Impact of Ground Warming on Pile-Soil Interface Strength in Ice-Poor Frozen Soils

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ABSTRACT
A surface roughness factor "m" has been used to represent the ratio between pile-soil adfreeze strength and cohesion strength of frozen soils. This factor is usually considered constant for a certain pile material but independent of soil type (i.e., ice-rich or ice-poor) and ground temperature. The current study summarizes results from an experimental program dedicated to evaluate the roughness factor "m" at different temperature exposures for steel piles in ice-poor frozen soils. The investigation was conducted in a walk-in environmental chamber using conventional and modified direct shear apparatuses. The results showed a significant reduction in shear strength of frozen soils as well as the pile-soil interface strength as the exposure surface temperature increased from -10℃ to -1℃. The roughness factor "m" varied slightly with temperature recording 0.7 at -10℃ and 0.75 at -1℃. A frictional factor was also introduced to represent the contribution of frictional resistance at the pile-soil interface based on friction angle of ice-poor frozen soils.

RÉSUMÉ
Un facteur de rugosité de surface ''m'' a été utilisé pour représenter le rapport entre la résistance au gel de l'empilage du sol et la résistance à la cohésion des sols gelés. Ce facteur est généralement considéré comme constant pour un certain matériau de pieu, mais indépendamment du type de sol (c'est-à-dire, riche en glace ou pauvre en glace) et de la température du sol. La présente étude résume les résultats d'un programme expérimental visant à évaluer le facteur de rugosité ''m'' à différentes expositions à la température pour des pieux d'acier dans des sols gelés pauvres en glace. L'étude a été menée dans une chambre environnementale sans accès à l'aide d'appareils de cisaillement direct conventionnels et modifiés. Les résultats ont montré une réduction significative de la résistance au cisaillement des sols gelés ainsi que de la résistance de l'interface pile-sol lorsque la température de la surface d'exposition est passée de -10 ℃ à -1 ℃. Le facteur de rugosité "m" variait légèrement avec l'enregistrement de la température 0,7 à -10 ℃ et 0,75 à -1 ℃. Un facteur de frottement a également été introduit pour représenter la contribution de la résistance au frottement à l'interface pile-sol en fonction de l'angle de frottement des sols gelés pauvres en glace.

1 INTRODUCTION

The principle of the design criteria for pile foundations in frozen soils was initially based on the allowable adfreeze strength obtained by dividing the ultimate adfreeze bond at the soil-pile interface by a factor of safety (e.g., Vialov 1959; Crory and Reed 1965; Penner 1970, 1974; Penner and Irwin 1969; Penner and Gold 1971; Dalmatov et al. 1973). Johnston and Ladanyi (1972), in contrast, reported a more appropriate method based on adfreeze strength that would cause an allowable displacement (or creep) rate over the life time of structures to accommodate for the time-dependent behavior of frozen soils. Other researchers including Nixon and McRoberts (1976), Morgenstern et al. (1980), and Weaver and Morgenstern (1981) suggested to check both adfreeze strength and creep behavior, and design based on the smaller value between them.

Parameswaran (1978) reported that ultimate capacity of piles depends mainly on the shaft resistance along the permanently frozen depth. The strength gained from end bearing was reported negligible for piles installed in ice-rich soils and could only be significant if the pile situated on ice-poor frozen ground such as bedrock or gravel and sand (Crory 1963). The US Army/Air Force (1967) suggested that the end bearing capacity should be neglected from design consideration for piles with base diameter smaller than 150 mm regardless of the soil type attributing that to the greater movement that is required for mobilizing the end bearing compared to the much smaller movement needed to overcome the adfreeze bond.

