Rationalizing the Design of Adfreeze Piles with Limit States Design

T. Ed Hoeve, P.Eng.
HoevEng Consulting Ltd., Yellowknife, NT, Canada
J. Richard Trimble, P.Eng.
Tetra Tech Canada Inc., Whitehorse, YT, Canada

ABSTRACT
Adfreeze steel pipe piles (adfreeze piles) are a commonly used foundation type in permafrost areas and are well suited to relatively light loads. The strength of the permafrost into which the piles are installed is time-dependent. Permafrost has higher resistance to short-term loads than sustained loads, all else being equal. Adfreeze piles are designed by assuming that they will settle in creep under load, over the life of the structure being supported.
The capacity of adfreeze piles is dependent on the pile configuration, the nature of the load being applied, the characteristics of the permafrost, and the allowable deformation of the structure being supported. Pile capacity can be optimized by taking into account the duration of the various types of loading be imposed on the pile. It is proposed that this be done by considering long-term and short-term loading separately.
In the context of Limit States Design, allowing for deformation is analogous to Serviceability Limit State (SLS). It is not practical to think of adfreeze pile capacity in terms of an Ultimate Limit State (ULS). Consequently, it is proposed that piles should be designed by referencing service (unfactored) loads, as designing for factored loads may be unnecessarily conservative. An approach to maintaining appropriate conservatism under SLS is proposed. Compression loading and frost-jacking resistance are considered.

RÉSUMÉ
Les pieux en acier du type qui favorise l'adhérence due au gel (pieux géli-adhérents) sont un type de fondation couramment utilisé dans les zones de pergélisol et sont bien adaptés pour les charges relativement légères. La force du pergélisol dans lequel les pieux sont installés est dépendante du temps. Le pergélisol offre une plus grande résistance aux charges à court terme qu'aux charges maintenues, tous autres facteurs étant égaux. Les pieux géli-adhérents sont conçus en supposant qu'ils vont se tasser par fluage sous la charge, au cours de la vie de la structure.
La capacité des pieux géli-adhérents dépend de la configuration du pieu, de la nature de la charge appliquée, des caractéristiques du pergélisol et de la déformation admissible de la structure étant soutenue. La capacité des pieux peut être optimisée en tenant compte de la durée des différents types de chargement qui seront imposés sur les pieux et en considérant le chargement à long terme et à court terme séparément.
Dans le contexte de la conception aux États Limites, la déformation admissible est analogue à la capacité structurale en service (ELS). Il n'est pas pratique de considérer la capacité de pieux géli-adhérents en termes d'état limite ultime (ELU). Par conséquent, il est proposé que les pieux soient conçus en utilisant les charges de service (non pondérées), car l'utilisation des charges pondérées peut être inutilement conservatrice. Une approche de conception utilisant la capacité structurale ELS est par consequent proposée. Les charges en compression et la résistance au soulèvement dû au gel sont considér.

1 INTRODUCTION

An adfreeze steel pipe pile (adfreeze pile) is constructed in permafrost by drilling an oversized hole, inserting a pile and backfilling the annulus and inside of the pile with a saturated granular backfill, which freezes following installation. The pile derives its capacity from the bond between pile and the frozen backfill and adjacent soil. For heated buildings, the permafrost is protected by extending the piles above grade and constructing the building over an air space that is maintained beneath the building, to permit natural air circulation. Alternatively, thermosyphons or thermopiles could be used.

Foundation design in permafrost has to-date not been thoroughly addressed in codes and standards. The details of design have been left to the experience and judgement of practitioners in this specialized field. This paper describes the current state of the design practice used by the authors. This practice is founded on a theoretical basis that developed between the 1970's and the 1990's, tempered with practical construction considerations.

The design methodology of adfreeze steel pipe piles is reviewed, with specific reference to characterizing the key geotechnical parameters considered in design and a rational consideration of the loads exerted on the pile.

2 ADFREEZE STEEL PIPE PILES

Adfreeze piles have been used in areas of permafrost for more than 50 years. They are typically suited to supporting loads in the range of 100 to 200 kN per pile. Earliest installations involved steam-jetting holes and installing timber piles. Problems resulting from deterioration (rotting) of inadequately treated timber piles have resulted in steel pipe piles being favoured for the last 10 to 20 years.

Ice, and therefore ice-rich soil, deforms under sustained load. With low stress, the deformation will attenuate (primary creep); under moderate stress, the rate of deformation is predicted to remain constant with time (steady-state or secondary creep); and under high stress the rate of deformation accelerates to failure (tertiary creep). The design premise for adfreeze piles is to determine the loading with relationships for secondary (steady-state) creep, such that the deformation can be maintained within a predetermined acceptable limit (Nixon and McRoberts 1976). Under the more common case of compression loading, the deformation would be settlement.

