout	Simulation Approach
1	Single pile only.	Axisymmetric around the axis of the pile surrounded by soils.
2	A single row of many external piles with a pile spacing of 5.2 m.	2D vertical plain with a continuous pile wall; equivalent pile thermal conductance used.
3	A single row of many external piles with a pile spacing of 3.5 m.	2D vertical plain with a continuous pile wall; equivalent pile thermal conductance used.
4	Infinite number of internal piles (under the building) with an equivalent pile thermal influence radius of 2.2 m.	Axisymmetric around the axis of the pile surrounded by a soil cylinder with a radius of 2.2 from the pile axis.

4.6 Boundary Conditions and Other Parameters

In the geothermal analyses, a ground-air boundary was applied to the ground surface to simulate the climatic conditions. A convective heat transfer boundary was applied inside of the thermopile surface to simulate the heat removal capability of the thermopile. A heat flux boundary was assigned to the bottom of the mesh to incorporate the geothermal gradient (0.046°C/m) at depth.

For Cases 1 to 3, a mean monthly snow condition was applied to the ground surface during winter. The mean monthly snow cover at Inuvik is 0.12 m in mid-October to a maximum of 0.58 m in mid-March. To consider the condition with a reduced snow depth under the building, the assumed snow depth for Case 4 was reduced to 50% of the mean snow cover. Based on the calibration analysis, a ground surface absorptivity of 0.75 was adopted for Cases 1 to 3. This value was reduced to 0.5 for Case 4 to consider the building shading effects for internal thermopiles.

The thermopile conductance was estimated based on an approach recommended by Yarmak (2012), the adopted thermopile radiator section geometry, and a design wind speed during winter at Inuvik. The measured mean wind speed at Inuvik ranged from 7.3 to 10.3 km/h during the period of October to April when the mean air temperatures are below the freezing point. The most frequent wind during the winter period at Inuvik is from the east. The longitudinal direction of the new detachment building is oriented in the east-west direction, which may reduce the speed of the wind flowing around the thermopiles below the new building. For this study, a design average wind speed of 4 km/h during the winter period was adopted in calculating the thermopile conductance.

The following parameters for the radiator section of each thermopile were adopted in calculating the thermopile conductance for the geothermal analysis:

- Effective length of the radiator section: 0.8 m
- Outside diameter of the radiator section of the steel thermopile: 219 mm (8.625”)
- Inside diameter of the radiator section of the steel thermopile: 203 mm (7.981”)
- Four sets of steel helix fins welded within the effective radiator section
- Vertical spacing of 76.2 mm (3”) for the helix fins
- Each helix fin with dimensions of 320.7 mm OD x 304.8 mm pitch x 6.35 mm thick (12.625” OD x 12” pitch x 0.25” thick)

4.7 Thermal Analysis Results

The predicted ground temperatures around a thermopile or a group of thermopiles vary with the thermopile location, vertical depth, and time. As an example, Figure 2 shows predicted ground temperature profiles on selected dates in the area around a thermopile for the Case 1 geothermal analysis.

