Load transfer of pile foundations in warming frozen ground

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ABSTRACT
An experimental research program has been undertaken to investigate the response of pile foundations to frost thawing and adfreeze bond in Leda clay. Field pile load tests were conducted at the Canadian Geotechnical Research Site No. 1 located in Gloucester, Ontario. The results of static pile load tests in frozen and unfrozen Leda clay showed dramatic reduction in pile capacities upon frost degradation. Load-displacement relationships for tested piles and theoretical estimation of long-term adhesion of frozen and unfrozen Leda clay are presented and discussed.

RÉSUMÉ
Un programme de recherche expérimentale a été entrepris pour étudier la réponse des fondations sur pieux au dégel et à l'adhésion due au gel de l'argile Leda. Les essais terrains de chargement de pieux ont été menés au site no 1 de recherche canadienne en géotechnique à Gloucester, Ontario. Les résultats des essais de chargement de pieux statiques dans l'argile Leda gelée et non gelée ont montré une réduction spectaculaire des capacités portantes des pieux lors de la dégradation du gel. Les relations charge-déplacement pour les pieux testés et l'estimation théorique de l'adhérence à long terme de l'argile Leda gelée et non gelée sont présentés et discutés.

1 INTRODUCTION

Permafrost, or perennially frozen ground, is ground remaining frozen for more than a year which is considered a critical component of the cryosphere and the Arctic system. Permafrost regions extend over 24% of the terrestrial surface of the Northern Hemisphere. Ground materials in their frozen state are stiffer than the materials in unfrozen state. In cold regions, however, frost heave and excessive settlement following ice melting in ice-reach soils are critical problems that can degrade the stability of infrastructure.

Lawrence and Slater (2005) discussed the evolution of permafrost in the Northern Hemisphere under global warming impact predicting present-day permafrost as well as permafrost state during the 21st century. The predicted permafrost projections on the International Permafrost Association (IPA) map showed 25% reduction in the global permafrost extent occurred between 1900 and 2000. Dramatic permafrost degradation also was predicted for 2100 which reported to contribute to 15% discharge increase into the Arctic Ocean from a total predicted discharge increase of 28%. Recently, permafrost regions have become economically important due to the existence of enormous natural resources in the north circumpolar region. This economic boom occurred following the development of raw materials' extraction methods and ease of transportation to population centers and consumers. The economic development, however, has been accompanied with increase of population and expansion of infrastructure such as civil facilities, hydrocarbon extraction facilities, transportation networks, communication lines, and industrial projects. In addition to the considerable expenses that the rapid and vast development has caused, however, Williams (1986) and Smith and McCarter (1997) suggested that these could be aggravated severely in both environmental and human terms by the effects of global warming on permafrost.

In cold regions, the permafrost conditions (e.g., warm permafrost or cold permafrost) dictate the selection of foundation type. Slurried pile foundations have commonly been used to support structures situated on frozen ground according to Design Manual for new Foundations on Permafrost (Fish, 1983). Piles are load bearing columns that interact with the surrounding soil through their end bearings and skin friction to carry the applied structural loads. In frozen ground, adfreeze strength developed during the frost formation along the pile shaft can significantly enhance the pile capacity. Early studies have adopted a design method based on the assessment of ultimate load capacity of piles related to the rupture of the adfreeze bond developed at the pile-frozen soil interface along the pile shaft. In more recent investigations (e.g., Nixon and McRoberts 1976; Morgenstern et al. 1980), the adoption of bearing capacity criteria for pile design in frozen ground was criticized claiming that excessive settlement may occur over the design life. Therefore, it was emphasised that ensuring tolerable pile displacements throughout the life of the structure is essential.

