AXIAL CAPACITY OF PILES FOUNDED IN PERMAFORST: A CASE STUDY ON THE APPLICABILITY OF MODERN PILE DESIGN IN REMOTE MONGOLIA

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ABSTRACT

A review of the most current and accepted practices in design of piles in permafrost is examined. Key input parameters necessary for the design of piles in permafrost are described with an emphasis on the characteristics of the permafrost itself. Regionally available resources and databases on permafrost characteristics are discussed along with the need for continued research, data collection, and data assimilation. A case study describing a bridge crossing over a large river at a remote permafrost site in Mongolia is presented. A unique aspect of the project involves the application of modern pile design procedures at this remote northern region in which primitive construction techniques are used to install timber piles during the winter though a frozen active layer.

INTRODUCTION

The objective of this project was to examine current practices in design and installation of piles in permafrost. The practice of designing and installing structural foundations in permafrost is not a new concept; however, the in-depth knowledge, understanding, and available data required for design in permafrost has only recently sprouted to the forefront of the engineering profession. Speculation for the recent increase in resources available to the engineering community includes the need to provide safer and more economical structures to support cold region activities such as developing roads, commercial structures, residential buildings, and military and mining support structures. Additionally, the global warming phenomenon has led to significant increases in research and monitoring of permafrost around the world. Scientists believe studying the changes in temperature, thickness, and land area covered by permafrost can provide evidence of the earth's changing climate. This research has lead to creation of promising resources in obtaining permafrost characteristics.

The effect and cause of frost heave is also examined in this paper. Frost heave is an important phenomena in designing any structure in cold regions. While an exact dollar amount is unavailable, frost heave is responsible for causing an extremely large amount of damage to structures resulting in reduction of structural safety, serviceability, or design life. Neglecting frost heave when designing in permafrost or cold regions can be a dangerous and costly oversight.

BACKGROUND

Mongolia's Darhad Region is located in the northern Hovsgol Province bordering Russia. The most prominent establishment in the Darhad Valley is the City of Rinchinlhumbe, which reportedly has a population between 10,000 and 30,000 people. The Darhad Valley is a wide, flat valley primarily composed of silty sandy soils deposited by the surrounding Khoridal Saridag Mountains, Bayan Mountains, and Hordil Mountains. Rivers in the Darhad Region are typically slow and meandering as a result of the wide and flat valley bottom. During peak runoff in July, the usually low volume and flow rivers transform into high velocity flows. This is a result of the melting snow from the high snow-covered mountains that contribute to the Darhad Valley watershed.

The indigenous people of the Darhad Region are often required to traverse these rivers while traveling between villages, grazing pastures, and food sources. To accommodate travel, several bridges have been installed by the local government. The most common bridge found in the region is composed of a wooden bridge deck supported on wooden piles. The design and installation process of bridges is not refined or standardized in Mongolia. Bridge failures occur all too often, and are likely a result of the combined effects of river scour, hydraulic loads, and frost heave. Unfortunately, the national government only designs, constructs, and maintains the roads in the region, while the local government, with little available funding, is responsible for design, construction, and maintenance of bridges. In other words, funding for bridge design and maintenance is not available to ensure the bridges in the region are capable of safely transporting pedestrians and cars across the many rivers throughout the year.

The everyday life of the Mongolian Nomad requires crossing the meandering valley rivers, regardless of whether a bridge exists. Alternate methods of traversing the region's many rivers include larger vehicles towing smaller vehicles across shallow sections of the river, makeshift ferries, and driving across the ice when the rivers are frozen. All these methods carry a risk to human safety. It is imperative to the culture and economy of the Darhad Region that safe travel across its many rivers is attainable.

LITERATURE REVIEW

Permafrost

Permafrost is any soil that has a continuous temperature below zero degrees Celsius. For purposes of this paper, the term "frozen" shall be synonymous with temperatures below zero degrees Celsius. Typically, for an area to be classified as permafrost, the ground must maintain a temperature of less than or equal to zero degrees Celsius for two consecutive winters and the corresponding summer (Brown and Kupsch 1974). The classification of permafrost is strictly based on temperature and therefore, as is often times confused, does not require the presence of ice or water. Figure 1 shows the typical section of a region containing permafrost. Permafrost is generally divided into two different types: continuous and discontinuous. As can be seen in Figure 1, continuous permafrost has no intermittent areas of unfrozen ground with depth, whereas discontinuous permafrost does contain intermittent areas of unfrozen ground. The general trend is for continuous permafrost to be found in the extremely cold northern climates and discontinuous permafrost to be found farther south near the interface of regions that do not contain permafrost.