Aside from the rotting timber pile issue, the performance of adfreeze piles has generally been excellent. The authors are not aware of a single adfreeze pile having failed under compression loading. This is because several design assumptions are considered to be conservative. This is further described in a later section.

One way in which adfreeze piles have performed less reliably is with respect to frost-jacking resistance – this is related to seasonal frost heave within the active layer. Most people involved with this pile type, including designers, installers and owners, can cite examines of piles exhibiting problematic uplift. This issue has been prevalent with foundations in tension, such as guy anchors for towers, but has also occurred with some regularity with piles that are lightly loaded in compression. The design for frost-jacking resistance is discussed in a later section.

3 LIMIT STATES DESIGN

Limit states design (LSD) has been used by structural engineers since about the 1970's. Geotechnical engineers continued working with the more familiar working stress design (WSD) approach and did not seriously begin to consider LSD until the 1990's. With respect to adfreeze piles, LSD was not employed with any regularity until probably about 10 years ago.

To briefly review, in the traditional WSD, an applied load, S_a, would be compared to an ultimate resistance, R_u, which was reduced by a factor of safety, FS, to account for a number of uncertainties in load effects and determining the actual resistance (Becker 1996). This can be expressed as:

$S_a \leq R_u$ / FS (from Equation 3b, Becker 1996a) [1]

In LSD, or more specifically with load and resistance factor design (LRFD), a factored (reduced) nominal resistance is compared to the sum of the factored (increased) nominal load effects:

$\Phi R_n \geq \sum \alpha_i S_{ni}$ (Equation 11, Becker 1996a) [2]

Where: ΦR_n is the factored resistance;
Φ is the resistance factor;
R_n is the nominal resistance through engineering analysis using characteristic (unfactored) values for geotechnical parameters;
$\sum \alpha_i S_{ni}$ is the summation of the factored overall load effects;
α_i is the load factor corresponding to a particular load, S_{ni};
S_{ni} is a specified load component of the overall load effects; and
i represents the various types of loads.

Because adfreeze pile design is based on a calculation of allowable settlement, it represents a serviceability limit state (SLS) condition. A conflict arises when a structural engineer assumes a recommended pile capacity to represent an ultimate limit state (ULS) when it actually represents an SLS condition (Becker 1996b). This was a source of confusion between structural engineers and geotechnical engineers when structural engineers were working with LSD and geotechnical engineers were working with WSD, but the disconnect has arguably increased as geotechnical engineers have transitioned to employing LSD for adfreeze pile design.

In WSD, deformation or settlement would have been estimated by assuming FS = 1. That is, the working load or service load would be compared to the ultimate resistance. In LSD, SLS criteria are evaluated using unfactored loads and unfactored resistance. In this sense, the methodology for calculation for SLS in LSD is analogous to that of WSD (Becker 1996b).

A conflict, or unnecessary conservatism, arose in adfreeze pile design when structural engineers compared factored loads to an allowable pile capacity that had incorporated a factor of safety. As geotechnical engineers began referring to factored geotechnical resistance, it became more natural for structural engineers to compare this to factored loads. Structural engineers should be referencing service, or specified (unfactored), loads when designing adfreeze piles. Geotechnical engineers should be clear in giving this advice.

4 DESIGN METHODOLOGY

The creep relationship is based on the flow law for ice and is applicable to ice-rich soil. It is conservative for soil that is not ice-rich, where there is significant particle to particle contact in the mineral soil. Nixon and McRoberts (1976) presented a simplified pile design procedure based on this flow law, where the strain rate could be defined through two temperature-dependent creep parameters:

$\dot{\epsilon}_e = B\sigma_e^{\,n}$ (Equation 8, Nixon and McRoberts 1976) [3]

Where: $\dot{\epsilon}_e$ and σ_e are strain rate and shear stress; and B and n are temperature dependent secondary creep parameters.

Morgenstern, et al (1980) further simplified the relationship by determining that n could be taken as 3 for the normal range of frozen ground temperatures. They further determined that the pile velocity could be expressed by normalizing the strain rate by pile radius with the following relationship:

$$\frac{\dot{u}_a}{a} = \frac{3^{(n+1)/2}B\tau^n}{n-1} \quad \text{(Equation 2 from Nixon 1988)} \qquad [4]$$

Where: a is the pile radius and the other parameters are similar to those defined above.

This secondary creep relationship is considered to be applicable to the range of stresses typically applicable to adfreeze piles, i.e. less than about 100 kPa.

Nixon and Lem (1984) showed that the creep parameter, B, is not only temperature-dependent, but also dependent on the porewater salinity, and to a lesser extent the type of the soil in which the pile is embedded.

