A STUDY OF SOME FACTORS AFFECTING THE ADFREEZE BOND OF PILES IN PERMAFROST

B. LADANYI and A. THÉRIAULT

Centre d'ingénierie nordique, École Polytechnique de Montréal
C.P.6079, Succ. A, Montréal, Québec, Canada. H3C 3A7

Abstract

Although a considerable amount of information on the adfreeze bond for pile design in permafrost is available in the literature, very little of it has been submitted to a careful and critical study. A recent review of available data and pile design methods has shown that certain improvements can be made in connection with the practical evaluation of short- and long-term adfreeze bond. This conclusion is based on the results of a number of direct double-shear tests, performed under cold room conditions. The tests included two kinds of pile materials (steel and aluminium) and two types of frozen soils (sand and silt). All the tests were made at a temperature of -2 °C.

The tests were designed primarily to help finding answers to two important questions of pile design in permafrost:

(1) What fraction of the shear strength of frozen soil represents the adfreeze strength for a given pile material and soil type, and

(2) Under which conditions the adfreeze bond, once broken by pile driving and settlement, can be reformed by healing, considering the influence of normal pressure, time and temperature.

Résumé

Malgré une quantité considérable de renseignements disponibles dans la littérature concernant l'adhérence du sol gelé au pieu en relation avec la conception des pieux dans le pergélisol, relativement peu de ceux-ci ont été soumis jusqu'à présent à une étude soigneuse et critique. Une revue récente des données disponibles et des méthodes conventionnelles de conception des pieux a montré que certaines améliorations sont possibles en relation avec l'évaluation pratique de l'adhérence à court et à long terme. Cette conclusion est basée sur les résultats d'une série d'essais de cisaillement direct, effectués en chambre froide. Les essais ont compris deux matériaux de pieux (acier et aluminium) et deux types de sols gelés (sable et silt). Tous les essais ont été effectués à une température de -2 °C. Leur but principal était de trouver les réponses à deux questions importantes concernant la conception des pieux dans le pergélisol, notamment:

(1) Quelle fraction de la résistance au cisaillement du sol gelé représente l'adhérence pour un matériau de pieu et un type de sol donnés, et

(2) Dans quelles conditions l'adhérence, une fois brisée par le battage et le tassement du pieu, peut être récupérée par cicatrisation, en considérant les effets de la pression normale, du temps, et de la température.

Introduction

PILE DESIGN IN PERMAFROST

Similarly as in unfrozen soil mechanics, the allowable load for a single pile embedded in frozen soil is determined by satisfying the criteria of tolerable settlement and safety against failure. When considering the latter, it is customary to use the so-called static pile formula, Eq. (1), expressing the fact that the total pile load, Q, is carried simultaneously by the pile base resistance, q_p, and by the sum of shear resistances, τ_a, along its shaft:

$$Q = q_p A + P\sum_{0}^{L} \tau_a \Delta L \qquad (1)$$

where A is the area of the pile base, P is the perimeter of the shaft, ΔL is the thickness of individual layers, and L is the total length of the pile embedded in frozen ground.

Clearly, when Eq.(1) is applied to a pile in frozen soil, in addition to other factors usually considered in connection with the bearing capacity of piles in unfrozen soils, one must also take into account the fact that, because of the presence of ice in the ground, both the shear and the adfreeze strength of frozen ground depend strongly upon the temperature, the ice content and the applied strain rate. When creep and strength data of frozen soils surrounding the pile are properly expressed as functions of temperature and strain rate, the pile capacity can be determined by following the procedure described, e.g., by Weaver & Morgenstern (1981b), which considers both settlement and bearing capacity criteria.

With very few exceptions, piles installed in permafrost have a circular or square cross-section, their shaft is straight, and their surface is usually smooth. It is well known that for such piles under service conditions, most of the applied load is carried by adfreeze bond, so that end bearing is often neglected in the design. Many careful studies of adfreeze bond (Parameswaran, 1978, 1979, 1985; Weaver & Morgenstern 1981 a & b) have shown that, under similar conditions, the adfreeze bond is affected by the pile material and by the method of pile installation.

Point resistance

As shown experimentally by Ladanyi & Paquin (1978) for frozen sand, and by Sego (1980) for ice, after a mobilization period, requiring a total settlement of about 30% of the base diameter, the point resistance of a pile in frozen soil or ice tends to become proportional to the penetration rate, as long as the temperature remains constant. Based on this finding, Nixon (1978) used a theory of creep settlement of circular footings in frozen soils, developed by Ladanyi & Johnston (1974), for determining the dependence of the pile point resistance, q_p on the penetration rate, s = ds/dt, where s denotes the amount of pile point penetration. This relation as the form:

$$q_p = \left[\frac{2n\sigma_{c\Theta}}{3}\right]\left[\frac{\dot{s}}{(D/2)\dot{\varepsilon}_c}\right]^{1/n} \qquad (2)$$

