Geotechnical Deep Foundation Design Challenges in Discontinous Permafrost of Northern Manitoba

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Challenges from North to South
Des défis du Nord au Sud

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ABSTRACT
Geotechnical foundation design challenges associated with the climatic and geotechnical conditions pertinent to northern Manitoba are presented and discussed. The challenges associated with a predicted warming trend for the degradation of frozen ground and its impact on the geotechnical foundation design was, in particular, a challenge at the Keewatinohk Converter Station site. Due to remoteness and cold climate at the site, the project owner's preference was for a relatively long service life and reduced maintenance during operation. The challenges included establishing a design freezing index, predicting and designing for frost depth and adfreeze forces. Driven steel tubular pipe piles were selected as a most feasible foundation option for the project. The paper also presents axial and lateral pile load and pile drivability analysis challenges and solutions, and construction considerations.

RÉSUMÉ
Les défis géotechniques de conception de fondations liés aux conditions climatiques et géotechniques concernant le nord du Manitoba sont présentés et discutés dans cet article. La tendance prédite du réchauffement, pour la dégradation des sols gelés, et son impact sur la conception de fondation étaient, en particulier, un défi au site Keewatinohk Converter Station. En raison de l'éloignement et du climat froid du site, les préférences du propriétaire du projet consistaient en une durée de vie de service relativement longue et un entretien réduit durant les opérations. Les défis incluaient l'établissement d'un indice de gèle de conception, la prévision et la conception de la profondeur de gel et des forces associées à celui-ci. Des pieux tubulaires en acier foncés ont été choisis comme meilleure option en termes de faisabilité pour le projet. Cet article présente également les chargements axial et latéral d'un pieu, les défis et solutions de l'analyse du fonçage de pieux, ainsi que les éléments à considérer lors de la construction.

1 INTRODUCTION

Northern Manitoba has recently seen a boost in hydroelectric power generation projects and with that a growing demand for infrastructure associated with power transmission. More than 3000 MW of hydroelectric potential could be developed in Manitoba, which includes 1380 MW at the Conawapa site, 630 MW at the Keeyask site, and 1000 MW at the Gillam Island site, all on the Lower Nelson River. Manitoba Hydro has embarked on a very ambitious project involving a new 500 kV high voltage direct current (HVDC) transmission line linking the northern power complex on the Lower Nelson River, the delivery system in Winnipeg and two converter stations, one on each end. The AC electricity generated is converted to DC for transmission through HVDC line to Winnipeg where the DC electricity is inverted back to AC electricity to feed into the delivery system. This HVDC transmission line is about 1400 km long.

This paper presents the geotechnical foundation design challenges associated with the Keewatinohk converter station (KCS) located at the generation end in northern Manitoba, about 80 km northeast of Gillam as shown in Figure 1. Keewatinohk is a remote site in the discontinuous permafrost and peat land classifications in the northern Hudson Bay Lowlands, south of Churchill. The other converter station at the terminus of the HVDC line in Winnipeg is called the Riel Converter Station.

The KCS site is spread over an area of about 1.0 km by 1.5 km. Construction activities began in 2013 with civil site preparation and development of camps. Foundation construction for the converter station is planned for late 2015 – early 2016. During the summer of 2014, the site was re-graded; peat, organics and vegetative cover was removed; deep open drains across the site were installed; and granular fill was placed. Hence, the ground surface conditions shall be considered levelled and well drained. The site has access to abundant and nearby good sources of granular borrow material (gravel pits).

Several foundation options were examined and tubular steel pipe piles were selected as the technically preferred alternative. This paper presents and discusses the geotechnical foundation design challenges associated with the climatic and geotechnical conditions pertinent to northern Manitoba. The challenges included designing foundations for transition from a discontinuous permafrost condition on northern portion of the site to seasonal frost conditions once the permafrost has degraded. The project's design consideration of longer service life and less maintenance during operation also had an impact on the design seasonal frost penetration depth and adfreeze forces. These impacts are discussed in detail with particular reference to Keewatinohk site climatic and geotechnical conditions and proposed foundation solution.