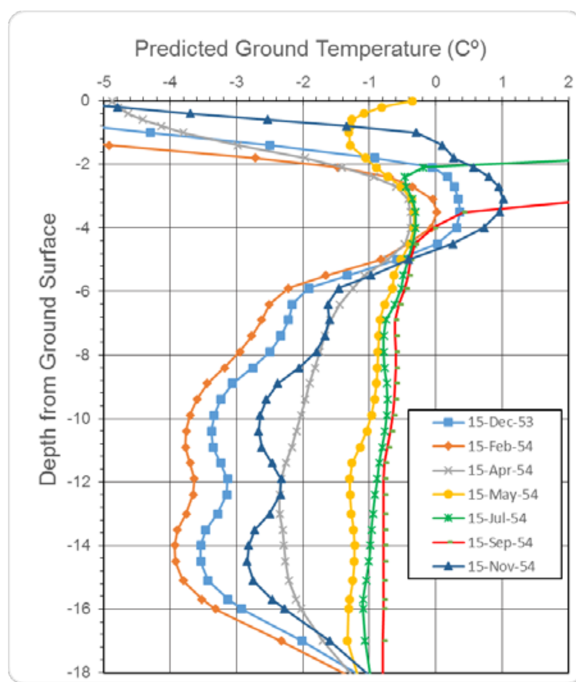

Figure 2. Predicted ground temperature profiles with time

4.8 Estimated Average Ground Temperatures for Geotechnical Evaluations of Thermopiles

Based on the thermopile layout plan provided, the thermopiles were classified into four categories for the geothermal analyses in this study. They are as follows:

- I: Corner thermopiles;
- II: External thermopiles in a row with a pile spacing of approximately 4.2 m to 5.6 m;
- III: External thermopiles in a row with a pile spacing of approximately 1.8 m to 3.5 m; and
- IV: Internal thermopiles with a maximum equivalent pile spacing of 4.4 m.

The geothermal analysis results from Cases 1 and 2 were used to estimate the ground thermal conditions around the corner thermopiles. The geothermal analysis results from Cases 1 to 3 were used to estimate the ground thermal conditions around the external thermopiles along the perimeter of the building foundation. The ground thermal conditions around the internal thermopiles were estimated based on the thermal analysis results from Cases 2 and 4. Engineering judgement was involved in estimating the average ground temperatures.

5 GEOTECHNICAL EVALUATIONS

5.1 Long-Term Pile Creep Settlement

The design of pile foundations in ice-rich permafrost is normally governed by long-term creep (serviceability). Pile design on an allowable settlement basis is commonly performed based on the secondary creep phenomenon to define the relationship between strain, time, and stress level. The design of an adfreeze pile in permafrost depends on the ground temperature, porewater salinity, and other geotechnical properties of the frozen soils around the pile. A thermopile is essentially an adfreeze pile, but with a heat removing capacity in winter periods when the air temperature is colder than the ground temperature.

The secondary creep settlement rate of a pile in ice-rich permafrost can be calculated based on the approach presented in Weaver and Morgenstern (1981).

5.2 Secondary Creep Parameters

To date, there have been few published measurements of creep parameters for warm, ice-rich frozen soils. The authors have used the available data for creep rates of ice and ice-rich permafrost to estimate a relationship between the B parameter, temperature, and salinity. The creep parameter n was assumed to be 3. Table 2 presents the values for the B parameter used in the thermopile creep settlement estimates in this study. A porewater salinity of 5 ppt was adopted. In reference to Table 2, the creep constants for various soil depth ranges, time periods, and pile categories can be interpolated based on the estimated ground temperatures.

Table 2. Ice-rich soil creep parameter B values adopted for thermopile creep settlement estimate

Average Ground Temperature (°C)	B (kPa^{-3}/year)
-0.5	8.0E-7
-1.0	1.8E-7
-2.0	5.9E-8
-3.0	3.2E-8
-4.0	2.3E-8
-5.0	1.7E-8
-6.0	1.4E-8

5.3 Minimum Pile Embedment Length to Limit Long-Term Creep Settlement

Due to various uncertainties in the thermopile design, installation, and operation, a factor of safety (greater than 1.0) is typically applied to determine the allowable pile shaft shear stress, which in turn determines the pile design load capacity or pile embedment depth. The shear stresses and settlement behaviour were considered at three effective diameters:

- The outside of the steel pile, neglecting the helix;
- An equivalent pile with an outside diameter of the helixes; or
- An equivalent pile with an outside diameter of the pre-drilled hole, rationalizing that the granular backfill in the annulus was likely to have a higher strength than the adjacent native soil/ice.

For this study, a different factor of safety for the allowable pile shaft shear stress was applied for each of the scenarios in consideration of the different risks and uncertainties associated with the scenarios. Table 3 presents the equivalent pile outside diameter and factor of safety for each of the scenarios in the geotechnical evaluations of the thermopiles.