Weaver and Morgenstern (1981) correlated the long-term adfreeze strength (τ_a) to the long-term shear strength of the frozen soil (τ_{lt}) using a roughness factor "m" that characterizes the pile surface and other pile surficial variables such as impurities. The long-term adfreeze strength respectively was expressed in the following formula:

$$\tau_a = m\,\tau_{lt} \qquad [1]$$

The long-term shear strength of frozen soils (τ_{lt}) has often been expressed using Mohr-Coulomb failure criterion as follow:

$$\tau_{lt} = C_{lt} + \sigma_n \tan \emptyset_{lt} \qquad [2]$$

where, C_{lt} and $\emptyset_{lt}$ are the long-term frozen soil strength parameters and σ_n is the normal stress acting on the shear plane. In frozen grounds, Weaver and Morgenstern (1981) stated that the normal stress on the shear plane is typically less than 100 kPa which may make the contribution of the frictional component to the total strength insignificant, thus, may be neglected. Therefore, the long-term adfreeze strength was reduced to:

$$\tau_a = m\,C_{lt} \qquad [3]$$

The roughness factor "m" was first extrapolated by Weaver and Morgenstern (1981) when data on cohesion strengths of frozen soils was compared to the data on pile-soil adfreeze strength. A roughness factor of 0.7 was suggested for uncreosoted timber piles based on the ratio between adfreeze strength of timber piles in ice to the long-term cohesion of ice obtained from field and experimental data of Voitkovskii (1960) and Vialov (1959). A roughness factor of 0.6 was inferred for steel and concrete piles based on data from field experiments conducted by Johnston and Ladanyi (1972) and Crory (1963). Weaver and Morgenstern (1981) stated that the long-term adfreeze strengths of steel and concrete piles in the used data were not well defined, however, 0.6 represented the lower boundary and could still be acceptable for conservative pile design.

Ladanyi and Theriault (1990) evaluated the soil-metal adfreeze bond at -2°C and reported that the long-term shaft resistance of piles in frozen ground does not solely depend on the long-term adfreeze, but also on the long-term friction angle at the interface, and respectively on the total lateral ground stress. Therefore, they improved the Weaver and Morgenstern's (1981) equation by adding the contribution of the long-term friction angle of frozen soil and proposed the following formula:

$$\tau_a = m\, C_{lt} + \sigma_{n_{total}} \tan \emptyset_{lt} \qquad [4]$$

Interestingly, the experimental results from Ladanyi and Theriault (1990) showed not only the frictional resistance was dependent on confining pressure but the roughness factor too. For steel-frozen sand interface, the roughness factor increased from 0.1 to 0.3 when the confining pressure was increased from 100 kPa to 1100 kPa but never reached the value of 0.6 that had been suggested by Weaver and Morgenstern (1981). Although, Ladanyi and Theriault (1990) were not confident of using their short-term study to argue about the roughness factor validation and suggested more investigations, continuation of using constant roughness factors in the way that proposed by Weaver and Morgenstern (1981) may have become questionable.

There are some uncertainties associated with the suggested roughness factors as they were extrapolated form very limited field and laboratory data that lacked some information about soils and piles in certain conditions. The used data from Johnston and Ladanyi (1972) and Crory (1963), for example, were obtained from their field studies on adfreeze strength of steel rods and steel pipe piles respectively, with no record of experiments on concrete piles. It is not clearly shown that the adfreeze strengths and cohesion strengths were obtained at identical conditions (i.e., pile material, soils type, stress history, and thermal boundary) that allows for reliable comparisons and accurate estimation for the roughness factors. The roughness factor for a steel pile in frozen sand, for instance, could be different from that for a steel pile embedded in frozen clay or silt. It could also be different for two identical steel piles embedded in the same soil but at different ground temperatures or different depths. The different surficial characteristics for soil-soil interfaces from that for pile-soil interfaces may result in different responses at different exposure conditions, thus different roughness factors.

The current study is aimed at investigating the exposure thermal impact on shear strengths of frozen sands and steel-frozen sand interfaces. It also reevaluates the roughness factor "m" under different thermal boundaries and load conditions. Moreover, the paper discusses the contribution of frictional resistance to the shaft capacity of steel piles in frozen ice-poor soils.