where D is the base diameter, and the remaining parameters refer to the creep law for frozen soils (e.g., Ladanyi & Johnston, 1974):

$$\varepsilon_e = (\sigma_e/\sigma_{c\Theta})^n\,(\varepsilon_c t/b)^b \qquad (3)$$

where the subscript e denotes the Von Mises equivalent stress and strain, t is the time, n and b are creep parameters. Finally, $\sigma_{c\Theta}$ denotes the reference stress at an arbitrary strain rate, ε_c, obtained from a test made at a temperature of Θ degrees below the freezing point. Normally, because of low settlement rates allowed for piles (usually less than 1 mm/year), the point resistance according to Eq. (2) is very low and is most often neglected.

Adfreeze bond

A synthesis of published data on long term adfreeze strength, made by Weaver & Morgenstern (1981), shows that, for saturated frozen soils, the adfreeze strength is inversely related to the ice content, and directly related to the roughness of the pile. In the limit, the adfreeze strength of very rough piles comes close to the shear strength of the soil. On the basis of the same data, these authors propose to relate the adfreeze streength of a frozen soil, τ_a, to the long-term shear stregth of the same frozen soil, τ_{lt}, by the relation:

$$\tau_a = m\ \tau_{lt} \qquad (4)$$

where m characterizes the type and the roughness of the pile surface. The long-term shear strength of a frozen soil is composed of both frictional and cohesive components:

$$\tau_{lt} = c_{lt} + \sigma_n \tan \phi_{lt} \qquad (5)$$

where c_{lt} is the long-term cohesion of the frozen soil, σ_n is the lateral stress acting on the surface of the pile, and ϕ_{lt} is the friction angle at the pile-soil interface. However, the same authors suggest that the frictional component can be neglected, because the lateral normal stress acting on the pile is usually relatively small. This enables Eq.(4) to be written as:

$$\tau_a \approx m\ c_{lt} \qquad (6)$$

By comparing published data on long-term shear and adfreeze strengths measured in different soils and pile materials, Weaver & Morgenstern (1981) suggest values for the coefficient m of 0.6 for steel and concrete piles, 0.7 for uncreosoted timber piles, and 1.0 for corrugated steel piles.

Although the assumptions on adfreeze strength made by Weaver & Morgenstern (1981) appear quite reasonable, it seems useful to take a critical look at certain aspects of the problem, and in particular that of the permanence of principal adfreeze bond parameters and their variation with time.

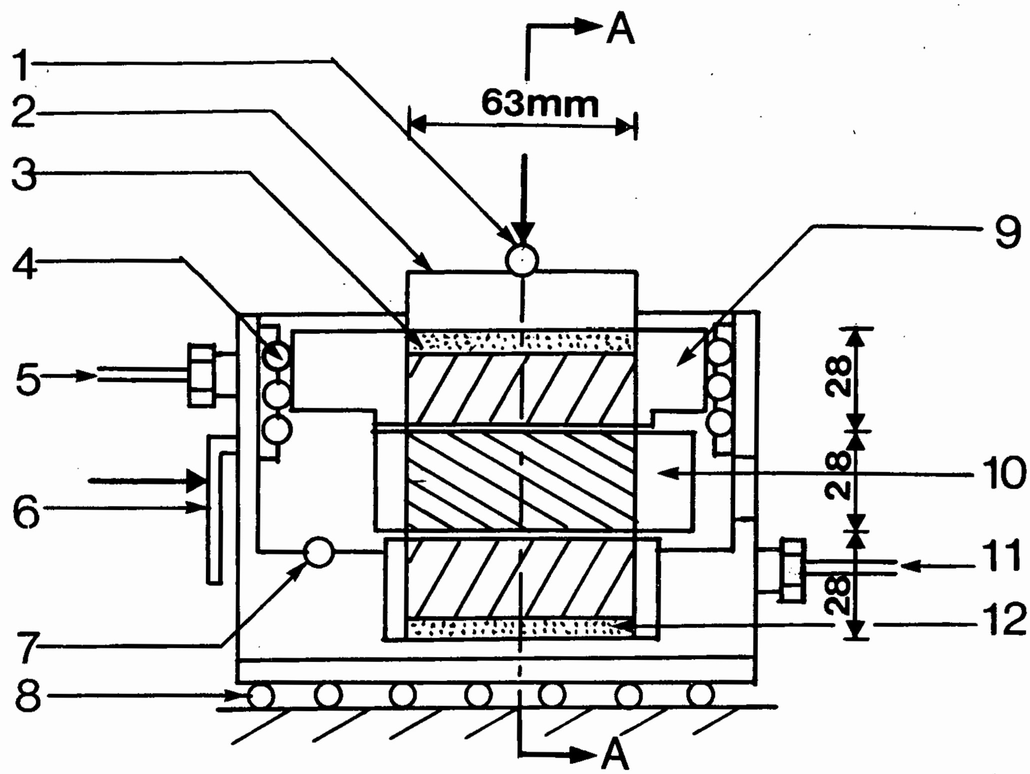
In order to get some more information about this problem, laboratory tests were made to determine the shear strength of two frozen soils and their adfreeze strengths against metallic surfaces, from which short-term experimental values of the coefficient m were determined. Special attention was paid to the phenomenon of bond healing as affected by time and normal pressure.