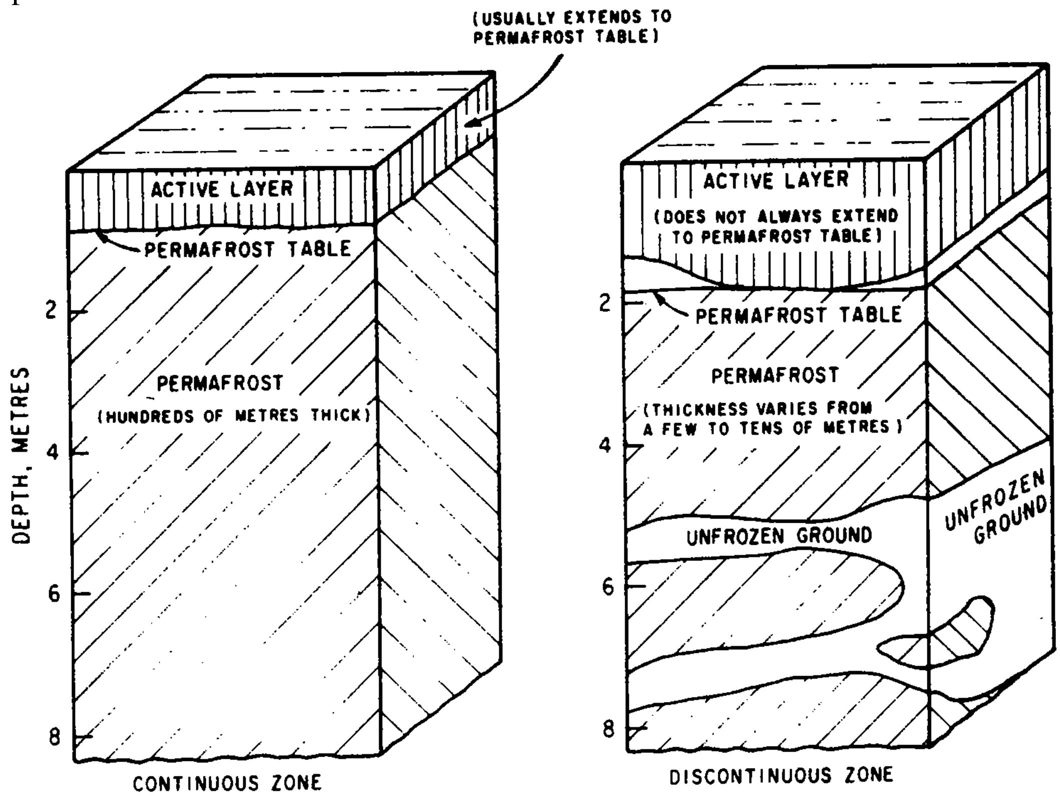


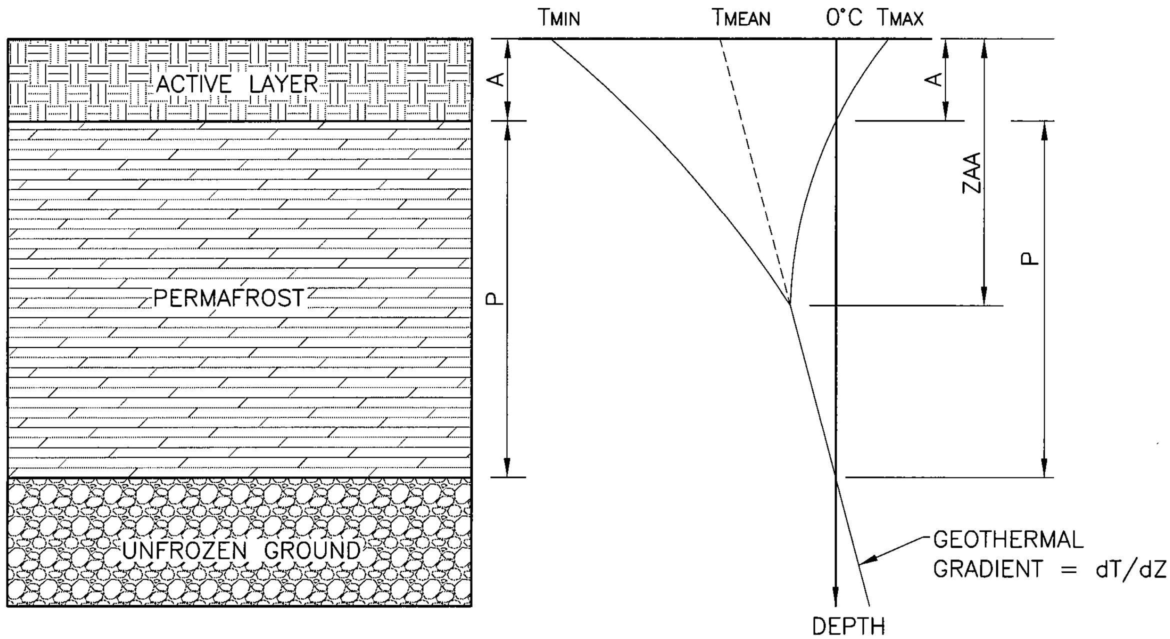
Figure 1. Typical section of permafrost (Brown et al. 1981).

Active Layer

The active layer is the layer of ground located above permafrost and is identified in Figure 1. The active layer does not exhibit a temperature of less than zero degrees year round. The active layer varies in thickness and typically follows a pattern of freezing during the winter months and thawing during the summer months. From an engineering point of view, the effects of the active layer are the primary source of structural problems and failures in permafrost regions. The freeze and thaw cycle of the active layer promotes conditions favorable for frost heave and thaw weakening. The thickness of the active layer varies depending on climatic influences and can be as little as a few centimeters to as many as several meters thick.

Thickness Determinations

The thickness of the active layer and the depth of permafrost are dependent on several factors that are site specific. The primary factors that govern the active layer and the permafrost thicknesses are the geothermal gradient and the ground surface temperatures. Figure 2 shows a plot of the maximum and minimum temperatures with depth at a generic site. This plot has been termed a trumpet curve due to its characteristic trumpet shape. A trumpet curve can be generated for any site based on in-situ ground temperatures at various depths.



Abbreviations

A	Depth of active layer
P	Depth of permafrost
ZAA	Depth of zero annual amplitude

Figure 2. Generic trumpet curve.

The trumpet curve is created by plotting the maximum and minimum ground temperatures over time on the horizontal axis for several depths. The depth of the active layer can be easily deciphered by determining the depth at which the ground temperature is never greater than zero degrees Celsius. The vertical axis or depth axis has been conveniently placed at a value of zero degrees Celsius to make this determination simple.