2 PHYSICAL PROPERTIES OF THE TEST SOIL AND STEEL INTERFACE

The test soil is classified as poorly graded non-plastic sand in accordance to the USCS classification system (ASTM D2487-17, 2011). Its maximum dry density (ρ_d max) and optimum moisture content (w_{opt}) were determined using Standard Proctor test (ASTM D698, 2005) and found to be 1.85Mg/m^3 and 10% respectively. The particle size distribution for the sand was determined in accordance with ASTM D422, (2005). The sand contained approximately 5% fines passing through 0.075mm sieve.

For pile-soil interface testing, steel plates were used to simulate the shaft surface of a typical steel pile. Steel piles are commonly used in permafrost region with different geometries including pipe piles, H-section piles, and helical piers. The interface test specimens were 90mm by 90mm square steel plates with a thickness of 25.4mm, machined to couple with the upper half of the direct shear box apparatus and provide a steel-soil interface area of 60mm by 60 mm (Figure 1). The total and the average surface roughness values for this particular type of steel were reported by Giraldo and Rayhani (2013) to be 9.7μm and 11.3μm respectively. The steel plates were equipped with thermocouples that inserted in tiny holes underneath the upper surface of the plates in order to track the temperature change at the steel-soil interface.

Figure 1. Conventional and modified shear box for the interface test.

3 EXPERIMENTAL PROGRAM

All shear strength tests were conducted using a direct shear test apparatus in accordance to ASTM

D3080/D3080M-11 and ASTM D5321-12. The apparatus was placed inside a fully controlled environmental chamber to enable testing at various freezing temperatures. The direct shear test apparatus consists of an electrical motor that enables applying a constant shear velocity to the lower part of the shear box, while the upper part is restrained by a digital load cell connected horizontally to it. The electrical motor connected to a gearbox to enable adjustment of the shear velocity to the desired rate. Horizontal and vertical displacements are measured through a Linear Variable Differential Transducers (LVDT) connected to a digital logging station using LabView software. The apparatus frame facilitates applying normal stress to the top of the test specimen by incorporating a steel bearing arm. An illustration of the shear apparatus is presented in Figure 2.

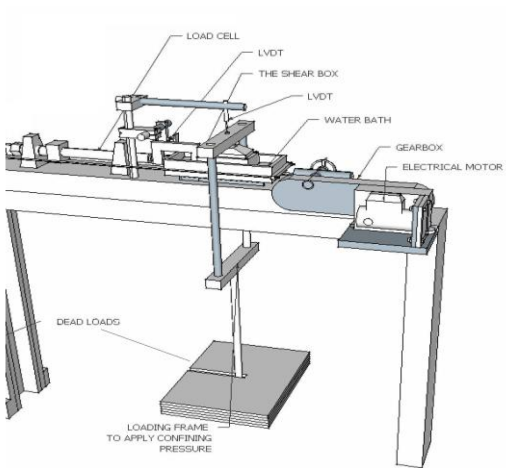

Figure 2. Direct shear test apparatus.

For frozen sand testing (soil-soil interface testing), a regular coupling shear box was used. This shear box had inner plan-view dimensions of 60mm by 60mm and depth of 40 mm. For steel-frozen soil interface testing, on the other hand, the modified shear box was utilized to simulate pile-soil interface characteristics.

The sand soil was prepared at a bulk density corresponding to its field capacity moisture content. This bulk density was determined by pouring the sand-water slurry into a known volume container and permitting water drainage and soil settling under gravity. When water flow stopped, the known volume soil sample was then weighed, and its density was determined to be 2.06Mg/m^3 at a field capacity of 13.5%. After keeping the test setup for 24 hrs. in the cold room, the shear load was applied at a constant shear velocity of 0.011mm/min (about 16.3mm/day). The shear tests were conducted at different temperature levels including -10℃, -7℃, -4℃, and -1℃,with an accuracy of ± 0.5℃. At each temperature, shear strength was determined under different normal stresses including 25kPa, 50kPa, 100kPa, and 200kPa.