Table 3. Factors of safety against shaft shear stress for thermopile creep settlement evaluation

Thermopile Settlement Scenario	Equivalent Outside Diameter (mm)	Factor of Safety against Shaft Shear Stress
S1: A pile without helix	219	1.2
S2: An equivalent pile with an outside diameter of the helixes	320	1.5
S3: An equivalent pile with an outside diameter of the pre-drilled hole	457	1.8

Table 4 summarizes the estimated minimum thermopile embedment lengths. These values were estimated based on the above-mentioned design assumptions and the following criteria:

- The maximum total thermopile creep settlement of 50 mm within 50 years of the building service life;
- The maximum thickness of the active layer of 3.0 m for pile load capacity calculation; and
- The unfactored maximum long-term service loads, as presented in Table 4.

Table 4. Estimated minimum thermopile embedment length required to meet creep settlement criteria under long-term service loads

Pile Axial Service Load Category Provided	Thermopile Category for Thermal Analysis			
	I	II	III	IV
P1 (50 kN)	8.5 m	N/A	7.3 m	N/A
P2 (100 kN)	11.2 m	9.6 m	8.9m	8.5 m
P3 (150 kN)	14.1 m	12.0 m	10.9 m	9.9 m
P4 (200 kN)	N/A	14.5 m	13.2 m	11.8 m
P5 (250 kN)	N/A	N/A	N/A	13.6 m

5.4 Minimum Pile Embedment Length against Frost Heave

Seasonal freezing of the active layer may impart potential heaving forces upon a conventional pile. It is understood that the thermopile can modify the frost action and reduce or eliminate frost jacking. The ground around the thermopile will freeze radially or horizontally along its entire evaporator section. As a result, the freezing front in the active layer tends to be vertical rather than horizontal, which will reduce the vertical uplifting frost jacking forces. The extent to which this is the case has not been quantified to our knowledge and past experience. Therefore, it was considered to be prudent to still consider frost-jacking resistance in the determination of pile embedment. A reasonably low factor of safety can be adopted in consideration of the low risk of frost jacking for the thermopiles.

The Canadian Foundation Engineering Manual (CFEM 2006) suggests that design adfreeze bonds for saturated gravel frozen to steel piles can be estimated at 150 kPa.

The following parameters were assumed for estimating the frost jacking force:

- Adfreeze bond stress between the backfill and thermopile surface: 150 kPa;
- Active layer thickness: 3 m;
- The outside diameter of the thermopile within the active layer (no helix in this zone): 0.219 m;
- The estimated maximum sustained frost jacking force is 310 kN.

The minimum thermopile embedment lengths against the frost jacking force were estimated based on the above-mentioned thermal analysis predictions and the following design criteria:

- The maximum cumulative total frost-inducted uplift deformation of 50 mm within 50 years of the building service life for each thermopile;
- The maximum frost-induced uplift deformation of 6 mm for a single frost cycle (6 months assumed);
- The design uplifting frost force of 310 kN;
- The adopted factor of safety against frost heave deformation; and
- Ignoring any vertical dead load on thermopile for the evaluations.

The owner of the proposed building consulted with an external expert who has experience on the performance of thermopiles installed in Alaska. The expert indicated that the risk of thermopile frost jacking is low. As a result, the factor of safety against frost heave deformation was adopted to be 1.0 for this study, which is relatively low for typical engineering design. This factor was applied in this study to estimate the minimum thermopile embedment length against frost jacking for the most likely deformation scenario (S2 in Table 3). The equivalent factor of safety would be less than 1.0 for the S1 scenario and greater than 1.0 for the S3 scenario. Table 5 summarizes the estimated minimum thermopile embedment lengths against the frost heave.

