Foundation Design in Talik Zone for a New School, Fort Good Hope, NT



Entzu Hsieh *AMEC Earth & Environmental, Calgary, Alberta* Dmitry Dumsky *AMEC Earth & Environmental, Calgary, Alberta* Ramesh Koirala *Public Works and Services, Government of the Northwest Territories*

ABSTRACT

The community of Fort Good Hope is located within the continuous permafrost zone. However a talik was encountered at the construction site. The permafrost table below the talik was found to be at depths from about 15 m to 17 m. A steel H-section pile foundation was proposed to support the new structure. PDA tests were conducted to confirm the pile resistance. Since the majority of the pile tips are positioned above or near the permafrost table, thawing of the permafrost or freezing of the talik below the school due to thermal impact of the building could result in differential settlement/heave of piles and cause damage to the school. As such, a skirted air gap equipped with adjustable ventilation windows was provided to control the circulation of cold air under the structure during winter months. A ground temperature monitoring program is implemented to collect ground temperature during operation of the school. Based on the temperature data collected, any necessary recommendations to adjust the opening area of the ventilation windows will be provided.

RÉSUMÉ

La Communauté de Fort Good Hope est située dans la zone de pergélisol continu. Néanmoins, un talik a été rencontré sur le site. Le pergélisol sous le talik se situe à une profondeur de 15 m à 17 m environ. Des fondations profondes constituées de pieux « H » en acier ont été proposées pour supporter la nouvelle structure. La capacité portante des pieux a été confirmée par des essais dynamiques à l'analyseur de battage de pieux (PDA). Étant donné que la pointe des pieux se situe généralement au-dessus ou près du pergélisol, la fonte de ce dernier ou encore le gel du talik présent sous l'école peuvent engendrer des tassements ou des soulèvements différentiels susceptibles d'endommager la structure. À cet effet, le bâtiment est surélevé par rapport au sol et pourvu d'un vide muni de fenêtres permettant de contrôler la circulation d'air froid pendant les mois d'hiver. Un programme de suivi a été mis en place pour recueillir des données sur la température du sol pendant l'opération de l'école. Sur la base des données recueillies, des recommandations pour ajuster l'ouverture des fenêtres contrôlant la ventilation seront fournies, si nécessaire.

1. INTRODUCTION

The existing Chief T'Selehye School in Fort Good Hope, Northwest Territories is currently approaching to the end of its operational life. The Public works and Services, Government of the Northwest Territories proposed a new school to serve kindergarten to grade 12 students and will replace the existing school. The new building is a single-storey steel frame structure with design load varying from 60 kN to 510 kN in each column. The project has a design build contract valued at \$20 million.

The community of Fort Good Hope is located within the continuous permafrost zone as shown in Figure 1 (Geological Survey of Canada). The permafrost table at the construction site was expected to be at a depth of 1 m to 2 m below the ground surface. However, the results of field investigation revealed that a talik exists on site and the permafrost table is located at depths from about 15 m to 17 m with permafrost temperatures varying from -0.1 °C to -0.6 °C. Such permafrost conditions are

considered to be unusual for the community of Fort Good Hope. A driven steel pile foundation was proposed to support the new structure. The pile resistance and blow count criteria during installation were assessed by Pile Driving Analyser (PDA) testings. In order to stabilize the current permafrost table after construction, a skirted air cap between the bottom of the structure and the ground surface was designed. The skirted air was equipped with adjustable ventilation windows providing a way to control the circulation of cold air under the structure. A ground temperature monitoring program is implemented to collect ground temperature to a depth of 24 m during operation of the school.

2. SITE CONDITION

Fort Good Hope is a community of 800 people and located at about 800 km northwest side of Yellowknife. The construction site is located at the north side of the old runway and is approximately 250 m by 250 m in size.

The new school building is situated at the southwest side of the construction site and occupies a gross area of $2,800 \text{ m}^2$. Figure 2 illustrates the location of the site in the community of Fort Good Hope.

The site prior to and during the investigations was occupied by Nav Canada. A signal tower and its associated facilities were located near the central area of the site. The site topography was generally flat, with slightly higher areas located in the northeast and southwest parts of the site. A gentle slope is located at the northwest portion of the site. Two small depressions were observed. The first depression was located near the south end of the site, and the second depression was identified to the east of the existing Nav Canada Tower.

Evidences of seasonal frost heave were observed in the southeast corner of the site, forming small heave hummocks, up to 0.5 m in height, and about 1 m in diameter.