Testing

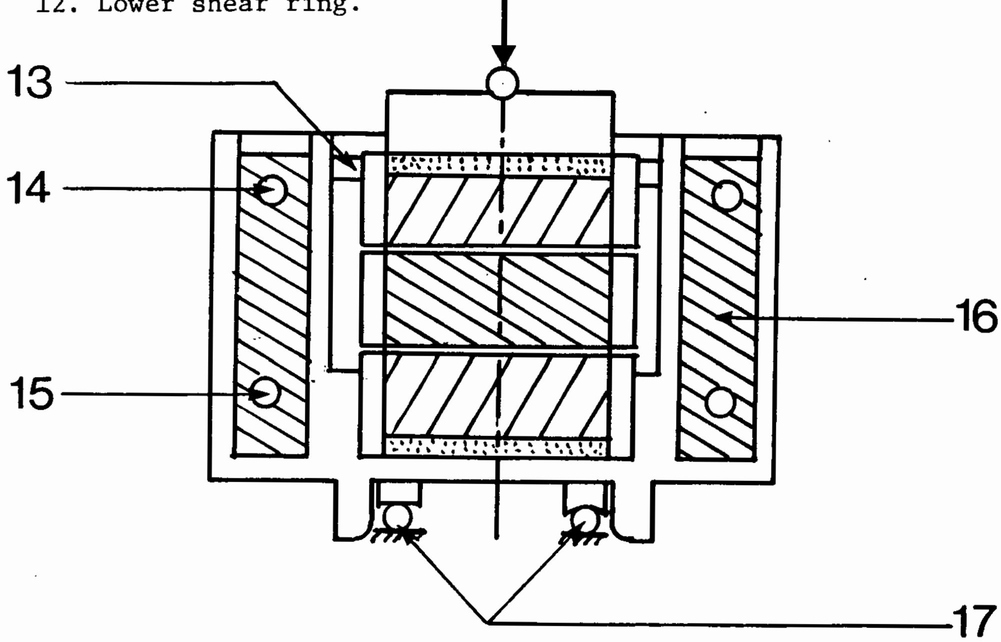
TEST EQUIPMENT

All the tests were carried out in a cold room where the temperature was set at -2 °C, with a variation of 1 °C. The direct shear tests were performed in a specially designed double-shear box, mounted on a conventional shear-rate-controlled testing frame. The double-shear box, shown in Fig. 1, assures horizontality of failure surfaces and enhances the precision of load and volume change measurements. A cooling system is incorporated in the shear box, in order to reduce the temperature variations. A temperature-controlled bath circulates antifreeze fluid into the chambers around the sample. A thermistor placed in the shear box permits temperature monitoring during the test. It was founf that, with this cooling system, the temperature variation within the box was less than 0.1 °C.

Horizontal displacements and volume variations were measured by displacement transducers Hewlett-Packard 24 DCDT 250, to an accuracy of 0.001 mm. Loads were measured by load transducers, to an accuracy of 1.85 N. All data, including the temperature, were recorded on a data acquisition system, and plotted subsequently. More information about the testing equipment can be found in Thériault (1988).



Longitudinal cross-section: 1. Vertical load application; 2. Loading platen; 3. Porous stone; 4. Cylindrcal bearings; 5. Antifreeze out; 6. Horizontal load application; 7. Thermistor; 8. Ball bearing; 9. Upper shear ring; 10. Central shear ring; 11. Antifreeze in; 12. Lower shear ring.



Cross-section A-A: 13. Flexible insulation; 14. Antifreeze out; 15. Antifreeze in; 16. Cooling chamber; 17. Ball bearings.

Figure 1. Double-shear box.

Test materials

Sand

The sand used in the tests was a fine sand from Joliette, Québec. It is the same sand as used previously in the tests described by Ladanyi and Eckardt (1983), and Ladanyi and Guichaoua (1985). Its grain size distribution is about 90% between 0.1 and 1.0 mm, with less than 2% below 0.1 mm, and less than 1% above 2 mm, giving a coefficient of uniformity of about 3. The maximum and minimum dry densities of the sand were 1810 and 1510 kg/m³, respectively. Triaxial tests performed with the dry sand gave peak shear strength angles of 45° at the maximum density, and 39° at a density of 1680 kg/mo.