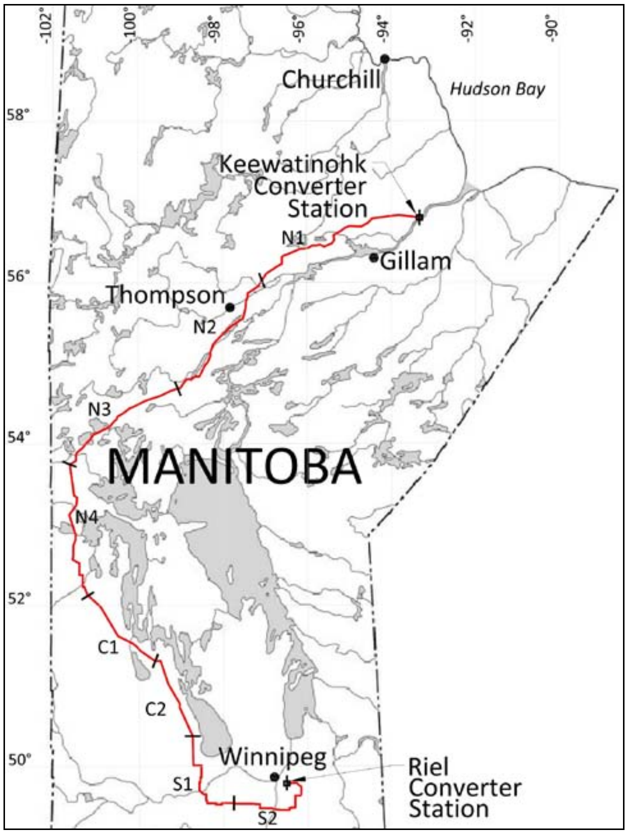

Figure 1. Location map showing HVDC transmission line and two converter station sites

2 SURFACE AND SUBSURFACE CONDITIONS

In general, the natural KCS Site conditions consist of swampy peat bog with discontinuous permafrost. Subsurface conditions comprise peat underlain by a variably ice-rich mixed zone of silt/clay/sand/till to several metres before encountering hard to very hard clayey till. The groundwater table is shallow and generally follows the peat/mineral soil interface. The subsurface conditions also include cobbles and boulders. Their size and frequency, and the correct strength assessment of the underlying hard till will be challenges to deal with for pile foundation installation and need to be considered during the design phase.

The existing geotechnical database for the KCS site included more than 100 test holes. These test holes were a compilation of several geotechnical investigation campaigns commissioned by Manitoba Hydro from 2009 to 2012: in 2009 (for site selection); in 2010 (preliminary investigations); 2011 (fire suppression investigation, 100 m deep boreholes) and 2011/2012 (foundation exploration). Some of the boreholes were instrumented with thermister strings (ground temperature monitoring) while some were instrumented with piezometers (groundwater level monitoring). Boreholes were logged for frozen soil classification. Laboratory testing included thaw consolidation testing. Some boreholes were drilled deep to 66.5 m (several metres into the bedrock). Standard penetration tests (SPT) at regular intervals were carried out in most of the test holes.

Within the KCS site is a large facility named the 230 kV AC Switchyard (KSY); the geotechnical database pertinent to this facility was further assessed for the foundation design development that is the subject of this paper. To further illustrate the subsurface conditions, the borehole layout plan within the KSY area is shown in Figure 2 and the site conditions prior to and during the site development works are shown by the photographs taken about a year apart in Figure 3 and Figure 4, respectively. A representative subsurface cross section through the longer axis of the KSY site area is shown in Figure 5.

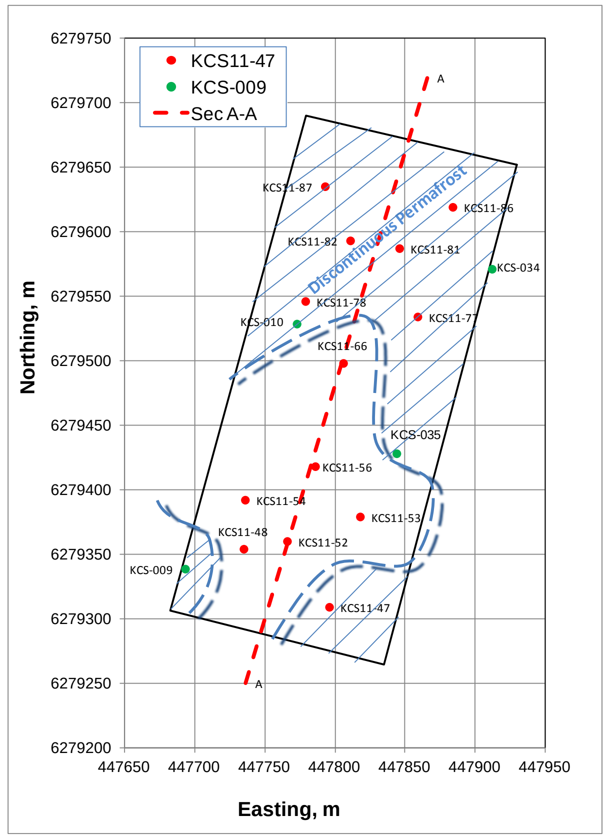

Figure 2. Keewatinohk switchyard area and borehole layout plan (with discontinuous permafrost delineated)

A number of pertinent subsurface property profiles were developed to support a detailed subsurface characterization and typically consist of the following:

2.1 Pit Run Gravel Fill

A 2 m compacted pit run gravel fill with occasional cobbles and boulders. Pit run gravel is well-graded material with limited fines that compacts well with several roller passes. The actual pit run gravel fill thickness is between 2 m to 3 m and locally thicker. Any portion of the fill below 2 m has been included with soil unit 2 below.