The depth of permafrost in certain regions has been recorded to depths on the scale of hundreds of meters. Permafrost can extend to great depths; however, subsurface temperatures ultimately will increase above zero degrees Celsius as a result of the Earth's heated core. The trumpet curve in Figure 2 is also an excellent way of graphically showing the thickness of permafrost. The top of the permafrost layer begins at a temperature where the ground is continuously below zero degrees Celsius. The bottom of the permafrost layer will end at the depth where the ground no longer maintains a continuous temperature below zero degrees Celsius. These two points and the corresponding permafrost thickness are easily determined by examining the trumpet curve.

The lower limit of permafrost is dependent on the heat generated by the Earth's core. This heat can be expressed using a geothermal gradient. The geothermal gradient is defined as the change in subsurface temperatures with depth as a result of heat generated by the Earth's core. The surface temperature and geothermal gradient essentially compete with each other until equilibrium is achieved. The thickness parameters of permafrost are defined by this equilibrium.

Depth of Zero Annual Amplitude

The depth of zero annual amplitude is the depth at which there is no subsurface temperature change throughout the seasons, as shown in Figure 2. The depth on the trumpet curve where the maximum and minimum subsurface temperatures converge defines the depth of zero annual amplitude.

Frost Action

Frost action is the process of soil freezing and thawing causing the formation of ice lenses during cold months and the corresponding thawing of ice lenses in warm months. During cold months, ice lenses will typically form in soil creating an upward force on foundations that may cause an upward heave of the ground and foundation. Ice lenses form perpendicular to the temperature front, which is typically parallel to the ground surface. The growth of ice lenses caused by migrating water to the freezing front, and corresponding expansion of freezing water creates an upward force on the soils above the lenses. A common misunderstanding is that the volume increase due to the phase change of water, from liquid to solid, is the primary cause of frost heave.

The melting of ice lenses created during the cold months creates additional design problems for structures and their foundations. Melting creates voids in the subsurface and increases in soil moisture content, causing a decrease in soil bearing capacity. The sudden increase in soil moisture is especially problematic in permafrost. As the active layer thaws and water is allowed to flow freely, a condition of oversaturated soil occurs in the active layer due to the impermeable underlying permafrost. This is why permafrost regions are commonly dotted with small ponds of standing water in the summer months that can make travel nearly impossible.

Frost heave can only occur if three factors are present: a source of water, freezing temperatures, and frost-susceptible soils.

Frost-susceptible Soils

Frost-susceptible soils are those that exhibit capillarity and high permeability. The most frost-susceptible soils are fined grained silty soils, which have both high capillarity and high permeability. Sands and gravels have low frost susceptibility as a result of the absence of capillarity. Clay is a fine grained soil with tremendous capillarity that exhibits low frost-susceptibility due to low permeability.

DESIGN OF PILES IN PERMAFROST

Adfreeze Strength

The basic strength parameter used in design of piles founded in permafrost is the adfreeze bond developed between the pile surface and frozen soil. A loose comparison can be made between the strength of adfreeze bonds in frozen soils and shear strength in unfrozen soils. Adfreeze strengths in frozen soils are defined using the same equation as used for unfrozen soils:

$\tau_a = c_a + p_n \bullet \tan(\varnothing_a)$ (1)

where: τ_a = adfreeze strength of frozen soil

c_a = cohesion at the soil pile/interface

$\varnothing_a$ = friction angle at the soil pile/interface

p_n = normal stress acting on the soil/pile interface

Similar to unfrozen soils, the adfreeze strength of frozen soils, τ_a, is based on shear parameters contingent on soil type and normal stresses acting at the soil/pile interface. Adfreeze strength in frozen soils is also dependent on several site specific parameters. These parameters primarily include but are not limited to the ice content of the soil, pile roughness, pile material, pile shape, and ground temperature.

The most widely used methods for pile design in permafrost do not include direct theoretical calculations, instead, most design procedures use quantitative empirical data based on years of pile testing and data acquisition. As a result of the recent increases in research and testing in cold regions, a significant data set of pile design graphs and tables are available to design engineers. The data set provides design professionals with strength parameters based on pile type, pile material, and soil type. Site specific parameters should be developed using specialized lab and in-situ tests for large projects or projects where risk to human health and safety is great.

There are generally two ways of determining adfreeze strength in frozen soils. The first and most straight forward is to use published tables or graphs that directly provide adfreeze strengths based on soil temperature, pile type, and soil type. Examples of these tables and graphs have been presented by Weaver and Morgenstern (1981) and Tsytovich (1975).

A second way of determining adfreeze bond strengths was presented by Weaver and Morgenstern (1981), who proposed that the adfreeze strength of frozen soils is related to the long-term shear strength, as shown in Equation 2.

$\tau_a = m \bullet \tau_{lt}$ (2)

where: τ_a = adfreeze strength of a frozen soil

m = roughness/pile surface parameter

τ_{lt} = long-term shear strength of frozen soil

Weaver and Morgenstern (1981) simplify the Mohr-Coulomb strength equation for soils (Equation 3) in their analysis by assuming the normal stress, σ, is negligible.

$\tau_{lt} = c_{lt} + \sigma \bullet \tan(\varnothing_{lt})$ (3)

where: c_{lt} = long-term cohesion

$\varnothing_{lt}$ = long-term friction

If normal stress is assumed to be small, Equation 2 and Equation 3 can be combined into the following equation:

$\tau_a = m \bullet c_{lt}$ (4)

Equation 4 is commonly used to determine the adfreeze strength of frozen soils if the long-term cohesion (c_{lt}) is measured or available.

Published graphs of long-term cohesion for various soil types at various temperatures are available in the literature. Figure 3 shows a long-term cohesion graph based on information obtained from Vialov (1959) and Voitkovskii (1960).