Figure 1. Location of Fort Good Hope and Permarfost Map



Figure 2. Location of the construction site

3. REGIONAL GEOLOGY

The community of Fort Good Hope lies between Jackfish Creek and the east bank of the Mackenzie River where the two water courses meet. Based on the published surficial geology information (Duk-Rodkin and Hughes, 1992), the majority of the community is located on a glaciofluvial terrace, while the northern portion of the community is situated on an undivided moraine/outwash plain). The moraine/outwash plain deposits consist of sand and gravel, with a veneer of eolian silty sand or sand. Glaciofluvial terrace deposits are noted to consist of sand and gravel, with silt and peat in some channels, and are noted to be 2 to 30 m thick.

4. CLIMATE & PERMAFROST

Climate records for the community of Fort Good Hope are not available. Mean data from the Inuvik weather station and Norman Wells weather station were used to evaluate the approximate thawing and freezing indices for the community. The approximate thawing and freezing indices were estimated to be about 1615 degree-days and 3990 degree-days, respectively. The mean annual air temperature was estimated to be about $-6 \degree C$ to $-7 \degree C$.

The changes of the mean monthly air temperatures for a northern portion of the Mackenzie River valley was reviewed based on data obtained from the Inuvik weather station from 1971 to 2000 and 2001 to 2008. Figure 3 below presents a comparison of the monthly air temperatures in Inuvik for the period from 1971 to 2000 and 2001 to 2008. It can be seen that there is a warming trend (about 2° C to 4° C) between September and March. Similar phenomenon was also observed in monthly air temperatures for data from Norman Wells weather station.





Based on previous work in the community, the thickness of the active layer varied from about 0.5 m to 2 m, depending on ground vegetative cover and surface disturbance. Ground temperature data within the community suggest that the range of mean ground temperature is from about -1.0 °C to -2.0 °C at a depth of about 10 m. However, the construction site is characterized by a mean annual soil temperature in an order of +0.5 °C to +2 °C.

5. SITE INVESTIGATION

During preparation of the drilling, the layout of the proposed structure within the site was not available. The advanced boreholes were located at the east, south and northwest corners of the site based on access availability and utility information.

The initial drilling program was carried out on February 20 and 21, 2007. A total of ten boreholes were drilled at the site to depths ranging from 5 m to 12 m using an air down hammer, track mounted drill rig. ABS thermistor conduits were installed in selected boreholes. The approximate locations of boreholes are presented in Figure 4.

A laboratory testing program was conducted on selected borehole samples to determine the natural water content and grain size distribution.



Figure 4. Locations of boreholes

6. SUBSURFACE CONDITIONS

6.1 Subsoil Conditions

Based on results of the drill program, the general soil profile encountered at the site consisted of a thin organic layer over sand which was underlain by silt. In the southwest corner of the site (BH-01 and BH-02), a gravel fill layer, 0.5 m to 0.7 m thick, was present above the native sand.

The organic layer was up to 0.2 m thick over the site. Sand was encountered in all boreholes under the organic layer or gravel fill. The sand was brown, coarse to fine grained, contained some gravel, trace to some silt and was wet. Moisture contents varied between 16 and 22 percent. The thickness of the sand ranged from 4 m to 9 m.

Silt, underlying the sand layer, was found in all boreholes with exception in the northwest corner of the site which was terminated in the sand. The silt was grey, contained some clay and sand, and inclusions of gravel. Moisture content of the silt was about 15 percent.

6.2 Groundwater Conditions

Groundwater levels in the construction site measured in standpipes (BH-03 and BH-06) were at depths of 1.4 m and 0.7 m, respectively. Groundwater was also encountered at similar depths during drilling in other boreholes.

6.3 Permafrost

Permafrost was not encountered in any of the initial boreholes drilled, which is considered unusual. Ground temperatures were measured on March 7, 2007 (14 days following completion of the drilling) in boreholes where thermistor conduits were installed. Results of the temperature measurements are provided in Table 1.

