Enhancement of Pile-Soil Adfreeze Strength in Warming Permafrost using a Freezing Liquid

by

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Abstract

Pile foundations are commonly used in permafrost area as load carrying elements for wide range of infrastructure systems mainly due to their high load carrying capacity in frozen soils. These foundations, however, are prone to loss of bearing capacity and increased displacements resulted from warm temperature in permafrost regions. In this thesis, a research program aimed at developing a new retrofitting technique for enhancing bearing capacity of steel pile foundations in warming permafrost area is presented. The main goal was to refreeze the thawed soils surrounding pile foundations in permafrost regions and develop adfreeze bond at the pile-soil interface level. In the proposed refrigeration technique, a low temperature freezing liquid (Antifreeze) is circulated into steel pile tubes to examine the possibility of reducing temperature of the pile and its surrounding soil, ideally induce freezing of the pile-soil interface, and enhance the bearing capacity of pile foundations in endangered warming permafrost regions.

A model pile load test setup was developed using steel pipe piles embedded in cohesionless ice-poor soils to examine the possibility of adfreeze bond formation induced by circulation of freezing liquid in the pile. The experimental results showed that local freezing is capable of decreasing temperature in the pile surrounding soil and developing adfreeze bond at the pile-soil interface. The pile shaft capacity significantly increased up to twice its initial shaft strength when the pile was exposed to circulation of antifreeze liquid for only 25 min. A finite element model was also developed using ABAQUS software and calibrated by the test results to investigate the effect of controlling parameters including pile diameter, thermal loads, and liquid circulation period on load carrying capacity of the piles.

Dedicated to my dear parents and my lovely wife

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CHAPTER 1: INTRODUCTION

1.1. Problem Statement

Permafrost is referred to any soil which remains in temperature below zero degree through more than one summer (Johnston, 1981). The temperature in permafrost is often near the melting point of ice and is very sensitive to the introduction of heat from man-made structures or the disruption of the insulating ground cover. Thawing may result in a significant loss of strength and ground settlements due to thaw-settlement in frozen soil. This could result in loss of bearing capacity of foundation systems and in turn failure of superstructures in permafrost. Pile foundations are mainly used to transfer load of the structures and infrastructure systems to the soil in frozen grounds. Preservation of the permafrost condition is as much a goal of foundation design as is bearing capacity.

Several types of pile foundations have been used in frozen ground to transfer superstructure and infrastructure load to frozen ground materials (Andersland and Anderson 1978). Timber piles used in permafrost foundation systems are often treated with wood preservative products to extend their service life spans (Andersland and Ladanyi, 2004). However, due to use of these treatments, the adfreeze bond at the pile-frozen soil interface may be reduced. Concrete piles are not the ideal choice to be used in frozen ground because of the relatively low tensile strength of concrete (Andersland and Ladanyi 2004). Steel piles are the most commonly used type of piles in frozen ground and permafrost region. They can be installed in both warm and cold permafrost. Pile foundations placed in frozen soil transfer their loads to the surrounding frozen soil principally through adfreeze bonding at the pile-soil interface. The ultimate bearing capacity of piles primarily

depends on the shaft resistance along the permanently frozen depth and the toe bearing contribution in frozen soil could be negligible (Parameswaran, 1978; Vialov, 1988).

The main strength characteristic determining the bearing capacity of a pile in frozen soil is the adfreeze bond strength between the pile shaft and its surrounding soil. For structures designed on frozen soils, the ground temperature is a significant feature for determining likely pile-capacities for resisting structural loads. Permafrost degradation has been predicted in the continuous permafrost zone. The load capacity of foundation systems in frozen soils depends upon maintaining the adfreeze bond strength at or below its design cold temperature. Breaking the adfreeze bond would lead to much weaker residual interface strengths. Foundation engineering practice in permafrost currently includes preserving the cold soils and protecting the adfreeze bond between soils and structural members. Several preservation and mitigation measures have been implemented to minimize heat transfer from the superstructures to frozen ground and maintain the adfreeze bond and, hence, avoid foundation strains (settlements or heaving). Open crawlspaces have been used to freely allow cold winter airflow below the entire building (Vyalov et al., 1993b, Perreault 2016). By use of insulations and crawl space technique, the heat generated from buildings which transfer to the soil under these structures have been prevented. Thermosiphons utilize passive soil refrigeration, and are used as a heat exchanger between the soil and the atmosphere to preserve permafrost in continuous and discontinuous permafrost zones (Holubec, 2008; Popov, Vaaz and Usachev, 2010; Zarling & Haynes, 1985). Thermosiphons and also thermal piles have been used to maintain the frozen condition of the soils. Cold Air Refrigerant Systems (CARS) have also been used to remove heat from the ground by circulating cold air in the pile foundation in winter to reduce the refreezing time (Johnston, 1981). The CARS can lower the freeze-back time for the cast-in-place piles and surrounding soil. Although these systems have shown to be

effective in minimizing heat transfer from superstructures into the frozen ground, their application in wide spread ground warming induced by climate change is fairly limited.

Due to temperature increase of soil and consequently thawing permafrost, bearing capacity and settlement of pile foundations are affected and buildings in permafrost regions are endangered. Previous techniques are working properly to keep the soil frozen against heat generated and transferred from the buildings, however, global warming is becoming the source of heat for these pile foundations in recent years. Therefore, current available techniques will not be capable of preventing heat transfer to the frozen grounds and loss of pile foundations' bearing capacity and settlement. More efficient design methods and retrofitting techniques will be needed for pile foundation design in warming permafrost and to maintain bearing capacity and displacement performance of pile foundations in thawing permafrost areas. This thesis examines the possibility of maintaining the shaft capacity of pile foundations in warming permafrost by circulating a freezing liquid into the piles.

1.2. Research Objectives

In this thesis, the aim is to increase the pile bearing capacity by inducing adfreeze bond between the pile and its surrounding soil through circulation of a freezing liquid into the pile. The cooling system is made by the use of antifreeze liquid and applying low temperature in a freezer. This hypothesis was examined through development of model pile load tests. The results from experimental tests were used to develop a numerical model in ABAQUS software to further investigate the controlling parameters of the systems. Pile foundations are being designed for different loading combinations including axial loadings, lateral loadings, and creep performance.

This research only considers axial loading conditions and the specific objectives of this study are listed below:

- To examine the possibility of decreasing pile and soil temperature through application of a cooling circulation method
- To explore the adfreeze bond development at the pile-soil interface by the cooling application and its impact on bearing capacity of pile foundations
- To understand the influence of controlling parameters such as pile diameter, temperature exposure time and thermal loading magnitude on performance of the proposed cooling method

1.3. Significance of The Work

Using antifreeze in pile foundations to reduce the pile and soil temperature and develop adfreeze bond at the pile-soil interface is an important mitigation technique to maintain high bearing capacity of pile foundations in thawing permafrost regions. This technique can be used as design method in new pile foundations as well as a retrofitting technique for existing pile foundations in cold regions exposed to global warming.

1.4. Structure of The Thesis

This thesis is written in a paper-based format and the contents of the thesis are divided in five chapters. Chapter 2 presents a comprehensive review of typical piling materials used in permafrost area and summarizes most of the design approaches used for pile foundation design in frozen grounds. In addition, the adfreeze bond characteristics of frozen grounds is discussed with

introducing the disadvantages of adfreeze when exposed to higher temperature. Moreover, current techniques to mitigate warming permafrost are explained in details and the need for new mitigation or retrofitting technique is proposed. Chapter 3 details the proposed novel technology along with experimental setup used for development and testing of the pile-soil system at different exposure temperatures. Chapter 4 presents the numerical model developed to simulate a steel pipe pile embedded in unfrozen sand and examine the possibility of adfreeze bond formation induced by circulation of freezing liquid in the pile. The finite element numerical models were developed employing ABAQUS software and calibrated based on the test results obtained from experimental model tests. Different aspects of model parameters including pipe pile diameter, initial temperature of soil and pile, and time exposure of pile and soil are presented in this chapter. Finally, conclusions of this research and recommendations for further investigations are summarized in Chapter 5.

1.5. Research Contribution

- Amanzadeh, A.H., and Rayhani, M.T., (2021). Enhancement of pile-soil adfreeze strength in warming permafrost using a freezing liquid. Innovative Infrastructure Solutions (submitted in Aug. 2021)
- Amanzadeh, A.H., and Rayhani, M.T., (2021). Improving load transfer of pile foundations in warm frozen ground through circulation of low temperature antifreeze liquid. International Journal of Geotechnical Engineering (submitted in Aug. 2021).

CHAPTER 2: LITERATURE REVIEW

2.1. Introduction

Frozen grounds in permafrost regions are often considered as durable bases for structural loads because of high bearing capacity of ice. Pile foundations have been widely used as foundation system in permafrost region for many years. These foundations have shown high bearing capacity and reliable performance for many infrastructure projects in frozen soils. High exposure temperature, however, can decrease the load carrying capacity of pile foundations and increase their displacement. To prevent transferring heat from buildings and structures into frozen soils, these structures are often built over frozen ground surface with a considerable gap (i.e., Crawl Space). Several techniques have also been implemented in recent years to maintain the freezing temperature and mitigate permafrost thawing including insulations and cooling systems, thermosiphon, coatings, air convection method, and sun sheds. Although most of these methods have shown to be successful in preventing the heat transfer from building to frozen ground, higher ambient temperature related to global warming could pose a new challenge for maintaining frozen ground temperature. This chapter reviews the current state of knowledge on techniques used for maintaining frozen ground temperature and previous research state in this field. This research aims at developing a novel refrigerant system to locally reduce ground temperature around existing and new pile foundations and, in turn, improve load transfer and creep.

2.2. Permafrost

Any soil which remains below zero degree through more than one summer is, by definition, permafrost or perennially frozen ground (Johnston, 1981). Approximately half of the surface area of Canada is underlain by permafrost in two forms of discontinuous perennially frozen patches, and continuous, in which the entire region is underlain by frozen soil. Discontinuous permafrost tends to occur where the mean annual air temperature is between -1.1°C and -8.3°C, while continuous permafrost exists where the mean annual air temperature is below -8.3°C. However, most discontinuous permafrost is not colder than -1°C and must be protected against thawing. The depth of permafrost may extend from a few meters up to hundreds of meters. For example, the permafrost in the Arctic Archipelago is approximately 460 meters deep (Johnston 1981).

The upper portion of permafrost close to ground surface may experience annual melting and refreezing (known as 'active zone') and is responsible for annual frost heave. The permafrost may consist of any combination of organic soils, mineral soils, and rock that are frozen. The ice content may range from negligible to bulk ice. Through gradual freezing, large ice lenses can develop within the soil (Johnston, 1981), and over time, the body of water may freeze forming a large amorphous ice mass within the permafrost. The temperature in permafrost is often near the melting point of water and is very sensitive to the introduction of heat from man-made structures or the disruption of the insulating ground cover. Thawing may result in a significant loss of strength and foundation settlements due to thaw-settlement in frozen soil. This could result in loss of bearing capacity and in turn failure of superstructures in permafrost.

Foundations must support load from the structure and transfer it into the underlying permafrost. At the same time, they must also avoid disrupting the thermal regime of the permafrost.

Permafrost can be disrupted either during the construction phase or throughout service life of the structure. Preservation of the permafrost condition is as much a goal of foundation design as is bearing capacity.

Frozen soil can be considered a highly complex four phase system that consists of solid soil particles, ice bound, low temperature liquid water, and air or other gas inclusions (Black, 2001). Frozen soil properties depend on the relative proportions of each constituent, their internal properties, and their interaction with the other components. This complex composition may show properties that are not representative of either ice or soil. Although frozen soil shows some of ice typical behaviour, it also shows frictional behaviour which depends on the soil particles physical properties. To simplify behavioural modelling, the water and gas components are often ignored to simplify it to a rheological two-phase material. This assumption is mostly valid for coarse-grained granular materials, as fine-grained (clay and silt) soils are mainly affected by unfrozen water content and it strongly influences strength and creep parameters of these soils.

Most fine-grained soils compose of silicate minerals, mostly in the form of small clay particles with high surface area. Silicate minerals tend to absorb the polar water molecules. Due to silicate and water bond formation, a layer of water molecules is held to the particle surface. Previous research (e.g., Jellinek, 1967; Weyl, 1951) has shown that ionic crystal formations (such as ice) are not fixed but converted and polarized. In the ice molecules case, this results in a liquid layer formation on its surface. Silicate and ice particles share the two layers of water bound which are in interaction. Ice formation in frozen soil occurs in the bound surface water layers (Anderson, 1968).

2.3. Pile Foundations in Frozen Ground

Several types of pile foundations have been used in frozen ground to transfer superstructure and infrastructure load to frozen ground materials. These piles could be classified in terms of their material, installation methods, their load transfer and the amount of displacement generated during installation.

2.3.1. Timber piles

Timber piles are the most affordable type of piles when they are in access locally, and also they are commonly used in northern region. Timber piles generally are tapered with larger diameter section at the base to provide greater uplift resistance against frost heave (Crory, 1966). Timber pile lengths range from 6 to 15 m, with top diameters from 150 to 250 mm, and base diameters from 300 to 350 mm. Timber piles are mainly built from local materials including spruce, Douglas fir, or pine. They generally remain well preserved if embedded in permafrost, however, they may experience decline in quality and as a result deteriorate when active layer exists. Therefore, wood preservatives are usually utilized as treatment for timbre piles to extend their service life spans (Andersland and Ladanyi, 2004). However, adfreeze bond at frozen soil-pile interaction may experience some reduction when some of these treatments are used. Possibility of ground temperature disruption is reduced, due to good insulation characteristics provided by timber piles. Although they provide a relatively high bond strength, timber piles have low structural capacity and may fail to resist against the mechanical driving loads into the frozen ground.

2.3.2. Concrete piles

The concrete piles, including precast and cast-in-place piles, are not the best option of pile systems in permafrost and frozen ground, because of low tensile strength of concrete materials. Tensile strength may be increased significantly due to frost heave in permafrost regions, and consequently generate cracks and expose the steel rebar in concrete piles to corrosion (Andersland and Ladanyi, 2004). Therefore, concrete piles are not often used in North American permafrost areas, however, in Siberia precast reinforced piles have been widely used with square and circular sections. They are either installed in pre-thawed ground or predrilled holes and then by the use of grouted concrete will be backfilled. Pre-stressed concrete piles can minimize the frost heave effect, and respectively enhance the performance of precast concrete materials. Additionally, easier transportation of the precast prestressed concrete piles compared to precast concrete piles is an advantage of prestressed concrete piles. Potential thermal disruption of frozen ground caused that concrete piles barely be cast in place in permafrost areas until recent years. However, usage of cast in place concrete piles has increased more than ever before in permafrost regions due to the development of a high-aluminous cement, that allows rapid grout curing and strength development at low temperatures as low as -10 °C (Biggar and Sego, 1990; Benmokrane et al., 1991).

2.3.3. Steel piles

The most commonly used type of piles in permafrost regions are steel pile because they can be either installed in warm or cold permafrost areas. The pipe pile and H-steel piles are the most used steel piles. Pipe piles are often used when laterally load resistance is needed and can be filled with concrete or sand to enhance their bearing capacity. Open-ended steel-pipe and H-steel piles used in warm permafrost are usually driven in deep frozen grounds in order to sufficient adfreeze bond

is acquired and resulted in high bearing capacity. In cold permafrost, usually closed-ended pipe piles are installed in oversized predrilled holes, and backfilled by grouted concrete. Linell and Johnston (1973) observed that steel piles are well preserved against corrosion when they are embedded in permafrost, however, small amount of corrosion was found along the active layer depth. Steel pipe piles can also be used as thermal pile if an exchange-heat tube installed inside them to maintain the thermal regime. By adding special lugs on steel pile sections (Andersland and Alwahhab, 1982, 1983) or corrugated surface manufacturing of steel piles (Thomas and Luscher, 1980), the overall load bearing capacity of these piles can be enhanced significantly.

2.3.4. Other piling techniques

Pre-stressed concrete piles are often used in permafrost because of high tensile capacity of pre-stressed materials to withstand stresses resulted from frost heave. Longer time of concrete freeze-back makes the cast-in-place concrete piles disadvantageous in frozen ground implementation. Additionally, adfreeze strength is decreased due to hydration heat generated in concrete curing process. Pile carrying capacities can be enhanced by section modification, including driving protruding spikes into the sides of timber piles, and welding steel pieces onto steel piles (Anderson and Alwahhab, 1983; Crory, 1966). Higher load carrying capacity of piles is achieved by forcing failure to occur in the frozen soil instead of along the pile surface when using modified section piles. Practically, it is time consuming and costly to make such modifications, but pile capacities can be increased significantly (Andersland and Ladanyi, 2004). With using corrugated steel pieces or irregularities on pile side surfaces higher levels of adhesion can be achieved in permafrost pile systems. Insulated or thermal piles can be used to maintain the temperature of permafrost at low levels (Phukan, 1980; 1985). Insulating piles from above ground structures is a commonly used technique to prevent heat transfer to the piles and ensure better long-term thermal stability of the

frozen ground around the piles. Thermal piles equipped with special thermal devices that are placed in a slurry within the piles. Thermal devices flow heat from the permafrost and transfer it to the atmosphere during the winter, consequently, the temperature of the permafrost remains low and the pile adfreeze is maintained.

2.4. Load Transfer of Pile Foundations in Frozen Ground

Pile foundations placed in frozen soil transfer their loads to the surrounding frozen soil principally through adfreeze bonding at the pile-soil interface. Adfreeze is a shear bond between the pile shaft and the adjacent frozen soil. Generally, adfreeze is considered a combination of the ice-to-surface bonding (adhesion), mechanical interaction, and friction (Weaver and Morgenstern, 1981). The proportions of each mechanism is related to the soil temperature, strain rate, ice, water and particle contents. Generally, adfreeze piles placed in permafrost regions are designed as friction piles due to limited contribution of end bearing component, making use of the available adfreeze between the frozen soil and shaft (Parameswaran, 1978).

Theriault and Ladanyi (1988) developed a numerical model which describes the load transfer mechanism in a friction pile in permafrost. Shaft bond failure mechanism was described for each given applied load in developed model. To simulate the confining pressure effect caused by freeze back during soil failure, a minimum shear strength (often called “residual” shear strength) was included in the numerical model to satisfy the frozen soil circumstances. The numerical model showed a uniform “residual” shear resistance along pile shaft length when pile was subjected to complete failure loads. Therefore, loads below the failure load were redistributed with time. Figure 2.1 illustrates the adfreeze failure progress of pile over time over the length of

the pile shaft. This model is similar to the solution proposed by Mattes and Poules (1969) with the exception that a yield criterion is included in Theriault and Ladanyi's model.

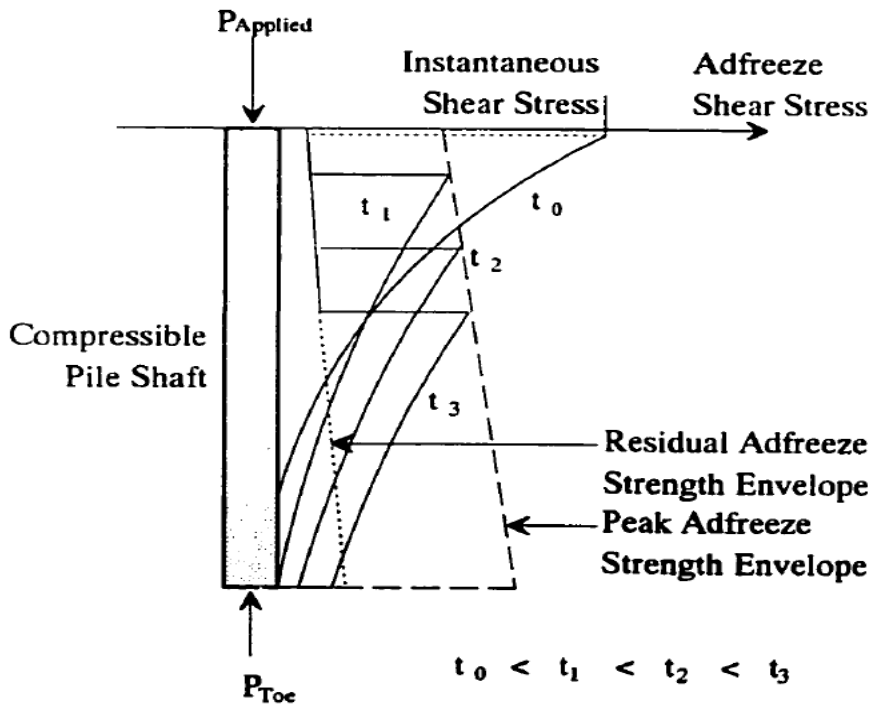


Figure 2.1: Distribution of adfreeze shear stress versus time (modified from Theriault and Ladanyi, 1988)

As most unfrozen soils exhibit limited creep displacement in comparison to frozen soils, the behaviour of long piles installed in both frozen and unfrozen soils may vary significantly. Short term creep settlement of the frozen soil along the pile shaft results in large shear strains occur along pile shaft without reaching failure (Black, 2001). Due to excessive strain capacity along the pile shaft, an applied load can be transferred further down the pile length. This can enhance the pile end-bearing capacity significantly. The end-bearing loads contribution was reported to range from a few percent to 24% of the applied pile load (Theriault and Ladanyi, 1988). Vialov (1988) reported an end-bearing load contribution of 8% for model steel pipe piles in hard-frozen loams at -4°C.

For plastic loams, the end-bearing loads were measured at 19% of the applied load. The structural characteristics of piles were also reported to contribute to load transfer mechanism of piles in frozen ground. Rigid piles showed more uniform load distribution in the soil, which caused the creep distribution along the entire shaft and the development of significant end-bearing loads (Vialov, 1988). This behaviour is slightly different from the long and slender piles. Mattes and Poules (1969) conducted a pile model analysis and found out that pile end-bearing load carrying capacity was a function of the length-to-diameter ratio and also pile shaft compressibility, which is a supportive fact regards to pile behaviour observed by Vialov (1988).

Modulus of elasticity of frozen soil is very high compared to unfrozen soil, however, the effect of pile stiffness in frozen ground is relatively unknown. Upon increase of soil elastic modulus induced by freezing, soil ability to strain against the shear stress from the pile reduces. Therefore, frozen soil is subjected to higher shear stresses, which will result in a very high load transfer rate near the ground surface. This resulted load transfer rate would be reduced with depth, resulting in negligible load transfer to the lower portion of the pile, while the adfreeze strength may be experienced a complete mobilization near the surface.

Pile surface characteristics were also found to have a significant effect on the adfreeze strength (Parameswaran, 1981). Figure 2.2 illustrates the load transfer for four piles with different surface roughness and shape. The H section cannot be considered as an ideal representative of the true steel pile behaviour because of its numerous corners caused stress concentrations, and reduced the potential bond strength. Additionally, the contribution of friction and interlock on total adfreeze strength is shown in Figure 2.2.