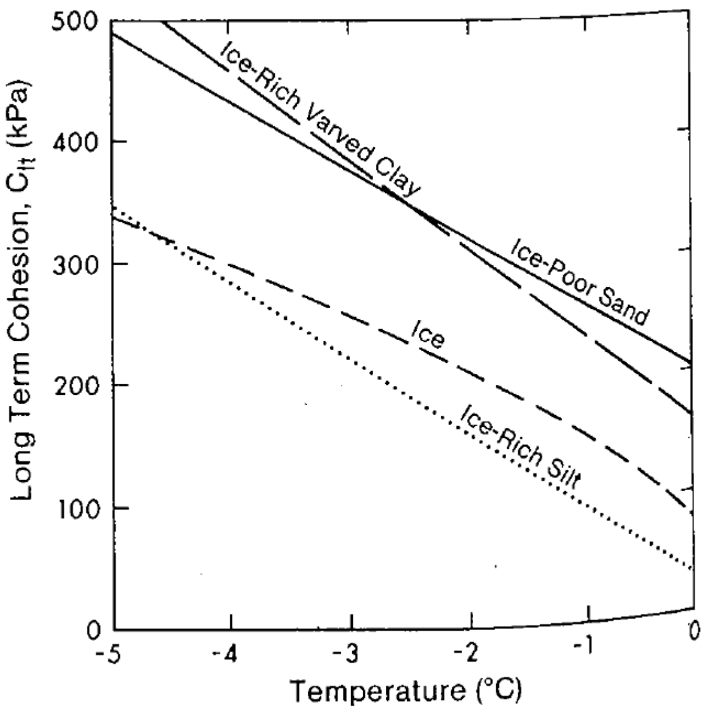


Figure 3. Long-term cohesion of frozen soils (Weaver and Morgenstern, 1981).

Roughness/surface parameters used in Equation 4 (m) for common pile materials are summarized in Table 1.

Table 1. m Coefficient (Weaver and Morgenstern, 1981)

Pile Type	m
Steel	0.6
Concrete	0.6
Timber (uncreosoted)	0.7
Corrugated steel pipe	1.0

The adfreeze strength is the critical parameter in analyzing piles in permafrost. An in-depth knowledge of the site's permafrost conditions, such as temperatures, thicknesses, soil types, and ice content is required in order to determine the adfreeze strength. Once adfreeze strength for a certain site is determined, the allowable bearing capacity, allowable pile capacity based on settlement, and the effects of frost heave can be examined for the structure and proposed piles.

Bearing Capacity of Piles in Permafrost

The bearing capacity of a pile in permafrost is determined by summing the forces developed at the tip of the pile and the accumulation of adfreeze forces working along the length of the pile. Figure 4(a) and 4(b) show the forces on a pile founded in permafrost during the summer and winter months respectively. In Figure 4, p is the applied load, p_p is the force due to the adfreeze bond, p_b is the force due to tip resistance, p_n is a force created by the consolidation of the active layer, and p_a is the force due to frost heave.

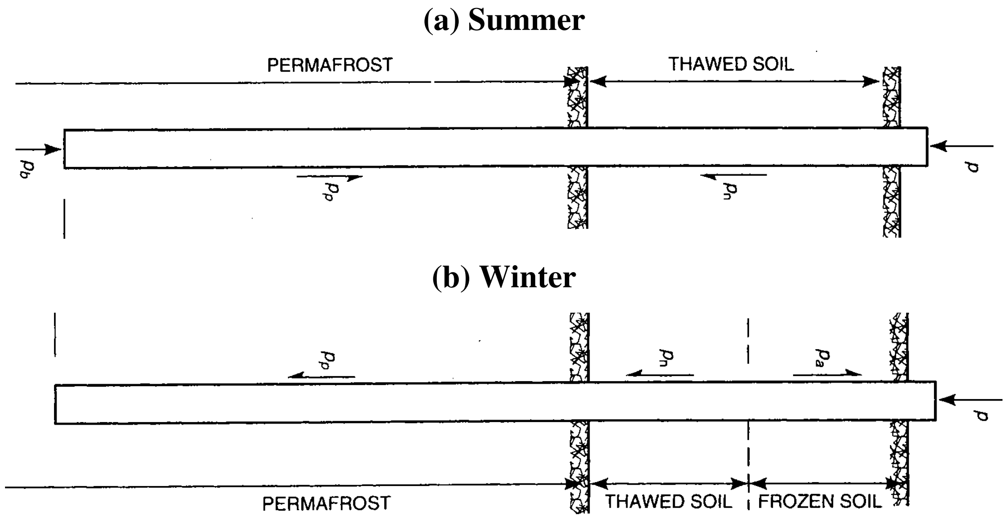


Figure 4. Pile forces (Andersland and Ladanyi, 2004).

For piles in permafrost tip resistance (p_b) or end bearing is neglected unless the pile is firmly situated on bedrock. The reason for neglecting end bearing forces is due to the large pile settlements required to fully develop end bearing and the small magnitude of end bearing resistance compared to adfreeze resistance (p_p).

Piles in permafrost develop their primary resistance to structural loads and frost heave from the adfreeze bonds between the pile and surrounding frozen soil. Adfreeze resistance is calculated by multiplying the adfreeze strength by the effective embedment depth and the pile circumference.

$$P_{ult} = \tau_a \bullet s \bullet D_{eff} \qquad (5)$$

where: τ_a = adfreeze strength of frozen soil
s = Circumference of the pile
D_{eff} = Effective embedment depth

The effective embedment depth is defined as the length of pile embedded in soil with a temperature continuously below minus one degree Celsius. Typically, an average temperature over the effective embedment depth is used in determining the adfreeze strength, even though temperature and resulting adfreeze strengths actually vary with depth. The basic calculations include finding the necessary effective embedment depth required to resist axial loads or finding the maximum allowable axial loads based on a given effective embedment depth. A safety factor of two to three is recommended in design.