Borehole	1	4	5	8	9		
Depth (m)	Temperature (°C)						
1	-8.2	-8.8	-0.2	-2.6	-10		
2	-1.1	-0.3	0.6	0.5	-2.5		
3	0.2	0	1.2	1.1	0.3		
4	0.4	0.4	1.3	1.6	1.1		
5	0.6	0.5	1.4	1.7	1.7		
6	0.6	0.6	1.5	1.8	2.0		
7	0.7	0.8	1.7	1.8	2.0		
8	0.5	0.6	1.5	1.7	2.0		
9	0.4	0.5	1.4		1.9		
10	0.3	0.4	1.3		1.7		
11	0.2						

Table 1. Ground Temperature Data

It was found that soil temperatures below the seasonal frost penetration depth were above 0°C. The ground temperatures were colder in the east side of the site (BH-01 and BH-04), while the ground temperatures in the south side of the site (BH-05, BH-06 and BH-08) were warmer. The pattern of the ground temperature distribution with depths suggests that permafrost may be present at about 18 m to 20 m below the ground surface.

7. ADDITIONAL INVESTIGATION

Since permafrost was inferred to be found at depths based on extrapolation of temperature data obtained within the upper 12 m depth of site, three additional boreholes were proposed and advanced at the site in July 2008 after a layout of the school footprint was available. The boreholes were advanced to a maximum depth of 20 m within the new school footprint using a truck mounted down-hole hammer drill rig. ABS conduits, 50 mm in diameter, were installed in all three boreholes to allow for measurement of ground temperature. The results of drilled holes revealed similar soil conditions as observed during the initial investigation. The temperature monitoring confirmed that the permafrost table is located at a depth of about 17 m and the permafrost temperature below this depth ranged from -0.1 °C to -0.3 °C.

Groundwater levels within the building footprint were measured at 1 m to 3 m below the ground surface, which were slightly lower than that from previous drilled hoes. It is expected that the groundwater level could be higher based on conversations with local residents who noted that it was not possible to drive a truck across the site in the summertime.

8. GEOTECHNICAL DESIGN

Based on geological and hydrogeological conditions described above, driven steel piles were considered to be a feasible foundation system for the new school. Due to potential existence of gravel and boulders or frozen layer at various locations, H-section piles were selected. Also each pile was equipped with a driving shoe to protect the pile tips against damage.

Since permafrost is encountered at depths below the foundation, one of the key geotechnical issues is to maintain the current permafrost level and avoid potential for thawing of permafrost or freezing of unfrozen materials. This issue was considered using a skirted air gap with adjustable ventilation windows in the skirt.

8.1 Driven Steel Pile

Shaft friction values were derived using effective stress approach as outlined in the CFEM (2006). It is expected that the piles would be embedded in silt stratum, a β value of 0.3 was applied to estimate the ultimate skin friction values. The end bearing resistance was ignored in the pile design.

The deformation of piles was expected to be less than 5 mm for soil encountered on site. Seventy percent of the shaft friction values were used in design of the driven steel piles in assessing the pile tensile capacity. Group effect was not considered in the design as the majority of pile spacing was greater than eight pile diameters.

Based on the design load in each structural column, the structural engineer completed the pile design and summarized as follows:

- HP 200x54 for P1 pile type and HP 310x79 for all others (P2, P3, P4 and P5 pile types);
- H piles to be conformed to ASTM A36 (f_y=250 MPa);
- depending on loadings, pile lengths varying between 9 m to 20 m from the ground surface;
- piles to be driven to the required blow counts, determined by field geotechnical personnel based on the results of load tests.

8.2 Skirted Wall with Adjustable Windows

In order to minimize the potential thermal impact from the heated structure on the existing permafrost, typical design is to provide an air gap between the bottom of the structure and the ground surface. However, the absence of snow in the air gap and shelter effect may cause a significant decrease in the mean annual soil temperature under the building, and hence result in aggradation of permafrost in the current unfrozen soil. As such, the air gap between the structure and the final grade should be skirted, but with openings which provide air ventilation to maintain the current location of the permafrost table.

The area of the openings is a function of such parameters as the structure room temperature. dimensions of structure, floor thermal resistance, and skirt wall thermal resistance. The ambient air temperature and soil thermal parameters also have impact on the area of openings. An empirical relationship (SNiP, 1989) using the above parameters and soil temperature was used to predict the total area of openings along the skirted wall of the building. The area was estimated to be 2 m² to 4 m². The required openings were spread evenly along the six skirted walls of the building via ventilation windows. It is understood that the empirical relationship is an approximate solution and adjustable windows will be required to control the amount of circulation of cold air under the structure. On this basis, extruded aluminum brick vents with opposed blade damper, 197 mm by 1219 mm in size, were adopted for the new building. Figure 5 presents the section of the extruded aluminum brick vent. The insect screen of the vent will be removed after installation to allow better circulation. The adjustable flap damper behind the windows provides a way to adjust the area of openings, where necessary.