5 DESIGN CONSIDERATIONS

The primary design inputs for adfreeze pile design are ground temperature, soil porewater salinity, short and long-term loading, pile diameter (with related minimum borehole diameter), active layer thickness at end of design life, and frost heave considerations. Adfreeze bond capacity in the seasonal active layer is neglected. The availability of slotted and un-slotted steel pipe is also a design consideration and is discussed in the following sections.

5.1 Mean Annual Ground Temperature

The design procedure uses mean annual ground temperature, and neglects seasonal variations or any gradient along the embedment length. The ground is likely to be colder when the heaviest loading cases are imposed (i.e. wind, snow, and ice loads).

5.2 Climate Change Considerations

Guidance described in CSA Plus 4011 is used to estimate air temperatures at the end of the service life of the structure being designed. That is, in the absence of modeling to provide better information, it is assumed that ground temperature changes in step with air temperature, at least until the point of phase change is approached. This assumption has been found to be reasonable at Inuvik (Hoeve 2012, 2015).

5.3 Porewater Salinity

Porewater salinity of the permafrost is determined by collecting soil samples in testholes along the pile length, and testing these samples in the laboratory. The slurry backfill is generally not tested, as this allows for the possibility of using native soil for backfill, which is often done. This is considered conservative, as the porewater in the native soil would be diluted during slurry mixing with potable water.

5.4 Long-term Resistance

Long-term pile capacity is based on long-term loads, which include dead load, and sustained live load, i.e. occupancy live load. It is typically assumed that the creep rate is 1 mm per year for the life of the structure, capped at 30 mm, but this can be varied in consultation with the owner, and in consideration of the performance requirements of the structure being supported by the piles.

5.5 Short-term Resistance

Short-term capacity based on combined short-term and long-term loads. Short-term loads typically include environmentally imposed live loads, such as wind, snow, and ice load. The typically assumed creep rate is 2 mm in four months. Seismic loads are generally not considered in this design, but would be just one more short-term loading case as determined by the structural engineer.

5.6 Effective Pile Diameter

By reviewing case histories, Morgenstern et al (1980) concluded that the effective pile diameter when a moist sand slurry was used could be assumed as the diameter of the bored hole. For saturated slurries, the assumed pile diameter should be the diameter of the pipe. Current practice is to take the effective diameter as the diameter of the pipe in essentially all cases. This is likely conservative.

There is a difference between unslotted and slotted piles, as shown in Figure 1. If slotted piles are used, it is reasonable to use the hole diameter or some intermediate diameter between the pile and hole, to reflect the use of the slots that theoretically promote a better/more continuous bond between pile and the permafrost. This can result in shorter piles being required.

In isolated northern communities, there is normally a supply of unslotted pipe available, and it has been shown to be generally cost-effective to install longer or more piles, as opposed to either having factory slotted piles shipped, or having slots cut by welders on site.

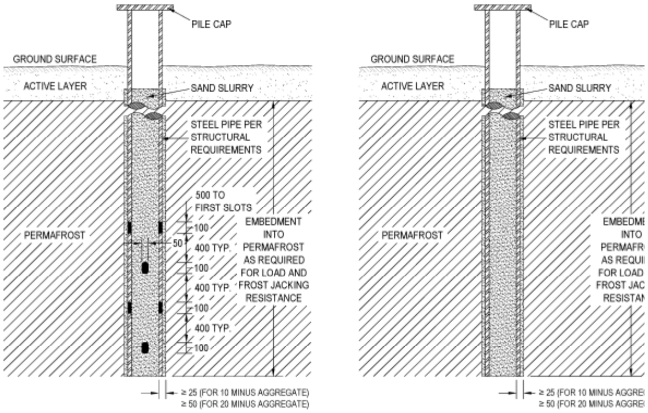


Figure 1: Typical Details of Slotted Piles vs Unslotted Piles

5.7 Frost-Jacking

Frost jacking resistance is often considered to be the primary design parameter, especially for piles that are loaded in tension or lightly loaded in compression. Past failures can be attributed to inadequate embedment. For design, an ultimate frost-jacking stress of 150 kPa through the active layer is used. This is compared to the ultimate (unfactored) short-term resistance to determine the minimum required pile embedment.

The common practice of using grease and poly wrap in the active layer is generally ineffective in reducing seasonal frost jacking forces. This may work in the short term (three to five years) but eventually becomes ineffective. It is always better to design adequate pile length in permafrost to resist anticipated frost heave forces.

5.8 Resistance Factors

Historically, when using WSD, it was acknowledged that design was based on acceptable settlement. Therefore, lower factors of safety were used than would typically be used in geotechnical practice for deep foundations in unfrozen soil. A FS of 2.0 was assumed for unslotted piles and FS of 1.5 for slotted piles, because the slots induce some degree of shearing within the backfill.