4 RESULTS

4.1 Shear Strength of Frozen Sand

Stress-strain curves at different temperatures and normal stresses for frozen sand are presented in Figure 3. The peak strength was typically observed at shear strain ranged from 1.5% to 4.0% representing a displacement range of 0.9mm to 2.4mm respectively. The variation of shear strain at peak was associated mostly with test temperatures, where soil samples tested at higher freezing temperatures failed at lower shear strain. As the test temperature was decreased, however, the shear strain at failure respectively increased. Most of the shear tests were continued after the failure up to at least 10% strain (6mm displacement) in attempt of characterizing the residual shear strength. After peak strength, most of the test samples exhibited strength reduction followed by a period of strain hardening that continued up to 7% strain. After this period, all test samples showed brittle failure, recording very small residual strength as small as 10% of the peak values. The largest residual strength was observed for samples tested under 200kPa recording values around 250kPa (see Figure 3). Brittle behavior becomes more pronounced as the test temperature and normal stress decrease, recording larger post-failure strength reductions.

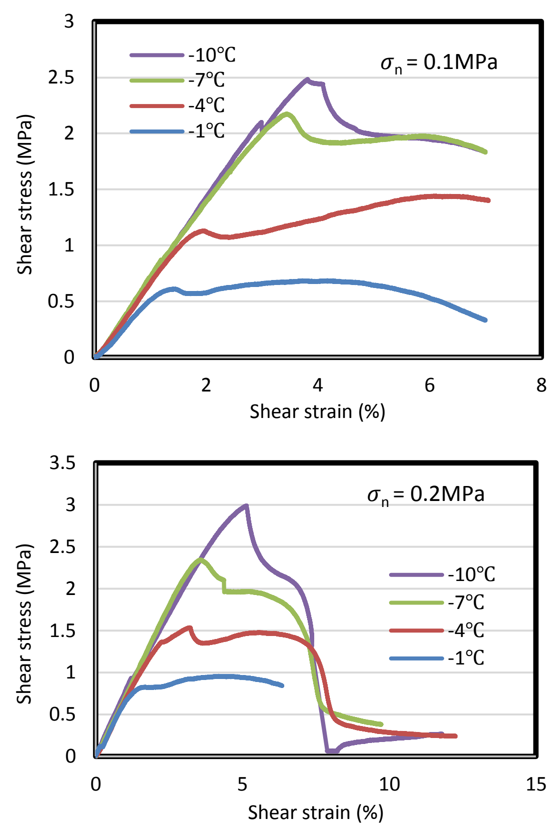

Figure 3. Sample stress-strain curves for frozen sand at different temperatures and normal stresses.

4.2 Shear Strength of Steel-Sand Interface

Steel-frozen soil interface testing program was aimed at studying the strength behavior of pile foundations in frozen ice-poor materials. The results from this experiment followed similar pattern exhibited by frozen sand with less stress-strain magnitudes at peaks (Figure 4). Interface peak strengths, for instance, happened usually at smaller shear strains compared to the frozen soil tests recording strain values ranged from 1% to 3%. As in frozen sand, higher strain at peak was recorded for the interface tests conducted at lower freezing temperatures and under higher normal stresses. This usually would refer to a higher ice bonding potential and larger frictional resistance which needed greater energy to be overcome. The brittle behavior dominated most of interface tests and was more pronounced compared to frozen sand.