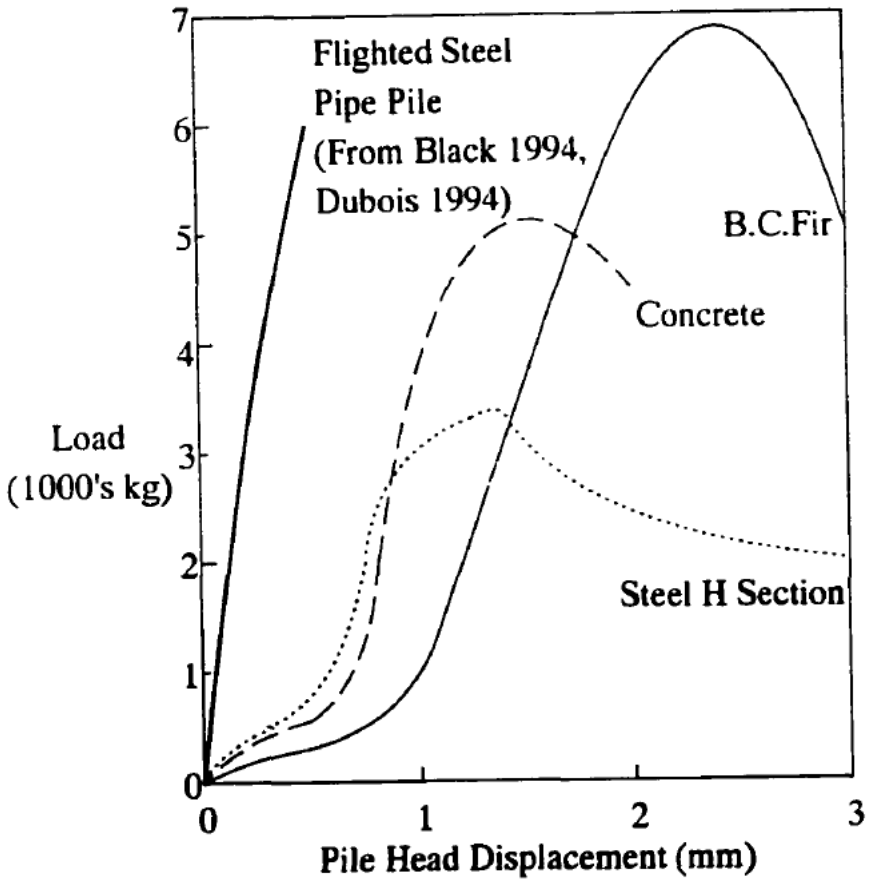


Figure 2.2: Typical load-displacement curves for piles with different surface conditions (from Parameswaran, 1981)

Parameswaran (1981) hypothesis regarding the increase of adfreeze strength was that it is a function of pile surface porosity increase (B.C. Fir vs. Concrete vs. Steel), thereby the possibility of bond area increase was proposed if water allowed to enter the pores of the concrete and timber piles. The pores may generate individual ice crystals extending across the potential failure surface. In rapidly frozen soil, the crystals are small, likely very smaller than the pores, however, the ice crystals would become larger in pores when soil exposed to normal freezing condition. Weaver and Morgenstern (1981) found that adfreeze shear strength is inversely related to ice content and directly related to pile surface roughness. For rough piles, the strength is determined away from

the immediate contact surface between the pile and soil and involves the interaction between the frozen soil and asperities. Later, Sego and Smith (1989) reported that sandblasting of the pile surface doubled the adfreeze strength at the steel pipe-backfill interface in frozen soil. The adfreeze bond between the pile and soil was also more ductile as the peak strength was reached at a much larger displacement (three times). Residual strengths, however, appeared to be comparable to strengths determined from other tests irrespective of the surface treatment. Pile material to which the ice is bonded, is also a significant factor in the bond strength. Coatings, rust, metal composition and the contamination of water may degrade the strength or change the behaviour of the adfreeze bond (Sego and Smith, 1989).

Early studies by Jellinek (1958, 1959,1962) demonstrated that the pile-frozen soil adhesion strength is a function of temperature. From 0°C to -13°C, shear failures were found to be at the interface between the steel pile and ice, while below this temperature, the failure occurred within the ice. On the other hand, the pile-frozen soil adhesion was found to be temperature dependent above -13°C, and independent below. The modulus of elasticity of frozen soil also increased with decrease in temperature.

The adfreeze bond strength between the pile surface and soil is controlled by the degree of contact at the pile-soil interface. This contact is controlled by four factors; thermal expansion different coefficients of the engineering material and the soil-ice system, the compressibility of the pile shaft and its lateral strain under compression, the soil-ice formation system at the pile interface, and the existing lateral pressure caused by the overburden. Adfreeze also depends on normal stress acting on pile surface and density in cohesion-less granular soils whose strength is essentially frictional in nature (Heydinger, 1987). Alternately, in fine grained soils, adfreeze mainly has cohesive nature. Ladanyi and Morel (1988) reported that, under similar density and

confining pressure, dilatancy (volumetric expansion during shear) could be much greater for frozen saturated sand than unfrozen sand. Ladanyi and Morel (1988) reported that unconfined dilation of frozen Ottawa sand at failure was four times greater than when it is unfrozen. Ice-cavitation was also demonstrated in frozen sand below the critical confining pressure at failure. Strain hardening occurs for frozen soils with soil dilatancy during failure (also known as dilatancy hardening).

2.5. Design Approaches for Pile Foundations in Frozen Grounds

Initially allowable load carrying capacity resulted from the ultimate adfreeze strength of pile-soil interface was the main design criterion for pile foundations in frozen soils (e.g., Vialov, 1959; Crory and Reed, 1965; Penner, 1970, 1974; Penner and Irwin, 1969; Penner and Gold, 1971; Dalmatov et al., 1973). An allowable load carrying capacity that would generate a tolerable displacement rate (or creep) over the structures lifetime was also proposed to incorporate the time-dependent behaviour of frozen soils (Johnston and Ladanyi 1972). Many researchers including Nixon and McRoberts (1976), Morgenstern et al. (1980), and Weaver and Morgenstern (1981) recommended to check both these load carrying capacities and design based on the lower value between them. The constitution components of soils in frozen ground could be an effective factor in pile-frozen soil failure criterion (Weaver and Morgenstern, 1981). Both the creep settlement and bearing capacity govern the failure criteria in ice-poor soils, while in ice-rich soils only creep settlement governs pile-frozen soil failure. In ice-poor soils primary creep governs settlement of pile under design load, while the secondary creep mostly governs pile settlement in ice-rich soils. Pile settlement in both cases must not exceed the displacement limit which can be tolerated along the design life of the supported structure in all cases. Therefore, pile foundation design protocol can be categorized under two groups; one indicates the design approaches based on ultimate bearing capacity, and the other demonstrates creep settlement design approaches.

2.5.1. Bearing capacity of piles in frozen grounds

The ultimate bearing capacity of piles primarily depends on the shaft resistance along the permanently frozen depth (Parameswaran, 1978). Crory (1963) claimed that the end bearing resistance would be considerable only for piles installed in ice-poor frozen soils like granular soils, and in ice-rich soils the contribution of end bearing strength could be insignificant. The end bearing capacity for piles with a base diameter smaller than 150 mm could be insignificant for design considerations regardless of the soil type (The US Army/Air Force, 1967).

The earliest approach for pile adfreeze strength estimation in frozen soils was proposed by Vialov (1959) through an empirical equation as a function of ground temperature:

$$\tau_{al} = \sqrt{1.65\,\theta} - 0.3 \qquad (2.1)$$

where: τ_{al} is the long-term adfreeze strength (kgf/cm^2) and θ is the positive value of the temperature below the freezing point (°C). In 1963, Crory (1963) developed different relations for estimating adfreeze strength based on a series of full-scale loading tests on 200mm diameter steel pipe piles installed in predrilled frozen silty sand and shaft-grouted with silt-water slurry at ground temperature of -4°C to 0°C:

$$\tau_{ul} = (0.0176\ T + 0.154)\ T + 0.005 \qquad (2.2)$$

$$\tau_{al} = (0.012\ T + 0.107)\ T + 0.002 \qquad (2.3)$$

where, τ_{ul} and τ_{al} are the ultimate and sustained adfreeze strength (MPa) respectively; T is the positive value of the freezing temperature (°C).

Weaver and Morgenstern (1981) were the first who correlated the long-term shear strength of the frozen soil (τ_{lt}) to the long-term adfreeze strength (τ_{al}) using a roughness factor "m" that characterizes the pile surface. The long-term adfreeze strength was expressed in the following formula:

$$\tau_{al} = m\ \tau_{lt} \qquad (2.4)$$

The long-term shear strength of frozen soils (τ_{lt}) usually has been expressed by Mohr-Coulomb failure criterion:

$$\tau_{lt} = C_{lt} + \sigma_n \tan\emptyset_{lt} \qquad (2.5)$$

where: C_{lt} and $\emptyset_{lt}$ are the long-term strength parameters of frozen soils and σ_n is the normal stress acting on the pile shaft. Weaver and Morgenstern (1981) concluded that long-term shear strength of the frozen soil would be negligible when the normal stress on the pile shaft is regularly less than 100 kPa. This makes the contribution of the frictional component to the total strength inconsequential, therefore may be ignored.

Based on the ratio between the long-term shear strength of ice and adfreeze strength of timber piles in ice, Voitkovskii (1960) and Vialov (1959) proposed a roughness factor of 0.7 for timber piles. Johnston and Ladanyi (1972) and Crory (1963) suggested a roughness factor of 0.6 for steel and concrete piles. For the long-term adfreeze strengths of concrete and steel piles, the roughness factor of 0.6 was found to be acceptable as the lower boundary for pile design (Weaver and Morgenstern, 1981). Through the soil-metal adfreeze testing at -2°C, Ladanyi and Theriault (1990) noticed that long-term shaft resistance of piles in frozen soil have been influenced by both

the adfreeze and friction angel of the soil. Thus, they proposed the following formula by adding long-term friction angle ($\emptyset_{lt}$) of frozen soil:

$$\tau_{al} = m\ C_{lt} + \sigma_{n\ total} \tan\emptyset_{lt} \qquad (2.6)$$

Ladanyi and Theriault (1990) also indicated that the frictional resistance was influenced by both normal stress (σ_n) and the roughness factor (m). When the normal stress was increased from 100 kPa to 1100 kPa for steel-frozen sand interface, an increase of 0.1 to 0.3 was observed in the roughness factor, but never reached the value of 0.6 that had been recommended by Weaver and Morgenstern (1981). However, Ladanyi and Theriault (1990) preferred to use the roughness factor of 0.6 proposed by Weaver and Morgenstern (1981) as they stated that further investigations are needed.

2.5.2. Deformation behaviour of frozen grounds and piles in frozen grounds

Glen (1952, 1955) was one of the first researchers that tried to realize the creep behaviour of frozen soils and examined temperature-dependent creep behaviour of polycrystalline ice. Three different stages were reported to control the ice deformation behaviour under compression stress. In stage one, a small elastic deformation was recorded immediately after the load application. During the second stage, which forms a transient stage, ice deformation occurs at slower rate with a decreasing trend over time. A steady state creep rate known as secondary creep is developed in the third stage. After showing the steady state creep rate, the ice samples were subjected to very high pressure and the ice experienced failure eventually. As ice is a crystalline material, therefore, its deformation was modeled using similar creep theories for metals (Hult, 1966; Odquist, 1966). The following creep law was proposed by Glen (1952, 1955) to demonstrate deformation of polycrystalline ice in simple compression:

$$\dot{\epsilon}_{zz} = A\ \sigma_{zz}^{n} \qquad (2.7)$$

where $\dot{\epsilon}_{zz}$ interprets axial strain rate; σ_{zz}^{n} denotes axial stress; A is constant for a given temperature and ice type; and n is an experimentally derived exponent. An exponent "n" value of 4 was picked through the log-log plot of strain ϵ_z against stress σ. By applying higher stresses, the greater exponent values could be obtained at higher stress levels. Mellor and Testa (1969b) reported a smaller exponent "n" of 1.8 for long-term low-stress test at -2°C and confined a power law relationship. Later, a power law relationship which includes wider range of stresses was proposed by Mellor and Testa (1969b) using combined results from others (Mellor and Smith, 1967; Mellor and Testa, 1969a). A linear viscose behaviour was reported through creep measurements under low stresses and temperatures close to freezing point Butkovich and Landauer (1959, 1960). On the other hand, Voitkovskii (1960) observed non-linear viscous behaviour in simple shear and torsion tests for polycrystalline ice subjected to higher stresses than 43 kPa and colder temperature than -1°C. Total creep strain of polycrystalline ice would include the combination of elastic, transient strain and the steady-state creep strain, and can be expressed in the following form (Barnes et al, 1971):

$$\epsilon = \epsilon_0 + B\ t^{1/3} + \dot{\epsilon}_s\ t \qquad (2.8)$$

where ϵ_o denotes the instantaneous elastic strain; B denotes an appropriate constant for transient creep; $\dot{\epsilon}_s$ denotes the steady-state creep rate; t is time.

In general, the creep response of frozen soil under an applied load can be categorized in three different stages depending on the ice content, stress level, and ground temperature. These stages are known as primary creep, secondary creep, and tertiary creep rate (Nixon and McRoberts, 1976). The creep rate decreases continuously in the primary stage, remains constant in the

secondary stage, and continually increases in the tertiary phase of the creep. Due to lack of creep data for frozen soils at the confining stress range of 0 to 138 kPa, Nixon and McRoberts (1976) used published uniaxial creep test data on ice to predict axial strain of frozen soil by developing analytical solution. For a temperature range of 0°C to -11°C, the creep parameters were developed using a log-log scale creep data as the following formula:

$$\epsilon_z = B_1\, \sigma_z^{\,n1} + B_2\, \sigma_z^{\,n2} \qquad (2.9)$$

B_1, B_2, n_1, and n_2 are temperature dependent fitting parameters, and σ_z is the applied axial stress. A series of approximate correlations (Figure 2.3) were developed by the authors to predict fitting creep parameters for ice and ice-rich soils at a given temperature as (Table 2.1):

$$B1 = 8E\text{-}3\ (T + 1)^{-2.37}\ ;\ B2 = 1E\text{-}6\ (T + 1)^{-1.9}\ ;\ n1 = 1.35\ (T + 1)^{0.2}\ ;\ n2 = 4$$

where T is the temperature below the freezing point. By substituting these parameters in the axial strain equation, the following formula was obtained:

$$\epsilon_z = 8E^{-3}\ (T + 1)^{-2.37}\ \sigma_z^{1.35\ (T+1)^{0.2}} + 1E^{-6}\ (T + 1)^{-1.9}\ \sigma_z^{\ 4} \qquad (2.10)$$

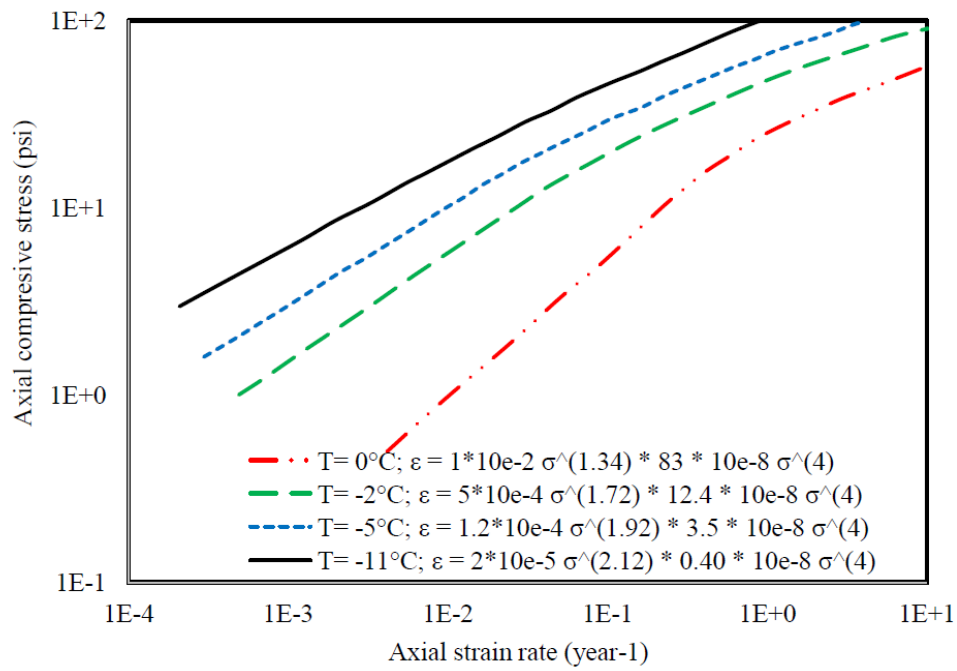


Figure 2.3: Axial stress versus axial strain for ice (modified from Nixon and McRoberts, 1976)

Table 2.1: Creep parameters for ice and ice-rich soils (Nixon and McRoberts, 1976)

Temperature (°C)	B_1 **(psi^{-1}. year $^{-1}$)**	n_1	B_2 **(psi^{-1}. year $^{-1}$)**	n_2
0.0	1.0E-2	1.34	83E-8	4.0
-2.0	5.0E-4	1.72	12.4E-8	4.0
-5.0	1.2E-4	1.92	3.5E-8	4.0
-11.0	2.0E-5	2.12	0.4E-8	4.0

Based on applied shaft shear stress (τ_a), Nixon and McRoberts (1976) predicted the normalized steady state creep rate of piles ($\frac{u_a}{a}$) in frozen ground by using ice creep law as the following equation:

$$\frac{u_a}{a} = \frac{3^{\frac{n1+1}{2}} B_1 \tau_a^{n1}}{n_1 - 1} + \frac{3^{\frac{n2+1}{2}} B_2 \tau_a^{n2}}{n_2 - 1} \qquad (2.11)$$

where u_a is the steady pile displacement rate, τ_a is the applied shaft shear stress in terms of the pile radius (a), and B and n are the temperature dependent parameters determined from uniaxial creep data for the frozen soil in question. Morgenstern et al. (1980) came up with different temperature-dependent parameters B and n to improve the axial strain model proposed by Nixon and McRoberts (1976) using additional creep data on ice (Table 2.2). A new flow law for piles in ice or ice-rich soils was given the following format:

$$\frac{u_a}{a} = \frac{3^{\frac{n+1}{2}} B \tau_a^{n}}{n - 1} \qquad (2.12)$$

A graphical chart was created using Nixon and McRoberts (1976) and Morgenstern et al. (1980) to indicate correlation between the normalized creep rate (h^{-1}) and the applied shaft stress (kPa) (Figure 2.4).

Table 2.2: Creep parameters for ice and ice-rich soils (Morgenstern et al., 1980)

Temperature (°C)	B (psi^{-1}. $year^{-1}$)	n
-1.0	4.5E-8	3
-2.0	2.0E-8	3
-5.0	1.0E-8	3
-10.0	5.6E-9	3

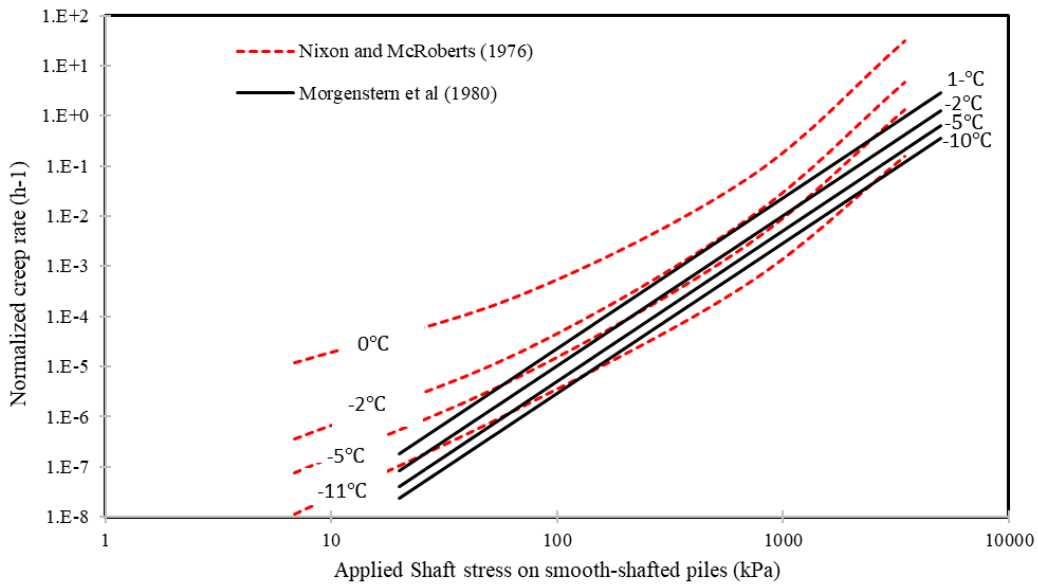


Figure 2.4 : Summary of pile creep models proposed by Nixon and McRoberts (1976) and Morgenstern et al. (1980)

Settlement should be the design criterion for piles in ice-rich soils, however, pile design in ice-poor soils should be governed by both strength criteria and settlement as suggested by Weaver and Morgenstern (1981) based on their comprehensive research on the pile performance. Thus, they proposed a creep law for ice-poor soils as follows:

$$\epsilon_1 = D(\sigma_1 - j\sigma_3)^c\, t^b \qquad (2.12)$$

where: $j = \frac{(1+\sin\varphi)}{(1-\sin\varphi)}$, and $D = [\frac{1}{w\,(\theta+1)^k}]^c$. φ is the internal friction angle, t is the time elapsed after the application of the load (h), θ is the temperature below the freezing point of water (°C) and w, b, c, k are material dependent parameters. Flow law model for piles in ice-poor situation was proposed using creep law model of ice-poor soil as the following equation:

$$\frac{u_a}{at^b} = \frac{3^{\frac{c+1}{2}} D\tau^c}{c-1} \tag{2.13}$$

Thaw settlement also could influence the creep behaviour of pile foundations in frozen grounds. The effect of temperature on pile creep and creep rate have been indicated in different studies. Ladanyi (1995) installed piles in ice-rich silt exposed to ground temperature of 1°C. A 35% increase in the creep settlement was estimated for a pile subjected to constant axial load. In a different study, a pile embedded 6m in frozen silty clay and subjected to 200 kN load at -1.3°C temperature was studied for 0.1°C warming temperature rate per year, and an increase in creep settlement of 30% was estimated over 25 years by Nixon (1990a).

Parameswaran (1979) was one of the first researchers who studied creep behaviour of piles in frozen grounds using test piles made of wood, concrete, and steel with different surface roughness in ice-poor soils The recorded creep behaviour was similar to steady state creep of viscoelastic materials at high temperature. Parameswaran (1979) also measured creep behaviour of piles in ice-poor soils affected by permafrost freezing. New thermal condition was applied by lowering the temperature from -6°C to -10°C and a reduction in the steady state creep rate of pile was observed when piles placed in -10°C for 24 hours. To evaluate the permafrost warming effect, the temperature was brought back to -6°C and the creep rate increased steadily at a lower rate than the original rate before freezing (Figure 2.5).

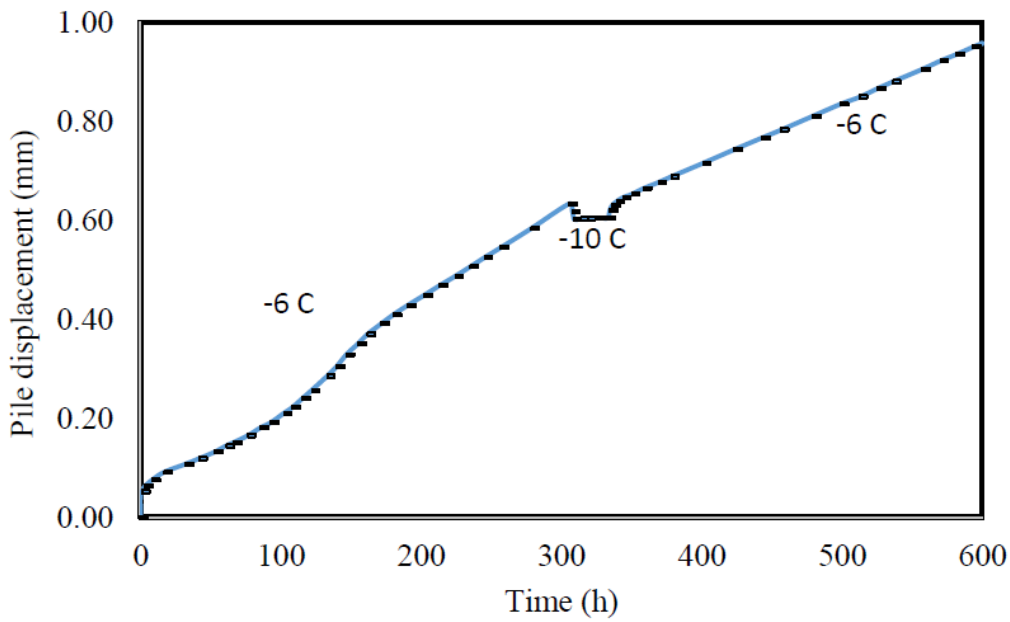


Figure 2.5: Impact of thermal change on creep rate of a model pile in frozen soil (after Parameswaran, 1979)

Parameswaran (1979) also derived the following equations to estimate pseudo-instantaneous displacement and the creep rate of helical piles by using creep data from tests performed on single helix helical piles in frozen silty clay:

$$F(q_{net}) = s_i = s_c^i \frac{C\,(q_{net} + \gamma D)}{q_{as}^i - (q_{net} + \gamma D)} \qquad (2.14)$$

$$G\,(q_{net}) = s^{\cdot} = s_c^{\cdot} \frac{c^{-}\,(q_{net} + \gamma D)}{q_{as}(q_{net} + \gamma D)} \qquad (2.15)$$

where $F(qnet)$ and $G(qnet)$ are the pseudo-instantaneous displacement and the creep rate respectively, and s_c^i and $s_c^{\cdot}$ are arbitrary constants. The other four parameters, C, q_{as}^i , C^{-}, and q_{as}, could be determined by plotting the field data for pseudo-instantaneous displacement and creep

rate for each creep stage in a linearized stress-strain plot format. The time required (t_{req}) to reach the measured allowable displacement (S_{all}) is then estimated using the following form:

$$t_{req} = \frac{S_{all} - F(q_{net})}{G(q_{net})} \qquad (2.16)$$

This hypothesis was supported by Johnston & Ladanyi (1974) observations, thus helical piles in frozen soils were concluded to show larger displacement before reaching their ultimate pull-out capacity.