Silt

The silt material used in the tests comes from Caen, France. For the tests, only the portion finer than 0.5 mm was used, so that the tested material contained about 70% of the grains between 0.1 and 0.01 mm, with 23.5% smaller than 0.01 mm and 17% smaller than 0.002 mm. For that silt, at a temperature of -2°C, Smith (1984) found that it contained about 14% unfrozen water by dry weight.

Sample preparation

For the tests, sand and silt samples 63 mm in diameter and 65 mm long were compacted at optimum water contents of 14% and 15%, respectively. A vacuum is first applied at the top of the sample, while water is supplied at the base. Saturation is completed by gravity flow for a period of 4 to 5 hours, and the sample is then placed in a cooler. Insulation assures unidirectional freezing from the top down, while water is available at the base of the sample, in order to get a more uniform water content distribution through the sample. After a few days, the sample is extruded from the mould, cut to final dimensions and wrapped in a plastic film. Samples are kept in the cooler, and 24 hours before testing, they are placed in the cold room to stabilize at the testing temperature of -2°C.

Samples of frozen silt had a void ratio between 0.6 and 0.7, average water content of 20.5%, degree of saturation between 75% and 98%, and dry density between 1855 and 1977 kg/m³. Samples of frozen sand had a void ratio between 0.62 and 0.70, water content of about 17.5%, degree of saturation of about 70%, and dry density of about 1880 kg/m³.

Test results

This investigation included 3 kinds of tests:

(1) Direct shear tests with the two frozen soils (14 tests with frozen silt and 7 tests with frozen sand),
(2) Direct shear tests between frozen sand and metallic surfaces (11 tests against steel and 8 tests against aluminium), and
(3) Bond-healing tests within the two soils, and between frozen sand and a steel surface (6 tests for the latter).

All the tests were made at a shear velocity of 16.9 mm/day.

In all the tests, normal pressures of up to 1.0 MPa were applied to already frozen samples, and the samples were sheared at a shear velocity of 16.9 mm/day, corresponding to about $2.0\text{x}10^{-4}$ mm/sec.

Shear strength

Typically, in both soils, failure (peak strength) occurred at a displacement of about 2.5 mm, but the tests were continued for about 5 mm more in the post-peak region, to determine also the residual strength.

Figures 2 and 3 show the peak and residual strengths of the two frozen soil materials. It is found that, at -2°C, the peak strengths of the two materials can be represented by Coulomb straight lines with average values of shear parameters: c = 0.4 MPa, θ = 17° for frozen silt, and c = 1.28 MPa, 0 = 45° for frozen sand.

It will be seen in Figs. 2 and 3 that for frozen silt a plastic-brittle transition is located at a normal pressure of about 1 MPa, but for frozen sand no such transition point has been found within the range of applied normal pressures. However, in the range of normal pressures acting on lateral pile surfaces, which are of the order of 100 kPa, both soil materials have a brittle behaviour with a considerable loss of strength after the peak.

Adfreeze strength

For smooth piles, failure occurs at the pile-soil interface. In order to study this problem, double shear tests betweeen frozen sand and both steel and aluminium surfaces were

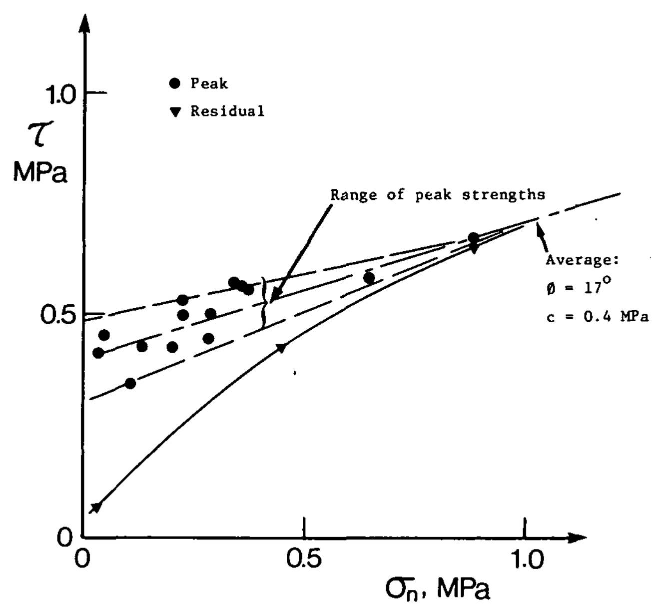


Figure 2. Peak and residual strengths of frozen silt at -2 °C.