Frozen soils have been classified into ice-reach and ice-poor soils, where ice-reach exhibits a frozen bulk density less than 1.7gm/cm^3, while ice-poor soil shows higher frozen bulk density (Nixon and McRoberts 1976). Frozen soils behave differently compared to unfrozen soils where the former show viscoelastic behaviour, while the latter exhibit elasto-plastic behaviour. Viscoelastic materials demonstrate creep upon exposure to loading, where creep is the plastic time-dependent deformation under continues loading. Deformation of ice-reach soils is governed by secondary creep, whereas in ice-poor soils primary creep affects the deformation as it was reported by Nixon and McRoberts (1976). Ice-rich soils are susceptible to excessive thaw settlement as a result of their excessive ice contents. Excessive ice is formed due to migration of unfrozen water from the vicinity under effect of cryosuction resulting in ice lenses and ice segregation (Zhang, Y. 2014). Many other factors control the formation of excess ice in permafrost such as thermodynamic and geological processes which makes the prediction of its extent and volume in the field complex (Bush et al.1998).

The creep behaviour of pile foundations in frozen soils is affected by thaw settlement. Different studies have shown the effect of temperature on the pile creep and creep rate. Ladanyi (1995) reported 35% increase in the creep settlement for a pile exposed to constant axial load installed in an ice-reach silt when the ground temperature increased by 1°C. Nixon (1990a) developed a mathematical expression to predict creep settlement corresponding to permafrost warming. For a pile carrying 200 kN concentrated load and embedded 6m in frozen silty clay at a temperature of -1.3°C, a creep settlement increase of 30% along 25 years was estimated when a warming rate of 0.1°C per year was applied. Nixon and McRoberts (1976) suggested using creep settlement criteria for pile design in ice-rich soils. However, pile design in ice-poor soils should satisfy both settlement and ultimate capacity criteria. The contribution of the end-bearing was always reported to be negligible for piles in homogeneous frozen ground compared to the skin friction.

Frozen ground could fail to maintain frozen condition in confrontation of global warming. In warm permafrost, a small temperature increase may be sufficient to cause extensive thawing. In cold permafrost, temperature increase by couple of degrees may result in significant increase in active layer depth (annual thaw depth), which can promote significant thaw settlement and increase the potential frost heave upon freezing. The thaw settlement will be even worse for ice-rich soils where the resulting thaw settlement and loss of bearing capacity can cause inclusive damage to the structures (Esch and Osterkamp, 1990). In the present study, attempts are made to investigate the ultimate bearing capacity of different piles installed in frozen Leda clay. The study also evaluates the adfreeze strength corresponding to the thaw action. Pile load test were carried out on piles in autumn and repeated again in winter in order to obtain shear strength parameter for frozen and unfrozen pile-soil interface and determine the load-displacement relationship. A comparison was then carried out to assess the impact of permafrost degradation on pile capacity in soft clay.

2 SITE PREPARATION AND CHARACTERISTICS

The experimental program was conducted at the Canadian Geotechnical Research Site No. 1 located in Gloucester, Ontario. The site mainly consists of about 1.2 m top sandy silt overlaying deep layer of marine clay known as Leda clay. Leda clay formed through marine deposition of the Champlain Sea, and now covers major parts of eastern Ontario and Quebec regions in Canada. The 1.2 m top soil was removed to reach the test level which will be called ground level from now on. Several in-situ tests were conducted to determine the geotechnical and thermal properties of the test soil.

The site investigation includes the determination of undrained shear strength profile (Cu) for Leda clay as well as temperature profile from ground surface along to the target depth. A 2 m deep thermocouple system, which measures temperature based on change in electric flow, was installed in the site to achieve the temperature profile by recording temperature readings at intervals of 10-25 cm (Figure 1). The undrained shear strength was determined by performing a vane shear test in accordance with the field vane shear test procedure outlined in ASTM D2573 (2008). The undrianed shear strength results show strength reduction with depth from a maximum value of 35 kPa at 25 cm depth to as low as 9 kPa at 2 m depth. The remolded shear strength was also determined and the results presented in Figure 2. The temperature profiles in both winter season and fall season were determined from ground surface along the depth of 2 m and the results are shown in Figure 1.