2.6. Adfreeze Bond in Warming Permafrost

The main strength characteristic determining the bearing capacity of a pile in frozen soil is the adfreeze bond strength. This bond strength is the maximum interface resistance of frozen soil to shear along a solid surface (i.e., a foundation pile). The adfreeze bond is the mechanical adhesive connection generated between the frozen soils and the foundation. The bond forms as the soils initially freeze against the foundation, is analogous to adhesion strength between dissimilar materials. Colder soils generally provide higher values of adfreeze bond strengths, and as the ground temperature warms, the adfreeze bond weakens and its long-term rupture-strength decreases. Under the same loading condition, the weaker bond may promote foundation strain (i.e., slow moving creep) and eventually interface failure (Vialov, 1973a).

For structures designed on frozen soils, the ground temperature is an important feature to determine structural load carrying capacity of piles. The adfreeze bond strength is a key design-factor used to determine the pile design size and depth in order to support a given design load. Based on Khrustalev (2001) reports, the climate warming will degrade the permafrost and weaken

the adfreeze bond capacity in warming permafrost regions. Many foundation-designs have safety-factors of less than 1.56 in Russia. If warming temperature reduces the adfreeze bond strength by 64% to 95%, those foundations will be at high risk of failure (Khrustalev, 2001). As an example, the adfreeze bond strength developed at -2°C (28.4°F) soils are triple the adfreeze bond strength as soils at -0.5 °C (31.1 °F) (Vialov, 1973b). Even a small permafrost warming may result in significant strength reductions, especially in warm permafrost.

It is being expected that permafrost in continuous zones experience degradation in future years. In the warmer, discontinuous permafrost regions, even complete disappearance has been predicted. Complete loss of load carrying capacity of pile foundations in permafrost areas will be expected if specific mitigation measurements are not implemented. The current average annual soil temperatures in warm permafrost range from 0°C to -0.6°C. From 40 % to 100 % strength loss is predicted in the Zone I (Fairbanks & Kotzebue) thermal regions (Vialov et al., 1998).

The load capacity of foundation systems in frozen soils depends upon maintaining the adfreeze bond strength at or below its design cold temperature. Breaking the adfreeze bond would lead to much weaker residual interface strengths. Preserving existing infrastructure (especially existing buildings on pile foundations in permafrost) will need preserving both the cold soils temperature and the adfreeze bond. The adfreeze bond was essentially brittle when tested under an exposure temperature of -2°C (Ladanyi & Theriault, 1990). Increase in ground temperature could lead to displacement in pile compared to the surrounding soil. A small displacement of less than 1 cm have been found sufficient to break the adfreeze bond (Anderson & Anderson, 1978; UFC, 2004b; Nidowicz & Shur, 1998). Once broken, the adfreeze bond does not reform rapidly, and the remaining strength (i.e., residual bond strength) would be significantly weaker. The

residual bond strength may be several times less than the long-term adfreeze bond strength (Nidowicz & Shur, 1998).

The thawing of frozen ground and the adfreeze bond is likely to be spatially non-uniform, with variable mechanical characteristics. Ladanyi & Theriault (1990) noticed that the adfreeze bond may partially recover (heal) under high lateral soil pressures combined with higher unfrozen water content. However, with warmer permafrost, low normal forces and drier soils, adfreeze bond healing may not occur at all (Ladanyi & Theriault, 1990). Mechanical cooling with refrigeration tubes (aka. heat pipes or thermosiphons) have been used to maintain the adfreeze bond and neutralize the warming effects on foundation systems. Development of three-dimensional models helped in determining the required amount of refreezing for repairing and stabilizing damaged adfreeze bonds (Dubina et al., 2003; Vialov et al., 1998). Refreezing was found to be able to repair the adfreeze bond up to one-half or less of the original adfreeze bond strength (UFC, 2004b).

2.7. Current Foundation Design Methods in Permafrost Regions

Foundation engineering practice in permafrost currently includes preserving the cold soils and protecting the adfreeze bond between soils and structural members. Several preservation and mitigation measures have been implemented to minimize heat transfer from the superstructures to frozen ground and maintain the adfreeze bond and, hence, avoid foundation strains (settlements or heaving). For new structures, a two-fold foundation design approach may be used to prevent foundation failure and minimize its settlement. The design safety factors can be increased to consider anticipated warmer soils. Khrustalev (2001) has outlined a risk assessment process to select increased first-costs for a stronger initial design that includes climate-warmed soils as a basic premise. Several active cooling measures such as below-floor forced air ducts or passive

cooling via thermosiphons are commonly used to maintain frozen condition. Open convective airflow technique which allows cold air to flow under the building may be used as an active cooling method example (McFadden, 2001). Thermosiphons are closed pressure vessels (pipes) in which a two-phase fluid (liquid and gas) is placed. Due to evaporation and condensation process of fluid, thermal energy conducts from the ground to the atmosphere (Holubec, 2008; Popov, Vaaz & Usachev, 2010; Zarling & Haynes, 1985).

Several insulation methods have also been used for maintaining the soils in their naturally frozen state using passive methods. The insulation thickness can be increased or decreased in response to actual temperature changes at a specific location. A number of other methods considering both natural and artificial means are used for the fundamental premise of keeping the soils below freezing.

2.7.1. Open Crawl Space Method

Open crawlspaces have been used to freely allow cold winter airflow below the entire building. Utilities, especially water and wastewater, are normally enclosed in heated utilidors outside of the building crawl spaces. Flow of water leakage into the permafrost soils has been prevented by use of utilidors. Building crawl spaces are often extended as close to outside winter cold temperatures as possible (Vialov et al., 1993b). Figure 2.6 shows an open crawl space below the Yuut Elitnaurviat learning center in Bethel, Alaska. Most commercial open-height crawl spaces are 1.2 m to 1.8 m tall (Perreault, 2016).

Figure 2.6: Open crawl space placed under Yuut Elitnaurviat learning center in Yupik Eskimo (Perreault, 2016)

Another way for cooling foundations below existing buildings is the use of completely opening an existing crawl space that is partly closed. In this case, existing utilities below the buildings may need to be relocated to inside the thermal envelope or to heated utility ducts (called utiliducts). Figure 2.7 shows insulated utilities at Yuut Elitnaurviat complex.

Figure 2.7: Utilities protected from freezing by enclosure within insulated arctic pipe (Perreault, 2016)

By providing only vent openings in a fully enclosed foundation system (i.e., vent openings in the crawlspace sidewalls), preserving the frozen state of soils will not be achieved. Maximum possible airflow below the buildings should be designed by considering the full openness space below the buildings (i.e., no sidewalls at all). Perreault (2016) saw a 'venturi tube' effect incorporated into the building shape. The bottom edge of the wall above the crawl space could also be curved to create a broader transition zone around the base of the building. This curved-shape edge helps to increase wind-driven airflow beneath the building. An example of this curved bottom-edge-wall shape is shown in Figure 2.8 which causes improvement of airflow under the building.

Figure 2.8: Improving convective airflow below a building with a curved wall-bottom shape (Perreault, 2016)

2.7.2. Thermal Syphons

Thermal syphons (commonly known as "thermosiphons") utilize passive soil refrigeration, and are used as a heat exchanger between the soil and the atmosphere to preserve permafrost in continuous

and discontinuous permafrost zones. As shown in Figure 2.9, a thermal syphon is a long and closed pressure vessel charged with a high pressure condensed gas (ammonia, carbon dioxide, propane, etc.). Gas pressures in the order of 500 kPa are used for thermosyphons with carbon dioxide. The portion of the pile which acts as an evaporator is embedded in the soil, while the segment above the ground surface acts as condenser. The evaporator part absorbs the heat from the soil around the embedded portion resulting the vaporization of the liquefied gas. The vapor rises to an above-ground radiator-condenser portion. In winter cold air temperatures, the vapor radiates its heat into the atmosphere. Then, as the vapor is cooled, it re-condenses into liquid form. Completing the cycle, the fluid falls by gravity down into the evaporator portion of the thermosiphon. The convection of high-pressure gas occurs only when air temperatures are lower than soil temperatures. According to Long (1982), a very slight temperature difference as little as 0.006°C cause the convection cycle to be started. Heat will be conducted downwards through the vessel walls when air temperatures are higher than soil temperature. Soil around the thermal syphon will remain frozen to lower temperatures in winter due to seasonal operation of the thermal syphon and as a result the soil remaining frozen throughout the summer. For a given size of thermal syphon, the volume of soil freezing and maintain frozen soil is dependent on soil conditions, climate and vegetative cover. Insulation installation at ground surface can reduce heat transfer from the frozen soil to the atmosphere during warm weather, therefore, a reduction in permafrost active layer thickness in the vicinity of the thermal syphon could be taken place.

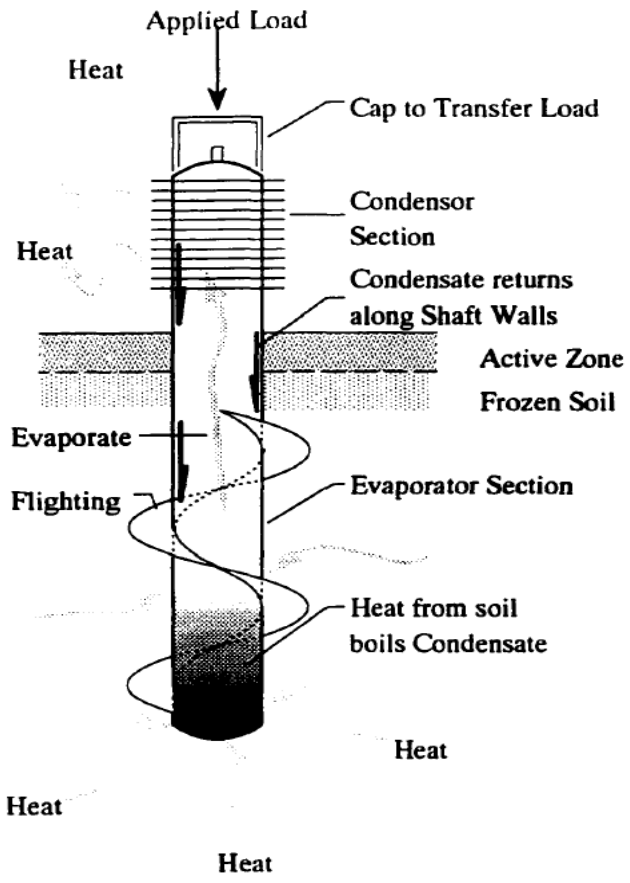


Figure 2.9: A schematic illustration of thermal syphons operation (Black, 2001)

In Alaska, soil cooling by thermosiphons has been reported since the 1960s. Thermosiphons may be installed vertically, horizontally, or inclined. Inclined thermosyphon installations occur at a minimum angle of approximately 3°C to 5°C relative to the horizontal which allows condensate functions and enclosed gas to work properly (Holubec, 2008). The "thermosiphon" term, is converted to "thermopile" when they are installed as a load bearing structural members (McFadden, 2001).

In order to preserve cold permafrost below heated buildings, seasonal cooling devices more commonly used from the end of the 1990s. The risk of ground subsidence will be reduced by the

use of mechanical tools in order to maintain the cold permafrost condition in warmer climates. One of the main technical solutions for building foundations is the use of a ventilated space under the building and the installation of seasonal cooling devices (Popov, et al., 2010).

Thermosiphons cool the soils enough in winter so net heating in summer is not generated. Cooling performance depends on variables such as winter air temperatures, maintaining pressure inside the tubes, and tube integrity to keep water out of the tubes. Spacing between cooling thermosiphons becomes a prominent feature. New models could become more conservative because of uncertainties associated with warmer climate projections (Holubec, 2008).

Three newer thermosiphon configurations have been installed on test site in Alaska. These configurations included undulating, flat loop, and hairpin thermosiphons. The Results showed that a completely buried system, would not be noticeable to traffic. Increasing condenser size recommended by Wagner (2014) and Wagner et al (2010). Evaporator sections were installed deeper in the ground for horizontal thermosiphon applications. The deeper evaporator section was separated from the closer to the surface condenser section by use of insulation. The insulation used to preserve the permafrost below from the thawing heat-source (building or outside warm air) above. Evaporator tubes of flat loop thermosiphons were snaked back and forth below the surface and were connected to a single radiator-condenser section in the next step (Holubec, 2008). Around the perimeter of smaller buildings, thermosiphons were installed vertically. Holubec (2008) showed that a “thermo-curtain” would be possible to be created by thermosiphons. He questioned the possibility of sufficient protection of interior regions related to building areas by employing only thermo-curtains in warming permafrost regions.

2.7.3. Thermal Piles

Foundation engineering in permafrost areas must contend with thermal degradation especially if the soil contains extensive ice lenses or ice masses. Frost heave resulting from re-freezing of the active zone may exert significant upward forces on a foundation system, forcing measures to prevent the heave. A foundation must meet the requirements for settlement, carrying capacity, frost uplift and permafrost preservation. One example of such foundation is a thermal pile, which is a thermal syphon designed to carry a structural load. Erwin Long was the first who developed Thermal piles over the period of 1956 to 1960. Many improvements have been made to its reliability and capacity over time.

Installation of thermal pile involves placing the pile into an oversized, pre-drilled hole and backfilling the space by sand slurry. Holubec (1990) suggested that boreholes should be large enough to provide a 50 mm annulus beyond the flighting tip to accommodate the placement of sand sludge. Slurry is placed in lifts and vibrated in order to minimize the potential void formation and to ensure higher densities. Steam thawing technique may take the pre-boring method's place in permafrost projects. Generally, attempts for thermal piles directly-driving installation method has been impractical and unsuccessful. The thermal pile and low ambient temperatures will quickly freeze the slurry and refreeze any surrounding soil that has been thawed when construction process was finished during winter. Then, the structure construction process can be begun following the foundation implementation. In case of summer construction, building construction often is delayed until winter time in order for refreezing process of soil to be completed taking advantage of low ambient temperature. If it was not possible to postpone the construction process until winter, loads on the thermal piles must remain below the capacity of the pile in unfrozen soil.

The pressure-vessel implemented in thermal piles contains a fluid that evaporates at temperatures below freezing. In the below-ground evaporator pipe, the fluid extracts heat from the ground, causing the fluid converted to vapor-form. Due to convection the vapor then rises the pressure vessel. Once this vapor reaches the portion above the ground, it condenses, and the heat of condensation is dissipated to the colder atmosphere. The heat removal cycle continues as condensate flows down and returns to the lower portion.

Figure 2.10 shows conventional thermosiphons (with radiator sections above the ground) installed next to the piles. Thermopiles and thermosiphons are commonly installed during the construction process before the building is constructed on pile foundation bases. Thermosiphons installation underneath existing buildings is more complicated process because it must be implemented from outside of the structure footprint. There are three frequently used alternative techniques to address this issue including use of thermal-curtains, inclined thermosiphons, or flat tubes (Perreault, 2016). Flat tube installation is more applicable to new construction than to retrofitting an existing structure. Design validation is more difficult due to protected property of design methods. Additionally, some calibration against recent ground temperature performance is needed for thermosiphon applications. One of the main concerns of performance of thermosiphons satisfactory is the lack of standards and clear temperature data to design thermosiphons properly. Refrigeration demand determination comes from climate modeling. If climate warming does not occur to the extent projected, it may result in a conservative refrigeration system and higher cost (Holubec, 2008).

Figure 2.10: Thermopile installation in Alaska. The thermopile radiator sections (tilted out from the vertical piles) are clearly visible (Perreault, 2016)

2.7.4. Cold Air Refrigerant System

Ground temperature around cast-in-place piles may increase due to the heat created as a result of hydration on cast-in-place pile foundations which also in turn may lower the bearing capacity of pile foundation. Previous findings show that it took two years to refreeze the cast-in-place pile foundation in Chalaping Bridge in China without any cooling system (Shang et al., 2017). A Cold Air Refrigerant System (CARS) can lower the freeze-back time for the pile and surrounding soil. The CARS removes heat from the ground by circulating cold air in the pile foundation in winter to reduce the refreezing time. This technique was reported to decrease the freeze-back time from 2 years to one year (Shang et al., 2017) (Figure 2.11).

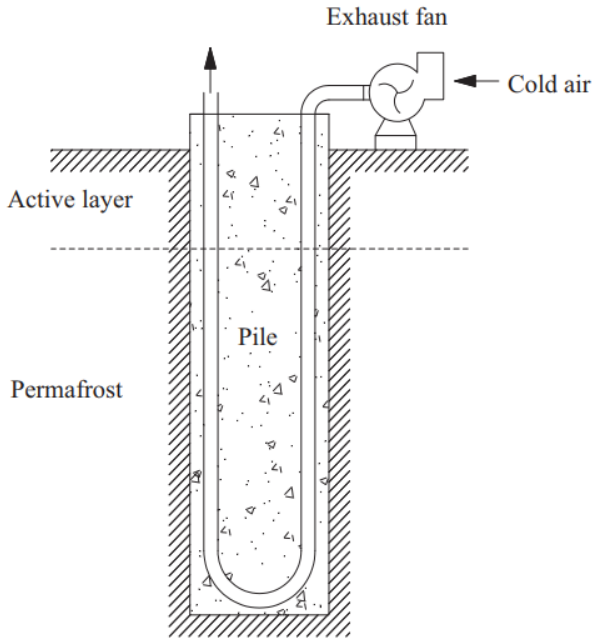


Figure 2.11: Cold Air Refrigerant System (CARS) (Shang et al., 2017)

The ground thermal regime is also disturbed by hydration heat of concrete around cast-in-place pile foundations and this temperature increment can cause thawing around the pile. Increase in temperature will have a reduction influence on the pile adfreeze shaft resistance and the point bearing capacity (Ma and Wang, 2014). Several investigations have been conducted on the hydration induced heat around cast-in-place piles and their freeze-back time in China (e.g., Wang et al., 2013; Wu et al., 2006). Cooling process and refreezing time of cast-in-place pile foundations in the thawing warm permafrost areas have also been analyzed (Yuan et al., 2005; Zhang et al., 2010). Li et al. (2005) and Wu et al. (2004) performed numerical analyses to estimate the freezing time in concrete. Tang et al. (2010) and Jia et al. (2004) developed single pile models to study thermal disturbance radius of hydration heat and the process of freeze-back of the cast-in-place pile foundation.

Johnston (1981) proposed the cold air refrigerant system (CARS) as one of the thermal pile systems which artificially cools the temperature around the metal piles and as a result decreases temperature in pile-frozen soil interface. The operation of CARS is in winter when air is colder than the permafrost, and the cold air is forced to be circulated in pile foundation to remove the ground heat. The cooling effect of CARS was evaluated on cast-in-place pile foundation through a three-dimensional FEM model during construction in Chalaping Bridge in China (Shang et al., 2017).

2.8. Summary

Pile foundations have been widely used as foundation system in permafrost region for many years. These foundations have shown high bearing capacity and reliable performance for many infrastructure projects in frozen soils. High exposure temperature, however, can decrease the load carrying capacity of pile foundations and increase their displacement. To prevent transferring heat from buildings and structures into frozen soils, these structures are often built over frozen ground surface with a considerable gap (i.e., Crawl Space). Several techniques have also been implemented in recent years to maintain the freezing temperature and mitigate permafrost thawing including insulations and cooling systems, thermosiphon, coatings, air convection method, and sun sheds. Although most of these methods have shown to be successful in preventing the heat transfer from building to frozen ground, higher ambient temperature related to global warming could pose a new challenge for maintaining frozen ground temperature. Therefore, a new refrigeration technique is needed to refreeze the thawed soils in permafrost regions in order to improve the soil bearing capacity and as a result enhance the pile foundations' load carrying capacity. This research aims at developing a novel refrigerant system to locally reduce ground temperature around existing

and new pile foundations and, in turn, improve load transfer and creep of steel pile foundations in frozen grounds.

CHAPTER 3: Effect of Freezing Liquid on Pile-Soil Adfreeze Strength

3.1. Introduction

Pile foundations have been widely used as foundation system in permafrost region for many years. These foundations have shown high bearing capacity and reliable performance for many infrastructure projects in frozen soils because of high bearing capacity of ice. However, a significant increase in permafrost temperature has been observed in the past decade which leads to permafrost degradation and reduction in shear strength of frozen soil (e.g., Jorgenson et al., 2006; Frey and McClelland, 2009). Pile foundations placed in frozen soil transfer their loads to the surrounding ground principally through adfreeze bonding at the pile-soil interface. The load capacity of foundation systems in frozen soils depends upon maintaining this adfreeze bond strength at or below its design temperature. Breaking the adfreeze bond between the pile and its surrounding soil would lead to much weaker residual interface strengths. Colder soils generally provide higher values of adfreeze bond strengths, and as the ground temperature increases, the adfreeze bond weakens and its long-term rupture-strength decreases (Vialov, 1973a).

Several studies have shown that change in exposure temperature can significantly impact the load transfer and displacement of pile foundations. Jellinek (1958, 1959,1962) demonstrated that the pile-frozen soil adfreeze strength is a function of temperature. For the exposure temperatures of 0°C to -13°C, shear failures were found to occur at the pile-soil interface between the steel pile and ice, while below this temperature, the failure occurred within the ice. The pile-soil adfreeze bond was also shown to significantly decrease as the ambient temperature in frozen

soil increases. Vialov (1973) reported that the adfreeze strength at a temperature of -2°C (28.4°F) could be about triple the adfreeze bond strength of the same soil at -0.5 °C (31.1 °F). A small temperature increase of 1°C was also reported to increase the creep settlement by 35% for a pile exposed to constant axial load installed in an ice-rich silt (Ladanyi, 1995).

Foundation engineering practice in permafrost regions currently includes preserving the frozen condition of the soils and protecting the adfreeze bond between soils and structural members. To prevent transferring heat from buildings and structures into frozen soils, the superstructures are often built over frozen ground surface with a considerable gap (i.e., Crawl Space). Several preservation and mitigation measures have also been implemented to minimize heat transfer from the superstructures to frozen ground and maintain the adfreeze bond and, hence, avoid foundation strains (settlements or heaving). Current mitigation techniques mostly include insulations and cooling systems, thermosiphons, coatings, air convection method, and sun sheds.

Although most of these methods have shown to be successful in preventing the heat transfer from buildings to frozen ground, higher ambient temperature related to global warming could pose a new challenge for maintaining frozen ground temperature. Therefore, new retrofitting techniques would be needed to preserve the frozen conditions of foundation soils in permafrost regions. A new refrigeration technique is explored here to refreeze the thawed soils in permafrost regions in order to improve the soil bearing capacity and as a result enhance the pile foundations' load carrying capacity.