The practice then evolved to employing resistance factors of 0.5 and 0.7 to develop recommended bond values. These resistance factors were initially taken as approximate inverses of the associated factors of safety.

A conflict arose when structural engineers compared the derived factored resistances with factored loads, in that the design became more conservative than would have been obtained previously under WSD. Comparing unfactored loads to the new factored resistances gave a result equivalent to that previously obtained with WSD.

It is acknowledged that applying resistance factors to calculated bond values is more conservative than a true SLS calculation, however this conservatism is considered to be justified because the ultimate resistance is somewhat subjective, based on the assumptions about the rate of creep.

Biggar 1991 discusses several ways to improve pile resistance: sandblasting, welded rings, and using grout backfill. While these are all effective to varying degrees, they are not commonly used in practice due to the cost. As previously noted, for relatively small projects in remote locations, it is more economical to install longer or more piles than to apply pile treatments.

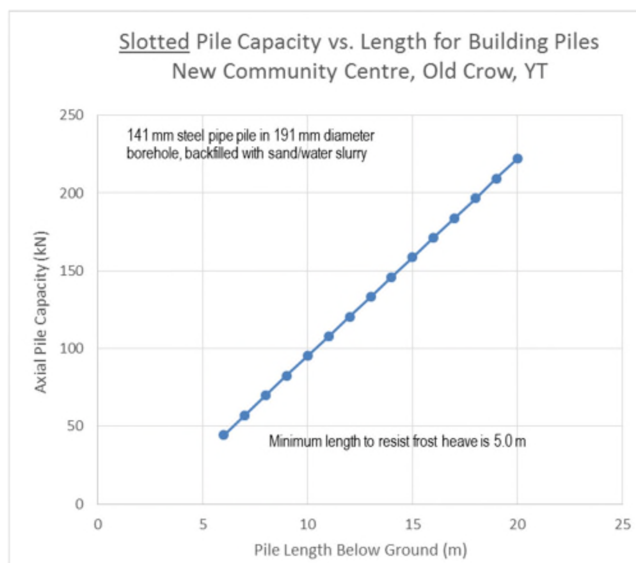
5.9 End-bearing

End-bearing is neglected. This is considered to be reasonable for ice, and ice-rich soil (Nixon and McRoberts 1976 and Morgenstern et al 1980) but is likely to be conservative in non-ice-rich soils.

Because of conservatism in design, predicted creep settlements rarely occur. But if the estimated creep settlements were to occur, perhaps as a result of ground warming, it is expected that some end-bearing would be mobilized to mitigate to some degree the decreasing bond stress along the shaft of the pile.

6 CONCLUSIONS

The chart in Figure 2 presents the results of an adfreeze pile design in Old Crow, Yukon, using the design process described above. It is critical to include the minimum length of pile required to resist frost heave, as sometimes using only the tension or compression loads on the piles will result in a shorter pile.



The procedure described herein presents a design process that has been proven over decades of practice and that, to our knowledge, has resulted in successful adfreeze pile performance at various sites and for various applications across northern Canada. While some aspects of the design remain proprietary and are not discussed in this paper, the theory upon which the process is based is described and referenced herein, and remains in the public domain.

7 REFERENCES

Biggar, K.W. 1991. *Adfreeze and Grouted Piles in Saline Permafrost*, Ph.D Thesis, University of Alberta, Edmonton, Canada.

Becker, D.E., 1996a. Eighteenth Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part I. An Overview of the Foundation Design Process. *Canadian Geotechnical Journal*, 33: 956-983.

Becker, D.E., 1996b. Eighteenth Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part II. Development of the National Building Code of Canada. *Canadian Geotechnical Journal*, 33: 984-1007.

Canadian Standards Association Plus 4011-10, 2010. *Technical Guide, Infrastructure on Permafrost: A Guideline for Climate Change Adaptation.*

Hoeve, T.E., 2015. The Inuvik airport runway – an evaluation of 50 years of performance. Proceedings of the 68th Canadian Geotechnical Conference, GeoQuebec, September 20 - 23, 2015, Quebec, PQ.

Hoeve, T.E., 2012. Climate change impact on the state of permafrost at Inuvik, N.W.T. Proceedings of the 65th Canadian Geotechnical Conference, GeoManitoba, September 30 – October 3, 2012, Winnipeg, MB.

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

Nixon, J.F., 1988. Pile loads tests in saline permafrost at Clyde River, Northwest Territories. *Canadian Geotechnical Journal,* 25:24-32.

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

Nixon, J.F., and Lem, G., 1984. Creep and strength testing of frozen saline fine-grained soils. *Canadian Geotechnical Journal,* 21(4): 518-529.