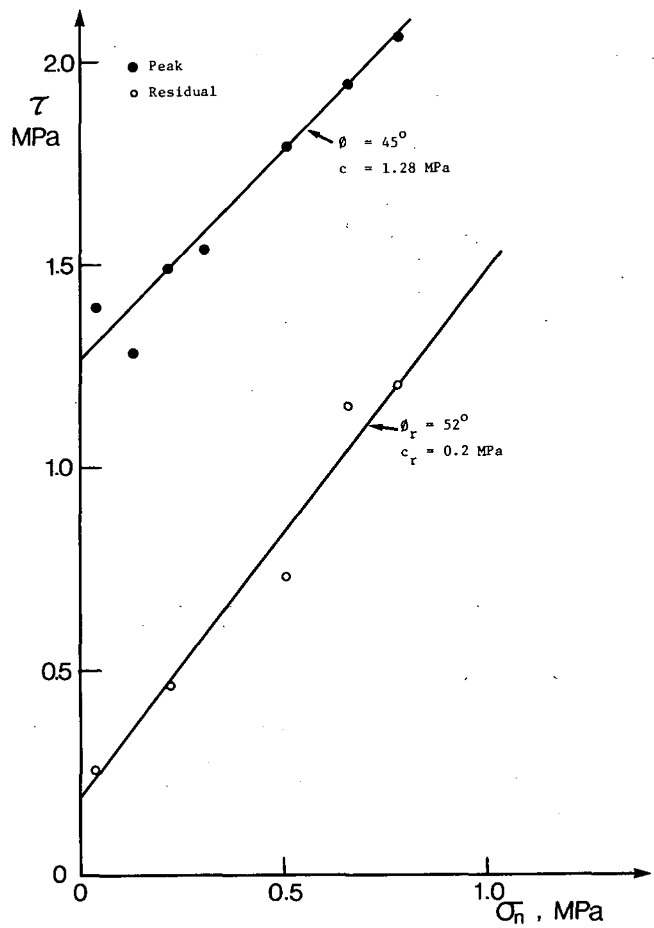


Figure 3. Peak and residual shear strenghts of frozen sand at -2 °C.

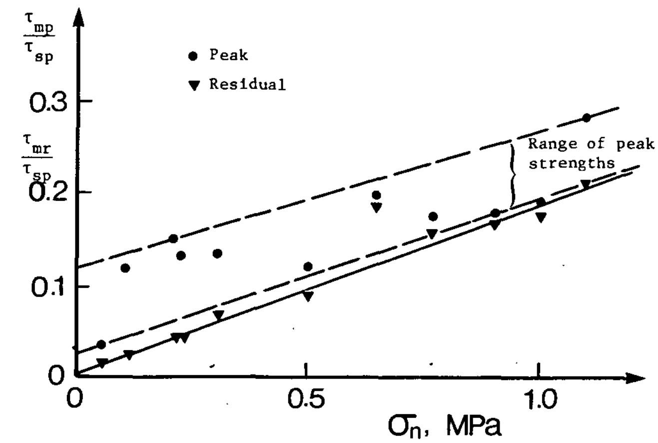


Figure 4. Adfreeze bond of frozen sand against steel, expressed as a fraction of shear strenght of frozen sand (T=-2 °C).

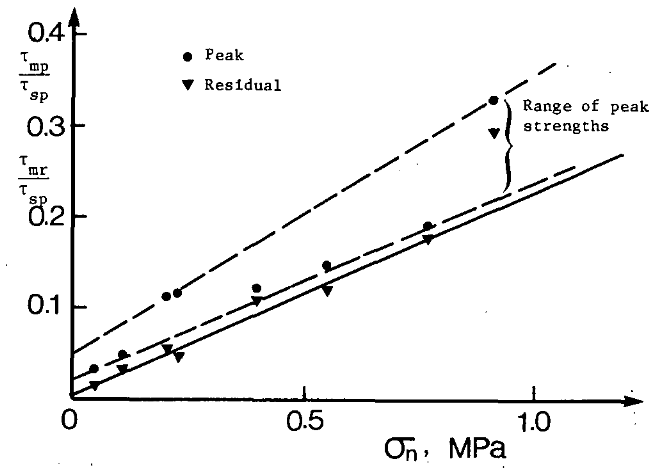


Figure 5. Adfreeze bond of frozen sand against aluminium, expressed as a fraction of shear strenght of frozen sand (t=-2 °C).

undertaken. For the tests, the same sand at a water content of 15 % was compacted in layers in a mould against steel or aluminium surfaces to a density of about 1957 kg/m^3, and then saturated. The samples were then frozen in a cooler for a few days, and finally extruded from the mould to be mounted in the double-shear box for the tests.

Figures 4 and 5 show a summary of these tests in a plot relating the ratios: τ_{mp}/τ_{sp} and τ_{mr}/τ_{sp}, respectively, with the applied normal pressure, where τ_{mp} and τ_{mr} denote peak and residual adfreeze strengths of frozen sand against the metallic surface, and τ_{sp} denotes the peak strength of frozen sand at the same normal stress.

It will be seen in these figures that the scatter of experimental points is fairly high for peak strength ratios, and slightly smaller for residual strengths. However, it is interesting to note that, in such short-term tests:

(1) the peak strength ratios obtained are rather low, being of the order of 10 % or less in the normal pressure range of 100 kPa, and
(2) they tend to increase with increasing normal pressure, following a slope of about 20° in the case of steel, and about 28° in the case of aluminium surfaces.