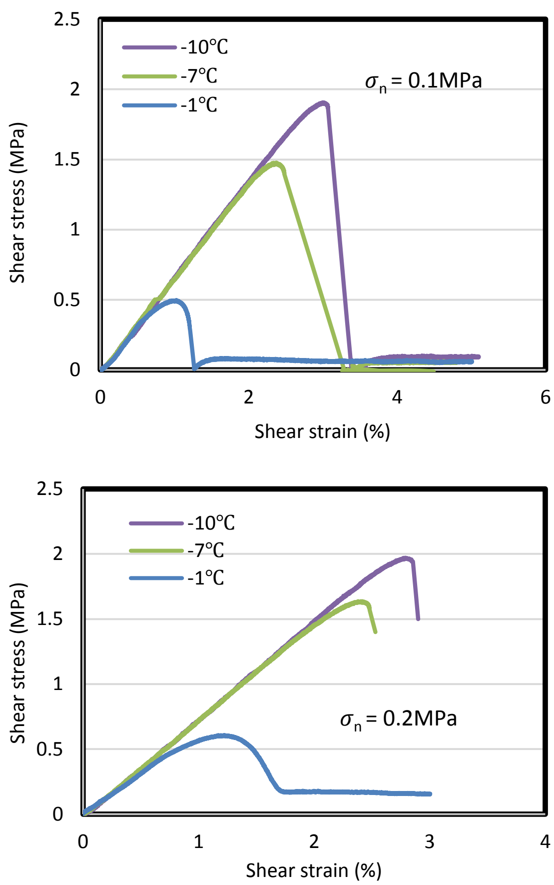

Figure 4. Sample stress-strain curves for pile-frozen sand interface at different temperatures and normal stresses.

Freezing temperature is directly proportional to the quantity of ice content and ice bonding concentration, thus to peak strengths too (Williams and Smith 1991). This was evidence of the higher peak interface strengths witnessed at lower freezing temperatures. At a normal stress of 25kPa and a test temperature of -10°C, interface peak strength was around 1.5MPa, however, the strength under a similar normal stress but at a temperature of -1°C, underwent significant reduction demonstrating ultimate shaft capacity of around 0.35MPa. This thermal change led to weakening shear strength of steel-soil interface by almost 77%. This, indeed, highlighted the significant effect of thermal change on the ultimate capacity of steel piles in frozen ground. Although, such dramatic temperature increase may not be encountered especially when considering macroscale level (i.e., global warming effects), it may happen in the microscale level due to different causes including improper thermal insulation measures and/or induced warming due to improper pile installation techniques. In the discontinuous permafrost zone, change from frozen to unfrozen condition is more likely to happen as the ground exists at relatively warm temperature.

5 DISCUSSION

The load carrying capacity of piles has been often directly correlated to the strength parameters of the surrounding soils using some correction factors. In unfrozen ground, for example, α – method is a commonly used method for pile design when undrained condition prevailed such as in cohesive soils and/or for quake loading. α is a correction factor that correlates shaft resistance of the pile to the undrained shear strength and can vary between 0 and 1.0 usually based on the ratio between undrained shear strength to the effective vertical stress. In frozen soils, in contrast, a roughness factor known as "m" is equivalent to the correction factor " α", however, "m" varies with pile materials only and is assumed to be irrelevant to the ground conditions. In this section, efforts are made to evaluate the suitability of this assumption by comparing peak cohesion of frozen soils with peak adfreeze strength of steel-soil interfaces at different temperature levels and normal stresses.

Using Mohr-Coulomb criterion, peak shear strengths versus normal stresses at different freezing temperatures were plotted for both frozen sand and steel-soil interface. The strengths of frozen soil as well as that of the interface samples were resultant of cohesion/adfreeze and friction resistance. At each temperature level, peak cohesions and peak adfreeze strengths were measured at the intersection point between Mohr-Coulomb straight lines and the vertical shear stress axis. Their values, the corresponding test temperatures, and the respective roughness factor "m" were all presented in Table 1.

At any given temperature, cohesion strength of frozen sand "c" was always higher than adfreeze strength of the pile-soil interface "c_a". Since sand soils are cohesion-less, adfreeze strength would be probably due to the existence of ice content. Therefore, the higher cohesion strength of frozen sand could be attributed to either higher ice content or lower unfrozen water content compared to pile-soil interface. Since steel has higher thermal conductivity, it would show lower temperature during freezing compared to the sand at any given time. Therefore, thermal gradient could exist and cause the liquid water to flow toward pile-soil interface resulting in higher residual inter-particles water contents adjacent to the shear plane.