Table 5. Estimated minimum thermopile embedment lengths against the frost heave

Thermopile Category for Thermal Analysis	Minimum Thermopile Embedment Length against Frost Heave (m)
I	12.3
II	11.6
III	10.6
IV	10.1

5.5 Minimum Pile Embedment Length to Withstand the Pile Design Axial Load

The minimum pipe embedment length of an adfreeze pile in ice-rich permafrost is seldom controlled by the pipe load carrying capacity to withstand the pile design axial load without shear failure. As a confirmation check-up, the minimum pipe embedment lengths to withstand the pile design axial loads were estimated, as presented in Table 6.

Table 6. Estimated minimum thermopile embedment length to withstand pile design axial load

Pile Axial Service Load Category (provided by Structural Engineer)	Minimum Thermopile Embedment Length to Withstand Pile Design Axial Load (m)
P1 (50 kN)	6.2
P2 (100 kN)	6.8
P3 (150 kN)	7.2
P4 (200 kN)	7.7
P5 (250 kN)	8.1

The values in Table 6 were estimated based on the following assumptions:

- The warmest predicted ground temperature profile around the thermopiles during the early stage of building operation;

- Equivalent shaft diameter of 0.219 m for 0 to 5.5 m below the ground surface and of 0.32 m for below 5.5 m;
- An effective friction angle of 32° for unfrozen silty sand/gravel (0 to 5.5 m);
- Long-term cohesion for permafrost soils (below 5.5 m) adopted based on Weaver and Morgenstern (1981) and predicted warmest ground temperatures;
- Ignoring the end-bearing load capacity of the thermopiles; and
- A factor of safety of 1.5 for the load capacity.

6 GEOTECHNICAL DESIGN PARAMETERS FOR THERMOPILES

Three design groups were adopted to optimize the pile designs while the total number of thermopile types is minimized, to facilitate thermopile manufacture and installation. Table 7 summarizes the recommended design parameters for each design group of the thermopiles. The design parameters were adopted based on the results of the geotechnical evaluations in Section 5, the pile layout plan, and design axial service loads that were provided by the structural engineer. Figure 3 presents the recommended thermopile geotechnical design details for the three design groups. A minimum pile center-to-center spacing of 1.4 m was recommended to reduce a potential negative influence of thermopiles on each other. Figure 4 presents the plan layout of the thermopiles and proposed ground temperature cable locations.