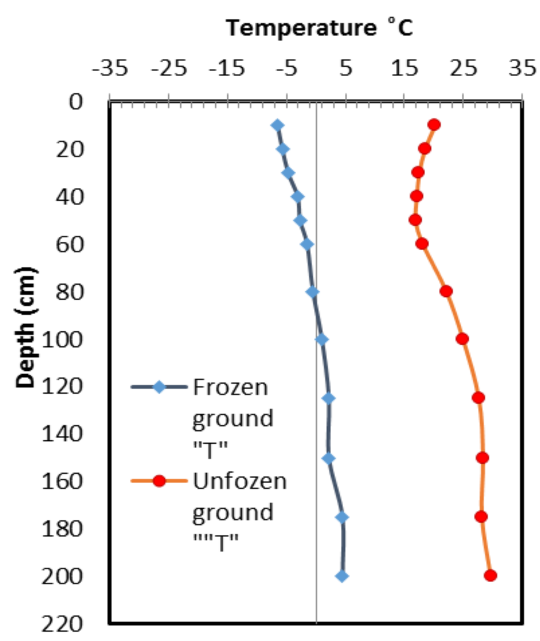

Figure 1. Temperature profiles in winter and autumn seasons.

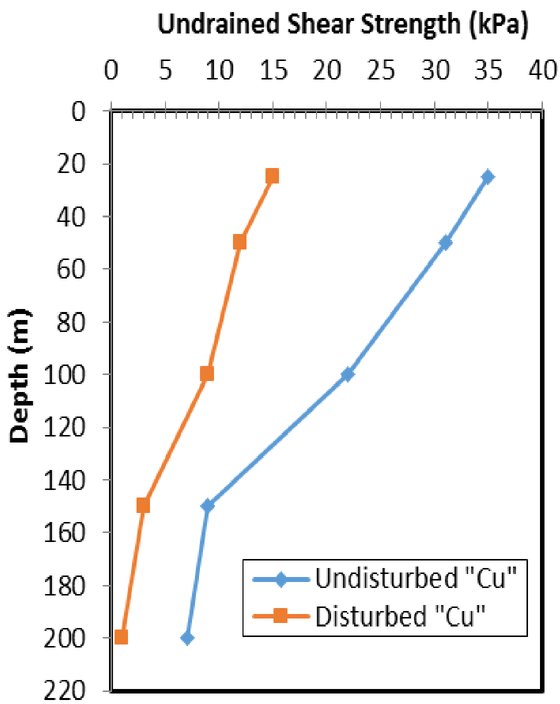

Figure 2. Undisturbed and disturbed shear strength profiles for Leda clay

3 PILE CHARACTERISTICS AND CONFIGURATIONS

The investigation was conducted utilizing circular section piles made of two different materials namely steel and reinforced concrete. The steel pile was an open-ended pile with a wall thickness of 0.5 cm, outer diameter of 10 cm and height of 210 cm. The precast reinforced concrete pile is a close-ended with 10 cm diameter and 210 cm height. Each pile was equipped with a 15 cm x 15 cm x 1 cm steel cap in order to facilitate the pile diving and instrumentation on the pile head during testing (Figure 3). For steel pile, the cap center was aligned perpendicular to the pile central axis and welded to its head. For concrete pile, the cap was first welded to the steel rebars, and then the concrete was casted where tight connection between the pile head and the steel cap was ensured. The pile caps were featured with 3/8 inch holes at the four corners. These holes were later used to attach another square plate which was connected to its center high-strength threaded steel bars with 19 mm diameter and 1.50 m length in order to perform uplift pile load tests.

Figure 3. Pile configurations and installation

4 PILE INSLALATION

Piles were driven using a 15.87 kg (35 lb) hammer falling from a distance of 0.5 m on the pile head utilizing a pulley and drop hammer setup. The falling distance was maintained constant and hammering action was conducted manually at a rate of 10-15 blows per minute. The penetration rate of the pile tip varies with depth from 2-5 mm/blow along the upper 1.0 m depth. The penetration rate increases significantly to be as high as 10 mm/blow through the rest of the depth. Piles were driven up to 2.0 m depth keeping 10 cm of the pile above the ground to ease the pile head instrumentation set up. Figure 4 shows cumulative blow count versus penetration depth of the steel pile.

