Bearing capacity and settlement of shaped piles in permafrost

Capacité portante et tassement des pieux crénelés et tronconiques dans le pergélisol

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SYNOPSIS Although in frozen soils most piles act as "friction piles", relying very little on end-bearing, not much attention has been concentrated up to now on the problem of improving their shaft resistance, e.g., by providing corrugations or giving a slight taper to their shaft. Realizing that a systematic study of such shaped piles would be useful, an experimental investigation was undertaken, including a large number of load-and rate-controlled tests on three types of model piles: smooth and corrugated straight-shafted piles, and smooth tapered piles, installed in frozen sand. This paper presents the principal test results, their analysis, and the comparison with theoretical predictions.

INTRODUCTION

With very few exceptions, most of the piles installed in permafrost have a regular cylindrical or square cross-section, their shaft is straight, and their surface is usually relatively smooth. It is well known that, under service loads, most of the load on such piles is carried by adfreeze bond, while the end bearing is small and is often neglected in the design. Many careful studies on the adfreeze bond (Parameswaran, 1978, 1979; Weaver and Morgenstern, 1981a,b) have shown that, under similar conditions, its intensity is affected by the pile material (being the highest for untreated wood, and the lowest for painted steel), and by the method of installation (i.e., whether the pile is driven, drill-driven, or installed in a slurried hole and refrozen). The adfreeze bond is usually assumed in the design to be a fraction of the time- and temperature-dependent frozen soil cohesion. One aspect, common to all straight-shafted piles, is that adfreeze bond fails in a brittle manner, leading to a high loss of pile capacity after failure. (Crory, 1963; Parameswaran, 1978).

In unfrozen soils, shaped piles have been used for many years for increasing their bearing capacity. Several types of such piles are now available commercially, and some design methods have been proposed (e.g., Nordlund, 1963).

In frozen soils, a quite extensive use of in-situ-corrugated straight-shafted piles has been made recently during the construction of the Alyeska Pipeline. However, although some reports on their full-scale performance have been published, showing their clear advantages (Black and Thomas, 1978; Thomas and Luscher, 1980; Luscher et al., 1983), the behaviour of such piles in frozen soils has not yet been systematically investigated and no practical design method has yet been proposed. The effect of corrugations on pulling capacity of anchors embedded in frozen sand was recently studied by Andersland and Alwahhab (1982), who made a series of model scale pulling tests on steel rods containing one or several lugs on their shaft. The effect of lugs, similar to corrugations, was found to considerably increase the pulling capacity of the rods. In addition, Weaver and Morgenstern (1981a) have studied the effect of the material roughness and the type of frozen soil on adfreeze strength in direct shear.

In order to get a better understanding of the behaviour of shaped piles embedded in frozen soil, an experimental study was undertaken in the laboratories of the Northern Engineering Centre of Ecole Polytechnique (Guichaoua, 1984). The study included a large number of both load- and settlement-rate-controlled tests on three types of model piles: smooth and corrugated straight-shafted piles, and smooth tapered piles. Like in practice, the piles were installed in oversized holes in frozen sand, surrounded by a compacted sand-water slurry, and let to freeze at -5°C. There was no pile-soil contact at the pile end.

In order to find a proper design method for the two types of piles, two theories were considered. The first one was ah adaptation of the Johnston and Ladanyi (1972) rod anchors theory, which assumes that a longitudinal shear distortion occurs in the soil around the pile, without shear failure at the pile-soil interface. The second theory, in turn, was based on the fact that a vertical displacement of a tapered piles produces a lateral expansion of the hole in which the pile is embedded, which enabled to relate the settlement rate of the pile with the rate of expansion of a cylindrical hole, following a solution by Ladanyi and Johnston (1973).

The paper presents and analyzes the principal test results obtained, and discusses the prediction of pile behaviour according to the two proposed theories.

TEST EQUIPMENT

All the tests were performed in a cold room where the temperature was controlled with an accuracy of ±1°C. In general, the temperature within the samples remained sensibly constant at -5°C. All load-controlled tests were carried out in cylindrical blocks of frozen sand, contained in a steel tank 762 mm in diam. and 460 mm high. The bottom of the tank was supplied with several water entry tubes to assure a proper sand saturation. In order to avoid the lateral pressure generation during freezing, the inside wall of the tank was covered with a layer of foamed plastic.

The rate-controlled tests, in turn, were carried out in cylindrical blocks of frozen sand 300 mm in diameter and 300 mm high, which were placed in an ordinary 50 kN soil mechanics testing machine, located in the cold room.