Figure 3. Predevelopment site condition July 2013

Figure 4. Site conditions during site works July 2014

2.2 Mixed Soil Layer

A 6 m thick variably frozen and locally ice-rich sand/silt/clay/till with occasional cobbles and boulders, designated as Mixed Soil. Mixed Soil is highly variable with materials ranging from silt to sand and predominantly frozen. SPT N-values range from 15 to more than 100 indicating medium dense to very dense sand or firm to very hard silt/clays. Moisture contents also exhibited a wide variability ranging from 8% to 30%. The frozen soil classifications varied from non/no-visible ice to visible ice.

2.3 Glacial Till

Glacial till layer exhibited very hard consistency across the site. Natural moisture contents were typically low at about ±10% and the material exhibited low plasticity. Till layer extended to 62 m depth, locally frozen to 12 to 14 m depth, occasionally with some discrete ice lenses. Occasional cobbles and boulders were present to about 11 m depth and then in increased frequency with increasing depth.

2.4 Bedrock

Bedrock is located at approximately 62 m depth, well below any foundation influence depth.

2.5 Groundwater

The groundwater table has been considered at 2 m depth (at about the gravel layer-mixed soil layer interface). However, this is a perched water table and impacts the soil above the till layer only and not the till layer since the till layer represents a confining layer for much deeper and permanent groundwater table in the region.

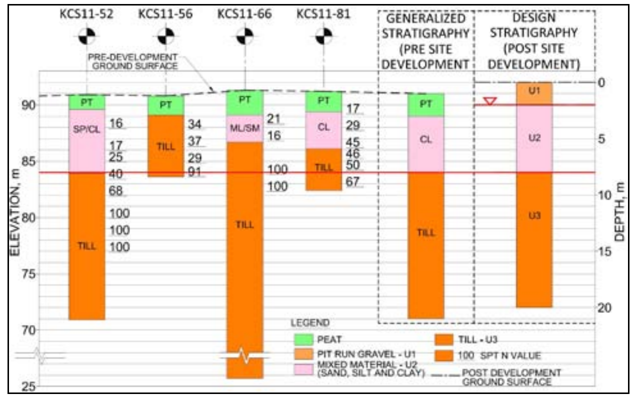

Figure 5. A subsurface cross section

2.6 Ground Temperature

Ground temperature profiles are being monitored on regular basis and some selected predevelopment site profiles are presented in Figure 6. The Figure 6 temperature profiles represent non-permafrost (KCS11-52) and discontinuous permafrost (KCS010) areas identified at the site. Temperatures in the top 1 to 3 m varied annually corroborate well with an assumed current active layer for the native ground conditions of about 3 m. An assumed thaw consolidation depth of 8 m below the final grade also corroborated well with the temperature profile measurement. Figure 7 shows frozen soil versus depth profiles. Below 8 m depth the frequency or the thickness of ice lenses varies. Two deep boreholes under the footprint of the proposed 230 kV AC substation (KCS11-82 and KCS-035) and two other boreholes KCS11-71 and KCS11-80A outside the proposed footprint indicated frozen soil and ice lenses up to about 12 m to 14 m depths.

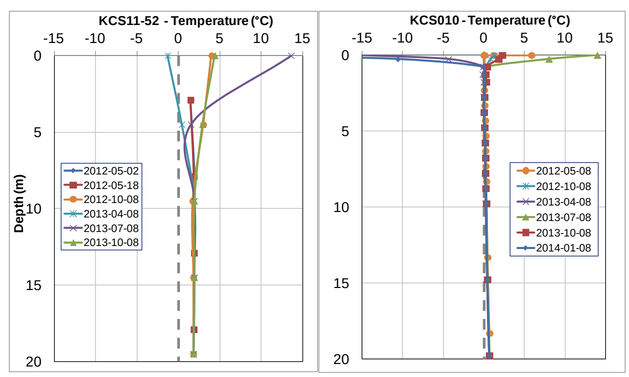

Figure 6. Temperature versus depth graphs for non-permafrost (KCS11-52) and discontinuous permafrost (KCS010) Areas

2.7 Thaw Strain

Figure 8 shows thaw strain profile data, together with an average thaw strain line estimated at about 5% through the 6 m layer of mixed soil ((silt/clay/sand/till) and therefore average thaw settlement is predicted to be on the order of 300 mm (5% of 6 m). There will be additional settlements post thawing (elastic and consolidation), however, these are expected to be very small compared with thaw settlement.