The proposed technology involves using an antifreeze liquid with below 0°C temperature to be circulated into steel pipe pile foundations. The hypothesis in this proposed technology was reduce the pile-soil interface temperature using a cold liquid with the hope that the pile and

surrounding soil temperature would be lowered when low temperature liquid was circulated within the pile. This proposed technology requires a cooling mechanism, such as a freezer, and it also requires a piping system so that the liquid inside the freezer can be transferred through the pile along the pile shaft. Two ports were used as inlet and outlet in the pile body, one directs the liquid inside the pile and the other transfers the liquid back to the freezer. In current cooling methods such as thermosiphons, passive soil refrigeration is used as heat exchanger between soil and atmosphere to protect soil's frozen condition in permafrost. However, this proposed technology uses electricity as energy source to reduce the liquid temperature in a freezer, and transfer the cold liquid into the pile to lower the pile-soil interface temperature. The proposed technology can be used as a retrofitting technique for existing pile foundations in warm permafrost or as new design method for new piles in permafrost area if the pile-soil interface reduction was possible by the use of this technology.

In this study, a low temperature freezing liquid (Antifreeze) is circulated into steel pile tubes to examine the possibility of reducing temperature of the pile and pile-soil interface, and ideally induce freezing of the soil around pile, to enhance the bearing capacity of pile foundations in endangered warming permafrost regions. This chapter presents details of this novel technology along with experimental setup used for development and testing of pile-soil systems at different exposure temperatures.

3.2. Model Soil Properties

A sandy soil with an optimum moisture content of about 14% was used for experimental testing of model pile-soil systems in this study. This soil was selected considering the simplicity of sample

preparation for model pile test as well as availability of material characteristics for numerical simulation. A sieve analysis test was performed to obtain grain size distribution of the soil according to ASTM D 422 (2003). D60, D30 and D10 were found to be 0.263, 0.185 and 0.130 respectively. Coefficient of uniformity (C_u) and Coefficient of curvature (C_c) were calculated to be 1.4 and 1.0 respectively. Based on results of sieve analysis test, this sand can be classified as poorly graded sand (SP) as per the Unified soil classification system (Figure 3.1). The maximum dry density of the soil was measured at 1900 kg/m3 with an optimum moisture content of 14% using a standard proctor test.

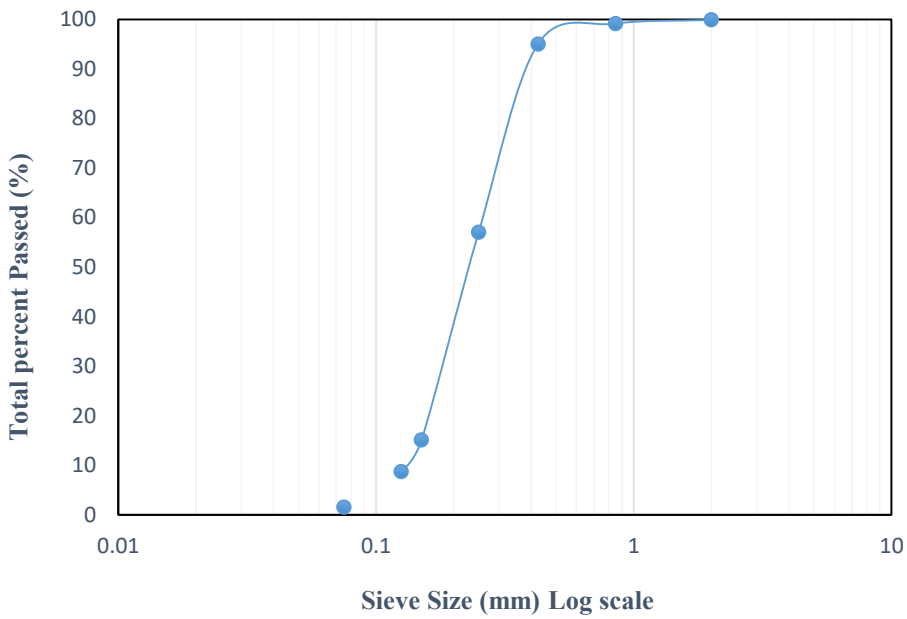


Figure 3.1: Particle size distribution curve of the soil

3.3. Model Piles

Steel is a common pile material used in cold region for manufacturing pile foundations in various geometries including pipe piles, H-sections, and helical piers. For relatively warm permafrost, open-ended steel-pipe and H-steel section piles can be driven deep enough in permafrost in order

to develop sufficient adfreeze bond and provide high bearing capacity. In cold permafrost, closed-end pipe piles may be installed in oversized predrilled holes and backfilled with sand-water slurry or grouted with concrete. In this experiment, steel pipes with 1375 mm in height and a diameter of 47 mm were employed for model pile load testing (inner diameter of 42 mm). The total and the average surface roughness values for this particular type of steel were measured by a FARO arm measuring device at 9.7 μm and 11.3 μm, respectively (Giraldo and Rayhani 2013).

The model pile was customized to allow circulation of the antifreeze within the pipe section to examine the possibility of decreasing temperature at the pile-soil interface level. Two holes were made in the body of the pile as inlet and outlet of antifreeze. By use of fittings and connecting a long piece of hose inside the pile to inlet fitting, the antifreeze was pumped into the pile from the bottom until it fills the whole pile to the top cap. Both the bottom and top of the pile were welded and all holes including fittings holes and strain gauge connection holes were sealed perfectly to avoid any possible leakage. The outlet was placed close to the top of the pile to ensure proper circulation of the freezing liquid along the pile length.

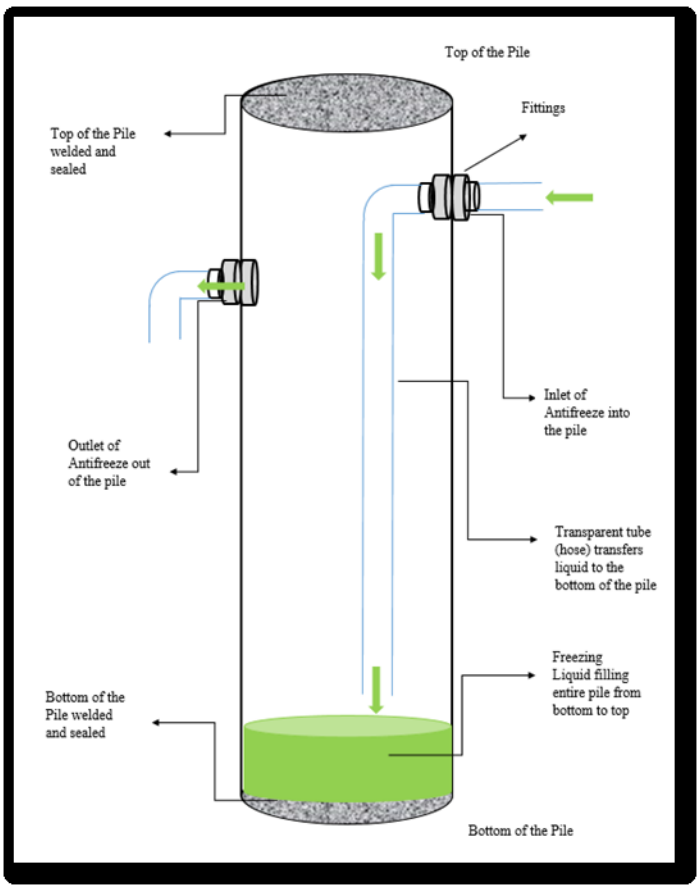
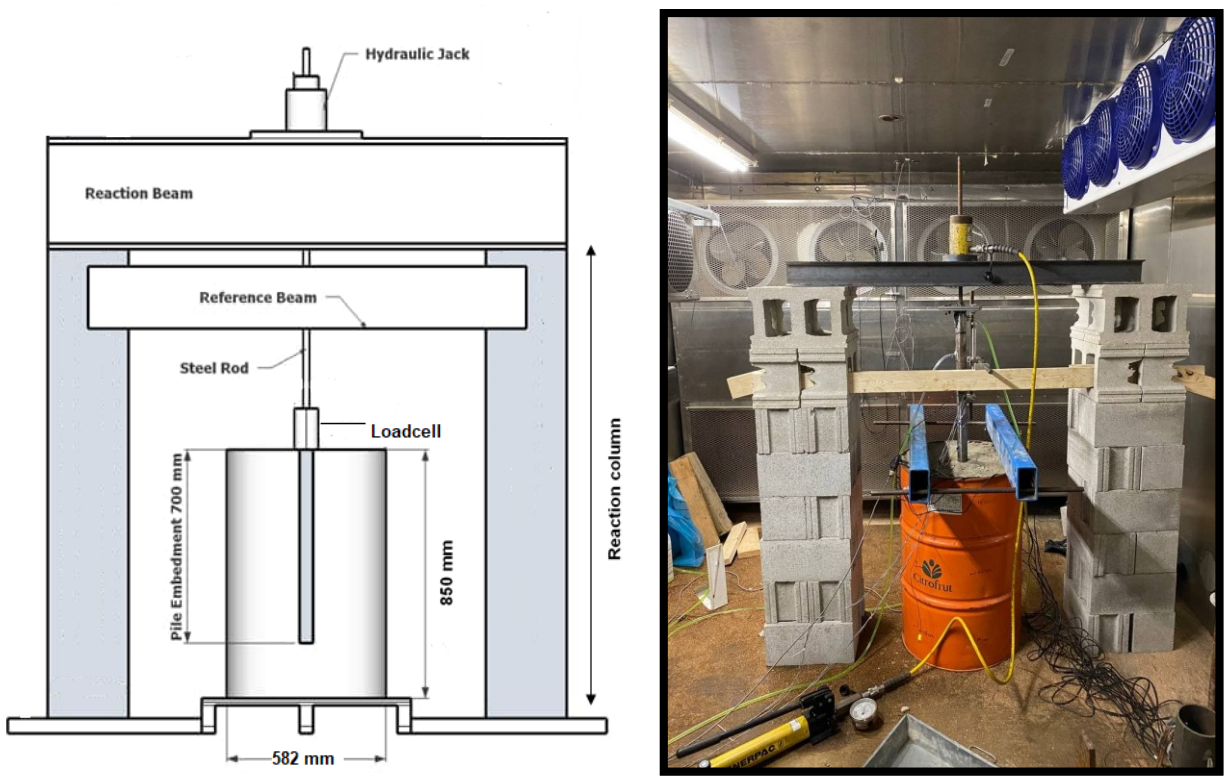


Figure 3.2: Schematic view of steel pipe with inlet and outlet for circulation of the freezing liquid

3.4. Experimental Setup

All model pile experimental tests were conducted in the environmental chamber of Carleton University. This environmental chamber has a dimension of 3m × 4m with height of 2.4m, and is capable of applying and controlling temperature within the range of -20˚C to 5˚C at an accuracy level of +/- 1˚C. A pile load testing frame was assembled in the environmental chamber to apply monotonic loading on model piles. A schematic view of the test assembly is shown in Figure 3.3. A reaction platform was assembled in the environmental chamber to facilitate the pull-out load testing. The reaction frame consisted of two concrete block columns placed on both sides of the test pile and barrel system. The reaction columns were connected to each other through two C

Channel-shaped steel reaction beams. A steel plate was placed across the channel section to serve as a reaction platform allowing for the application of axial pull out loads. Axial pull-out loads were applied using a hydraulic jack coupled to a steel rod connected to the pile head which runs vertically aligned with the pile axis. Force measurement was carried out using a load cell placed at the connection of steel rod-pile head cap to ensure proper alignment. The load cell was connected to a data logging station to measure axial resistance, while axial displacement at the pile head and the soil surface was recorded by two LVDT devices connected to a reference beam. The reference beam was a 2-inch x 4-inch timber beam that was connected to the reaction columns to hold the LVDTs used for monitor the displacement of pile during pull-out and soil settlement.



a b

Figure 3.3: a) Schematic view of experimental setup, b) A view of the model pile and the soil container (barrel) used in this experiment

A barrel container with 850 mm in height and a diameter of 582 mm was used to contain the soil model. The model closed-end pipe pile was embedded within the poorly graded sand in the barrel at a depth of 700 mm (Figure 3.3). This model in experimental setup was scaled based on geometry conditions, and the stress condition was not considered in model scaling. For model soil preparation, the sand was mixed with about 14% moisture content and the mixture was kept within a sealed container for about 24 hours to ensure uniform moisture distribution. The soil was then placed within the model test container in layers with a thickness of about 100 mm and compacted to achieve a dry density of about 1800 kg/m3 (95% of maximum dry density). The sample uniformity was controlled by monitoring the thickness of each soil layer. After compaction of each layer of the soil in the barrel, 5TE moisture sensors and thermocouples were placed simultaneously at each soil level to monitor moisture content and the temperature of the model soil during testing (Fig. 3.4).

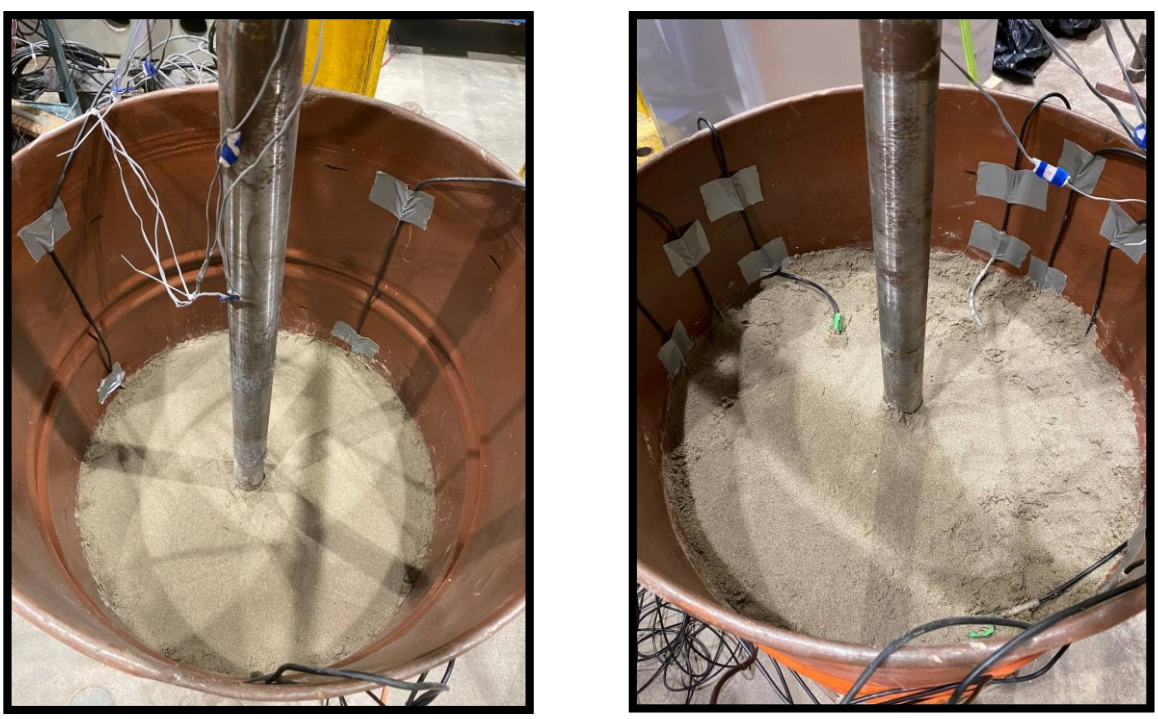


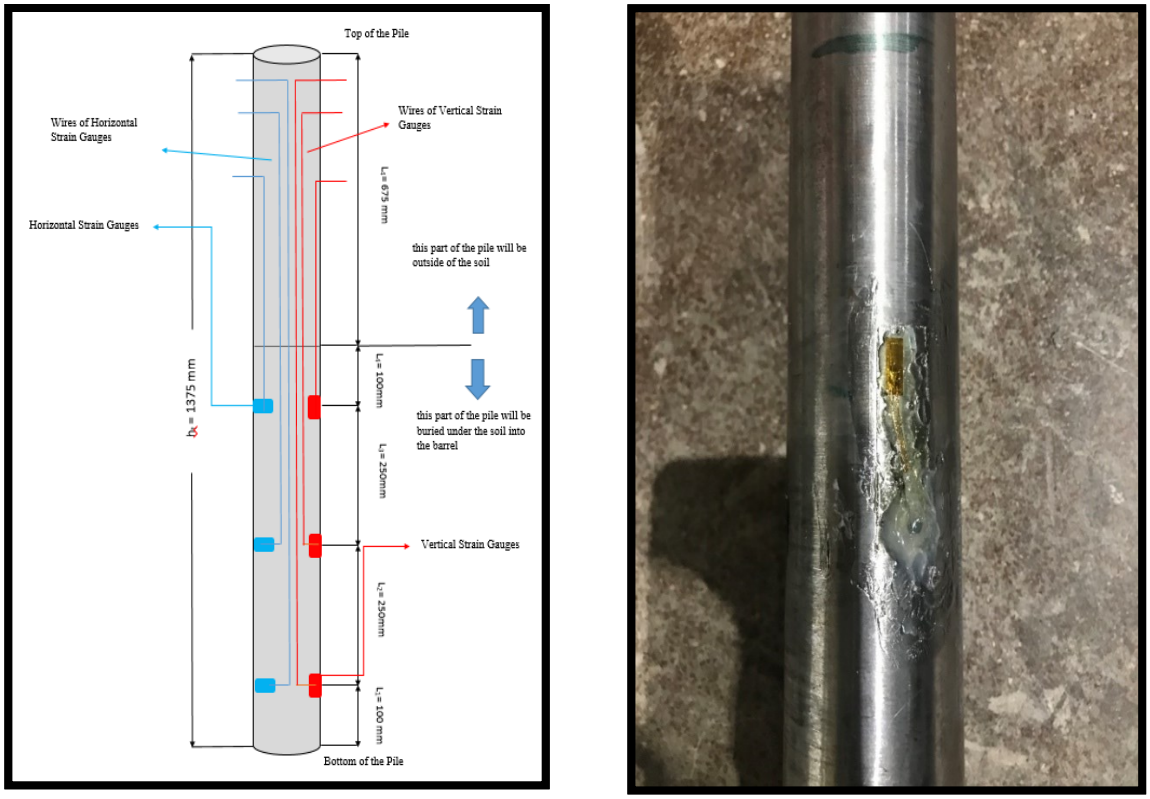
Figure 3.4: Model soil preparation and instrumentation

3.5. Instrumentation

A series of strain gauges, Linear Variable Differential Transducers (LVDTs), 5TE temperature sensors, and thermocouples were used to monitor response of the soil and the model pile during freezing and pile load testing. In the next following sections, the role of each instrument along with calibration and installation procedure are explained.

3.5.1. Strain gauges

The model pile was instrumented with several strain gauges to detect pile's strain during the load transfer test. Figure 3.5 demonstrates the location of 3 vertical strain gauges and 3 horizontal strain gauges that were installed on the model pile at three different levels to measure lateral and axial strains experienced by the pile during the test. This gauge configuration was established to provide adequate resolution of the expected pile strain profile along the pile length. Strain gauges (Tokyo Sokki Kenkyujo Model) were mounted to the exterior of the model pile using M-Bond 200 glue while the wires were passed to the interior of the model pile through a small hole. After gauge attachment, the holes were sealed by multiple surface treatment of M-Coat polyurethane and a coat of silicon protection to waterproof the gauges. Upon mounting the strain gauges, the input wires were passed through inside of the pile to the top section of the pile and connected to spring terminal of National Instrument cDAQ module (NI-MAX). The data acquisition system was connected to DASYLAB platform to enable reading gauge data. The strain gauges were pre-calibrated through a task created in NI MAX system. Strain gauge readings were carried out during circulation of antifreeze into the pile as well as pile load testing.



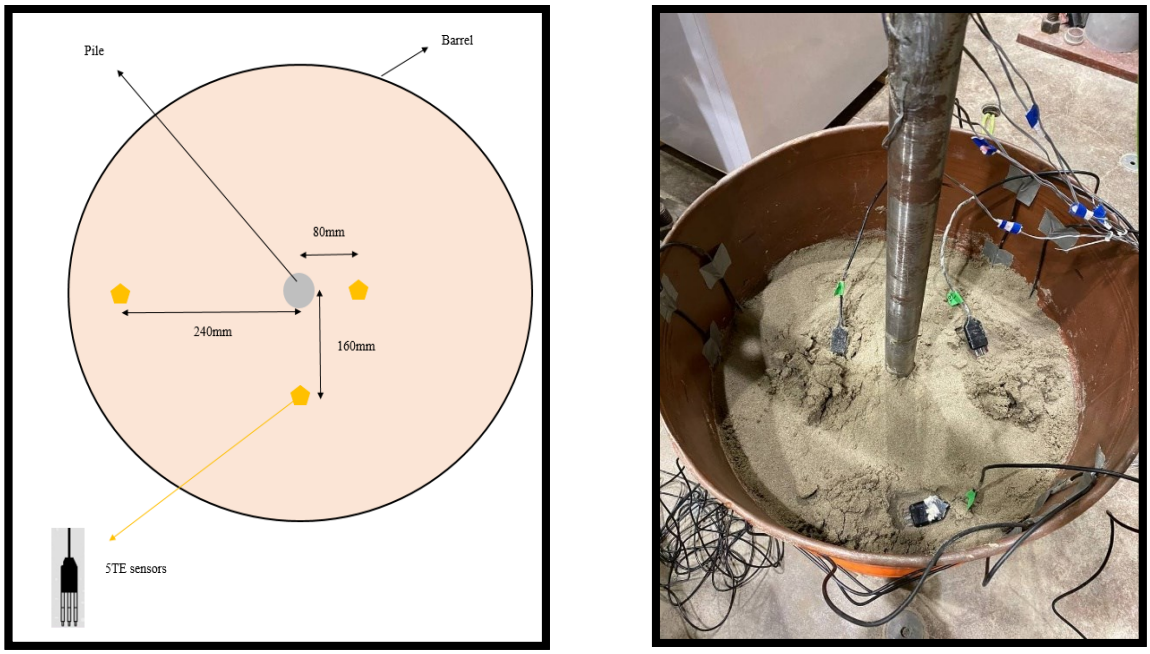
(a) (b)

Figure 3.5: a) Strain gauges' location on model pile, b) strain gauge attachment and sealing

3.5.2. Automatic temperature measurement sensors "5TE"

Twelve sets of 5TE probes were employed to measure soil's temperature at different depths and distances from the model pile during testing. The 5TE probe uses an electromagnetic field to measure the dielectric permittivity of the surrounding medium. The sensor can provide measurements of electrical conductivity (EC), volumetric water content (VWC), and temperature. As per the manufacturer's specification, the sensor can provide reliable measurements at the temperature range of -40 to 50.0°C. In this experiment, only temperature measurement feature of these sensors was used. The sensors were installed within the soil around the pile at four different

elevation levels and three different distances from the pile to monitor temperature variation in both vertical and horizontal directions. As shown in Figure 3.6, 5TE sensors were placed at four different elevations with a 200 mm vertical space between each set of sensors. In lateral direction, the sensors were positioned at 80 mm, 160 mm, and 240 mm distances from the pile (Fig. 3.6b). The sensors were pushed into the soil medium after compaction the soil at each layer.



(a) (b)

Figure 3.6: a) 5TE sensor layout at each depth, b) Placement of 5TE sensors within the soil

3.5.3. Thermocouples

A series of thermocouples (Model TT-T-30-SLE-500) were also utilized to measure the temperature at the pile-soil interface level and in the soil. These thermocouples are capable of measuring temperature below the freezing point. Four sets of these thermocouples were positioned close to pile surface at different depths of 100mm, 300mm, 500mm, and 700mm. The remaining

eight sets of them were placed within the soil at the space between the 5TE sensors and the pile to capture temperature profile closer to pile's surface and monitor variation of soil's temperature in both vertical and lateral directions. In terms of the lateral distance, the thermocouples were spaced at 10 mm, 20 mm, and 30 mm distance from pile. Similar to 5TE probes, thermocouples were also inserted into the soil in each layer and connected to National Instrument data acquisition system through a specific module designed for temperature measurement. Using a task created in NI MAX software, thermocouples' data were then monitored in DASYLAB platform and recorded in an excel file.