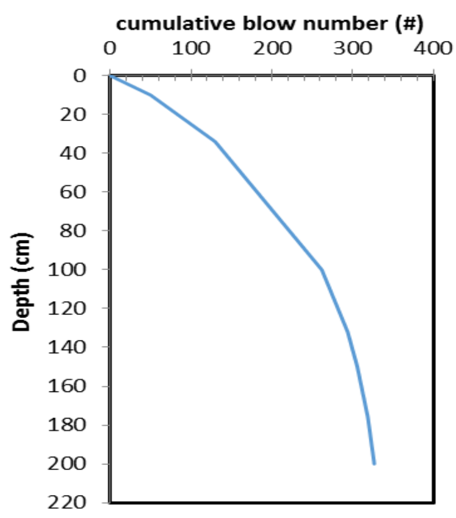

Figure 4. Pile driving and installation

5 LOAD AND DISPLACEMENT MEASUREMENTS

Load measurements were facilitated utilizing a vibrating wire load cell. The load was applied using a 103 kPa

(15 psi) manual hydraulic pump and a 100 kN hollow hydraulic jack. The data was recorded electronically using a data acquisition system. The vertical pile displacement under uplift loading was recorded using Linear Variable Differential Transformers (LVDTs) with accuracy of ±0.01 mm and maximum mechanical travel of 100 mm. The displacement sensor was mounted on a dial indicator holders attached to a circle-section steel reference beam. The reference beam was independently supported with two steel angle-sections positioned far enough from the reaction frame to eliminate the disturbance of displacement measurements during testing. The displacement transducer freely bears on the steel cap parallel to the vertical axis of the pile.

6 LOAD TEST REACTION FRAME

In order to obtain valid comparison between pile capacities in frozen and unfrozen grounds, piles were exclusively examined against uplift loading to eliminate the contribution of piles' end-bearing. The end-bearing capacity was reported to be negligible for piles in homogeneous frozen ground (Nixon and McRoberts 1976; Morgenstern et al. 1980). The reaction frame for pile load testing consisted of two reaction steel open-ended piles, each of 15 cm diameter, 250 cm length, and wall thickness of 0.5 cm. The reaction piles were connected to each other using two 2''*12''*12' plywood reaction beams. The reaction beams were connected to the reaction piles using high-strength threaded steel bars of 19 mm (¾ inch) diameter. The threaded steel bars were passed through pre-drilled holes in the reaction beams and piles, then, tightened carefully utilizing steel nuts. The reaction system has a theoretical capacity of approximately 3.0 times the capacity of the test piles. The uplift test was performed by applying uplift force at the pile head. This was done by attaching the steel square plate to the pile cap using four 6.35 mm (¼ inch) bolts and jacking the test pile upward through the 1.5 m high-strength threaded steel bar that was connected to the plate by a steel nut welded to its center. The uplift force was applied using the hydraulic jack which was positioned on a 30 cm x 40cm x 2cm hollow steel plate at the upper surface of the reaction beams. The load cell was situated on top of the jack where the whole system was sealed by a circular steel ring tightened by a steel nut at the very top (Figure 5).

7 TESTING PROSEDURE

Uplift load testing for the piles was performed in accordance with ASTM D1143/D1134M (2007), following the Quick Test Method for individual piles. The final capacity of the test piles was estimated theoretically based on the total stress analysis for cohesive soils using the α-method. Uplift load was applied in increments of 10% of the theoretical estimated load with a time intervals of 5 minutes between the loading steps. The applied load was measured using the calibrated load cell connected to a data acquisition system, whereas the pile head displacement was measured utilizing the LVDTs.