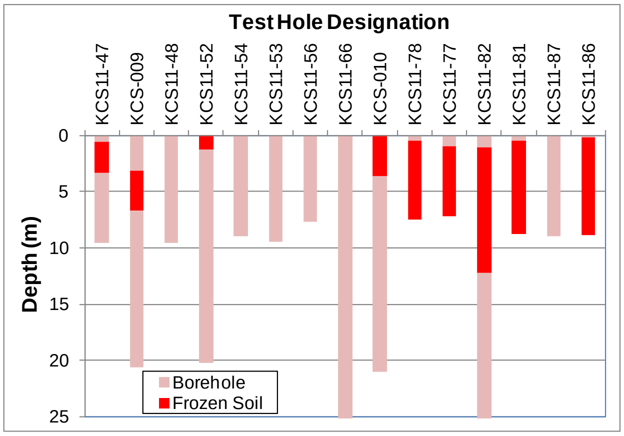

Figure 7. Frozen soil versus depth profile

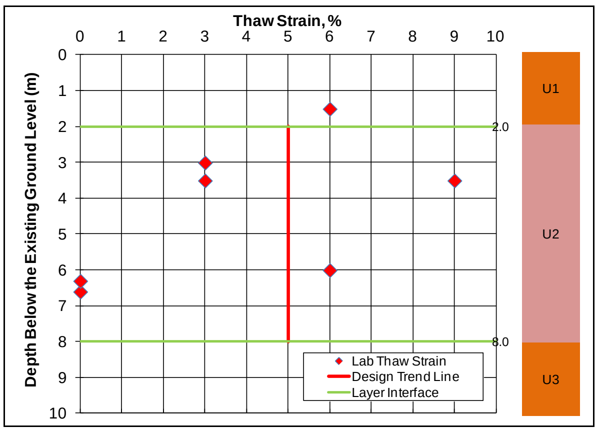

Figure 8. Thaw consolidation test – thaw strain profile

2.8 Engineering Properties

The pertinent engineering properties used for deep foundation design axial and lateral bearing capacities estimates are presented in Table 1.

Table 1: Recommended Engineering Properties

Soil Unit	Depth Range	γ (kN/m^3)	Φ' (°)	S_u (kPa)	K (kN/m^3)
Unit-1: Pit Run Gravel	0-2m	20.0	35	-	30000
Unit 2: Mixed Soil	2-8m	19.6	25	-	5000
Unit 3: Hard till	8-62m	22.5	35	500	540000
Bedrock	>62m	NA	-	-	-

Notes: γ: total or bulk unit weight; Φ: angle of internal friction; Su: undrained shear strength; K: horizontal modulus of subgrade reaction; Unit 2 in thawed condition.

3 CLIMATIC CONDITIONS

Keewatinohk is located in the subarctic climate zone. The historical design freezing index for the general project area is about 3300°C-days (based on the 30 year return period period 1931 to 1960) and 3750°C-days (based on the 50 year return period 1958 to 2007). The owner's preference for a relatively long service life (75 years) and reduced maintenance during operations is merited due to the remoteness and cold climate of the site. Therefore, it was desirable to have a minimum freezing index return period of 75 years as a design basis. Figure 8 presents the plot of freezing index versus return period. The design freezing index would project to well over 4000°C-days. With higher freezing index and site development works replacing all peat cover with more thermally conductive pit run gravel fill, the design freeze-thaw penetration depth of 4 m has been considered for the project.

The nearest meteorological station is Gillam Airport, which maintains a comprehensive long-term record (Environment Canada, 2015). Figure 9 presents the average annual air temperatures at Gillam for the period of 1971- 2013; the average mean annual air temperature (MAAT) for this period is ~-3.9°C. A linear regression trend line is overlain on the data indicating a general increasing trend in MAAT and hence will likely increase the long-term thaw index and lead to degradation of the frozen ground during the service life of the project. Similar trends in MAAT increase in northern Manitoba have been reported by others (Dyke and Sladen 2010; and French and Egorov 1998).

Therefore, the thawing of currently frozen ground is an important geotechnical foundation design consideration. Thawing will manifest into ground settlement resulting in negative skin friction and inducing downdrag loads on piles; weakened ground conditions for pile lateral load resistance; and poor bearing capacity and excessive settlements for shallow foundations.

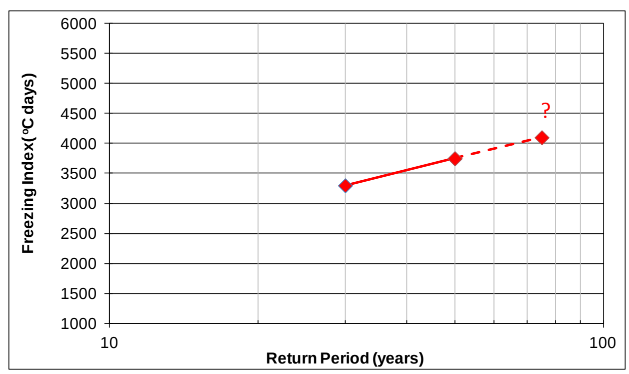

Figure 8. Plot of design freezing index v/s return period

Figure 10 presents 1981-2010 temperature climate normals for Gillam together with extreme maximum and extreme minimum recorded since the early 1970s. These normals represent averages for periods of 30 consecutive years. The most extreme maximum and the most extreme minimum recordings are: +36.8°C in June 2002 and -46.1°C in January 1975. For winter months of December, January and February, the average daily temperature ranged from - 21.4°C and - 24.4°C. The extreme cold temperatures are a consideration for constructability since the foundation construction is scheduled to take place during the 2015 - 2016 winter months.