3.5.4. Linear variable differential transducers (LVDTs)

The displacements of pile and soil surface were monitored using Linear Variable Differential Transducers (LVDTs) with an accuracy of 0.01 mm. The test data was recorded in a frequency of one data point per second using a data acquisition system and plotted instantaneously on a DASYLAB platform. These sensors were calibrated before testing through the same data acquisition system (i.e., National Instrument, NI) and DASYLAB platform. As shown in Figure 3.7a, two LVDTs were used in this experiment set up. One was installed on the joint plate of pile to record the displacement of the pile during pile load testing. The other one was placed on the soil surface near the pile to measure the settlement and possible frost heave of the soil when subjected to freezing temperature by antifreeze circulation into the pile. A view of the whole experimental setup including the liquid circulation system, experiment instruments, and pile load test assembly is shown in Figure 3.7b.

(a) (b)

Figure 3.7: a) LVDTs installation locations in experiment setup, b) a view of the whole experimental setup (green tubes used for liquid circulation).

3.6. Experimental Procedure

The objective of this research was to evaluate the possibility of decreasing temperature at the pile-soil interface level. To achieve this objective, an industrial antifreeze liquid was circulated within the pipe pile to reduce the pile's temperature and induce freezing at the pile-soil interface level. A full container of the antifreeze liquid was placed within a freezer with a set temperature of -20°C to reduce the temperature of the liquid well below the freezing point. Figure 3.8 shows the freezer and the antifreeze used for this experiment.

The pile and soil system were exposed to 0°C temperature in the environmental chamber to simulate the ground temperaure in warming permafrost regions. Using a 1/12 HP pump, the antifreeze liquid was pumped through a 7/16-inch transparent tube from antifreeze container inside the freezer to the pile's inlet system. A layer of insulation was wrapped around the transparent tube to minimize the exposure of the liquid to ambient air temperature and maintain the desired applied temperature. The antifreeze liquid was continuously pumped into the pile through the inlet which transfers the liquid to the bottom of the pile. After circulation into the pile shaft, the antifreeze liquid was circulated back to the freezer through the outlet tube. This process was continued over the period of the test. All the holes and fittings along the pile shaft (e.g., strain gauges, inlet, outlet, bottom, and top cap of the pile) were sealed by welding and silicon coating.

Figure 3.8: The freezer and Antifreeze as source of cold temperature

3.7. Pile Load Testing

Axial pull-out load tests were carried out on the model pile before and after freezing application following a modified method based on ASTM D1143. A pile load test was initially conducted on the model pile when the pile-soil model exposed to a surface temperature of 0°C to measure the load carrying capacity of the pile in unfrozen condition. To perform the test, the pile head plate mechanism was used to connect a steel threaded rod to the hydraulic jack and reaction frame allowing for the application of pull-out axial loads. The loading procedure was designed to bring the pile to failure by applying incremental loading at the lowest rate manually possible intervals allowing for interface forces to reach equilibrium. Based on De Nicola and Randolph (1999) recommendation, a pile head displacement equivalent to 10% of pile diameter is often considered as failure criteria to determine the ultimate pile capacity.

To secure and fix the barrel against movement causing by the pull-out test, two steel box beams were placed over the barrel edges (Figure 3.9). Using two long rods and extra loads the beams were connected to each other to keep the barrel fixed during pile pull-out test. The reaction system has a theoretical capacity of approximately 3.0 times the capacity of the test piles, and it was utilized to serve pull-out load testing. The pull-out load was applied using a 103 kPa (15 psi) manual hydraulic pump and a 100 kN hollow hydraulic jack. For the pull-out test, the hydraulic jack was mounted on top of the reaction beam and the pull-out force was transmitted from the jack to the pile head through a steel square plate. The test was performed in accordance with ASTM D1143/D1134M (2007) following the Quick Test Method for single piles.

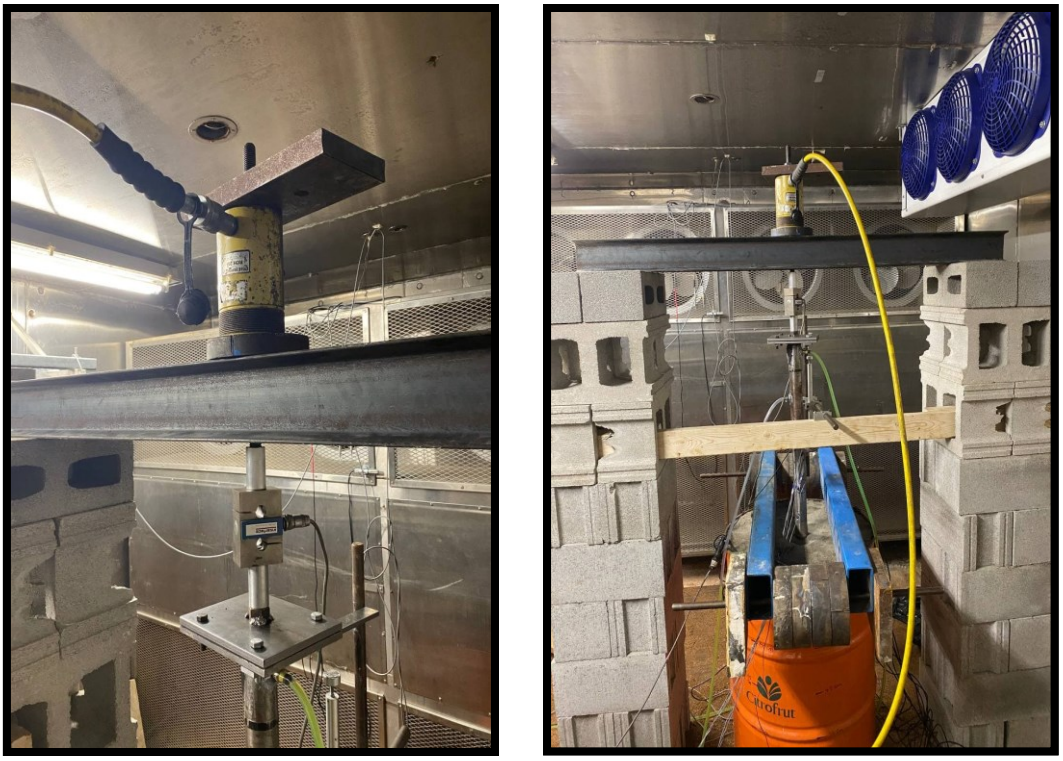


Figure 3.9: Pile pull-out testing assembly and setup

After measuring the initial load transfer of the pile, the antifreeze liquid exposed to a temperature of -20°C was circulated into the model pile over a period of 25minutes (Fig. 8). When the temperature equilibrium was reached at the pile soil interface level, a similar pile load testing was carried out to measure the load carrying capacity of the model pile under frozen condition.

3.8. Results and Discussion

3.8.1. Pile pull-out bearing capacity without use of cooling system

The pull-out bearing capacity of this hollow 2-inch steel pipe pile was initially measured in unfrozen condition when the temperature of the soil was around 0˚C. Figure 3.10 shows the load transfer of the model pile in unfrozen sand. The maximum pull-out capacity of the pile was found

to be 326 N at a pile head displacement of about 3 mm. The pile capacity was observed to remain approximately constant over a pile head displacement of 3-6 mm indicating the pile-soil interface failure. Considering the weight of the pile and the pile cap square plate (about 68 N), the actual shaft frictional resistance of the model pile would be 258 N in unfrozen condition. This frictional resistance was observed to engage at low displacements ranging from 3 mm to 3.5 mm corresponding to 6-7% the pile diameter. This value is slightly less than the 10% diameter criterion proposed by De Nicola and Randolph (1999).

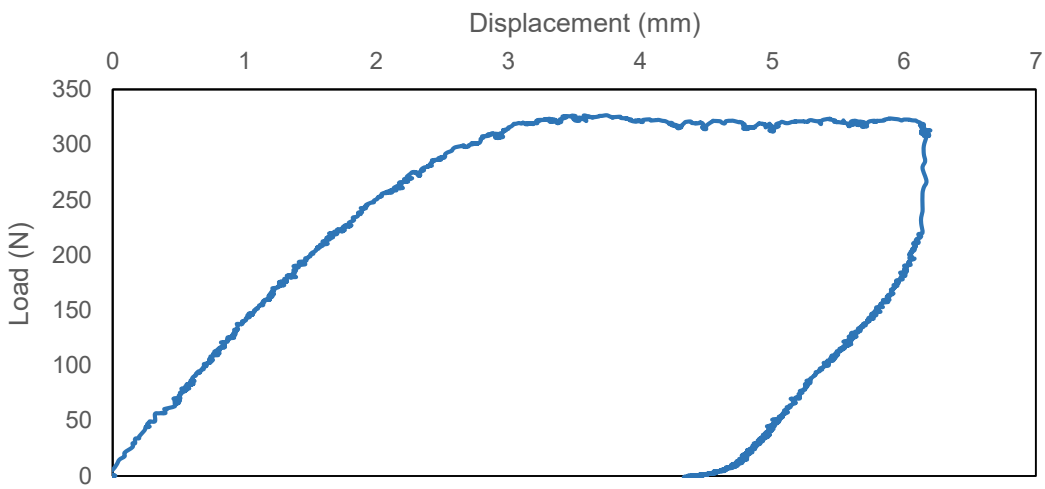


Figure 3.10: Pull-out bearing capacity of the model pile in unfrozen condition

Bearing capacity of piles in cohesion-less soils is developed primarily in the form of soil-pile frictional resistance, also referred as pile shaft resistance, in drained conditions (a.k.a β method). In this method (e.g., CFEM, 2007) pile shaft capacity is estimated as a function of pile geometry (length and diameter) and soil-pile interface frictional resistance. The shaft resistance (Q_s) is typically estimated from the following equation:

$$Q_s = CLq_s \qquad (3.1)$$

where C is pile circumference, L is pile embedment and q_s is the average shear strength, also known as unit shaft resistance, along the soil-pile contact area. The average shear strength q_s can be expressed in terms of the pile-soil interface friction angle (δ), effective stress of the soil (σ'_v), and coeficoent of lateral earth pressure (K_s). An empirical factor, β, is also used in pile design practice to account for pile-soil interface friction and coefficient of lateral earth pressure. q_s is estimated by the following equation:

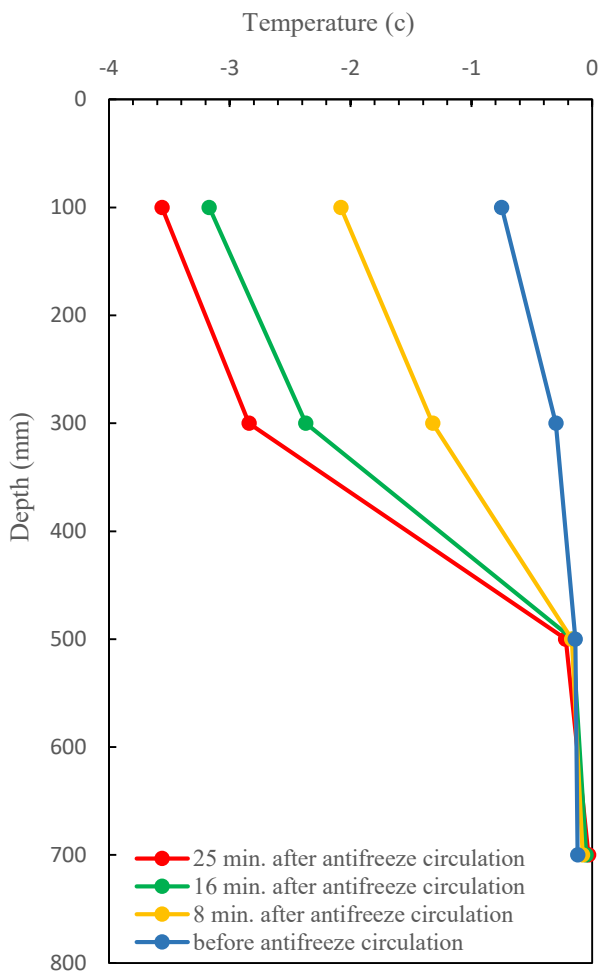
$$q_s = K_S \sigma'_v tan\delta = \beta\sigma'_v \qquad (3.2)$$

The average unit shaft resistance, q_s, for this model pile was estimated at about 2.35 kPa from the measured pile capacities by dividing the load transferred to the soil between the pile head and toe by the surface area of the pile. The corresponding β-coefficient for the tested pile was also estimated to be about 0.33 considering the average effective stress around the pile shaft.

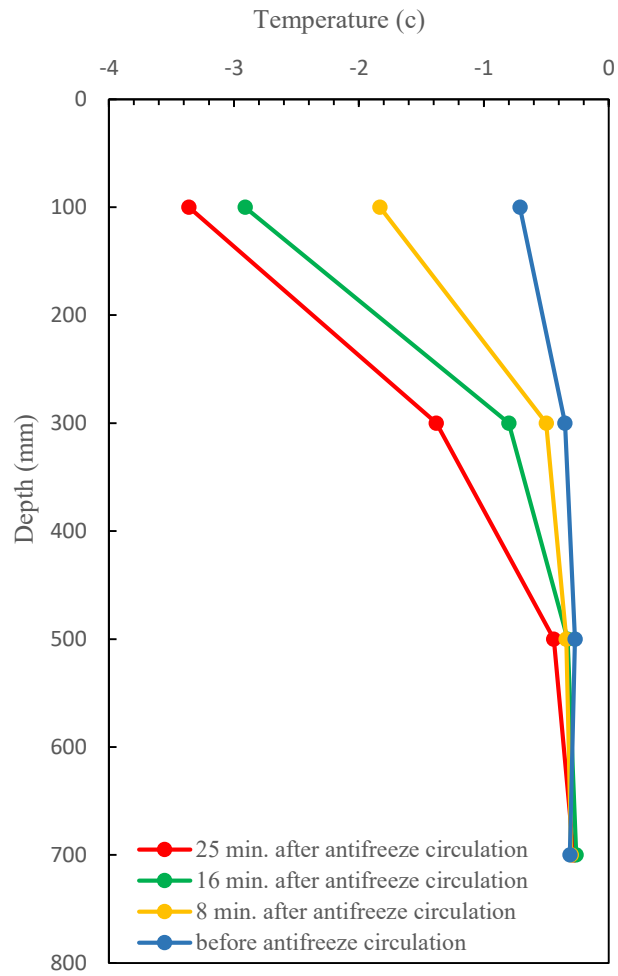
3.8.2. Temperature profile of soil subjected to antifreeze circulation into the pile

To examine the influence of local freezing on load transfer of piles in warming permafrost, a cold temperature liquid was circulated into the pipe pile to induce freezing at the pile-soil interface and increase the load capacity of the pile. The antifreeze liquid that was exposed to a temperature of -20°C in a freezer was continuously pumped into the pile while the temperature of the surrounding soil and the ambient temperature was in the range of +1˚C to -0.5˚C. In the first 25 minutes of antifreeze circulation, the temperature of the soil around the pile dropped from about 0˚C to -3.5˚C. Temperature profiles were measured at 10 mm, 20 mm, and 30 mm distance from the pile shaft in four different soil depths of 100 mm, 300 mm, 500 mm, and 700 mm from the soil surface. Figure 3.11 demonstrates the temperature profiles at different distances from the pile. These profiles show

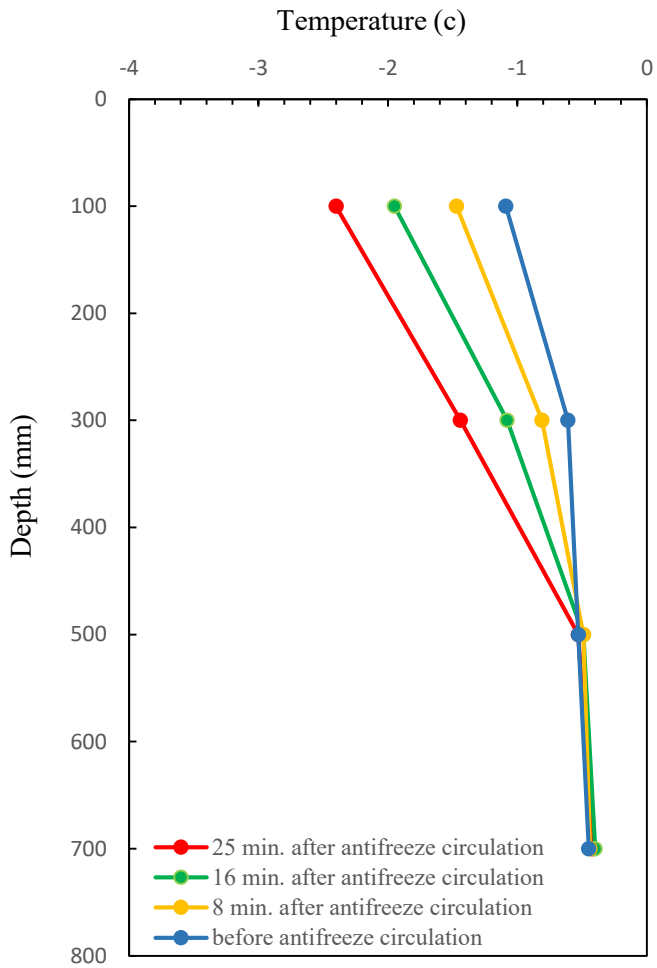
that the novel refrigeration technique developed in this research is capable of reducing the soil temperature around the pile and as a result could enhance the adfreeze bond between the pile and the surrounding soil.



a) 10 mm distance from pile



b) 20 mm distance from pile



c) 30 mm distance from the pile

Figure 3.11: Soil temperature profile at different distances from the pile during antifreeze circulation

As it is shown in Figure 3.11, by use of the freezing liquid inside the steel pile foundations, it is possible to lower the soil temperature below the freezing point and preserve the frozen condition of the soil in waring permafrost regions. The temperature reduction of the soil was started from the soil surface and then continued to affect the deeper soils accordingly. Based on the temperature profiles, the soil temperature dropped significantly at a depth of 100 mm from the surface in a very short time. The temperature at this depth reduced more than 3˚C after only 25

minutes of circulating the freezing liquid through the pile. Within the same time frame, soil temperature at a depth 300 mm also reduced to about -2.8 ˚C which is much lower than the initial soil temperature of about 0˚C. At deeper portion of soil, however, the temperature reduction was not significant. This is possibly because of scalability, including pile diameter, embedment depth, and also can be related to the limited amount of freezing liquid which got warmer in a relatively short period of operation time, and consequently fails to maintain the low temperature for a long time. Numerical simulations will be used to investigate the scale effects on the surrounding soil temperature, and also to eliminate the system flaws related to limited available freezing liquid and limited operation time period.

To understand the variation of soil temperature over time, Figure 3.12 shows temperature distribution over antifreeze circulation time for three different distances at a depth of 100 mm. Soil at 10mm distance from the pile was the first vertical bond that experienced temperature reduction after 4min of freezing system operation. Upon 25min use of refrigeration system soil temperature was reduced to -3.5˚C. Following the first soil area at 10mm distance, soil at 20mm and 30mm distance from the pile began to reduce in temperature level after 6min and 10min respectively. Operating freezing system for 30min resulted the temperature level of soil at 20mm and 30mm distance from the steel pile to drop up to -3.4˚C and -2.5˚C respectively.

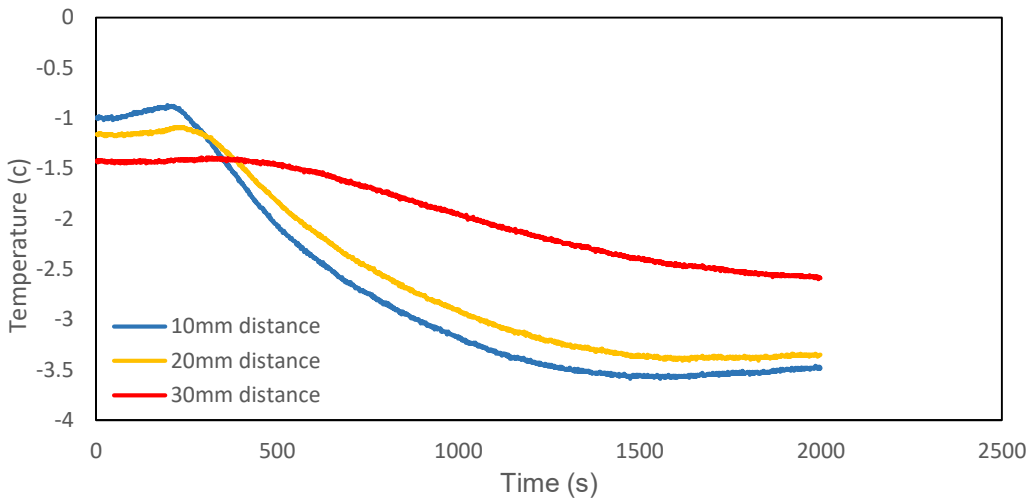


Figure 3.12: Variation of soil temperature over time at a depth of 100 mm

The temperature data shown in Figures 3.11-12 were collected by thermocouples installed at 10 mm, 20 mm, and 30 mm distances from the steel pile. The "5TE" sensors installed at 80 mm, 160 mm, and 240 mm distance from the pile did not record any changes in temperature when antifreeze was circulated into the pile. This means that the cold liquid circulation used in this experiment was only capable of freezing the pile surrounding soil in close proximity to the pile. This could be related to the limited volume of the antifreeze source (i.e., 10 gallons of -20˚C antifreeze liquid). As a result of liquid circulation, the antifreeze absorbs the heat the surrounding soil around the pile and gets warmer over time. The warmer liquid was transferred back to the freezer for temperature reduction but, due to continues circulation of the liquid, the freezer could not significantly decrease and maintain the temperature of antifreeze because of limited exposure time in the freezer. Higher volume of antifreeze liquid and a larger freezer would be required for freezing wider surrounding soil around the pile.

3.8.3. Pile pull out bearing capacity achieved by the use of cooling system

To examine the impact of freezing liquid circulation on load transfer of the model pile, a pull-out pile load testing was also conducted on the model pile while circulating the freezing liquid into the pile. After 25 min of liquid circulation, the soil temperature at a distance of 10 mm from the pile dropped below the freezing point (about -2.5˚C to -3.5˚C) up to an embedment depth of about 500 mm. Assuming that this change in temperature has created an adfreeze bond between the pile and the surrounding soil over a portion of the pile's embedment depth (up to 500 mm), the pull-out load test was carried out 25 min after start of the circulation. Similar to the previous load test, the pull-out process was implemented by the use of hydraulic jack and the load cell system. The load-displacement curve for the load test is shown in Figure 3.13 and, as it is noted, the partial pile-soil adfreeze bond created by freezing liquid has significantly improved the load transfer of the pile. The maximum pull-out capacity of the model pile was measured at 616 N with a pile head displacement of about 4-5 mm. Since the weight of the pile and the pile cap system is included in this measured value of 616 N, subtracting their equivalent weight (about 68 N) would lead to the ultimate shaft capacity of 548 N in partially frozen condition.