8 RESULTS AND DISCUSSION

Pile load test and temperature measurements were conducted in fall season and winter season respectively. Piles were installed in September 20, 2014 and tested 15 days after the installation in order to account for pore water pressure dissipation (known also as pile set up effect) (Afshin and Rayhani, 2014). Ground temperature measured in the fall shows a reduction from 20 °C at 10 cm depth to 16.9 °C at half a meter depth from ground surface. The temperature, then, increased reaching 29.6 °C at 200 cm depth from ground surface (Figure 1). The temperature change with time was tracked during the winter season until constant readings were observed and recorded in February 25, 2015. At 10 cm depth below ground surface, the temperature drops from 20 °C in the fall to -6.5 °C in winter. However, the winter temperature profile shows a linear increase with depth intersecting the zero degree Celsius at a depth of 85 cm below ground surface and 4.5 °C at depth of 200 cm (Figure 1). Zero degree Celsius is the theoretical freezing point at which the phase change of water from liquid to solid occurs. Therefore, the portion of the ground that shows temperature equal or below freezing point is considered as frozen state. Although there is no permanent permafrost observed recently in Ottawa region, a seasonal freeze/thaw depth between 0.8-1.2m has been reported. In this study, the depth of the frozen ground (active layer) extents from ground surface up to 85 cm depth falling within the previous reported data.

The load-displacement relationship was determined from pile load test results for piles in unfrozen and frozen ground. The pile head displacement at failure and the maximum corresponding uplift capacity of the piles were estimated and presented in Table 1. For piles in unfrozen soil, the maximum uplift loads were 4700 N and 5500 N recorded for steel and concrete piles respectively. The corresponding displacement at failure varies from 0.8 mm for the steel pile to 1.3 mm for concrete pile (Figure 5). The higher uplift capacity for concrete pile might be attributed to the pile's higher surface roughness and/or generation of suction force at clay-unsaturated concrete interface resulting in water absorption and increasing the pile-soil interface strength (Giraldo and Rayhani, 2013).

Table 1. Settlement and pile capacity at failure.

Characteristics at failure	Unfrozen Leda clay		Frozen Leda clay	
	Concrete	Steel	Concrete	steel
Displacement (mm)	1.3	0.8	5	5
Pile capacity (kN)	5.5	4.7	24	26