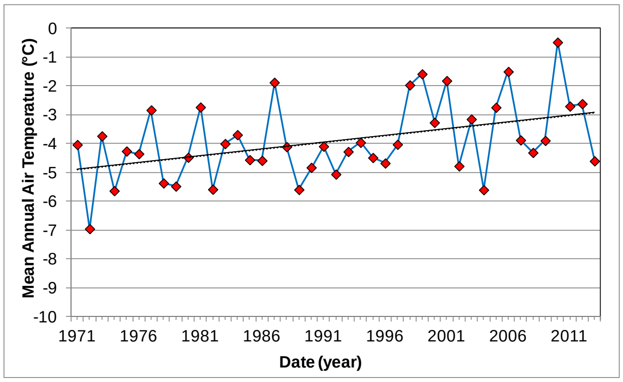

Figure 9. Mean annual air temperature for Gillam (Environment Canada period of record 1971-2013)

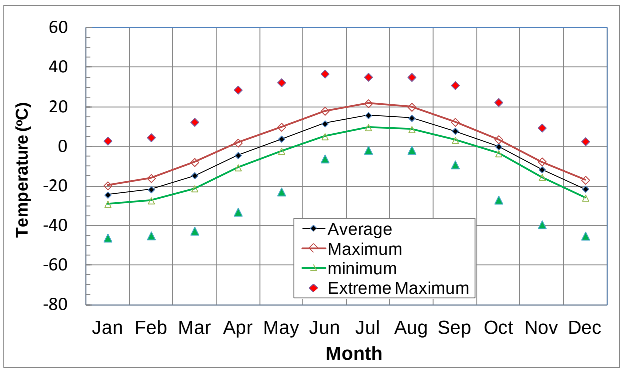

Figure 10. Temperature climate normals and extremes at Gillam, 30 year period 1981-2010 (Environment Canada)

4 FOUNDATION OPTIONS

4.1 Shallow Foundations

The permanent shallow foundations have to meet the following cold regions geotechnical requirements:

a) Frost penetration depth of 4 m;
b) Potential thaw consolidation of variably ice rich mixed material (average thaw strain of mixed soil layer under self weight = 5%). If assumed completely thawed during the 75 years service life, it may result in settlements in excess of 300 mm.

Permanent shallow foundations would not meet the serviceability criteria unless a massive ground thawing and ground improvement programs were implemented prior to foundation construction. Thawing of the entire frozen ground mass and then following with a ground improvement program are both time consuming and costly options. Hence, permanent shallow foundations are not a feasible option at the KCS site.

Further, all slabs or beams on grade may also be subjected to frost jacking, hence would need to be supported on deep foundations and placed over grade leaving void spaces between the underside of the slab or beam and the ground surface. Wherever shallow foundations are considered it would need to allow for a minimum 300 mm differential settlement if interfaced with piled foundation structure since piled foundation settlements would be small. A foundation depth of 4 m or shallower, if appropriate insulation is provided to meet with design consideration of frost penetration depth of 4 m. Groundwater being shallow; dewatering and form work will also be required. Hence shallow foundations do not provide a commercially feasible solution.

4.2 Deep Foundations

The schematics of axial loads acting on the deep foundation system during extreme seasons (summer and winter) are illustrated in Figure 12.

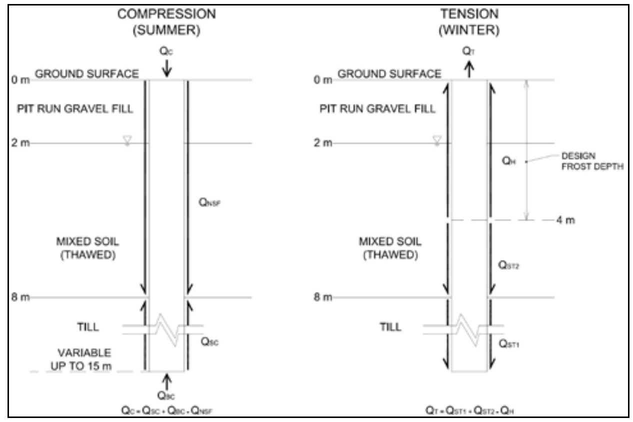

Figure 12. Schematics of axial loading on deep foundation system during extreme seasons (summer and winter)

The deep foundation design for the project is based on the following additional concepts relative to conventional deep foundation design:

a) Adfreeze forces (tension) may be generated within the design frost penetration depth of upper 4 m (Q_H);
b) Intermediate ice-rich mixed soil layer may thaw and consolidate at some point during the service life of the project and may result in:
   - Negative skin friction on the deep foundation resulting from all layers above the dense till layer (full 8 m thick layer, Q_{NSF}); and
   - Thaw weakening for lateral support (assume thawed ground) of the entire mixed soil layer.
c) Vertical capacity in compression will have to be mobilized from underlying till layer (Q_{sc}+Q_{BC}).
d) Vertical capacity in pullout or tension will have to be mobilized from underlying dense till layer (Q_{ST1}) and a limited thickness of mixed soil layer (i.e. 4 m thickness of mixed soil layer, from frost interface at 4 m depth to till layer at 8 m depth, Q_{ST2}). The mixed soil layer will be assumed to be thaw weakened for estimating frictional resistance to pullout.
e) For drivability assessment worst soil profile is considered as: till immediately underlying pit run gravel fill and till is considered as very hard to weak rock (UCS = 1 to 3 MPa).
f) Subsurface conditions include frozen soil and the presence of cobbles and boulders though out the

depth of interest and will pose challenges to pile driving operations.