Creep-Settlement of Piles Founded in Permafrost

Settlement and creep of a pile founded in permafrost can exceed design tolerances creating failure at capacities less than the bearing capacity of the permafrost. One of the primary reasons for this is the ability for ice to flow or creep. The general theory of pile settlement does not change between piles in frozen and unfrozen ground. The components of settlement at the top of a pile are the summation of the compressibility of the pile itself, the immediate elastic settlement of the subsurface material, and creep. Creep is the long-term effect of constant shear stress on the subsurface material in contact with the pile. Testing and observation of pile behavior in permafrost has brought about the generally accepted practice of ignoring the settlement caused by compressibility of the pile itself and the immediate elastic settlement of subsurface soils. These components of the overall pile settlement are typically negligible compared to creep settlement.

Constant steady-state creep of piles founded in ice or ice-soil combinations have been studied in depth by many researchers including Odqvist (1966), Weaver (1979), Nixon and McRoberts (1976), and Vialov (1962). Creep settlement in ice or ice-rich soils can be expressed using the following equation developed by Odqvist (1966):

$$\varepsilon = B \bullet \sigma_e^{\,n} \qquad (6)$$

where: ε = effective shear strain rate

σ_e = shear stress

B and n = temperature dependant soil constants

Values of B and n are defined in Table 2:

Table 2. B and n Soil Constants (Weaver and Morgenstern, 1981)

Ground Temperature	B	n
-1°C	4.5×10^{-8}	3.0
-2°C	2.0×10^{-8}	3.0
-5°C	1.0×10^{-8}	3.0
-10°C	5.6×10^{-9}	3.0

Creep settlement in ice-poor soils has been researched and theoretically expressed using an equation developed by Ladanyi (1972) based on multi-axial stresses on a pile. Due to a lack of experimental data, Ladanyi's equation is only theoretical and has not been validated (Weaver and Morgenstern, 1981).

A practical and design-friendly equation has been developed (Nixon and McRoberts 1976) to approximate long-term creep settlement of piles founded in permafrost. This equation utilizes adfreeze bond strengths along with the soil constant parameters, B and n, developed by Odqvist (1966). This simplified pile creep settlement equation is as follows:

$$u_a/a = 3^{(n+1)/2} \bullet B \bullet \tau^n/(n-1) \qquad (7)$$

where: u_a = pile steady-state displacement rate (m/yr)

a = pile radius (m)

τ = average applied adfreeze load (kPa)

B and n = see Table 2

Equation 7 is based on the following assumptions outlined by Weaver and Morgenstern (1981):

- Soil above the tip of the pile deforms as a result of load transferred from the pile shaft
- Soil below the tip of the pile deforms as a result of pile end load
- The analysis is insensitive to changes in normal stress on the lateral surface of the pile (reasonable for ice-rich soils but not for ice-poor soils)
- Slip does not occur at the pile-soil interface
- Gravity forces are negligible
- Soil is homogenous and isotropic and soil properties are constant with depth

The above model and assumptions appear to limit the applicability of this equation; however, Equation 7 appears to provide solutions within the acceptable tolerances of more rigorous numerical solutions and it is applicable in both ice-rich and ice-poor soils.

Similar to bearing capacities of piles in permafrost, safety factors with respect to settlements of two to three are recommended.

Frost Heave Forces

Pile design includes determining the minimum allowable load based on bearing capacity and settlement. An additional component to designing piles in permafrost involves checking the design with expected frost heaving forces. Frost heave forces may be large enough to control the design of pile foundations. Limited literature is available on estimating frost heave forces. The extensive research performed during the writing of this paper uncovered one equation used to estimate frost heaving forces: Dalmatov's equation presented by Tystovich (1959). Dalmatov's equation is as follows:

$$F = s \bullet h_a \bullet (c - bT_m) \quad (8)$$

where: F = upward force due to frost heave (kgf)

s = perimeter of foundation in contact with frozen soil (cm)

h_a = thickness of frozen zone or active layer thickness (cm)

c and b = parameters determined experimentally

T_m = minimum soil temperature in the frozen zone (°C)

Values of c and b have been incorrectly referenced by Andersland and Ladanyi (2004) as 40 to 70 kilopascals for c and 10 to 19 kilopascals for b. Values for c and b based on Tystovich's (1975) original research can be seen in Table 3.

Table 3. c and b Parameters (Tsytovich, 1975)

Reported by	Soil Type	c (kgf/cm^2)	b (kgf/cm^2)
B. I. Dalmatov	morane loam	0.5	0.12
B. I. Dalmatov	silt loam	0.4	0.1
B. I. Dalmatov	heavy silt loam with sand	0.4	0.16
Yu. D. Dubnov	silty loam	0.356	0.147
V.I. Puskov	sandy loam	0.7	0.22
V.I. Puskov	silty loam	0.5	0.18

These values are site specific and should be used as an initial estimate. If frost heave forces cannot be avoided through prevention, then testing should be conducted for specific soils at a specific site using specific pile parameters to determine appropriate frost heave forces for design.

The active layer of permafrost can be several meters thick. The amount of frost heave potential created in a layer this thick can be extremely large. Frost heave forces can be counteracted by increasing the depth of pile foundations or by increasing the applied load. Increasing the applied load to counteract large frost heave forces may create conditions where the loads are larger than the bearing capacity or allowable settlement capacity of the pile. In addition, large frost heave forces could require that piles be driven to uneconomical and unrealistic depths in order to create adequate resisting forces. Alternatives to these methods are discussed in the following section.