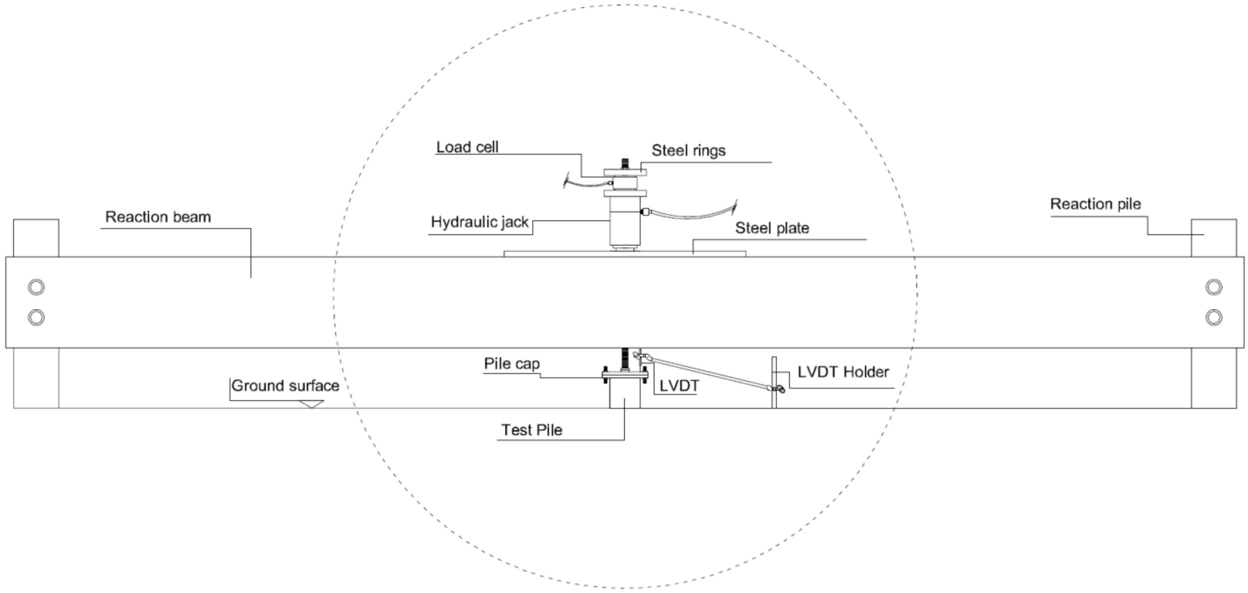

Figure 5. Pile load test instrumentation and setup

In frozen soil, piles showed continuous increase in their uplift capacity with proportional displacement without exhibiting any signs of failure. The performance of steel and concrete piles was quite similar up to 2 mm displacement and corresponding capacity of 7500N. Beyond this stage, and at given displacements, the steel pile exhibited higher capacity compared to the concrete counterpart. However, both piles showed linear increase in their capacities until the test was terminated when 5 mm displacement was reached (Figure 7). Selection of 5 mm as limiting displacement was based on 5% pile diameter failure criteria suggested by the Federal Highway Administration (FHWA) for driven piles. FHWA criterion is applicable if plunging of the shaft cannot be achieved, thus the failure load is considered at a pile displacement of 5% the shaft diameter (Paikowsky, 2004). In the current experiment, the corresponding uplift capacities at termination (i.e., 5% D displacement) were 26 kN and 24 kN for the steel and concrete piles respectively. Although the failure criteria for piles in frozen ground is governed by creep analyses (Nixon and McRoberts 1976), the objective of the current study is to investigate pile load transfer in frozen Leda clay and the frost degradation effect on pile bearing capacity in warming frozen ground.

Figures 6 and 7 show the load–displacement relationships obtained from pile load tests in unfrozen and frozen ground respectively. In unfrozen ground, the load–displacement was typical exhibiting elastic-perfectly plastic behaviour along three distinct zones. The first zone was linear-elastic with sharp slope indicating large elastic modulus. The second zone exhibited transitional nonlinear behaviour where the displacement and load increment were considerably disproportional. The final zone was flat linear with almost zero slope indicating reduced stiffness and showing strain softening with lower residual strength compared to the peak strength at failure. In contrast, the load–displacement relationship for piles in frozen ground was totally different demonstrating continues linear behavior (Figure 7).