5 DEEP FOUNDATION DESIGN CHALLENGES

5.1 Axial Capacity Analysis - Compression

The permanent deep foundations have to meet the following cold regions geotechnical requirements:

Negative Skin Friction or Downdrag Load

In temperate climates the negative skin friction (NSF) or downdrag forces on piles are associated with piles being installed through a clay deposit that is subjected to consolidation and resulting in downward movement of the clay around the pile. At the Keewatinohk site it is anticipated that, due to an exceptionally warm summer or due to long-term permafrost degradation the frozen soils will be subjected to thawing during the service life of the facility and result in thaw consolidation. Further, during the post thawing period, this predominantly fine grained layer will be subjected to normal consolidation processes under the fill load (pit run gravel fill layer placed across the site will act as surcharge load for consolidation process). Thawing and consolidation processes will result in downward movement of the entire stratigraphy above the very hard till layer (i.e. Soil Unit 1: Pit run Gravel Fill + Soil Unit II: Mixed Material) inducing downdrag forces on the piles. Relative to the thaw strain and consolidation movements, the movements associated with the very hard till layer are insignificant and for current pile load analysis the underlying till layer can be considered non-yielding. The unit NSF is computed in the same way as the positive unit shaft friction. Using the soil properties given in Table 1 and other appropriate design parameters, the unit NSF profile is shown in Figure 13 together with a computed average value.

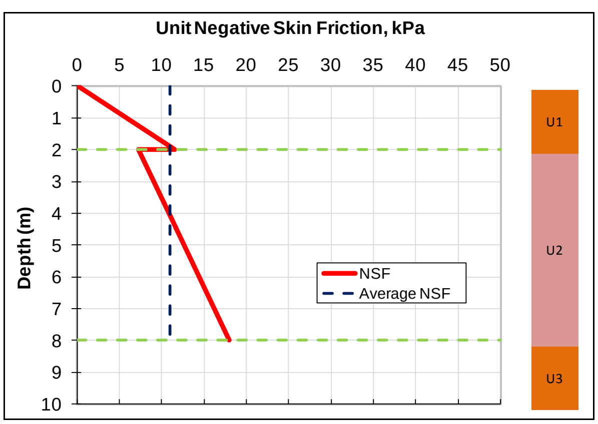

Figure 13. Negative unit skin friction profile

Axial Compression Capacity

Pile capacity in compression will be mobilized by the portion of the pile embedment in very hard till below 8 m depth. Soils above till layer are considered to induce NSF or downdrag force on the pile and no resistance to axial compression loading is considered through the NSF length (8 m). The ultimate capacity of pile for compression loading is computed by using both the shaft friction and end bearing. Based on assuming very hard clay characteristics the unit skin friction would be higher than assuming it as very dense sand. Conservatively, we have limited the unit skin friction for the till layer to 120 kPa upper limit for "sand". Assuming the piles will be driven open-ended but during driving through very hard/ dense till, it will become plugged and hence end-bearing is also considered. Considering the till layer as a sand will produce a higher unit end bearing than assuming it is clay. Conservatively the clay model is assumed for unit end bearing and a limiting unit end bearing of 4.5 MPa (9 x Su) is used. Based on the above design assumption, pile capacities are summarised for different pile penetration depths in Figure 14.

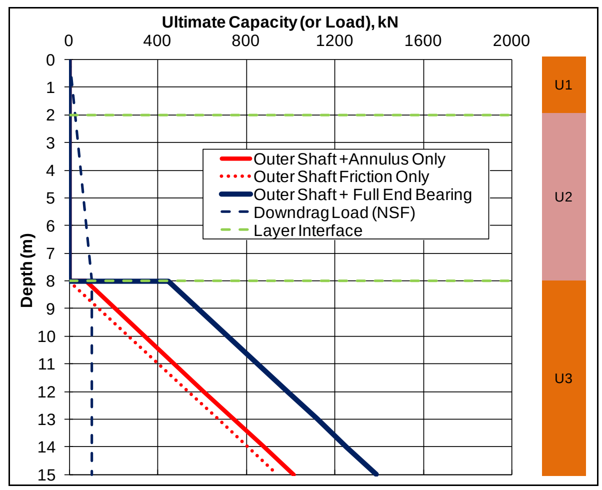

Figure 14. Axial pile capacity profile – compression (356 mm outer diameter pipe pile)

5.2 Axial Capacity Analysis – Tension/Pullout

The following design assumptions are made to estimate the axial uplift capacity of a single driven tubular pile:

Adfreeze Stress and Frost Jacking Load

Frost is an important factor in designing deep foundations in cold region. Adfreeze forces on piles or pile frost jacking loads are usually the governing loads for lightly loaded piles in cold region areas, particularly for piles supporting unheated structures or raised buildings, or perimeter piles supporting a heated structure. Frost susceptible soils, if located within the seasonal frost depth can heave due to the action of soil expansion on freezing and transmit uplift forces to piles. These forces are a manifestation of adfreeze bond stress mobilized between frozen soil and pile by heave. These forces are computed in a similar manner as shaft friction in compression or tension. Adfreeze bond stress is taken as an average for the frost susceptible layer than as a function of depth in shaft friction. Top 2 m pit run gravel fill is above the groundwater table and relatively dry and is considered not to contribute to frost jacking; any resistance provided to frost jacking is also ignored. The soil immediately below the top 2 m pit run gravel fill is considered saturated and the full layer to the design frost penetration depth as frost susceptible and considered to contribute to frost jacking. The average adfreeze bond stress for saturated mixed material was taken as 100 kPa, a typical value recommended by Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 2006) for design adfreeze stress for fine grained soil frozen to steel. Hence, the total adfreeze force for 356 mm outer diameter pipe pile would be equal to 224 kN.

Axial Tension or Pullout Capacity

Pile capacity in tension will be mobilized predominantly by the portion of the pile embedment in very hard till below 8 m depth. No resistance will be mobilized through the active layer or design frost penetration depth of 4 m. Some resistance will be mobilized through the mixed soil layer below the design frost depth, however, the soil will be considered in a weakened state due to long-term thawing. The unit skin friction in tension is computed in the same way as the positive unit shaft friction or resistance and is a function of the effective stress acting on the pile. Unit skin friction in tension through the remaining 4 m of mixed soil were determined based on the recommended engineering properties in Table 1 and computed as about 36 kPa, on average. The unit skin friction in tension through the till layer is assumed as 120 kPa. Sustained tension loads are very small compared to design adfreeze forces, hence no reduction to skin friction in tension compared to compression has been applied. Only outer skin friction is considered and no end bearing or pile weight is considered in geotechnical capacities.

Based on the above design assumption, pile capacities in tension are summarised for different pile embedment depths in Figure 15.

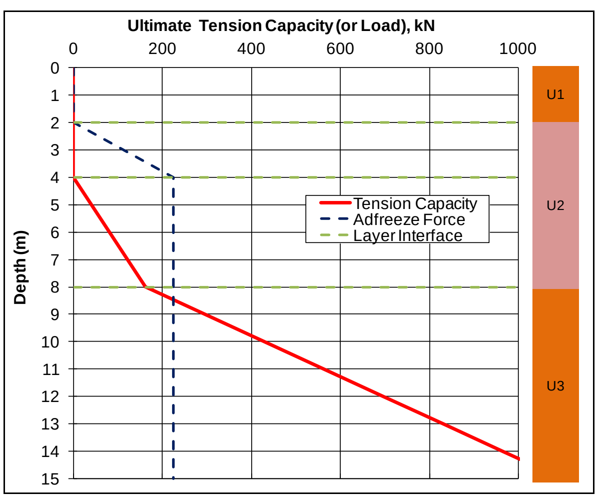

Figure 15. Axial pile capacity profile – tension (356 mm outer diameter pipe pile)

5.3 Lateral Load Analysis

Design parameters required for lateral analysis are presented in Table 1. It is noted that the parameters for the mixed soil zone correspond to the worst-case scenario of being completely thawed and unconsolidated. Consistent with LPile, a computer program for the analysis of deep foundation under lateral loading (Ensoft 2010) a representative value for $\varepsilon_{50} = 0.004$ for till layer is recommended. These parameters will establish the worst-case pile displacement but for internal pile moments, the pit run gravel layer shall also be considered as frozen and modelled as a soft rock with UCS of 10 MPa.

5.4 Pile Drivability

Pile driveability analysis is based on a combination of a wave equation analysis and the soil resistance to driving (SRD) estimates. Computation of the soil resistance to pile driving is analogous to the computation of ultimate axial capacity by the static method. The resistance to driving is the sum of the shaft resistance and the point resistance. The shaft resistance is computed by multiplying the average unit skin friction during driving and the embedded surface area of the pile. The point resistance is computed by multiplying the unit end bearing and the end bearing area. Driving resistance is the lesser of summation of outer shaft friction plus full end bearing or outer and inner friction plus pile wall end bearing. Soil parameters representing moderately conservative profile are used as the basis for SRD estimates.

Wave equation analysis was performed using the commercially available GRLWEAP software (PDI 2005). The software's bearing graph and pile driveability method of analyses were used. The analysis facilitates the predictions of blow counts as a function of driving resistance for a fixed percentage of side friction in the bearing graph analysis method (75% was used), and blow counts and driving stresses as a function of pile penetration in pile driveability analysis method.

Wave equation analysis of pile driving is based on the discrete element idealization of the hammer-pile-soil system formulated by Smith in 1960 (PDI, 2005). The parameters or design basis used in the wave equation analysis can be divided into three major groups: Pile Model; Hammer Model; and Soil Model.