Cohesion and adfreeze strengths experienced dramatic reduction as the exposure temperature increased toward freezing point. This possibly is due to the reduction in ice content and the increase in unfrozen water content as the temperature increased. However, the ratio between adfreeze strength and cohesion strength at different temperatures varied within a small range represented by the roughness factor "m" shown in Table 1. The correlation between roughness factor and the ground temperature plus the unity (T+1) was established and presented in Figure 5.

Table 1. Roughness factor "m" and frictional factor "n"

T °C	Soil-soil interface		Pile-soil interface		m	n
	C (MPa)	$\tan(\emptyset)$	C_a (MPa)	$\tan(\delta)$		
-10.0	2.19	3.47	1.54	2.38	0.70	0.69
-7.0	1.8	2.9	1.15	2.50	0.64	0.86
-1.0	0.44	1.76	0.33	1.40	0.75	0.80

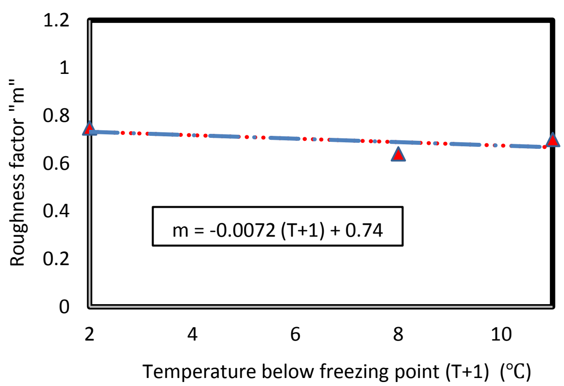

Figure 5. Variation of roughness factor with ground temperature

The graphical demonstration in Figure 5 shows a reduction trend of the roughness factor corresponding to temperature reduction. However, this reduction is very small compared to relatively large thermal change from -10°C to -1.0°C. Therefore, the roughness factor could be considered constant and insensitive to ground temperature for steel piles in frozen ice-poor soils and can be given an average value of 0.7. This supports the assumption of using a constant roughness factor of 0.6 proposed by Weaver and Morgenstern (1981) which may be considered conservative for the examined soil, pile material, and the test temperature.

The shaft resistance of the steel pile in frozen sand, however, was not solely depending on adfreeze strength but on frictional resistance and, thus, on the applied normal stress too. Generally, at high strain rates, a higher strength corresponding to higher confining pressures in frozen soils is expected. This was stated by Jones (1982) when increasing confining pressure up to 34MPa increased the strength directly by minimizing cracking occurrence. However, lower strength increase rate was recorded as the normal stress exceeded 34 MPa which was attributed to the pressure melting phenomenon; which transformed the ice to liquid at very high pressures. At low strain rates, however, less strength increase was observed with increasing confining pressure. This was attributed to the plastic deformation and predominant cohesive strength demonstrated at low strain rates.

Practically, installation of pile foundations in frozen grounds could induce horizontal stress increase at any depth along the pile shaft. This happens following to slurry freezing around piles installed in oversized holes, or by soil freeze-back occurrence after driving a pile directly in frozen ground or in a slightly undersized hole. Ladanyi (1988) theoretically showed that the mobilized lateral pressure due to pile installation in ice-rich soils may be relaxed within 24 hrs. falling to less than 1% of the before-installation lateral pressure. Nevertheless, a rapid dissipation of the mobilized lateral pressure may not occur in frozen soils because of their extremely low permeability and long consolidation time that could be longer than the service life of the structure. Therefore, Ladanyi and Theriault (1990) suggested that lateral stresses around a pile in frozen ice-rich are improbably to become less than the total lateral stress in the ground. Subsequently they proposed that the ratio between total lateral stress acting on the pile shaft to the total vertical stress could vary from k_0 for dense frozen soils to 1 for ice-rich soils and crystalline ice.