TEST MATERIAL

The sand used in the tests was the same as in a previous study described by Ladanyi and Eckardt (1983). Its grain size distribution was about 90% between 0.1 and 1.0 mm, with less than 2% below 0.1 mm, and less than 1% above 2mm, giving a coefficient of uniformity of about 3.0. The maximum and minimum dry densities of the sand were 1810 and 1510 kg/m^3, respectively. Triaxial tests on the dry sand gave peak shear strength angles of 45° at the maximum density and 39° at a density of 1680 kg/m^3.

SAMPLE PREPARATION

The sand is first compacted in the tank with a vibratory compactor and is then saturated from below with distilled de-aerated water. During subsequent freezing, and during the tests, the sand temperature is measured with a set of thermistors embedded at 3 different levels in the sample. About 160 hours were necessary for a sample to freeze and attain a constant temperature of -5°C.

In permafrost practice, the piles are usually installed in predrilled holes with their diameter 50% larger than that of the pile. They are then surrounded by a compacted sand-water slurry and frozen-in. A similar method was used for installing the model piles in this investigation. However, the holes were not drilled, but were obtained by placing either steel tubes with 5 cm diameter (Tests 1 to 12, Table I) or plastic tubes with 6 cm diameter (Tests 14 to 26, Table II), in the sand before freezing. The tubes were silicone-lubricated and wrapped in a thin plastic sheet, to enable their easy retraction.

The piles, with a diameter of 2 to 3 cm, were put in the holes, their ends resting on a cushion of foamed plastic. The sand-water slurry prepared at optimum density, was then placed around the piles, compacted manually and saturated. Before placing, the slurry was cooled to below 4°C.

LOADING AND DISPLACEMENT CONTROL

In load-controlled tests, 8 piles were installed in each sand block, so that their distance from the lateral surface or from another pile was not smaller than 15 cm. The loads were applied and held constant up to over 300 hours by using Belofram pneumatic jacks. In rate-controlled tests, Table III, only one pile was tested in each block. The available rates of penetration were 0.030, 0.061, and 0.300 mm/min. In the tests, the loads were controlled by load cells, and the displacements by DCDT Transducers with 25 mm range, so that all the data, including the temperature, were recorded on a data acquisition system, and plotted subsequently.

TYPES OF PILES

The three types of piles used in the tests, were: smooth, corrugated and tapered piles. All the piles were machined from high strength aluminium. The smooth straight-shafted piles had the diameter of 2 cm and were 21 to 23 cm long. The corrugated piles had the shape of a screw with a thread sloping down at 10° angle. The thread had a triangular shape with 12.7 mm long base and an ascending angle of 10°. This gave in the longitudinal section of the pile indentations 12.7 mm long and ascending at 10° in the direction of the pile movement. The sizes of the corrugated piles are given in Table I.

All the tapered piles had a taper angle of α = 2.1°. They were about 20.3 cm long and their maximum and minimum diameter was about 3.5 and 2.0 cm, respectively. Data about their embedded lengths and diameters are given in Tables II and III.

TABLE I

Results of load-controlled creep tests with straight-shafted piles

Test No.	Slurry	Length	Aver. Diam.	Aver. Shear Stress	Min. Settl. Rate	Total Creep Time
	w %	L cm	D_m cm	τ kPa	$\dot{S}$ 10^{-3} mm/h	h
(A) Smooth piles						
2	16.0	16.2	2.0	43.8	0.05	186
10	15.2	14.8	2.0	62.4	0.08	120
12	15.2	15.4	2.0	69.0	0.43	170
7	16.0	14.1	2.0	75.7	—[1]	0.02
4	16.0	15.7	2.0	90.6	—[1]	0.11
(B) Corrugated piles						
1	16.0	16.1	2.23	118.6	0.21	186
6	16.0	16.3	2.20	147.9	0.15	250
5	16.0	15.3	2.22	190.9	0.25	250
3	16.0	18.2	2.09	224.9	4.31	67
8	15.2	17.2	2.23	263.6	132.50[2]	60
11	15.2	15.1	2.22	277.0	2.70	170