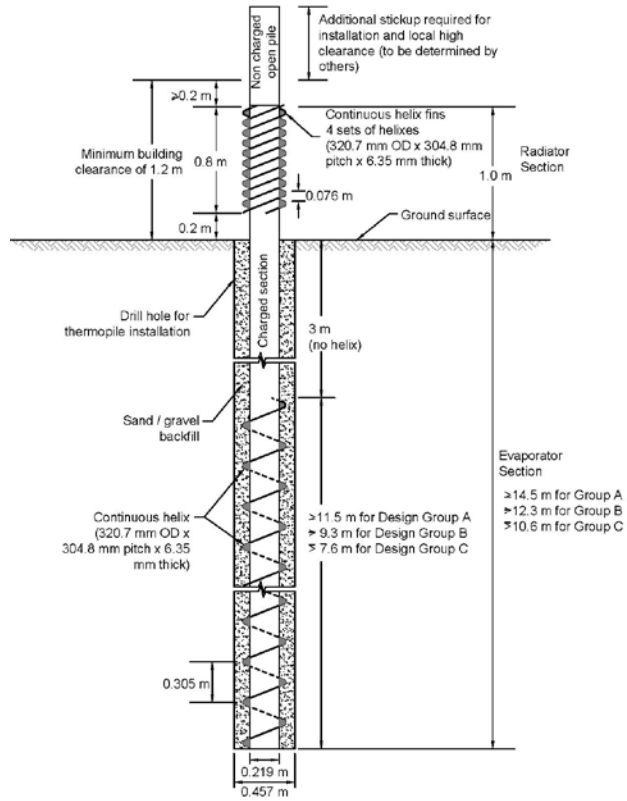

Figure 3. Recommended thermopile design details

Table 7. Recommended thermopile design groups and parameters

Design Group for Thermopiles	Pile Axial Service Load Category (P1 to P5) & Thermopile Category (I to IV) for Thermal Analysis	Total Number of Thermopiles	Minimum Thermopile Embedment Length below Ground Surface (m)	Design Length of Radiator Section above Ground Surface (m)	Radiator Section with Helix Fins
A	P3 & Category I, P4 & Category II, P4 & Category III, or P5 & Category IV	62	14.5	1.0	0.2 m to 1.0 m from ground surface
	P1 or P2 & Category I, P2 or P3 & Category II, P3 & Category III, or P4 & Category IV	36	12.3	1.0	0.2 m to 1.0 m from ground surface
C	P1 or P2 & Category III, or P2 or P3 & Category IV	30	10.6	1.0	0.2 m to 1.0 m from ground surface

7 SUMMARY

A thermopile foundation has been designed for a new building in Inuvik. A set of geothermal analyses were carried out to estimate the building foundation ground temperatures with depth and time under normal working conditions of the thermopiles and climatic conditions considering projected climate changes over the building design life of 50 years. Geotechnical evaluations were conducted to estimate the required pipe embedment depths in consideration of specified pile long-term creep settlement criteria, design criteria against potential frost jacking, and pile design axial loads. A total of three design groups of thermopiles were recommended for the building foundation.

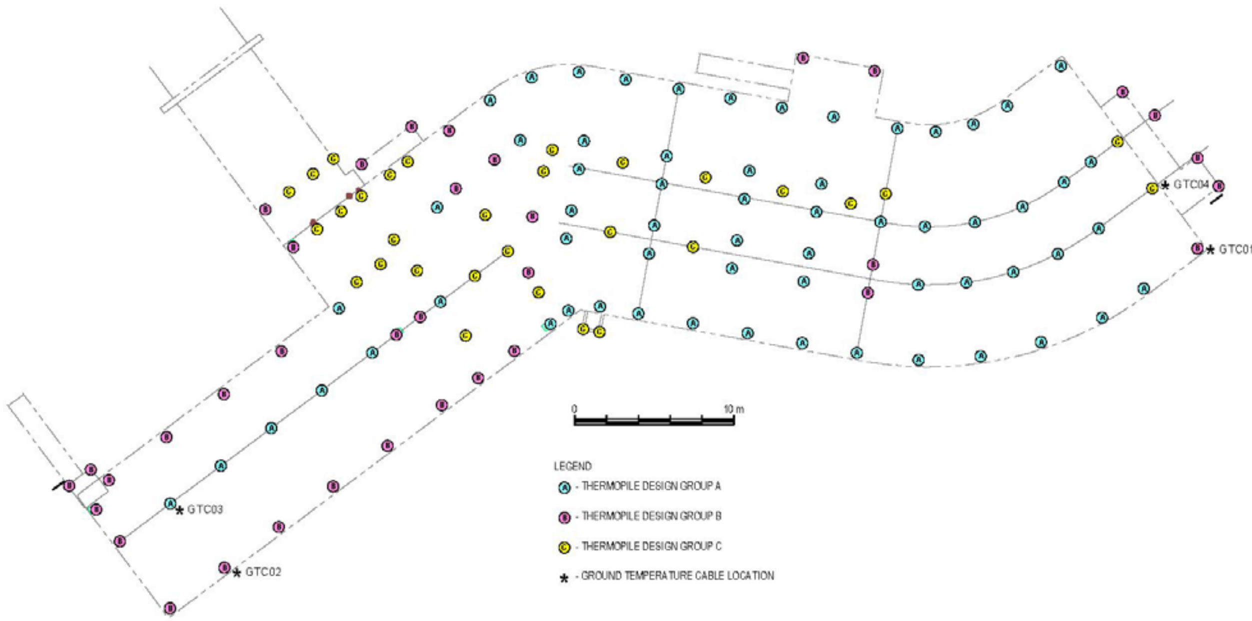

Figure 4. Plan layout of the thermopiles and proposed ground temperature cable locations

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