A single pile configuration (356 mm by 15 mm wall thickness) was used and maximum length (15 m) was considered. A Bermingham B550 C/-OE Diesel hammer with continuous driving for full target penetration, operating at 80% efficiency was assumed. SRD predictions were based on assuming very hard till through full penetration depth of 15 m and the following soil model was assumed:

- Skin Friction: 150 kPa;
- Unit End Bearing: 9 x Su (Su of 500 and 1500 kPa) or 4,500 kPa and 13,500 kPa range.

Damping and quake factors appropriate to the soil conditions assumed were used (quake for shaft and toe as 0.0025 m and damping for shaft and toe as 0.656 s/m and 0.49 s/m, respectively). Bearing graph was generated with GRLWEAP using appropriate hammer and proportional shaft resistance at the end of driving and is presented in Figure 16.

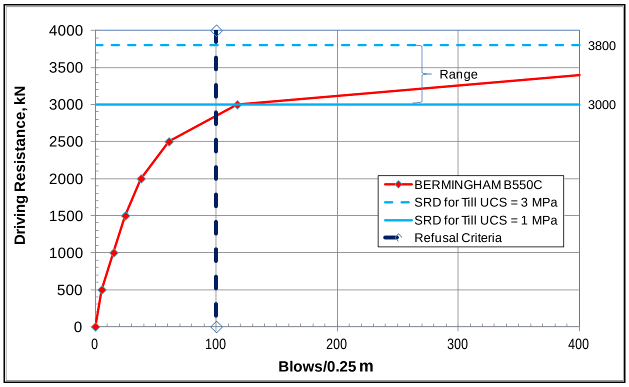

Figure 16. Pile driving analysis - bearing graph results

This graph also shows the estimated range of SRD and assumed driving refusal criteria. The graph indicates that a hammer with energy rating of ±120 kJ-m and assumed UCS strength of till layer as 1 MPa can drive the given pile to 15 m depth with no difficulty but if the till strength UCS approaches 3 MPa driving will be difficult after 10 m. This is based on an open-end pipe pile driven as a plugged pile (not in coring mode). Hence, it would be applicable to closed-end pipe pile as well.

Blow counts versus depth and dynamic stress (compression and tension during driving) versus depth predictions were generated using GRLWEAP's driveability analysis option and these results are shown in Figure 17.

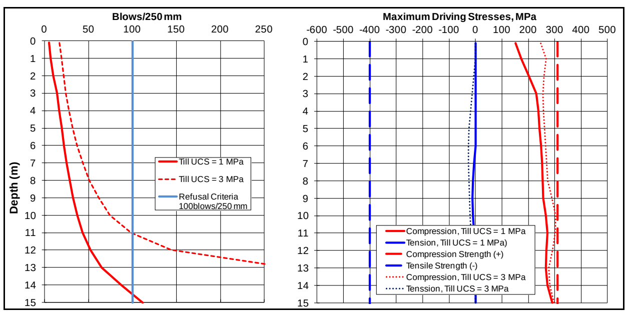

Figure 17. Pile drivability analysis results

The analysis results indicate that:

- If the till UCS = 1 MPa, a hammer with rated energy of ±120 kJ-m can drive the given pile to 15 m without exceeding the limit of blows/meter or the allowable stress of steel pile. The total number of blows is about 2000 and time for driving is less than 1 hour;
- If the till UCS = 3 MPa, the blow/meter increases significantly after 10 m driving but the stresses remain within limits.

The presence of cobbles and boulders is not possible to model with the GRLWEAP analysis. Pre-production pile installation (driving with and without pre-boring) will be carried out to determine the best installation methodology. Preproduction pile driving including pile driving analyzer (PDA) testing is currently proposed to finalize the most suitable technique for pile installation or alternatively pre-boring to almost full depth is considered.

6 CONCLUSIONS

Climatic and geotechnical conditions at the Keewatinohk Converter Station site, located 80 km northeast of Gillam, northern Manitoba are presented. It includes a comprehensive temperature climate record of the nearest meteorological station at Gillam Airport, subsurface characterization with a focus on thaw strain, and ground temperature monitoring data. The impact of a long proposed service life and reduced maintenance considerations for the KCS facilities resulted in deeper design frost depth and full degradation of permafrost soils for foundation design. Given the depth of frozen ground and its considerable thaw strain, shallow foundations that would experience large settlement (more than 300 mm) were not considered feasible. Tubular steel pipe piles were considered a robust foundation option mobilizing pull out resistance and axial compression by penetrating deeper into the hard clayey till encountered at site. Foundation design challenges and solutions for deep foundation due to climatic and subsurface ground conditions are presented and discussed. A minimum pile embedment depth of 10 m is required to just provide the resistance required for the loads imposed by seasonal climatic changes and potential warming during the service life of the project. Further plans are made for geotechnical investigation to better characterize the till for driveability and a pre-production piling and pile load testing for achieving a best methodology for pile installation and refining design assumptions and parameters.

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