Preventing Frost Heave

The simplest and most economical way to avoid frost heave is to isolate the foundation or pile from the frost-susceptible soils. The typical approach to achieve isolation is to use a non-frost-susceptible soil as backfill around the pile in the active layer, which corresponds to the area where frost heave forces are developed. Backfill composed of a soil-oil-wax combination have been proven effective in preventing frost heave. Wood piles soaked in creosote have also been shown to have lower frost heave forces.

More advanced and active methods of preventing frost heave include keeping the soil around the pile permanently frozen or permanently thawed by thermally treating the soil though heating or refrigerant tubes. Frost heave and soil weakening occur during the freezing or thawing processes, respectively. Thermally treating soil around the pile will prevent either scenario from occurring, eliminating the potential for heave or weakened subsurface soils.

Other ways of reducing frost heave forces on piles include using corrugated shaped piles, installing spikes in wooden piles, and installing piles with flared ends in an attempt to create larger frictional and passive soil resistant forces to counteract frost heave uplift.

RESEARCH AND DATA ANALYSIS

Prior to analyzing the potential causes of the bridge foundation failures in Mongolia's Darhad Region, research and data acquisition was necessary. The available funding, project submittal timeline, and location of the bridge sites unfortunately made it unrealistic for a site visit. For most geotechnical projects, a site reconnaissance visit and subsurface investigation is crucial. "One hole is worth a thousand opinions" a wise engineer once said. Fortunately for this project, there are available resources for local surface, subsurface, and permafrost data acquisition that, in light of the preliminary nature of this case study, will suffice for feasibility studies. At the forefront of these available resources are organizations devoted to studying and recording long-term permafrost characteristics. One organization's database proved to be exceptionally valuable to this project.

The Global Terrestrial Network for Permafrost (GTN-P) was initiated by the International Permafrost Association (IPA) as a means of recording and monitoring permafrost data in an attempt to compare and possibly forecast climate changes. To date, GTN-P has accumulated permafrost data from 15 counties and Antarctica. Permafrost characteristics and subsurface data is collected from existing boreholes by local scientists, engineers, and academics and submitted to GTN-P for inclusion in its database. Included in the GTN-P database is a vast amount of information on permafrost in Mongolia. Information obtained from the GTN-P for 16 boreholes in the vicinity of the Darhad Region of Mongolia was used to estimate the likely permafrost conditions at the bridge sites. It should be noted that this is an approximation based on the best information available. It is understood that subsurface conditions may vary significantly from site to site, even when they are in close proximity to boreholes. Additionally, the effects of bodies of water on ground temperatures have been studied in depth and are known to influence permafrost in their proximity. This indicates permafrost characteristics in and around bodies of water may or may not be similar to the regional trends provided by the GTN-P database.

Following is a summary of the likely subsurface conditions at the Mongolian bridge site based on information obtained from the GTN-P database.

- Active Layer Depth: 3 meters
- Depth of Permafrost: 80 meters
- Average Temperature of Permafrost: -2.5 °C

- Depth of Zero Annual Amplitude: 10.5 meters

Additional data necessary for this project included pile material and construction techniques. This data was primarily provided by Professor Clifford Montagne through personal interview and observation of pictures from his work in Mongolia.

Pile materials used for construction of bridge pile foundations include 0.25 to 0.30 meter diameter wood piles. At the time of driving, the piles conditions varied from as-harvested to roughly debarked. The driving procedures used by the local nomadic villagers are very primitive. Piles are driven using an A-frame bracing over which a rope and anvil are placed. Figure 5 shows the typical pile driving equipment and anvil used in the Darhad Region of Mongolia.

Figure 5. Pile driving A-frame and anvil, photograph courtesy of C. Montagne.

The rope is raised and released through the repeated process of tying the rope to a vehicle, driving a certain distance, and cutting the rope. This process is typically repeated until the primitive driving methods can no longer progress the pile downward. Pile driving is usually conducted during the colder months. The reason for driving piles during the colder months is for ease of vehicular travel, which is necessary for pile installation at the bridge sites. During the summer months when the active layer is thawed, conditions are wet and muddy, creating difficult, if not impossible travel across the permafrost.

DISCUSSION OF FINDINGS

Based on analyses conducted using best estimates of the site conditions, it appears that the material and primitive installation techniques being used by the native nomadic villagers of Mongolia to drive piles into permafrost are inadequate. Three primary reasons for probable bridge failure include shallow pile depths, frost heave, and river scour.

The current pile driving techniques have been found to be insufficient at progressing piles to sufficient depths. The axial strength of a pile to resist forces from live loads, dead loads, and frost heave is a function of the adfreeze bond between the pile and subsurface materials. Cyclical freezing and thawing of the active layer results in discontinuous adfreeze strength within this layer. Axial pile strength is dependent on the permafrost characteristics and the depth the pile is driven into the permafrost, also known as the effective embedment depth. The pile driving techniques utilized by the Mongolian bridge builders may at best, allow the piles to be driven through the active layer to the permafrost interface. The situation can be modeled much like a pile resting on bedrock. The hardened permafrost layer will provide some bearing capacity in the form of tip resistance; however, excessive settlement will likely occur in order for this capacity to fully develop. Because the piles are not driven into the permafrost, the most current and generally accepted design equations outlined in the literature review do not apply to the Mongolian bridge piles.

Allowable capacity is essential to pile design, but with the expected loads being relatively small, a more critical factor is uplift forces. The native's inability to drive piles into the permafrost and develop adfreeze bonds between the pile and subsurface results in a scenario with zero resistance to uplifting frost heave forces. The estimated frost heave forces for an active layer three meters in depth are in excess of all resisting forces. Preliminary calculations indicate an effective embedment depth of 4.1 meters is necessary to resist frost heave forces with a factor of safety equal to one. This would require the native's to drive the pile a total of 7.1 meters of which 4.1 meters would be through solid frozen ground. These depths are impossible given the pile driving techniques being used.