Figure 5. Extruded aluminum brick vent in skirted wall

8.3 Frost Heave

During winter period, it is anticipated that the subgrade soil will be frozen to about 1.5 m to 2 m depth. Frost heave (jacking) forces on the foundation piles would develop as the result of the freezing of saturated soils around piles. For long term consideration, the resistance to frost heaving forces can be provided by the shaft friction below the depth of frost penetration and by sustained vertical load. Hence a bond breaker to reduce the frost heaving force is not required. For short term condition (during the first winter after pile installation and prior to completion of the new structure), since all piles were driven to depth deeper than the minimum embedded length to resist frost heave, polyethylene sheets with heavy grease applied over the exposed surface of the pile within the active zone were not required.

The skirted wall, having contact with the ground surface will be subject to the action of frost heave. Frost heave pressure under the skirt can be significant. Thus, a 200 mm thick void forming product between the soil and the underside of the skirt was provided to reduce the pressure.

8.4 Gravel Pad

In order to provide both accessible and stable working conditions for construction equipment, a 500 mm thick gravel pad was constructed within the building footprint. The construction was started in August 2008 prior to pile installation.

Shrubs and small trees were firstly removed to expose organic/native surficial soil. A layer of non-woven polypropylene geotextile was placed over the native soil with overlapping a minimum of 500 mm for separation purpose. Gravel was placed in approximately 300 mm thick lifts, onto the geotextile and spread with a bulldozer. The gravel available on site is well graded, 75 mm minus sand and gravel. Depending on the gravel moisture, either compaction via smooth drum vibratory roller was undertaken or the gravel was allowed to dry, upon which it was compacted with vibration.

Compacted areas were then proof-rolled with a fully loaded gravel truck to check gravel deflection. If the deflection of the gravel was minimal (typically less than 12 mm), the gravel was considered adequately compacted. If additional gravel was needed to bring the area to the required grade(s), another lift of gravel was placed and the placing, compacting, and proof rolling sequence was repeated.

9. PILE INSTALLATION AND PDA TEST

Driving of piles was carried out by Delmag D19-32 single acting diesel hammer. During driving, the hammer was operated in the highest energy setting (58 kJ). The pile installation began on April 29, 2009, and finished on June 10, 2009.

A preliminary blow count criteria was provided to the contractor using the result of WEAP analysis based on pile properties, assumed efficiency of the proposed hammer and soil parameters. The blow count criteria were re-established after completion of the PDA tests. The pile type, test load and acceptable criteria are shown below in Table 2. Due to the low design load, no refusal criteria were used for type P1 piles, however a minimum embedment length of 9.0 m was required to withstand frost heave forces. The results for P2 and P3 from PDA tests were judged to be questionable. Hence, WEAP

analysis was used and calibrated using the results obtained from CAPWAP for P4 and P5. The blow count criteria for P2 and P3 were derived from the results of WEAP analyses.

The accumulated shaft resistances along test piles obtained from CAPWAP were compared to the estimated shaft resistances using effective stress approach as assumed during the design stage. Figure 6 presents the results of comparison. It was found that the effective stress approach lies below the lower bound values derived from CAPWAP analyses when the pile length is longer than 10 m.

Table 2. Summary of Acceptable Criteria

Pile	Design	Test		
Type	Load	Load	Criteria	Remarks
. , po	(kN)	(kN)		
P1	60	120	Min. 9 m	
P2	240	480	20/250 mm	WEAP
P3	300	600	25/250 mm	WEAP
P4	400	800	28/250 mm	CAPWAP
P5	510	1050	42/250 mm	CAPWAP



Figure 6. Comparison of CAPWAP analyses with effective stress approach

10. CONSTRUCTION ISSUES

During installation, boulders were encountered in some areas, especially in the southern portion of the building. No drilled hole was conducted in this area at the time of investigation. As per conversation with local people, this area might have been used for fill placement and piles typically hit refusal at 1.5 m to 2.5 m. After discussion with the contractor, it was decided to excavate down to 2 m to 2.5 m depth at some selected pile locations. All encountered rock fragments were removed and the excavations were backfilled with the excavated materials and compacted to satisfaction of field representative. Then the pile was driven at the surveyed location to achieve acceptable criterion.

Several piles were out of plumbness (1%) during piling. The contractor and structural engineer decided that these piles were unusable, therefore they were cut and abandoned. Two additional piles were installed on either side of the original piles.