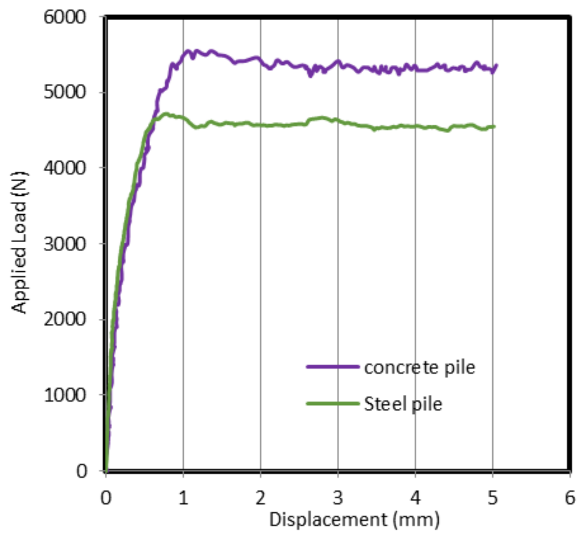

Figure 6. Load-displacement curves for piles in unfrozen Leda clay

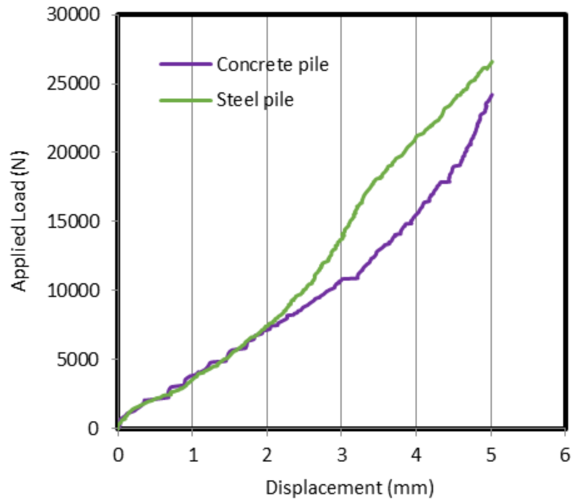

Figure 7. Load-displacement curves for piles in frozen Leda clay.

This behaviour has been characterized as viscoelastic behaviour which distinguishes frozen grounds with frozen water content (Weaver and Morgenstern, 1981). Such behaviour was observed to exhibit creep displacement and can be analyzed using secondary creep analysis for ice-rich soils or primary creep for ice-poor soils. Although piles in frozen ground showed greater capacity against uplift force in large displacement region, their capacities were lower within low displacement region compared to piles in unfrozen soil. For example, at 1 mm displacement, piles in unfrozen ground showed maximum capacities of 4.7 kN and 5.5 kN for steel and concrete respectively. However, at the same displacement of 1 mm, the capacity of both piles in frozen ground was almost identical of 3.7 kN representing 20-30% lower capacity compared to the steel and concrete piles in unfrozen ground respectively. On the other hand, beyond 1 mm displacement, piles in unfrozen ground maintained constant capacity regardless of displacement increase, while in frozen ground the capacity continually increased until the test was terminated at 5 mm displacement showing 4.5 and 5.5 times greater capacity for the steel and concrete piles respectively. This comparison highlights the importance of proper selection of the displacement region upon which the piles are designed in warming frozen ground. It is worth to mention that the adfreeze bond formed only along the upper 85 cm of the pile shaft, while the rest of the pile was within unfrozen clay. Therefore, the loss of pile capacity due to frost thawing could be even worse if the adfreeze bond was formed along the entire shaft. In cold region the contribution of adfreeze strength is an essential component of pile design, however, it might be critical if permafrost degradation occurs. Pile capacity could descent five times or more upon losing the adfreeze bond. This could promote excessive deformation and lead to extensive damage in the supported structures and facilities.

The experiential results obtained from pile load tests in frozen state can be utilized to develop theoretical solution to estimate the long-term pile-soil interface shear strength of frozen Leda clay. Weaver and Morgenstern, (1981) proposed an equation that relates the adfreeze strength 'τ_a' to the long-term shear strength of frozen ground 'c_{lt}' using a roughness-dependent parameter 'm' as follow:

$$\tau_a = m * c_{lt} \quad \ldots\ldots\ldots\ldots\ldots \qquad [1]$$

Based on observation reported by Weaver and Morgenstern (1981), the parameter "m" was found to be equal to 0.6 for steel and concrete, 0.7 for timber and 1 for corrugated steel. In the current analysis, two scenarios were adopted for failure based on frozen ground condition. First scenario assumes permanent permafrost where shaft plunging is not achievable, thus adfreeze strength corresponding to 5 mm displacement is selected for analysis. Second scenario assumes warming permafrost where shaft plunging could occur, thus unfrozen shear strength is used for the steel and concrete piles where parameter "m" equal to 1 is selected resulting in long-term adhesion for warming frozen Leda clay equal to the unfrozen shear strength (Table 2).

Table 2. Theoretical estimation of long-term adhesion of frozen Leda clay and effects of frost thawing.