Pictures of the failed bridges show several of the foundation piles, within the river's extent, washed out as shown in Figures 6.

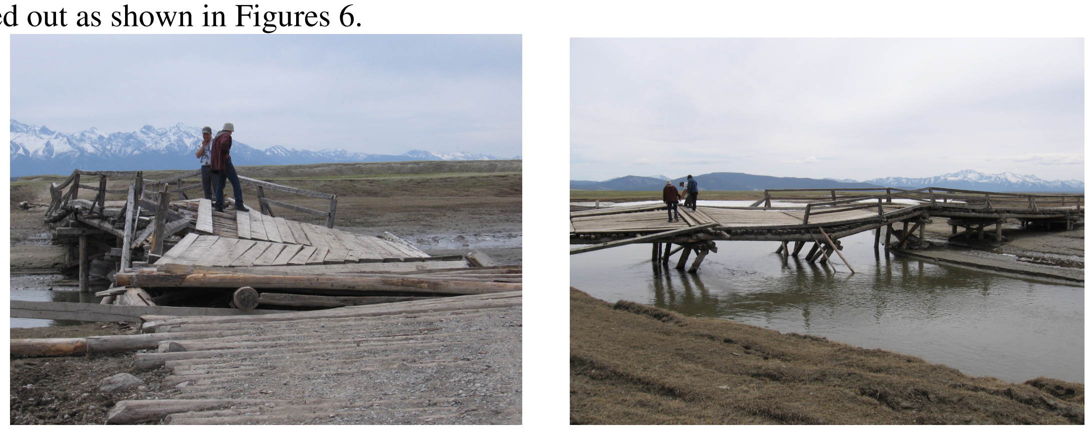


Figure 6. Examples of bridge failure, photograph courtesy of C. Montagne.

Washout of the piles likely occurred as a result of large runoff flows scouring the piles that were in a weakened state due to frost heave. This combination of axial uplifting forces and then lateral hydraulic forces acting on the pile is larger than the resisting forces created using the primitive installation techniques.

CONCLUSIONS AND RECOMMENDATIONS

As a result of the primitive pile driving techniques employed by the local nomadic villagers in remote Mongolia, bridge failures will continue to occur if the Mongolians continue using their primitive techniques for installing bridge piles. Until more advanced equipment is economically and locally available to the natives of the Darhad Region, bridge failures will continue to occur as a result of frost heave and scour. Given the economic conditions of the local communities in the Darhad Region and the unlikelihood of mobilizing advanced equipment into the area, efforts should be focused toward alternative solutions, prevention techniques, and ways of extending the useful life of bridge piles considering the techniques of their installation.

Installing non-frost susceptible backfill around bridge piles would be the first recommendation to increase the design life of remote Mongolian bridges. Installing non-frost-susceptible backfill would minimize the effects of frost heave by isolating the bridge piles from the native frost susceptible silts. Backfilling around the piles with non-frost-susceptible soils will also lessen the affects of frost heave by preventing ice lenses in the highly saturated active layer from forming in the vicinity of the pile. This approach to limiting frost heave effects should be attainable at a low cost. Quality non-frost-susceptible backfill should be abundant in the region, considering the close proximity of mountainous terrain. To make it possible for backfill placement around the pile, manual excavation will be necessary in order to provide space for backfill. If excavation is impractical due to frozen ground conditions, creosote could be applied to the piles prior to driving.

It is apparent from examining Mongolian bridge failures that hydraulic forces during the peak runoff are large enough to scour and washout pile foundations. Hydraulic forces could be lessened and the useful life of bridges increased by armoring the piles located within the river limits. An inexpensive method of producing the necessary armoring could be achieved using wire meshing and boulders. Armoring walls could be constructed by the local bridge builders using techniques similar to those used for gabion basket retaining walls on highway abutments in the United States. If wire mesh is unattainable, the bridge piles should be protected by surrounding them with the largest boulders capable of being moved into place. This would provide some protection from river scour in addition to lessening the lateral hydraulic forces acting on the pile.

In order to develop the adfreeze bond strength necessary for creating the resistance to outside forces, piles must be driven into the permafrost. The installation techniques used by the Mongolian bridge builders cannot progress piles into the permafrost. Possible advances in the installation process such as metal toe caps on the piles or higher energy driving methods should be considered. Adjustments to the existing pile installation techniques that could increase the driving energy include heavier anvils and/or larger drop heights. Although these adjustments may seem insignificant, any increase in pile depth will increase the useful life of the bridges.

Continual experimentation with alternative bridge construction methods is encouraged. Currently, the local Mongolian bridge authorities are working on experimental bridge construction techniques utilizing pontoons. These experimental bridges may yield satisfactory results. Creative thinking and alternative bridge design could greatly enhance the ease and safety of crossing Mongolian rivers. However, even with alternative bridge designs, a sturdy support foundation will be required.

An in-depth site investigation at the bridge site by an engineer experienced with designing in cold regions is highly recommended. Several factors are involved with the design and installation of piles in permafrost that second-hand information and pictures may not reveal. Possible funding strategies through charitable organizations should be considered. If funding can be acquired, the most important and frequently used bridges should be constructed using drilling equipment capable of penetrating into the permafrost.

The current construction procedures for installing piles in permafrost include utilizing drilling equipment to core into the permafrost layer. After a pilot hole has been drilled to the necessary effective embedment depth, the pile is inserted into the hole and backfilled with a slurry composed of water and soil. As the backfill freezes, it becomes a part of the surrounding permafrost. This freezing of the backfill slurry creates the essential adfreeze bonds necessary to resists axial loads.