Bond healing tests

In bond healing tests, each shearing test was stopped in the post- peak region, after a displacement of about 5 mm, and it was resumed after different periods of time, under the same normal pressure.

Figure 6 shows for frozen silt the percent fraction of shear strength recovered after waiting up to 95 hours at a given normal stress. In the brittle failure range, i.e., for $\sigma_n <$ 1 MPa, it is found that the amount of bond healing shows an increase with increasing normal pressure, and that, under favorable conditions, it may even exceed the initial peak strength of the material. This behaviour of frozen silt is considered to be due to the presence of about 14 % of unfrozen water at the test temperature of -2 °C.

On the other hand, in frozen sand, which contains little or no unfrozen water at that temperature, no strength recovery with time has been observed in similar tests, which means that its post-peak strength remained at its residual value after any time period under normal pressure.

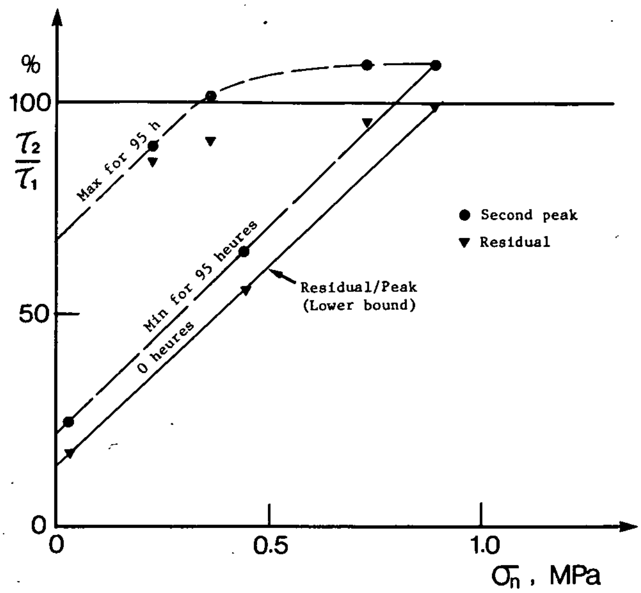


Figure 6. Bond-healing within frozen silt at -2 °C.

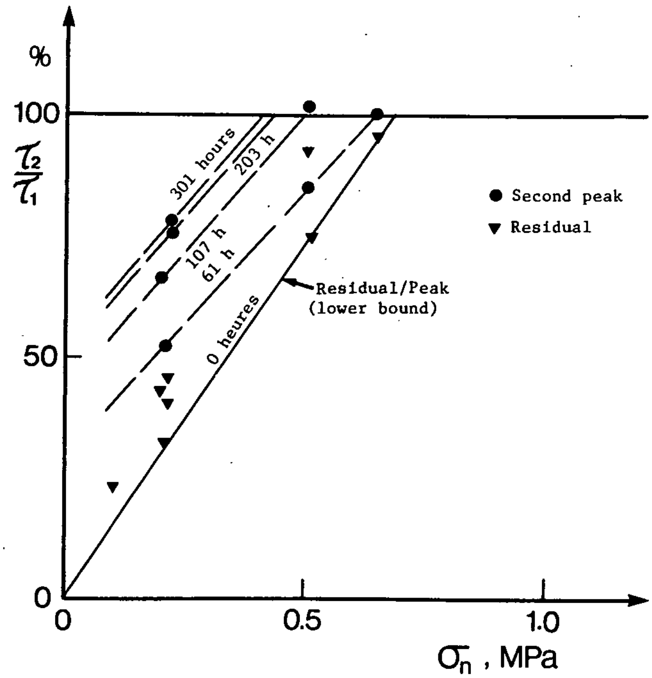


Figure 7. Shear tests with frozen sand against a steel surface : Bond-healing as a function of pressure, with time as a parameter. (T=-2 °C).

Finally, Figs. 7 and 8 show the results of bond-healing tests carried out with frozen sand against a steel surface. In that case, a clear bond recovery with time was observed, tending to a complete recovery for times over 300 hours and pressures over 200 kPa. Figure 8 drawn from Fig. 7, represents a tentative rate of bond recovery with time under normal pressures varying from 100 kPa to 500 kPa.

This difference in post-failure healing, on one hand within frozen sand, and on the other, at the frozen sand–steel interface, is considered to be due to the pressure thawing of ice at the contact of sharp sand grains and the flat steel surface, resulting in water migration and refreezing in the pore space in contact with the metal.