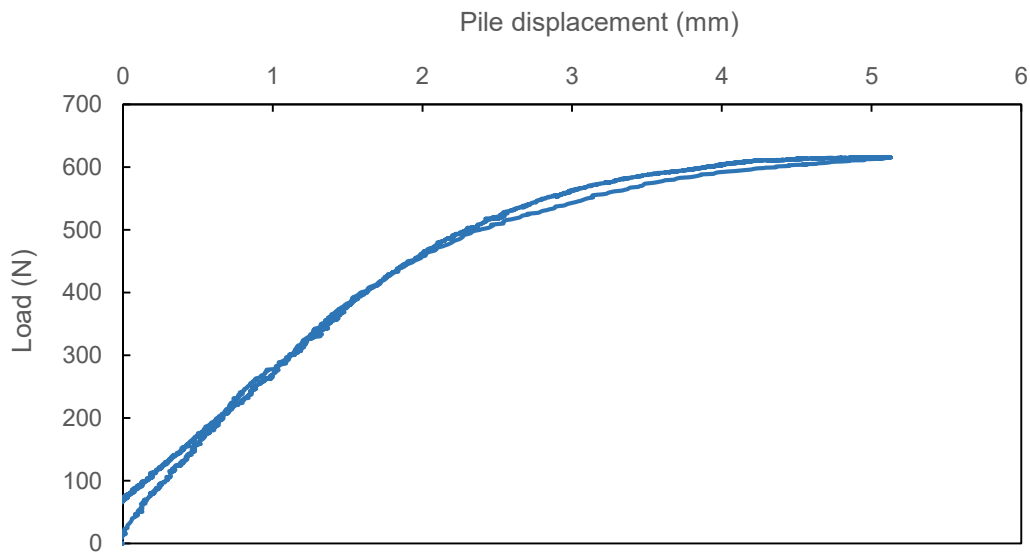


Figure 3.13: Pull-out bearing capacity of the pile after antifreeze circulation

The bearing capacity of pile foundations in ice-poor frozen soil is often estimated based on the adfreeze strength of the pile-soil interface as proposed by Weaver and Morgenstern (1981):

$$\tau_{al} = m\,\tau_{lt} \qquad (3.3)$$

where "m" is an empirical roughness factor that characterizes the pile surface and pile's surficial variables such as impurities, and "τ_{lt}" is the long-term shear strength of frozen soils that can be expressed using Mohr-Coulomb failure criterion as follow:

$$\tau_{lt} = C_{lt} + \sigma_n \tan \phi_{lt} \qquad (3.4)$$

where: C_{lt} and ϕ_{lt} are the long-term strength parameters of the frozen soil and σ_n is the normal stress acting on the pile shaft. Weaver and Morgenstern (1981) stated the second term in this equation may be neglected as the normal stress acting on the pile shaft is typically less than 100

kPa which makes the contribution of the frictional component to the total strength insignificant. Therefore, the long-term shear strength of the frozen soil may be reduced to:

$$\tau_{al} = m\, C_{lt} \qquad (3.5)$$

The embedment depth of the model pile in frozen part of the soil was about 500 mm. Considering a proportional shaft capacity of about 74 N for the unfrozen portion of the pile (i.e., 200 mm) based on unfrozen pile load test, the shaft capacity within the frozen section of the pile would be about 474 N. A roughness factor of 0.7 was inferred for steel piles in ice-poor frozen soils based on data from interface testing conducted by Aldaeef and Rayhani (2019). Assuming the same roughness factor for the model steel pile used in this study, the long-term adfreeze strength for this experiment would be about 8.6 kPa which is more than twice the unit shaft capacity measured in unfrozen condition.

3.8.4. Exposure temperature effect on pile bearing capacity

The pile load tests conducted before the use of the cooling system, and after liquid circulation were compared to investigate the role of this proposed cooling system on load transfer of the pile (Figure 3.14). As noted in the figure, the pile shaft capacity has significantly increased from an initial strength of about 326 N to about 616 N when the pile was exposed to circulation of antifreeze liquid for only 25 min. The freezing liquid circulation was able to drop the initial temperature at the pile-soil interface level from about -0°C to an exposure temperature of -about -3°C. This change in pile-soil interface temperature was capable of increasing the pile shaft capacity significantly due to development of adfreeze strength at the pile-soil interface level. Previous studies (e.g., Aldaeef and Rayhani, 2019) have shown the pull-out bearing capacity of piles in frozen sands could be 4 to 6 times the shaft capacity in thawed soils depending on the exposure

temperature. However, the shaft capacity in artificially frozen soil was only about twice the initial capacity in unfrozen condition. This limited effect of the proposed cooling system is partially related to the limited volume of the freezing liquid as well as the small size of the freezer used here. The freezing potential can be increased by the use of more liquid and larger freezer system.

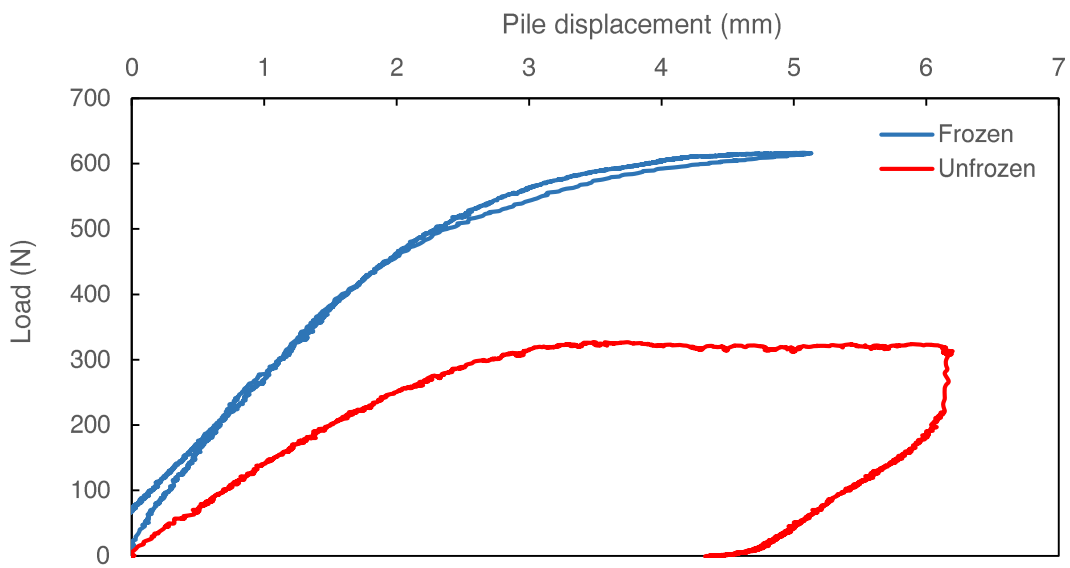


Figure 3.14: Comparison of pile bearing capacity in frozen and unfrozen conditions

The load-displacement curves for both the pile load test also show that loading behaviour was mostly governed by elastic response of the pile-soil interface under frozen condition. The elastic feature of frozen soil which is created by the use of refrigeration system caused the pile to go back to its original place when the pile was unloaded. However, the 5mm displacement was maintained for unfrozen condition test.

3.9. Conclusions

A new refrigeration technique was explored in this chapter to refreeze the thawed soils in permafrost regions in order to improve the soil bearing capacity and as a result enhance the pile foundations' load carrying capacity. The objective of this research was to evaluate the possibility of decreasing temperature at the pile-soil interface level. To examine the impact of freezing liquid circulation on load transfer of the model 2-inch pile, a pull-out pile load testing was conducted on the model pile while circulating the freezing liquid into the pile, and the results were compared to the model pile pull out testing without use of freezing system. The test pile was evaluated under pull-out loading in unfrozen and frozen soil condition and the following conclusions can be made:

- In the first 25 minutes use of refrigeration system, the temperature of the soil around the pile dropped from about 0˚C to -3.5˚C. This shows that the novel refrigeration technique developed in this research is capable of reducing the soil temperature around the pile and as a result could enhance the adfreeze bond between the pile and the surrounding soil. By use of the freezing liquid inside the steel pile foundations, it is possible to lower the soil temperature below the freezing point and preserve the frozen condition of the soil in warming permafrost regions.
- The temperature reduction of the soil due to utilizing freezing system starts from the soil surface and then continues to affect the deeper soils accordingly. Also, the cold liquid circulation technique used in this experiment is only capable of freezing the pile surrounding soil in close proximity to the pile in a short time period.
- This change in temperature creates an adfreeze bond between the pile and the surrounding soil over a portion of the pile's embedment depth (up to 500 mm). The partial pile-soil

adfreeze bond created by freezing liquid has significantly improved the load transfer of the pile.

- The actual shaft frictional resistance of the model pile was found to be 258 N in unfrozen condition. This frictional resistance was observed to engage at low displacements ranging from 3 mm to 3.5 mm corresponding to 6-7% the pile diameter. The average unit shaft resistance, q_s, for this model pile was estimated at about 2.35 kPa from the measured pile capacities in unfrozen condition. Considering the average effective stress around the pile shaft in unfrozen condition, the corresponding β-coefficient for the tested pile was also estimated to be about 0.33.
- The pile shaft capacity was significantly increased from an initial strength of about 258 N to about 548 N when the pile was exposed to circulation of antifreeze liquid for only 25 min. The ultimate shaft capacity of the model pile was measured at 548 N with pile head displacement of about 4-5 mm in partially frozen soil condition. The long-term adfreeze strength for this experiment was measured to be about 8.6 kPa which is more than twice the unit shaft capacity measured in unfrozen condition. The shaft capacity in artificially frozen soil was only about twice the initial capacity in unfrozen condition, while, the pull-out bearing capacity of piles in frozen sands could be 4 to 6 times the shaft capacity in thawed soils depending on the exposure temperature.
- The load-displacement curves for both the pile load test also show that loading behaviour was mostly governed by elastic response of the pile-soil interface under frozen condition. The elastic feature of frozen soil which is created by the use of refrigeration system caused the pile to go back to its original place when pile was unloaded. However, the 5mm displacement was maintained for unfrozen condition test.

CHAPTER 4: NUMERICAL SIMULATION OF PILE SOIL INTERFACE

4.1. Introduction

Pile foundations have been widely used to transfer the structural loads to the ground in permafrost region mainly due to their high load carrying capacity in frozen ground. These foundations are often designed based on the strength characteristics of frozen ground materials and the adfreeze bond at the pile-soil interface. However, pile foundations located in frozen grounds may experience significant reduction in their load carrying capacity when exposed to higher ambient temperature due to permafrost thawing and loss of pile-soil adfreeze strength. Foundation engineering practice in permafrost regions currently attempts to preserve the frozen condition of the soils by preventing the heat transfer from buildings and structures into frozen soils through several mitigation measures such as insulation, thermosiphons, coatings, air convection method, and sun sheds. However, thawing of adfreeze bond posed by higher ambient temperature related to global warming in frozen grounds is a major problem for pile foundations specially for existing piles in warming permafrost area. Therefore, a new retrofitting technique is required to be employed to maintain the frozen state of pile-soil adfreeze bonding around the piles.

Chen et al. (2020) proposed the use of freezing liquid in concrete piles (Fast Freezing System) to lower the temperature of piles locally while casting in place, reduce the refreezing time of concrete, and ideally enhance the freezing force of cast-in-place piles in permafrost regions. FFS was reported to be effective to accelerate the refreezing process of cast-in-place pile, and also improved its freezing force in permafrost. Chen et al (2020) observed that FFS had a significant

cooling effect which can reduce the interface temperature of pile foundation to below the natural ground temperature in a few days. An experimental setup was also developed here and discussed in previous chapter to investigate the possibility of local pile-soil interface freezing using a freezing liquid circulated into steel pipe piles. The experimental results showed that local freezing is capable of decreasing temperature in the pile surrounding soil and development of adfreeze bond at the pile-soil interface. Consideration the limitations of experimental model tests, this chapter aims at developing a numerical model based on the model test results obtained from experimental testing discussed earlier.

This chapter presents the numerical model developed to simulate a steel pipe pile embedded in unfrozen sand and examine the possibility of adfreeze bond formation induced by circulation of freezing liquid in the pile. The finite element numerical models were developed employing ABAQUS software and calibrated based on the test results obtained from experimental model tests. The models were then used to investigate other parameters controlling the response of the pile-soil system to local freezing. These parameters include pile geometry (diameter and length), ground temperature, exposure freezing liquid temperature and other pile and soil characteristics. This chapter presents the development details of these numerical models as well as parameter analysis in this study.

4.2. ABAQUS Model Characteristics

Finite element method of analysis is widely used to simulate soil stress-strain characteristics in a variety of geotechnical engineering problems. The numerical model that can simulate pile foundations in warming permafrost would require specific features including 3-Dimensional

modeling, ability to model interface elements and soil behaviour, auto-mesh control, and ability to adopt temperature dependent characteristics. A three-dimensional finite element was developed using ABAQUS software to simulate axial pull-out load tests in cohesion-less soils exposed to different temperatures. ABAQUS is a highly sophisticated 2D-3D program capable of providing features mentioned above as well as characterizing static, dynamic, thermal, and acoustic cases with linear or nonlinear solutions. In this model, distinctive characteristics which influence the outcome of the analysis were considered individually based of the soil and pile properties to achieve the best and most accurate results. These characteristics include element types, degrees of freedom, number of nodes and the material, constitutive law.

4.3. Model Components

Material properties are one of the most important aspects of finite element modelling to ensure model efficiency and accuracy. To simulate the response of the soil within elastic region, the model components were defined using an elastic modulus, Poisson's ratio, and density. The plastic behaviour for each material was also defined in a manner in which the material was expected to fail. An axisymmetric finite element model was developed to predict the load displacement response of steel pipe pile foundations in homogeneous unfrozen and frozen sandy soil. This model incorporates the following simulation techniques: the selection of mesh boundaries and element type, constitutive model for soils, simulation of pile-soil interaction, and simulation of initial stress conditions in soils. The soil was considered as an elasto-plastic medium following the Mohr-Coulomb constitutive model, while the steel pipe pile was assumed to act as a linear elastic material. The finite element modeling techniques were verified by analyzing experimental cases

given in previous chapter. It is important to note that parameter units must be in the same standard system.

A barrel domain of 600 mm diameter and 850 mm height was modeled to simulate the soil container while a solid pile element with a diameter of 50.8 mm and an embedment depth of 700 mm was modelled to simulate the model pile. The diameter of the soil domain was sufficiently large (twelve times) compared to the diameter of the pile to minimize the boundary effects on pile-soil model behaviour. The soil around the pile was modeled using a modified form of Mohr-Coulomb failure criterion. The modification involves the variation of mobilized angle of internal friction and dilation angle with plastic shear strain. Table 4.1 shows the model properties for the model pile as well as the soil properties used for modelling both unfrozen and frozen soil conditions. The model soil was a poorly graded sand with a fine content of about 1.55% (Figure 3.1). Normally for cohesion-less soils, the cohesion parameter is considered as zero but due to moisture content of the sand used in this model, a small cohesion of 1 kPa was considered to represent the apparent cohesion in the soil. The friction angle of the soil in unfrozen condition was measured at about 40° using direct shear test. The soil density in unfrozen condition was also measured at 1800 kg/m^3.

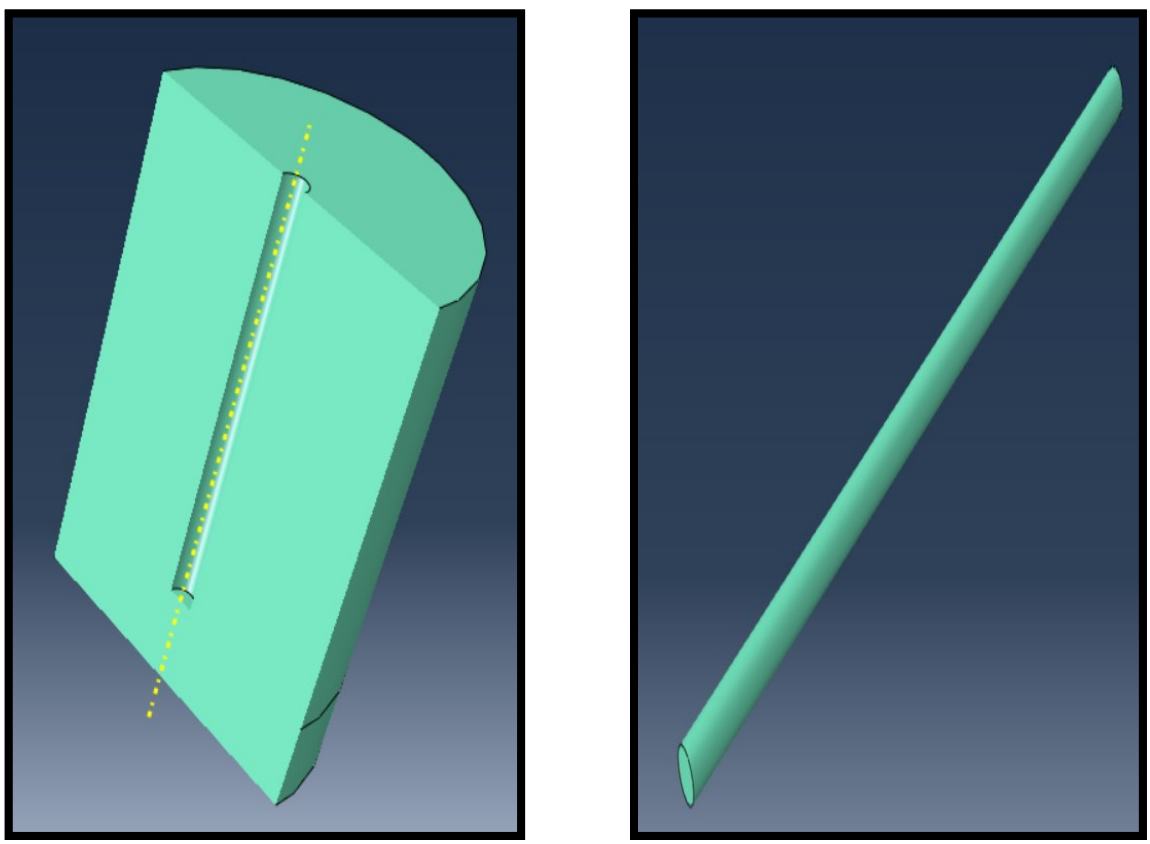
A solid steel pile of 1380 mm length and 50.8 mm diameter was modeled to simulate pile-soil in unfrozen and frozen soils. Considering the mass of the pile and pile head system, the density of the pile element was considered as 3369 kg/m^3 to simulate the equivalent weight of the pile compared to reality. The pile-soil interface can be modelled through different interaction mechanisms of surface to surface, self-surface, or acoustic impedance in ABAQUS. The interface system in this case was modelled using a predefined hole in the soil and connecting the pile element

to internal surface elements of the hole through the surface-to-surface connection mode. This solution was found to be more appropriate for the pile-soil interface simulation.

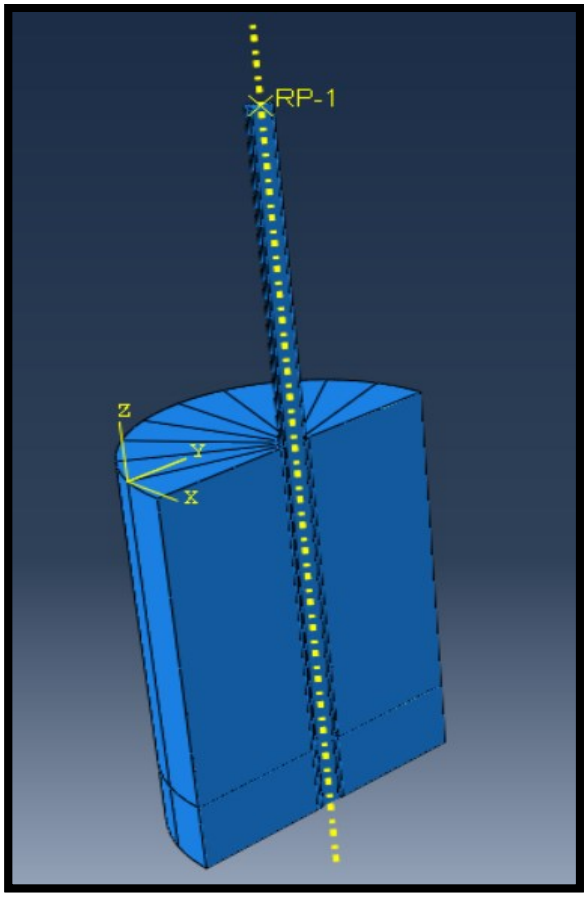
Table 4.1: Model properties used in numerical analysis

Soil properties (Unfrozen)		
Mechanical parameter	Value	Reference
Modulus of elasticity (E_s)	40,000 kPa	Aldaeef and Rayhani (2019)
Poisson's ratio (ν_p)	0.3	
Density (ρ)	1800 kg/m^3	
Cohesion (C)	1 kPa	
Internal friction angel (ϕ')	40˚	
Dilation angel (ψ)	10˚	
Steel pile model properties		
Modulus of elasticity (E_p)	210×10^6 kPa	
Poisson's ratio (ν_p)	0.15	
Density (ρ)	3369 kg/m^3	
Soil model properties (frozen)		
Modulus of elasticity (E_s)	60,000 kPa	Aldaeef and Rayhani (2019)
Poisson's ratio (ν_p)	0.3	
Density (ρ)	1800 kg/m^3	
Cohesion (C)	440 kPa	Aldaeef and Rayhani (2019)
Internal friction angel (ϕ')	40˚	
Dilation angel (ψ)	10˚	
Specific heat (C_s)	722 Joule/˚C	Tarnawski et al (2009)
Thermal conductivity (k_s)	0.274 W/m˚C	Tarnawski et al (2009)

The interaction between two bodies in ABAQUS is often defined through contact between two interacting “master” and “slave” surfaces known as “contact pair” (see Figure 4.2). The master surface should be a surface which is more rigid than the slave surface. The nodes of the master surface (i.e., pile) could penetrate the slave surface (i.e., soil) (Figure 4.2). ABAQUS simulates two types of contact behaviour for surface-based contacts. One is the tangential friction between the two surfaces, and the other one is the load transfer between the two surfaces in normal direction. The pile-soil interface was selected as tangential and isotropic behaviour, and a friction method was employed to control potential shear strain between the pile and the soil based on friction values.



(a) (b)



(C)

Figure 4.1: Geometry of the soil (a), pile (b), and pile-soil assembly (c) model in ABAQUS

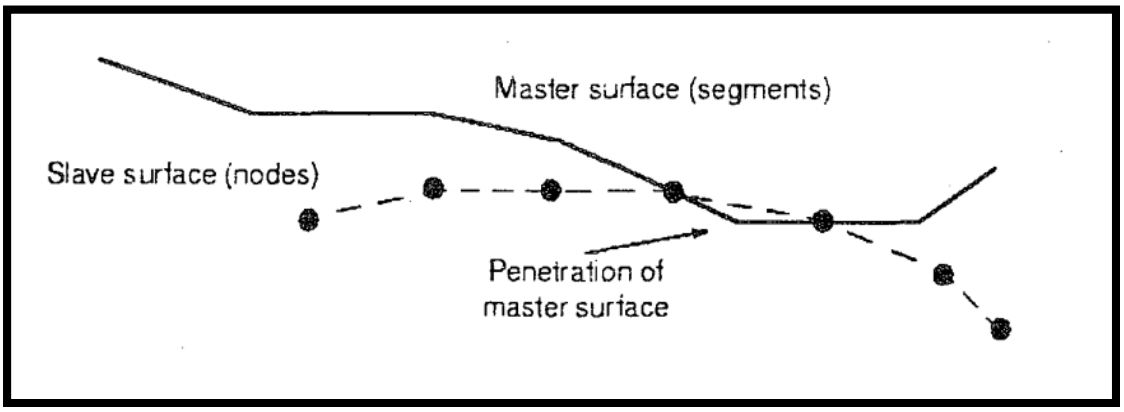


Figure 4.2: Master and slave surface interaction- (ABAQUS Manual, 2006)

Frictional values were assigned based on steel-soil shear strength values considering Coulomb's friction model. This model defines the friction coefficient (μ) as μ=tan (ϕ_μ), where ϕ_μ is the pile-soil interface friction angle. This interface friction angle (ϕ_μ) was considered as 70% of the friction angle of the soil (0.7ϕ') in this analysis, and a friction coefficient (μ) value of 0.56 was used for interface simulation of the steel pile in unfrozen sand. A similar ratio is often used in practice for estimating the interface shear strength of pile foundations. An elastic slip of 3.1 mm was defined as absolute distance to specify the maximum elastic slip of the pile in soil based on experimental observation of the model pile test. For frozen soil, the shear strength parameters were obtained from earlier experimental tests conducted by Aldaeef and Rayhani (2019) on the same soil (Table 4.1). The pile-frozen soil interface friction angle, ϕ_μ, was also assumed as 70% of friction coefficient (μ) in frozen sand (0.86). Temperature dependent properties including thermal conductivity and heat capacity of the soil was obtained from Tarnawski et al (2009).