Characteristics (%)	Permanent permafrost		Warming permafrost		Strength loss due to frost degradation (kPa)		Percentage of strength loss (%)	
	Concrete	Steel	Concrete	Concrete	Steel	Steel	Concrete	Steel
Effective depth (m)	0.85	0.85	2	2	-	-	-	-
Pile circumference (m)	0.314	0.314	0.314	0.314	-	-	-	-
Pile capacity (kN)	24	26	5.5	4.7	-	-	-	-
Adfreeze strength (kPa)	90	97.4	8.8	7.5	81.2	89.9	900 %	1200%
"m" parameter	0.6	0.6	1	1	-	-	-	-
Long-term C_{lt} (kPa)	150	162.4	8.8	7.5	-	-	-	-

9 CONCLUSION

A series of pile load tests in unfrozen and frozen Leda clay was conducted to investigate the impact of permafrost degradation and strength behaviour of frozen Leda clay. Load-displacement curves for piles in frozen Leda clay show linear increase of pile capacity with proportional increase of displacement without exhibiting any failure signs. This supports the findings reported by other researchers (e.g., Weaver and Morgenstern, 1981) that the failure criteria in frozen ground is governed by creep rather than ultimate load capacity. Frost thawing minimized the adfreeze bond between piles and frozen soils and resulted in pile capacity loss of about 10 times their capacity in frozen state. Therefore, the long-term performance of piles in warming frozen grounds was assessed based on shear strength parameters of the ground in its unfrozen state. Long-term shear strength of frozen Leda clay was theoretically estimated implementing the adfreeze bond measured in field using Weaver and Morgenstern (1981) equation. Further investigations and preferably numerical analyses are needed to support the findings and validate the theoretical results.

10 ACKNOWLEDGEMENTS

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11 REFFERENCES

Afshin, A., and Rayhani, M.T. 2014. Evaluation of bearing capacity with time for small-scale piles driven into Leda clay. *International Journal of Geotechnical Engineering*, DOI: 10.1179/1939787914Y.

ASTM. 2007. Standard test methods for deep foundations under axial compressive load. ASTM standard D1143M-07. *ASTM International*, West Conshohocken, Pa.

ASTM. 2008. Standard test method for field vane shear test in cohesive soil, ASTM standard D2573, *ASTM International*, West Conshohocken, Pa.

Bush, E., Etkin, D. A., Hayley, D., Hivon, E., Ladanyi, B., Lavender, B., and Smith, M. 1998. Climate change impacts on permafrost engineering design. *Environment Canada, Environmental Adaptation Research Group*, Downsview, Ontario.

Esch, D.C. and Osterkamp, T.E. 1990. Cold regions engineering: Climatic warming concerns for Alaska. J*. of Cold Regions Eng*. 4(1):6-14.

Fish, A. M. 1983. Comparison of USSR codes and US Army manual for design of foundations on permafrost. *Cold Regions Science and Technology*,*8*(1), 3-24.

Giraldo, J., & Rayhani, M. T. 2013. Influence of fiber-reinforced polymers on pile–soil interface strength in clays. *J. Adv. Civ. Eng*. Mater, 2(1).

Ladanyi, B. 1995. Civil Engineering concerns of climate change in the Arctic. *Transactions of the Royal Society of Canada,* VI (VI): 7–19.

Lawrence, D.M., & Slater, A.G. 2005. A projection of severe near-surface permafrost degradation during the 21st century. *Geophysical Research Letters*, 32(24).

Morgenstern, N.R., Roggensack, W.D., and Weaver, J.S. 1980. The behavior of friction piles in ice and ice-rich soils. *Canadian Geotechnical Journal*, 17: 405-415.

Nixon, J.F. 1990a. Effect of climate warming on pile creep in permafrost. *Journal of Cold Regions Engineering,* 4(1): 67–73.

Nixon, J.F., and McRoberts, E.C. 1976. A design approach for pile foundations in permafrost. *Canadian Geotechnical Journal*, 13: 40-57.

Paikowsky, S. G. 2004. Load and resistance factor design (LRFD) for deep foundations (No. 507). *Transportation Research Board*.

Smith, E. A., McCarter, J. eds. 1997. Contested Arctic: Indigenous Peoples, Industrial States, and the Circumpolar Environment. University of Washington Press, Seattle, US.