Discussion

When extended to pile behaviour in permafrost, the results of this study lead to the conclusion that the adfreeze bond against a smooth metallic surface is essentially brittle, as found earlier by Crory (1963), Sadovskiy (1973), and Parameswaran (1978). Consequently, during pile driving, or after large pile settlements, such as in a pile loading test, the adhesive portion of adfreeze bond will be strongly reduced or completely destroyed, the remaining residual bond strength being mainly due to the residual friction, depending on the largely unknown normal pressure against the pile shaft.

As for the latter, it is found that, although relatively high lateral stresses are generated either by slurry refreezing around a pile in an oversized hole, or by driving a pile directly in frozen ground or in a slightly undersized hole, these stresses will relax rapidly with time, at a rate depending on the constitutive equation of frozen soil. A recent theoretical study (Ladanyi, 1988) shows that in an ice-rich soil, the mobilized lateral pressure on the pile will relax very rapidly, falling to less than 1 % of its initial value after only a day. This leads to the conclusion that lateral stresses mobilized either by slurry refreezing, or by pile driving, should not be considered in pile design for long-term loads. They should, however, be taken into account when performing and evaluating the results of pile loading tests in permafrost.

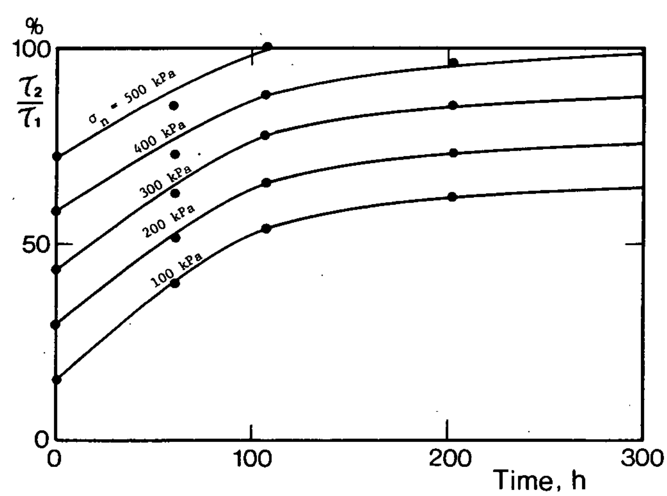


Figure 8. Shear tests with frozen sand against a steel surface : Bond-healing as a function of time, with normal pressure as parameter. (T=-2 °C).

Postulated long-term behaviour of piles in permafrost.

Although this experimental study involved only short-term tests, it may be of interest, nevertheless, to see its possible implications for general pile design in permafrost.

In unfrozen soils, long-term behaviour of piles is explained by assuming that some time after the pile driving or installation, the pore pressure generated will dissipate, and that lateral stresses will eventually return to their original effective state. This philosophy is unlikely to work in frozen soils, because of their extremely long consolidation time, which may exceed the useful service life of the structure. In other words, lateral stresses around a pile in permafrost are unlikely to fall below the total lateral stress p_o in the ground and attain its effective value.

Unfortunately, at present, very little is still known about the lateral stress, which has up to now never been directly and systematically measured in situ. Tentatively, it can be assumed that, below the level of seasonal temperature variation, the ratio of total horizontal to total vertical ground stress is located somewhere between the total K_o value in dense frozen soils, and 1 in ice-rich soils and in pure ice. Above that level,closer to the ground surface, the lateral stress is expected to show a seasonal variation, varying from 0 when tensile cracks appear, to more than 1, when the cracks fill with water and refreeze. It follows that, along a pile embedded in permafrost, lateral stresses may vary not only with depth, but also with time and temperature.

In current pile design practice in frozen soils, it is usually considered that long-term pile capacity can be predicted by extrapolating either laboratory or field creep testing information to the service life of the pile. However, the stress relaxation phenomena in frozen soils point to the fact that, at long periods of time, the pile behaviour may be governed by factors quite different from those obtained by simple extrapolation of short-term data. It is postulated that at long term, the pile shaft capacity will depend not only on the long-term cohesion of the frozen soil, but also on the residual friction angle at the soil-pile interface, and the total original lateral ground stress. The probable long-term value of the shaft resistance will then be:

$$\tau_{a,lt} = m\, c_{lt} + \sigma_{n,tot} \tan \phi_{lt} \qquad (7)$$

implying that at long term the frictional contribution to the shaft resistance may not be negligibly small with respect to the cohesive adfreeze bond.

Effective lateral stresses may be considered only in those frozen soils which contain large quantities of unfrozen water, such as saline soils in offshore permafrost regions.

This experimental investigation has confirmed many previous findings that, in the case of straight-shafted piles in permafrost, the adfreeze bond is essentially brittle, leading to a large loss of pile capacity after only a small axial displacement. Although the test results indicate that a portion of the original cohesive bond may be recovered with time under favorable conditions, and especially in soils containing large amounts of unfrozen water, a determination of the long-term cohesive bond from short-term data by a simple extrapolation in time or strain rate, seems quite dubious for such piles.