Meshing 3D elements in numerical modelling can sometimes be challenging, depending on the type of element and the size in the region, where stress and deformation are expected to be relatively high. Automatic meshing might lead to errors in most of the cases and also it takes a lot of time to run an "explicit" model in ABAQUS, therefore, the soil and pile geometry models have been manually meshed by the use of partition creation in soil and pile critical contact regions for the sake of accuracy and time. The size of the mesh has a significant effect on finite element modeling. Often a finer mesh yields more accurate results, but computational time is higher. For successful modeling of load-displacement behaviour of piles under axial load, a denser mesh should be used near the pile. As shown in Figure 4.3, smaller soil elements were used near the pile and the size of the elements were increased with radial distance from the center of the pile. A 3D stress family mesh of standard element library (C3D8R) have been chosen for the pile-soil

interface and pull-out model tests. However, for temperature analysis models, a heat transfer family mesh of standard element library (DC3D8) was selected.

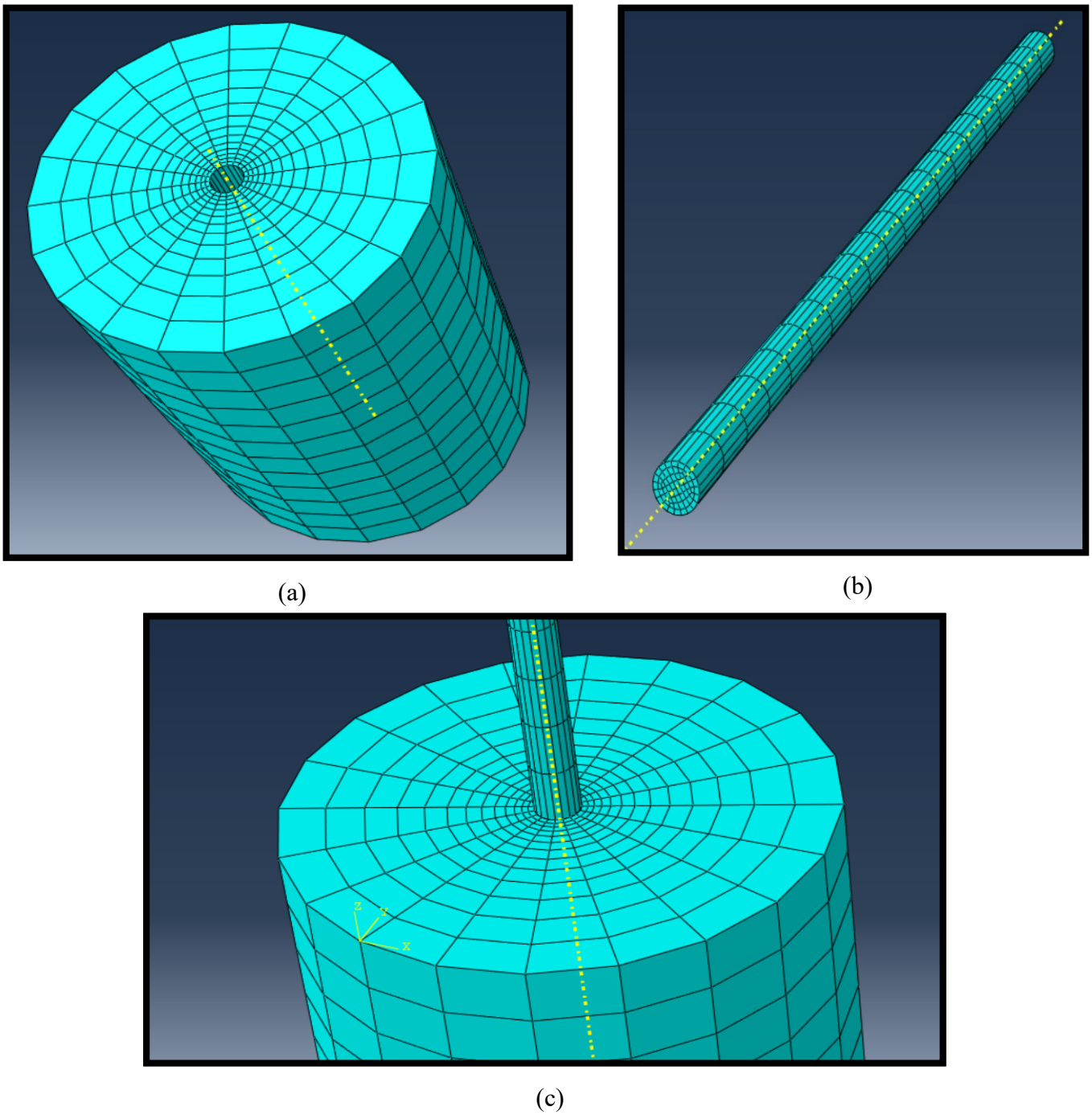


Figure 4.3: Manually meshed soil (a), pile (b), and pile soil assembled (c)

A series of load and temperature boundary conditions was employed to simulate the model container and define degrees of freedom in two separate models including thermal analysis and pile pull-out loading analysis. Other part of the model can be sufficiently meshed to transfer stresses from one node to another. For simulation of pile pull-out load tests, “Geostatic” and “Loading” steps were defined in step module as “Static-General” analysis, and then loads of gravity, confinement pressure of soil, and pull-out load as a displacement control form had been assigned to these steps. Pile displacement rate and amplitude was assigned to the pull-out loading model based on the experimental tests in lab. Displacement boundary conditions were applied to the model where any movement or rotation at the bottom of the soil was constraint to prevent any movement in any direction. In thermal analysis simulations, “Geostatic” step was replaced with “Heat Transfer” analysis. Temperatures were applied through the pile-soil interaction surface as a form of thermal boundary condition to the soil. Once the geometry of the soil and pile models were designed, initial thermal boundary conditions of the surrounding soil were added to the model. The boundary at the bottom of the soil was constrained to prevent any movement or rotations in any direction.

4.4. Numerical Models

4.4.1. Thermal analysis models

The aim of thermal analysis models was to investigate the temperature distribution within the soil and the pile-soil interface under various thermal exposures. Temperature load was applied through the predefined pile hole in soil where soil and pile were in contact. For unfrozen soil condition, an initial temperature of 1˚C was applied to both the soil and pile models. Thermocouples installed at pile-soil interface in experimental tests were showing pile’s surface temperature in the range of -

8˚C to -10˚C after 25 minutes of freezing liquid circulation. To simulate the temperature distribution during local freezing, a thermal load equivalent to the pile temperature of -8˚C was applied to the pile-soil contact surface for 25 minutes (Figure 4.4). After comparison of the model with experimental results, a total of 6 thermal analyses were conducted to investigate the impact of various controlling parameters including pile diameters (2, 4, and 5-inch piles), initial soil temperatures, and exposure time effects on temperature distribution in pile-soil systems.

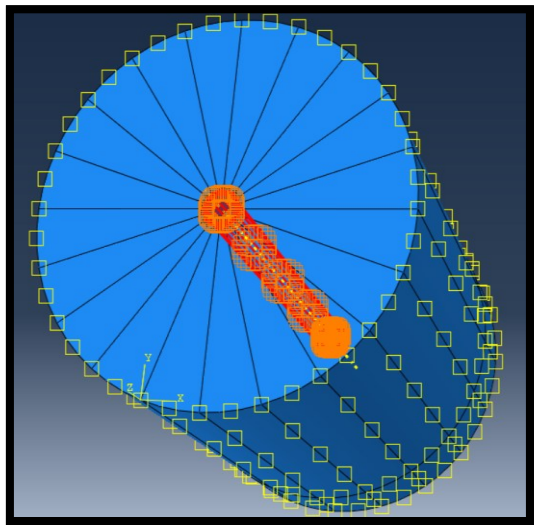
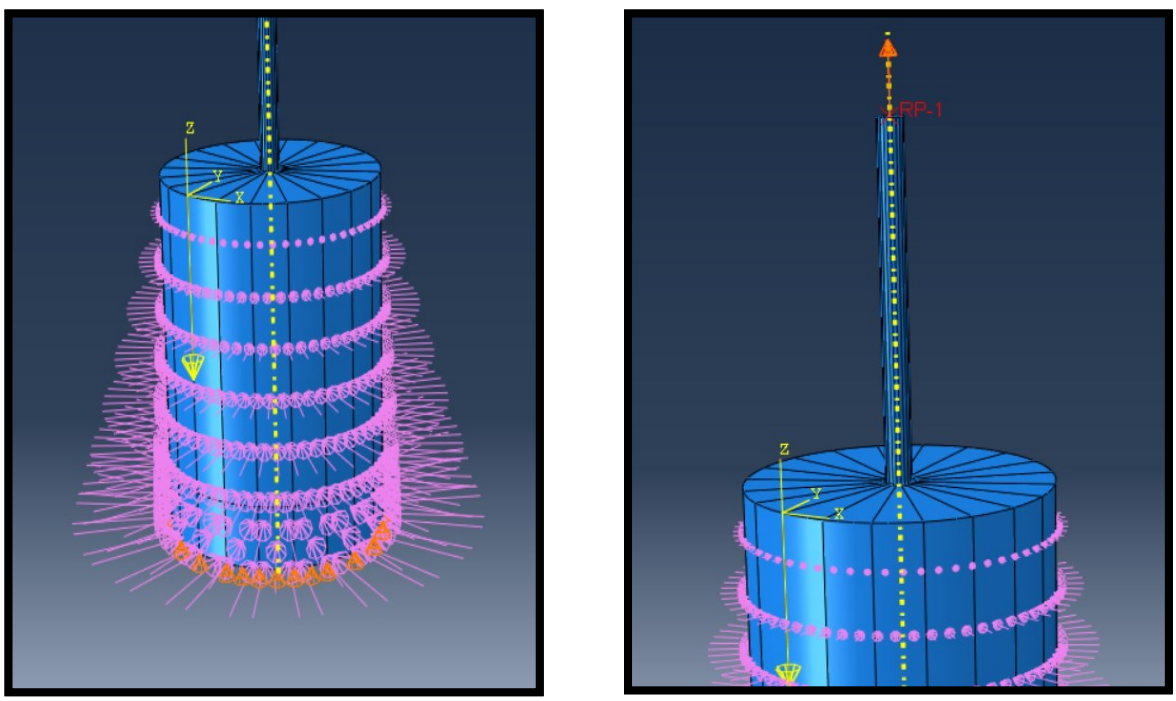


Figure 4.4: Thermal load application to the soil through pile-soil contact surface

4.4.2. Pile load test models

A series of pull-out test models were also developed to simulate the pile-soil interface behaviour and examine possible changes in the shaft load carrying capacity of a steel pile when exposed to local pile temperature variance. The pile was initially embedded in unfrozen soil before use of any freezing liquid technique, and after utilizing freezing system, the surrounding soil was in frozen condition up to an embedment depth of about 500 mm. Considering the soil density of 1800 kg/m^3 and soil depth of 0.85 m, a confinement pressure of 8075 N/m^2 was applied linearly from the soil

surface to the bottom of the soil model in unfrozen condition. Due to slight increase in volume upon soil freezing in the barrel, and expansion pressure resulted from the side soil, the confinement pressure for frozen condition was considered 1.4 times of the unfrozen one. After defining the boundary conditions, material properties, and interactions, the gravity was imposed on the model to simulate the real field condition for the pile-soil model. The pile pull-out loading and unloading were applied to the pile head through displacement-controlled method in the axial directions. A total of 6 models were carried out using various pile diameters (2, 4, and 5-inch pile) in both frozen and unfrozen soils.

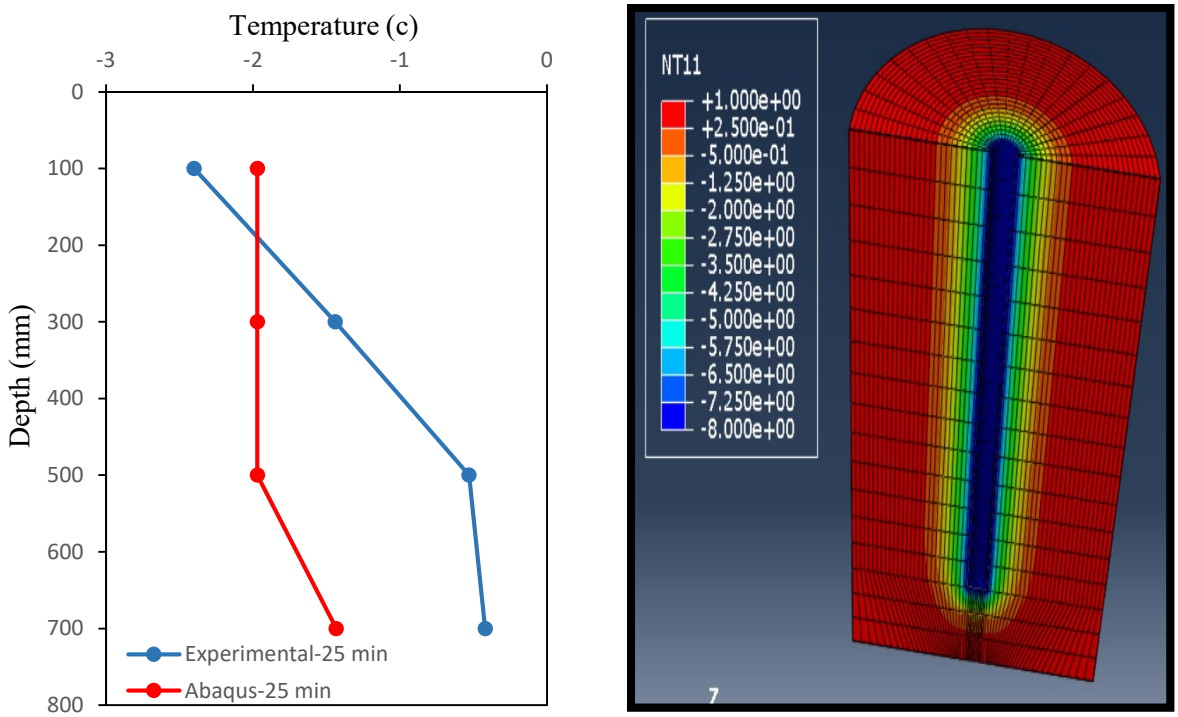


(a) (b)

Figure 4.5: Application of (a) confining pressure and gravity load, and (b) pull-out load

4.5. Model Results and Verification

Thermal model results can be divided into two types of outputs from the software including temperature data with respect to the time for each selected nodes of the model, and also spatial field temperature outputs (Figure 4.6). The spatial field temperature results are useful to show the big picture of the whole model and what happens when the model was subjected to different exposure conditions. To validate the thermal analysis developed in ABAQUS, temperature profiles measured in the surrounding soil at 30 mm distance from the 2-inch pile was compared with the results obtained from numerical models. As shown in Figure (4.6), after 25 minutes of using freezing liquid, the soil profile at 30 mm radius distance from the pile cooled down to -2˚C. In numerical analysis, the temperature dropped from 1˚C to -2˚C which was comparable to the experimental lab results. However, the variation of the temperature with depth between the two models is not consistent.

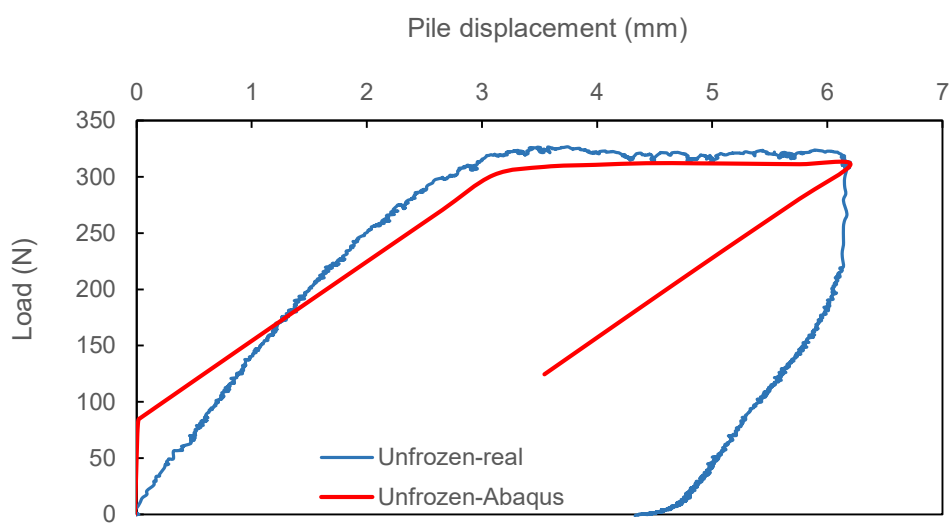


(a) (b)

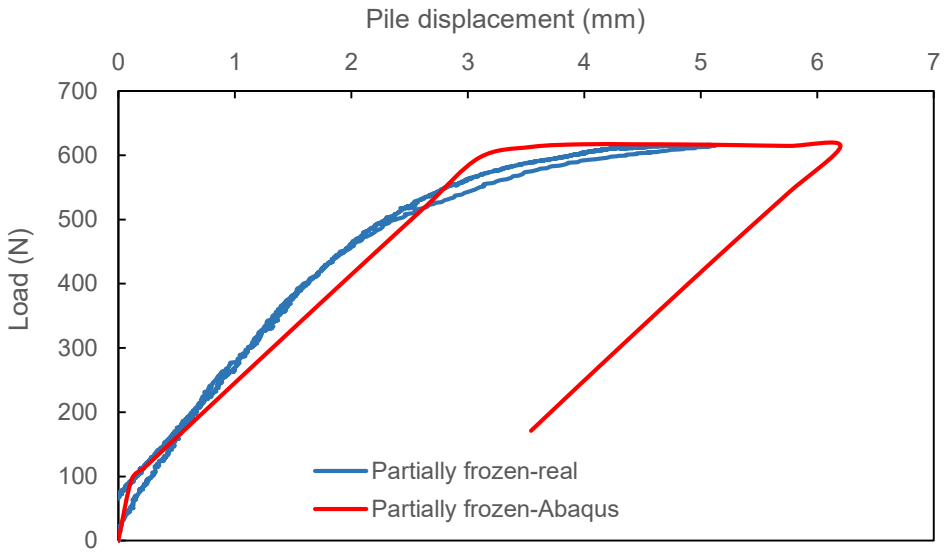
Figure 4.6: Soil temperature profile at 30mm distance from the 2-inch pile, (a) data, (b) spatial field temperature

Simulation of elasto-plastic behaviour in geomaterials, specifically cohesive-frictional materials, has experienced extensive progress in the form of research (Houlsby, 1991). However, discrepancy still occurs between theoretical models and in-situ soil behaviour (Houlsby, 1991). Each of the simulated experimental pile load tests were loaded until plastic failure was observed. Each load in the developed numerical models was also applied until the permanent plastic deformation in the soil-steel pile system was reached similar to the procedure used for axial pull-out load test in the lab. Altogether, 6 pile load tests and 3 different temperature distribution models were conducted consisting frozen and unfrozen sand.

Figure 4.7 shows the pull-out bearing capacity of the steel pile embedded 700 mm in unfrozen and semi-frozen sand obtained from both the measured load tests and the finite element models developed in ABAQUS. As discussed earlier, the finite element models were developed using material properties of unfrozen and frozen soils, pile-soil interaction values, and loading patterns similar to experimental tests. A 2-inch pile diameter with 700 mm embedment depth was modeled in ABAQUS, and the cooling performance of refrigeration system was investigated by monitoring soil profile temperature distribution in different times. After operating the cooling system, soils around the pile experienced a temperature of -2˚C and formed a partially-frozen bond (adfreeze bond) around the pile. Due to higher pile-soil interface strength induced by the adfreeze bond, shaft bearing capacity of the pile enhanced. As it is shown in Figure 4.7, the finite element model results for both unfrozen and semi-frozen sand are very close to measured experimental values with a very negligible difference in yield resistance of the pile shaft capacity. These verified models were then used to investigate the influence of pile and soil geometries, cooling temperature and boundary conditions on performance of the developed cooling system and pile-soil behaviour.



(a) Unfrozen Soil



(b) During freezing liquid circulation

Figure 4.7: Comparison of lab experimental test vs. ABAQUS model results of 2-inch pile pull-out capacities in (a) unfrozen sand and (b) semi-frozen sand conditions

As it is shown in Figure 4.8, the shaft bearing capacity of the 2-inch model pile in unfrozen condition was 312 N. As a result of partial-adfreeze bond formation around the pile in 25 minutes' period, the shaft bearing capacity of the pile increased up to 617 N. Based on pile dimensions and unit weight in numerical model, the pile weight was calculated to be about 69 N, and this amount should be reduced from the total value that obtained from the software. This will result to a shaft capacity of 243 N for the 2-inch pile in unfrozen sand. This value is comparable to the shaft capacity of 263 N estimated based on the β method for this pile in cohesion-less soils. Similarly, a pile shaft capacity of about 548 N was calculated for the partially-frozen sand based on the adfreeze bond parameters obtained from Aldaeef and Rayhani (2019) and design procedure by Weaver and Morgenstern (1981). Therefore, it can be concluded that shaft capacity of a 2-inch diameter pile will be enhanced by 2.2 times if the proposed refrigeration technique is used to improve the adfreeze bond between the pile and its surrounding soil.

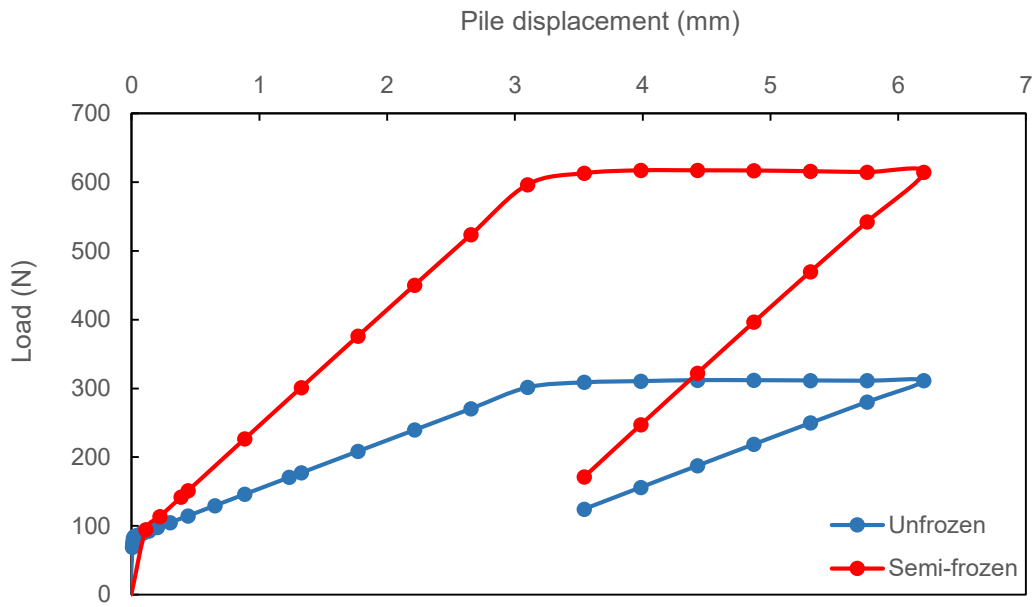


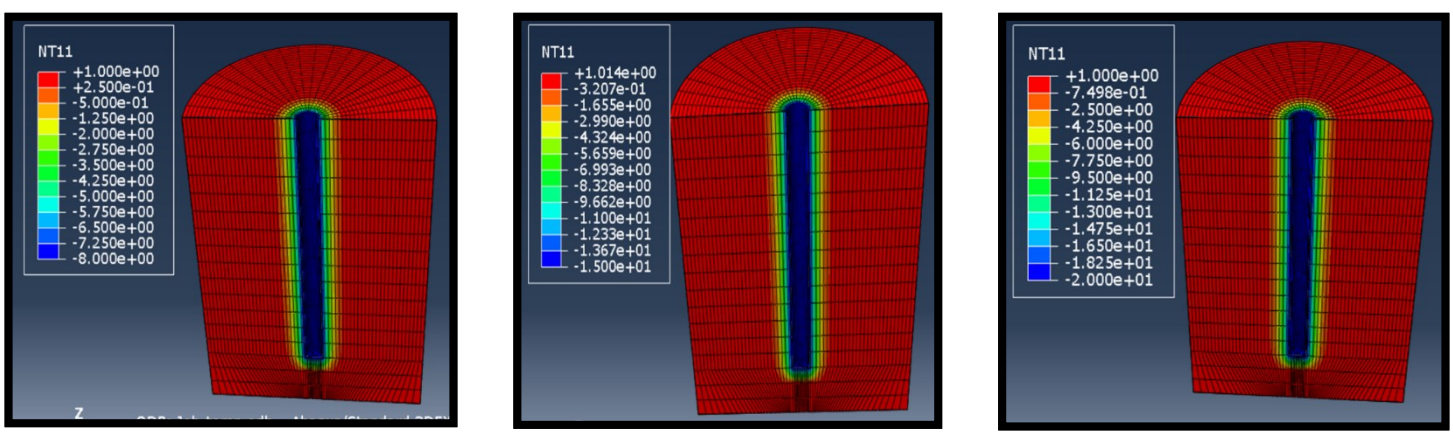
Figure 4.8: Pile shaft bearing capacity in unfrozen and partial frozen conditions

4.6. Analysis and Discussion

The pile and soil medium used in the finite element model were identical to a small experimental model pile with 50.8 mm in diameter embedded up to a depth of 700 mm in a sand medium to ensure proper validation of the model. This model was only exposed to a cooling pile temperature of -8˚C for 25 minutes, while the initial soil temperature was about 1°C. To expand the practical aspects of this cooling system, the calibrated model was used to explore other controlling parameters including freezing liquid temperature, initial soil temperature, circulation time, and pile dimensions, and optimize efficacy of the proposed cooling technique. This section summarizes results of these parametric analyses.