For piles did not reach the initial driving criteria, the piles were re-struck a few days later. Piles which reached the criteria on re-strike were then accepted. It was found that the re-strike blow count could be increased by more than 50%. If the piles were out of plumb and could not be re-stricked, the pile was abandoned and two other piles at about 1 m spacing from the original pile were installed on either side.

11. TEMPERATURE MONITORING SCHEDULE

Driving records compiled during pile installation indicate pile embedment lengths varying from about 9 m to 16 m, suggesting that most of the pile tips are positioned above or close to the permafrost table. As discussed previously, thawing of the permafrost below pile tips due to thermal impact from the building could result in differential settlement and cause damage to the structure. On the contrary, the aggradation of permafrost will create uplift loads on piles, lead to formation of ice-rich material under the structure and differential heaving of the structure.

On this basis, a temperature monitoring program was proposed to collect ground temperature data to a depth of 24 m over the first 3 to 5 years of building operation. Based on a regular assessment of the temperature data any necessary recommendations for adjustment of ventilation window area can be provided, such that no aggregation or degradation of the permafrost will develop under the building. To procure sufficient temperature information, three locations including centre, north end and south end of the building were selected for temperature monitoring. Thermistor cables were installed in the boreholes and extended into a defined location inside the building.

The first series of temperature measurements were undertaken on June 11, 2009 following the completion of pile installation. The measured temperatures corresponding to depths below the ground surface are presented in Table 3. The permafrost temperatures ranged between -0.1 °C and -0.6 °C. No noticeable shift of the permafrost table was observed since temperature measurements were taken in July, 2008 during the additional geotechnical investigations.

Table 3: Temperature Monitoring

Location	MB-1	MB-2	MB-3
Depth (m)	Temperature (℃)		
4	0.1	0	0.3
5	0.3	0.2	0.4
6	0.4	0.2	0.5
7	0.5	0.4	0.5
8	0.6	0.5	0.6
9	0.6	0.6	0.6
10	0.6	0.4	0.5
11	0.4	0.4	0.3
13	0	0.3	0
15	-0.2	0.3	-0.1
16	-0.2	0.1	-0.2
17	-0.3	0	-0.3
18	-0.3	-0.1	-0.3
19	-0.3	-0.2	-0.4
20	-0.4	-0.4	-0.4
21	-0.4	-0.3	-0.4
22	-0.5	-0.4	-0.5
23	-0.6	-0.5	-0.5
24	-0.6	-0.5	-0.5

The recommended schedule for the temperature measurements and reporting during school construction and operation is presented in Table 4. Based on the monitoring information, recommendations for adjustment of the area of ventilation windows will be determined and provided to Government of the Northwest Territories (GNWT) if the ground temperature readings indicate that significant change of the permafrost table has occurred. It is expected that the ground temperatures will achieve an equilibrium state in a few years and monitoring program will be completed after 3 to 5 years of the temperature monitoring.

Table 4. Temperature Measurements

	Schedule		
Phase	Temperature Measurements	Reporting	
Construction	Monthly	End of Construction	
First Year of Operation	Monthly	End of First Year	
Second Year through Fifth Year of Operation	Quarterly	End of Second, Third, Fourth and Fifth Year.	

12. SUMMARY

Permafrost table was found in the construction site at depths ranging from about 15 m to 17 m with permafrost temperatures varying from -0.1 °C to -0.6 °C. Such permafrost conditions are considered to be unusual, taking into account that the location of Fort Good Hope is within the continuous permafrost zone. A driven steel H-section pile foundation was proposed to support the new structure based on encountered soil and permafrost conditions. The pile resistance and blow count criteria during installation were evaluated by PDA tests. In order

to stabilize the current permafrost table after construction, a skirted air gap equipped with adjustable ventilation windows was designed and a temperature monitoring program was implemented to collect ground temperature to a depth of 24 m during operation of the school. Based on the temperature data collected, any necessary recommendations to adjust the size of the ventilation windows will be provided.

13. ACKNOWLEDGEMENTS

The Authors acknowledge the support of our colleagues Dr. A. Tchekhovski, P. Eng. who was involved in design and geothermal modeling of the foundation, and Mr. K. Spencer, P. Eng. who provided valuable input during design and construction phases of the project. Field support was provided by Ms. K. Hincks, P. Geol and Mr. R. Mateff, E.I. T. We would like to thank personnel of the Department of Public Works and Services, Government of the Northwest Territories, who helped in organization of field investigation and development of monitoring program for this project.

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