In this respect, as shown by Ladanyi and Guichaoua (1985), the use in permafrost of corrugated or tapered piles, installed in predrilled holes and slurried, seems to offer some clear advantages, because of their more plastic response to short-term loads, and much better possibilities for prediction of their long-term behaviour.

Conclusions

The tests performed in this study have given at least some partial answers to the two questions asked in the beginning. In particular, it was found that at short term and at a temperature of -2 °C, the adfreeze bond is essentially brittle and once broken after a small displacement, it represents only a small fraction of the peak shear strength of frozen soil, which tends to increase with increasing normal pressure.

As for the adfreeze strength recovery by bond healing, the repeated double-shear tests show that, at -2 °C, after the bond is broken, its complete recovery may occur with time only under relatively high normal pressures of the order of 500 kPa. At lower normal stresses, the bond healing is only partial, and may even fall to zero if no other measure is undertaken. The bond healing seems to be favorized by the unfrozen water content of the soil, and at the testing temperature of -2 °C, it was found to be more pronounced in frozen silt with 14 % unfrozen water content that in frozen sand with practically no unfrozen water.

As for the design of piles in permafrost for long-term loads, this study only indicates that, for straight-shafted piles, the relevant long-term cohesive portion of adfreeze bond depends not only on time, but also on many other factors, so that, without additional proof, its determination by extrapolation from short-term creep data seems to be highly questionable.

Finally, it is clearly recognized that this investigation has furnished answers only to a limited number of questions concerning the adfreeze bond problem. For more clarification, further studies of that kind and large scale observations should be made in the future, covering the effects such as the density and ice saturation of frozen soil, the character of the soil-pile interface after freezing, and the effect of time on bond healing for much longer time periods.

Acknowledgements

This study was financially supported by the grants No. A-1801 and G-1566 of the National Science and Engineering Research Council of Canada. The authors would like also to thank the reviewers of the paper for their useful critical comments.

References

CRORY, F.E. 1963. Pile foundations in permafrost. Proc. Int. Conf. on Permafrost, Lafayette, IN, USNAS, 1278 : 467-476.

LADANYI, B. 1988. Short- and long-term behaviour of axially loaded bored piles in permafrost. Proc.Int.Symp.on Deep Foundations on Bored and Auger Piles, Ghent, Belgium, pp. 121-131.

LADANYI, B. and ECKARDT, H. 1983. Dilatometer testing in thick cylinders of frozen sand. Proc.Fourth Int.Conf. on Permafrost, Fairbanks, Alaska, Nat.Academy Press, Washington, 1 : 677-682.

LADANYI, B. and GUICHAOUA, A. 1985. Bearing capacity and settlement of shaped piles in permafrost. Proc. 9th ICSMFE, San Francisco, pp. 1421-1427.

LADANYI, B. and JOHNSTON, G.H. 1974. Behaviour of circular footings and plate anchors embedded in permafrost. Canad. Geotech.J., 11 : 531-553.

LADANYI, B. and PAQUIN, J. 1978. Creep behaviour of frozen sand under a deep circular load. Proc. Third Int.Conf. on Permafrost, Edmonton, 1 : 679-686.

NIXON, J.F. 1978. Foundation design approaches in permafrost areas. Canadian Geotech. J., 15 : 96-112.

PARAMESWARAN, V.R. 1978. Adfreeze strength of frozen sand to model piles. Canad. Geotech. J., 15 : 494-500.

PARAMESWARAN, V.R. 1979. Creep of model piles in frozen soil. Canad. Geotech. J., 16 : 69-77.

PARAMESWARAN, V.R. 1985. Attenuating creep of piles in frozen soils. Proc. ASCE Spring Convention, Denver, Session: Foundations in Permafrost, pp. 16-28.

ROGGENSACK, W.D. and MORGENSTERN, N.R. 1978. Direct shear tests on natural fine-grained permafrost soils. Proc. Third Int.Conf.on Permafrost, Edmonton, 1 : 728-735.

SADOVSKIY, A.V. 1973. Adfreeze between ground and foundation materials. Proc. 2nd Int. Conf. on Permafrost, Yakutsk, pp. 650–653.

SEGO, D. 1980. Deformation of ice under low stress. Ph.D. Thesis, Univ. of Alberta, Edmonton, 350 p.

SMITH, M.W. 1984. Soil freezing and frost heave at the Caen Experiment. Proc. Seminar on Pipelines and Frost Heave, Caen, France, pp.19-21.

THÉRIAULT, A. 1988. Capacite portante des pieux dans le pergélisol. Thèse de Ph.D., École Polytechnique, Montréal, 337 p.

WEAVER, J.S. and MORGENSTERN, N.R. 1981a. Simple shear creep tests on frozen soils. Canad. Geotech. J., 18 : 217–229.

WEAVER, J.S. and MORGENSTERN, N.R. 1981b. Pile design in permafrost. Canad. Geotech. J., 18 : 357-370.