4.6.1. Effect of cooling temperature on soil temperature profile

In the initial numerical model, a cooling freezer at -8˚C was applied to the soil-pile contact area and distribution of temperature in the surrounding soil was investigated. To extend this investigation, colder cooling liquid including -15˚C and -20˚C were applied to soil-pile contact area to study the influence of colder freezing liquid on soil and pile's response. As it is shown in Figure (4.9a), after 25 minutes of thermal application at -8˚C, a radial distance of more than 30 mm from the pile-soil interface experienced temperature reduction up to -2˚C. Decreasing the thermal load from -8˚C to -15˚C decreased the surrounding soil temperature to -4˚C at the same radial distance of 30 mm over the same time period. Further drop in the applied freezing temperature to -20˚C was shown to induce much lower temperature of -6˚C in the surrounding soil (Figure 9c). These improvements in surrounding soil temperature is expected increase the adfreeze bond between the pile and the surrounding soil and, hence, lead to higher pile shaft bearing capacity.

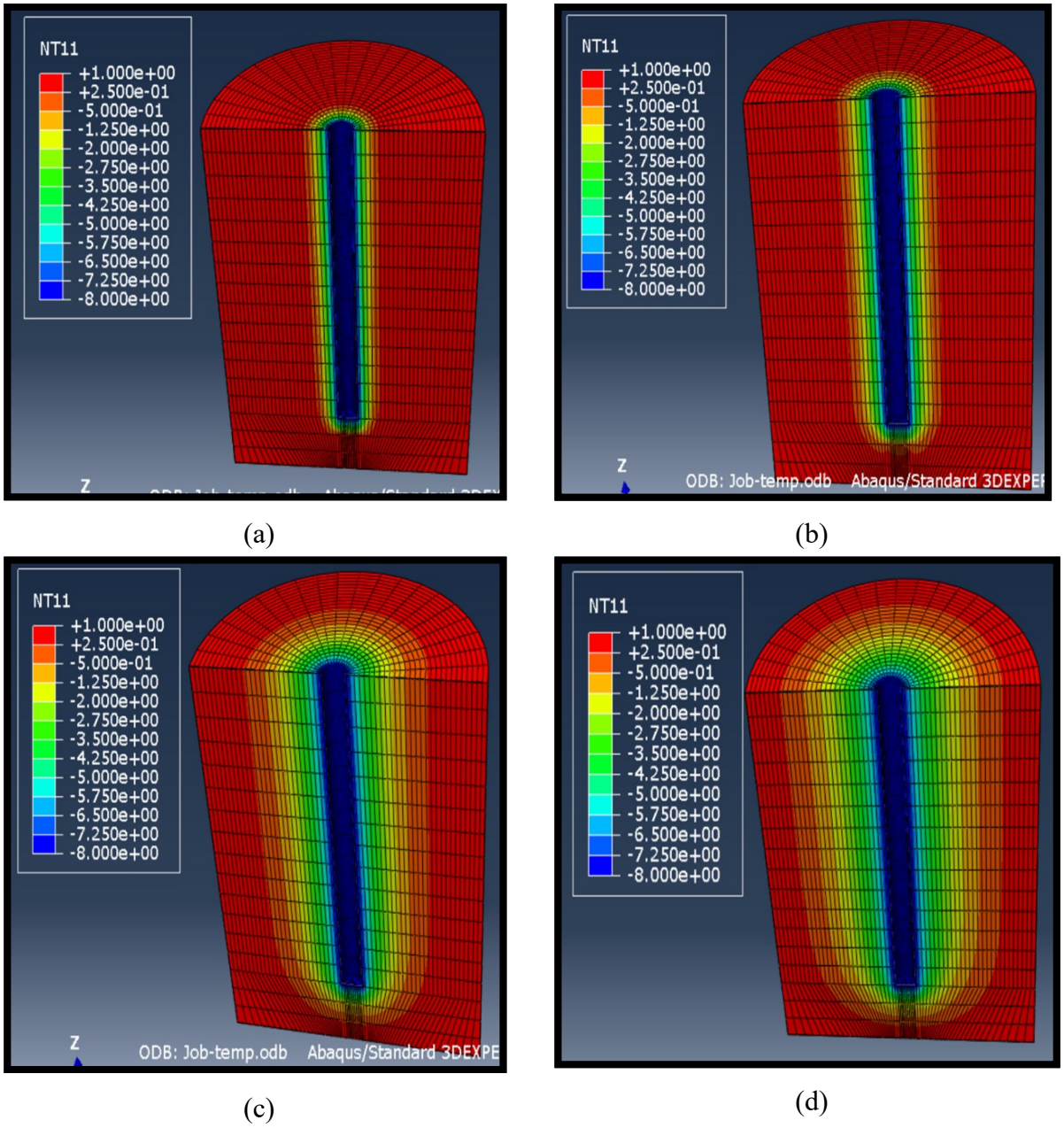


(a) (b) (c)

Figure 4.9: Temperature distribution in soil after 25 minutes of cooling exposure at (a) -8˚C, (b) -10˚C, (c) -20˚C

4.6.2. Effect of cooling exposure time on soil's temperature profile

The cooling circulation period was limited to 25 min in experimental testing due to the small size of the freezer and low volume of the freezing liquid. This setup led to freezing the pile surrounding soil up to a radial distance of 30 mm from the pile's surface. To examine the freezing behaviour of the pile-soil system over longer period of cooling operation, the thermal load application in the numerical model was extended over different time periods and the soil temperature was monitored at various depths and radial distances from the pile. As illustrated in Figure (4.10, b), cooling thermal application over 60 minutes would lead to freezing the pile surrounding soil up to a radial distance of 50 mm. After 60 min of cooling application, the temperature of the surrounding soil experienced a 3˚C reduction in temperature, and a soil temperature of -2˚C was achieved within the 50 mm distance near the pile. If the cooling system is operated for 5 hours in this pile-soil system, a minimum radial distance of 100 mm would be affected with a soil temperature of less than -1.2˚C. The critical radial distance around the piles in permafrost areas are mainly considered as two times the pile diameter (Dubina, Chernyakov, and Teslenko, 2003; Vyalov et al., 1998). If the soil temperature within this distance remains below the freezing point, then it is expected that the pile would maintain its maximum shaft bearing capacity depending on the specific soil temperature. As shown in Figure (4.10, c), freezing this critical radial distance would be achieved after 5 hours of cooling application for the 2-inch pile used here. Figure 10 (d) also shows the temperature distribution in the surrounding soil after 10 hours of cooling application. As it is noted, the freezing distance would be extended up to 200 mm radius distance around the pile and within this range the soil will experience -2˚C freezing temperature.



(a) (b)

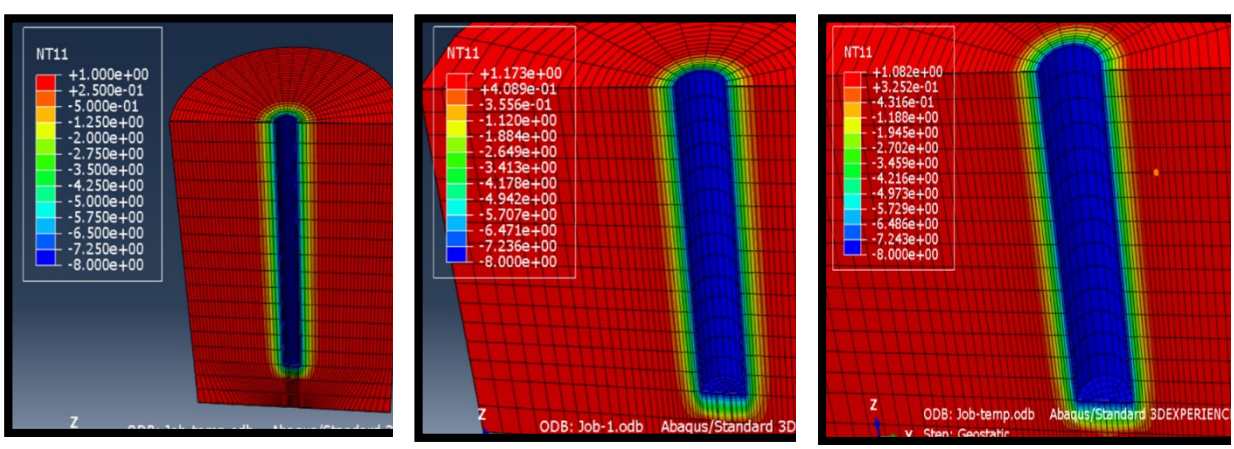
(c) (d)

Figure 4.10: Temperature distribution in soil with the applied exposure temperature of -8˚C, over (a) 25 minutes, (b) an hour, (c) five hour, (d) ten hours of time exposure

4.6.3. Effect of pile diameter on temperature distribution in soil

The pile used for development and calibration of the finite element model was a 2-inch diameter steel pipe pile which was embedded about 700 mm in soil. To investigate the effect of this novel

freezing system on performance of real pile in the field, two different pile models with 4 inch and 5 inch in diameters were modeled and exposed to similar thermal loading conditions. The diameter of the soil around the pile was also increased up to 6 times the pile diameter to achieve a similar soil/pile diameter ratio of 12 and satisfy the boundary condition criteria. The 4-inch pile diameter was modeled in a soil medium with 1.2 m diameter and a similar embedment depth of 700 mm. The pile-soil model was exposed to cooling application for 25 minutes at exposure temperature of -8˚C. As shown in Figure (4.11, b), increasing the pile diameter widens the radial thickness frozen soil from 30 mm up to about 50 mm under similar thermal loading exposure over 25 minutes. This means that when pile diameter increased from 2-inch to 4-inch -in the same temperature conditions- the created adfreeze bond will be enlarged by about 1.6 times. For the 5-inch pile diameter and under similar thermal loading conditions, the surrounding soil was found to freeze up to a radial distance of 70 mm (Figure 4.11, c). Compared to the initial 2-inch pile, the radial frozen soil thickness of the 5-inch pile would be about 2.3 times that of the 2-inch pile.



(a) (b) (c)

Figure 4.11: Soil temperature distribution piles with (a) 2-inch, (b) 4-inch, (c) 5-inch diameters

4.6.4. Effect of pile diameter on pile capacity during cooling operation

To examine efficiency of the cooling system in piles with different geometries, two extra piles with -inch and 5-inch diameters were developed in ABAQUS and tested under similar thermal loading conditions. To maintain similar boundary conditions, the diameter of the soil model was also increased to 1.2 m and 1.5 m respectively. Figure 4.12 shows the load transfer of the piles for both unfrozen condition and when they were exposed to cooling application (a.k.a., semi-frozen). The shaft capacity of the initial 2-inch pile was about 312 N in unfrozen condition and about 617 N in semi-frozen condition as a result of cooling operation for 25 minutes (Figure 4.8).

For the 4-inch (101.6 mm) pile diameter, the maximum pull-out capacity was reached at a pile displacement equivalent to 7% of the pile diameter in both unfrozen and semi-frozen conditions (7.0 mm). The peak shaft capacity of the 4-inch pile in unfrozen condition was estimated to be 668 N, while it reached a significantly higher shaft resistance of 952 N when subjected to cooling application. This is attributed to the larger surface area of the 4-inch pile as well as the wider frozen part of the surrounding soil. Therefore, using refrigeration system in 4-inch steel piles for 25 minutes is expected to increase the shaft bearing capacity of the pile by about 1.5 times that of the smaller 2-inch pile. The 5-inch model pile demonstrated a shaft capacity of 1354 N when it was exposed to cooling application. For unfrozen soil, the shaft capacity of this pile was about 958 N (Figure 4.12). These values were reached at a pile head displacement of about 8 mm (equivalent displacement of 6.5% pile diameter).

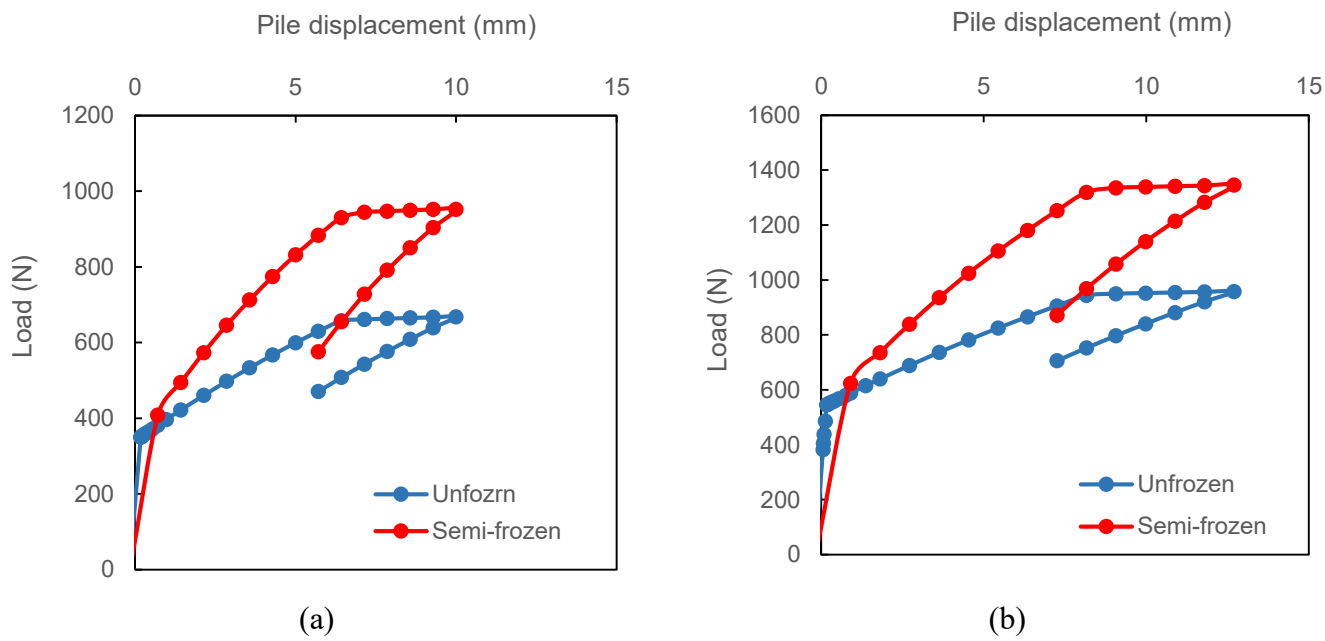


Figure 4.12: Load transfer of (a) 4-inch and (b) 5-inch piles under unfrozen and cooling application

Table 4.2 summarizes the load carrying capacity of all the three model piles when exposed to cooling application. As noted, increasing pile diameter seems to lead to better development of adfreeze bond between the pile and the soil and wider frozen area in the surrounding soil. However, the rate of increase in load capacity with pile diameter is not linear. The adfreeze improved load capacity for the initial 2-inch pile was about twice that of the unfrozen soil. This rate for the 4-inch and 5-inch piles was about 1.4-1.5 times the pile capacity in unfrozen soil condition.

Table 4.2: Pull-out bearing capacity of model piles with different diameter

Model Pile	Pile Shaft Capacity (N)		
	2-inch Pile	4-inch Pile	5-inch Pile
Unfrozen Soil	312	668	958
Cooling Application	617	952	1354

4.7. Summary

A finite element numerical model was developed for a 2-inch steel pipe pile embedded in cohesion-less soil in ABAQUS software to investigate the role of a cooling liquid circulation within the pile on bearing capacity of the pile. A thermal load with a temperature of -8˚C was applied to the pile-soil interface for 25 minutes to simulate the cooling liquid's temperature within the pile. The pile was subjected to a pull-out test to measure the shaft capacity of the pile for both unfrozen soil and when the pile-soil was exposed to cooling application. The loading test was simulated in ABAQUS by defining an interface between the pile and soil and applying an axial pull-out load to the pile head. Both the temperature model and the pile loading simulation were validated by comparing the software results with experimental lab results. After validation of all three models related to 2-inch pile, a detailed parametric analysis was conducted to investigate the influence of several controlling factors including pile diameters, cooling application period and the level of applied thermal load on performance of the cooling procedure on load transfer of the pile.

When the duration of cooling application was increased from 25 min to longer periods of one to five hours, the initial frozen area in the pile surrounding soil significantly widened from 30 mm up to 100 mm. A similar behaviour was observed when the temperature of the cooling liquid was dropped from -8˚C to colder thermal loads of -15˚C and -20˚C. Both the extend of the frozen surrounding soil as well as the soil temperature within the frozen region was improved by changing the thermal load and its duration.

Two numerical models were also developed 4-inch and 5-inch pile diameters to explore temperature distribution in the pile surrounding soil for piles with larger diameters and examine

possible changes in load transfer of the piles. Increasing the pile diameter had a significant effect on both temperature distribution and pile shaft capacity. The radial distance of frozen area in the pile surrounding soil increased from the initial distance of 30 mm up to 70 mm from the pile's surface. The load carrying capacity of the piles also increased from 617 N for the 2-inch diameter pile to 1354 N for the pile with 5-inch diameter during the cooling application. This was attributed to higher surface area of the pile and, hence, development of stronger adfreeze bond between the pile and surrounded frozen soil.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

In this thesis two studies were presented exploring the performance of a new refrigeration technique to refreeze the thawed soils surrounding pile foundations in permafrost area, and as a result enhance the pile foundations' load carrying capacity. In the first study an experimental test setup was carried out aimed at evaluating the possibility of decreasing temperature at the pile-soil interface level using freezing liquid circulation within a model steel pipe pile. To examine the impact of freezing liquid circulation on load transfer of the model 2-inch pile, a pull-out pile load testing was conducted on the model pile while circulating the freezing liquid into the pile, and the results were compared to the model pile pull-out testing without use of freezing system. In the second study, a finite element numerical model was developed for a 2-inch steel pipe pile embedded in cohesion-less soil in ABAQUS software to investigate the role of a cooling liquid circulation within the pile on bearing capacity of the pile. The pile was subjected to a pull-out test to measure the shaft capacity of the pile for both unfrozen soil and when the pile-soil was exposed to cooling application. Both the temperature model and the pile loading simulation were validated by comparing the software results with experimental lab results, and then, a detailed parametric analysis was conducted to investigate the influence of several controlling factors including pile diameters, cooling application period and the level of applied thermal load on performance of the cooling procedure on load transfer of the pile. Presented below are the major findings of both the research programs.

- During the first 25 minutes of refrigeration application, the temperature of the soil around the pile dropped from about 0˚C to -3.5˚C. This shows that the novel refrigeration technique developed in this research is capable of reducing the soil temperature around the pile and as a result could enhance the adfreeze bond between the pile and the surrounding soil. By use of the freezing liquid inside the steel pile foundations, it is possible to lower the soil temperature below the freezing point and preserve the frozen condition of the soil in warming permafrost regions.
- The temperature reduction of the soil due to utilizing freezing system starts from the soil surface and then continues to affect deeper soil layers accordingly. The cold liquid circulation technique used in this experiment was only capable of freezing the pile surrounding soil in close proximity to the pile in a short time period due to limited volume of the liquid used. This limited frozen area, however, was able to create an adfreeze bond between the pile and the surrounding soil up to an embedment depth of 500 mm. The partial pile-soil adfreeze bond created by freezing liquid has significantly improved the load transfer of the pile.
- The actual shaft frictional resistance of the model pile was found to be 258 N in unfrozen condition. This frictional resistance was observed to engage at low displacements ranging from 3 mm to 3.5 mm corresponding to 6-7% the pile diameter. The pile shaft capacity was found to significantly increase up to about 548 N when the pile was exposed to circulation of antifreeze liquid for only 25 min. The long-term adfreeze strength for this experiment was measured to be about 8.6 kPa which is more than twice the unit shaft capacity measured in unfrozen condition. This increase in load transfer of the pile was achieved when only a portion of the pile profile was frozen. The pull-out bearing capacity of piles

in full frozen pile-soil system could be 4 to 6 times the shaft capacity in thawed soils depending on the exposure temperature.

- The load-displacement curves for both the pile load tests also show that loading behaviour was mostly governed by elastic response of the pile-soil interface under frozen condition. The elastic feature of frozen soil which is created by the use of refrigeration system caused the pile to go back to its original place when pile was unloaded. However, the 5 mm displacement was maintained for unfrozen condition test.
- The initial frozen area in the pile surrounding soil significantly widened from 30 mm up to 100 mm when the duration of cooling application was increased from 25 min to longer periods of one to five hours. A similar behaviour was observed when the temperature of the cooling liquid was dropped from -8˚C to colder thermal loads of -15˚C and -20˚C. Both the extend of the frozen surrounding soil as well as the soil temperature within the frozen region was improved by changing the thermal load and its duration.
- The radial distance of frozen area in the pile surrounding soil increased from the initial distance of 30 mm up to 70 mm from the pile's surface when the pile diameter was increased from 2'inch to 5-inch. The load carrying capacity of the piles also increased from 617 N for the 2-inch diameter pile to 1354 N for the pile with 5-inch diameter during the cooling application. This was attributed to higher surface area of the pile and, hence, development of stronger adfreeze bond between the pile and surrounded frozen soil.

5.2. Recommendations for Future Work

The results presented in this thesis indicate the possibility of preserving thawed sandy soils in permafrost regions by use of freezing liquid as cooling system. Further work is needed in the form

of full-scale field pile load tests and also temperature distribution tests in order to corroborate the results in realistic field conditions. A comprehensive field instrumentation and testing programs are recommended for studying the effect of freezing system on pile-soil adfreeze strength as well as temperature change in surrounding soil profile. Furthermore, different steel pipe pile diameters equipped with this new refrigeration system is needed to be investigated experimentally in lab as well as real field for different soil materials such as clay. It is being recommended to employ a high-capacity freezer with more than 100 gallons of antifreeze as freezing source in order to be able to extend the experimental process time period, hence, artificially fully frozen soil around the pile would be achieved.

The numerical simulation in this study was conducted based on frictional properties of soil in unfrozen and frozen condition, and the soil interaction with steel piles. However, proper modeling should include simulation of adhesion in the pile-soil interface. Investigating the creep behaviour of these steel piles after use of refrigeration system would also be beneficial. Creep characteristics of the steel piles in unfrozen soil and artificially frozen soils can be compared in future investigations.

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