REFERENCES

Andersland, O. B., B. Ladanyi, 2004. *Frozen Ground Engineering*, 2nd Ed. New Jersey: John Wiley & Sons, Inc.

Bioregions International 2008, 2008. http://www.bioregions.org/index.php?view=home.

Brown, R. J. E., G. H. Johnston, J. R. Mackay, N. R. Morgenstern, and W. W. Shilts. 1981. Permafrost distribution and terrain characteristics. Chap. 2 in Permafrost Engineering Design and Construction, ed. G. H. Johnston. New York: Wiley, pp. 31-72.

Brown, R. J. E., and W. O. Kupsch. 1974. Permafrost Terminology. Natl. Res. Counc. Can., Tech. Memo. 111.

Chamberlain, E. J., P. N. Gaskin, D. Esch, R. L. Berg. 1984. Survey of methods for classifying frost susceptibility. Tech. Counc. on Cold Regions Eng. Mono. New York: ASCE.

Crory, F. E., R. M. Isaacs, E. Penner, F. J. Sanger, J. F. Shook, 1984. Designing for frost heave conditions. Tech. Counc. on Cold Regions Eng. Mono. New York: ASCE.

Global Terrestrial Network for Permafrost. 2007-2008. http://www.gtnp.org/index_e.html.

Ladanyi, B. 1972. An engineering theory of creep of frozen soils. Can. Geotech. J. 9(1): 63-80.

Montagne, C. Personal Interview. 12 December 2007.

Nixon, J. F., and E. C. McRoberts. 1976. A design approach for file foundations in permafrost. Can. Geotech. J. 13(1):40-57.

Odqvist, F. K. G. 1966. *Mathematical Theory of Creep and Creep Rupture*. Oxford; Mathematical Monographs. Oxford: Clarendon Press.

Tsytovich, N. A. 1959. *Principles of Geocryology*, part 2, chap. 3. Nat. Res. Counc. Can Tech. Transl. TT 1239, pp. 28-79.

Tsytovich, N. A. 1975. *The Mechanics of Frozen Ground* (trans.), eds. G. K. Sinzow and G. P. Tschebotarioff. New York: Scripta/McGraw Hill.

U.S. Army Corps of Engineers, 1965. Soil and Geology – *Pavement Design for Frost Conditions*. Department of the Army Technical Manual TM 5-818-2.

Vialov, S. S. 1959. *Rheological Properties and Bearing Capacity of Frozen Soils*. Moscow: USSR Academy of Science. Translation, U.S. Army Cold Regions Research and Engineering Laboratory, SIPRE TL 74, 1965.

Vialov, S. S. 1962. *The Strength and Creep of Frozen Soils and Calculations for Ice-Soil Retaining Structures*. U.S. Army Cold Regions Research and Engineering Laboratory Translation SIPRE TL 76, 1965.

Voitkovskii, K. F. 1960. The mechanical properties of ice. U.S. Air Force Cambridge Research Laboratories. Bedford, Massachusetts, AFCRL 62-838, AMS-T-R-391.

Weaver, J. S., and N. R. Morgenstern. 1981. Pile design in permafrost. Can. Geotech. J. 18(3): 357-70.

Appendix - Example Calculations

Given: Active Layer Depth (h_a): 3 meters
Depth of Permafrost: 80 meters
Average Temperature of Permafrost: -2.5 °C
Minimum Soil Temperature (T_m): -15 °C
Pile Diameter (2a): 0.25 meters
Pile Circumference (s): 0.785 meters
Soil Type: Sandy Loam to Silty Sand
Dead Load of Bridge: 133 KN
Live Load: 70 KN
Design Life: 20 years
Allowable Settlement: 0.0508 meters

Find: The necessary effective embedment depth for bearing capacity, allowable settlement, and frost heave.

<u>Calculate Adfreeze Strength:</u>

$\tau_a = m \bullet c_{lt}$

m = 0.7 (From Table 1 for Timber Pile)

c_{lt} = 350 kPa for Ice-Poor Sand(From Figure 3)

τ_a = 245 kPa

<u>Calculate Necessary Effective Embedment Depth for Bearing Capacity:</u>

Bridge Load = (τ_a) (D_{eff}) (s)

203 KN = (245 kPa) (D_{eff}) (0.785 meters)

D_{eff} = 1.056 meters

<u>Calculate Necessary Effective Embedment Depth for Allowable Settlement:</u>

u_a = (0.0508 meters) / (20 years) = 0.00254 m/yr

n = 3 (From Table 2)

B = 1.83×10^{-8} (Interpolated from Table 2)

$u_a/a = 3^{(n+1)/2} \bullet B \bullet \tau^n/(n-1)$

(0.00254 m/yr) / (0.125 meters) = $3^{(3+1)/2}$ (1.83×10^{-8}) $(\tau)^3$ / (3-1)

τ = 63 kPa

Bridge Load = (τ) (D_{eff}) (s)

203 KN = (63 kPa) (D_{eff}) (0.785 meters)

D_{eff} = 4.105 meters

<u>Calculate Necessary Effective Embedment Depth for Resistance to Frost Heave</u>

c = 0.7 kgf/cm^2 (From Table 3 Sandy Loam)

b = 0.22 kgf/cm^2 (From Table 3 Sandy Loam)

$F = s \bullet h_a \bullet (c - bT_m)$

F = (78.5 cm) (300 cm) (0.7 kgf/cm^2 - (0.22 kgf/cm^2) (-15 °C))

F = 94200 kgf = **924 KN**

Frost Heave (F) = (τ_a) (D_{eff}) (s) + Dead Load

924 KN = (245 kPa) (D_{eff}) (0.785 meters) + 133 KN

D_{eff} = 4.113 meters