In the current study, the inclination in Mohr-Coulomb's straight lines indicates higher strengths for frozen sand and steel-soil interface samples as the normal stress becomes greater. This confirms the contribution of the frictional resistance to the ultimate strength for both frozen sand and shaft resistance of steel piles in frozen sand. The frozen sand, in addition, showed decreased friction angles ($\emptyset$) with increasing temperature recording 74° at -10°C and around 60° at -1.0°C. Steel-soil interface friction (δ), furthermore, followed similar pattern but with lower values at any given temperature exhibiting 67° at -10°C and around 54° at -1.0°C (Figure 6).

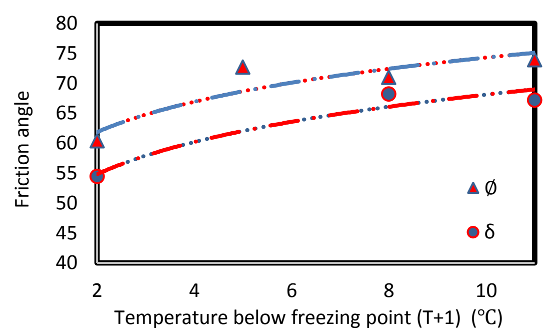

Figure 6. Friction angles and interface frictions at different ground temperatures.

Frictional resistance is predominantly due to soil particles interlock force which increases with increasing normal stress. Fractured ice crystals, moreover, contributes to the overall frictional resistance. At lower temperatures, therefore, higher friction angles were recorded due to a higher ice content. The smooth surface of steel piles would yield lower frictional resistance at the pile-soil interface compared to the soil-soil interface in frozen soils. This explains the lower interface friction angles recorded at the pile-soil interface compared to the friction angles exhibited by the frozen sand at any given normal stress.

For design purpose, it would be usual to provide correlation between the frictional resistance of the frozen soils with the frictional resistance that would be developed at the pile surface. This correlation can be analogous to the one provided by the roughness factor which correlates adfreeze strength to the cohesion strength of frozen soils. Therefore, by obtaining information about friction angle and cohesion strength of the frozen soil, the carrying load capacity of the piles can be accordingly estimated. For this reason, a frictional factor "n" similar to the roughness factor 'm" is introduced in this study to express the ratio between tan(δ) and tan($\emptyset$) with respect to temperature. The frictional factor "n" versus temperature is presented in Table 1 and Figure 7 respectively. Although, the data showed an average reduction trend of the frictional factor with decreasing the test temperature, the change is very small considering the large change in thermal boundary from -10°C to -1.0°C. The frictional factor "n" can be considered constant and given a value of 0.7 for the examined soil, pile material, and ground temperature range.

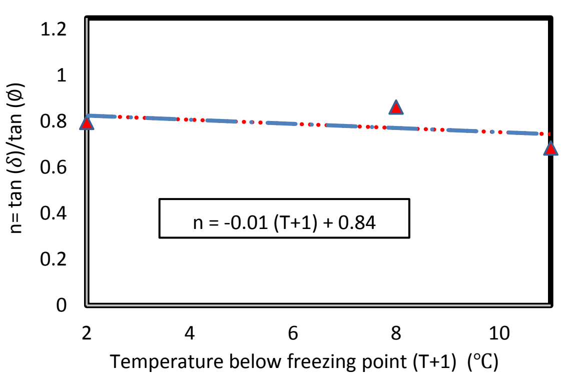

Figure 7. Variation of frictional factor with ground temperature for steel piles in ice-poor soils.

6 CONCLUSION

The current paper reviewed the design approach used for pile foundations in frozen ground. The ultimate shaft capacity of pile foundations in frozen sand is a resultant of adfreeze strength and frictional resistance developed at the pile-soil interface. A constant roughness factor of 0.7 was found to be reasonable to correlate the adfreeze strength of the steel pile shaft to the cohesion strength of the frozen sand with respect to ground temperature. A frictional factor "n" with a value of 0.7 was introduced to account for frictional resistance developed at the pile-soils interface based on the frictional resistance of the frozen soil with respect to ground temperature.

7 ACKNOWLEDGMENTS

This study was financially supported by Carleton University and the Natural Sciences and Engineering Research Council of Canada (NSERC). The authors would like to thank both agencies for this support.

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