[1]Instantaneous failure
[2]Low slurry compaction

TABLE II

Results of load-controlled creep tests with tapered piles

Test No.	Slurry w %	Embedded Length L cm	Diameter D cm	d cm	Load Q kN	Aver. Shear Stress τ kPa	Settl. Rate $\dot{s}$* 10^{-3}mm/h	Total Creep Time h
14	17.6	15.0	3.08	1.98	3.12	262	0.84	191
15	17.6	13.6	3.00	2.00	4.01	376	5.88	191
16	17.6	14.2	3.07	2.03	2.90	255	0.49	335
17	17.6	13.4	3.03	2.02	3.39	321	0.24	335
18	17.6	14.3	3.03	1.98	1.78	159	0.09	263
19	17.6	12.6	2.98	2.05	8.92	897	22.00	263
20	17.8	10.4	2.74	1.98	2.68	348	6.95	263
21	17.8	13.1	2.89	1.93	2.90	292	9.87	263
22	17.8	10.4	2.74	1.98	2.45	318	2.23	266
23	17.8	10.8	2.90	2.03	8.92	978	1.94	266
24	17.8	14.8	3.08	2.00	2.01	173	3.28	236
25	17.8	9.6	2.73	2.02	7.58	1045	4.53	236
26	17.8	11.9	2.93	2.05	5.60	600	5.12	240

*at 120 hours

TABLE III

Results of rate-controlled tests

Test No.	Slurry w %	Length L cm	Aver. Diam. D_m cm	Settl. rate $\dot{s}$ mm/h	Aver. peak Shear Stress τ kPa
(A) Smooth Piles					
IX	18.3	11.81	2.0	1.80	853
III	15.7	15.58	2.0	3.66	429
VI	15.7	15.48	2.0	18.30	438
(B) Corrugated piles					
VIII	18.3	12.34	2.23	1.80	1790
II	15.7	14.04	2.20	3.66	1380
V	15.7	13.23	2.09	18.30	2010
(C) Tapered piles ($\alpha = 2.1^o$)					
VII	18.3	14.41	2.55	1.80	(1460)*
I	17.5	16.96	2.64	3.66	(716)*
IV	15.7	13.26	2.51	18.30	(2040)*

* Steadily increasing after 2 cm settlement.

TEST RESULTS

Altogether 33 pile tests were carried out, 24 of which were constant load creep tests, and 9 constant settlement rate tests. Tables I, II and III present the main data and some results of the tests. For each test, the tables give the water content of the slurry and the embedded length and the diameter of the pile. Further, for creep tests, Tables I and II give the value of the applied average shear stress τ (and the total load Q in Table II), the resulting minimum settlement rate, and the total creep time. For rate controlled tests, Table III gives the value of the applied settlement rate and the resulting average peak shear stress τ.

Figures 1 to 4 show some typical results obtained in the experiments. Figure 1 shows creep curves of a corrugated and a tapered pile obtained under similar conditions. A smooth pile would have failed nearly instantaneously under this load. The curves show a long primary creep portion, with some apparent tendency to steady state after about 100 hours. However, when the results of some most typical pile creep tests are plotted in a log-log plot, Fig. 2, after subtracting the instantaneous response, one gets rather the impression that the settlement of all the piles tended not to a steady state, but to a stationary creep with an attenuating trend. A similar conclusion was made by Weaver and Morgenstern (1981b) for piles in "ice-poor" soils, and by Luscher et al. (1983) on the basis of full scale tests on corrugated piles in Alaska.

This kind of creep test results can be processed in two different ways: First, one can concentrate only on the steady-state portion, or the minimum creep rate, neglecting the attenuating character of the curves. When the quasi-steady-state creep rates so obtained are plotted against the applied stresses in a log-log plot for a series of pile creep tests, it is usually found that the rate sensitivity of the shear strength along the shaft is rather low, i.e., the exponent of stress n in the creep equation Eq. (2) shown further is high. Figure 3 shows one such plot obtained from the present results with smooth and corrugated piles, containing also the results of rate-controlled tests. It will be seen that the lines drawn through the test points give the values of the exponents n of about n = 4 and 5, respectively. It is also found that the bearing capacity of corrugated piles is from 2 to 3.5 times higher than that of smooth piles at the same settlement rate. The above values of n compare well with those found by Parameswaran (1978, 1979) (n = 4.5 for steel) under similar test conditions. However, as stated by Ladanyi (1972), and Johnston and Ladanyi (1972), this kind of plotting has a practical value only if used in connection with the determined pseudo-instantaneous displacements, which include all the preceding elastic and primary creep strains. But, even then, the assumption of steady-state creep settlement of a pile usually leads to excessive settlement rate predictions and over-conservative design.

Another alternative is clearly to draw conclusions from the log-log plot of creep curves, as in Fig. 2. As shown in the following, these curves can be approximated by a more general creep equation, such as Eq. (2). Only very long term full-scale tests could tell which of the two methods will be able to give more correct long-term predictions.

Finally, in Fig. 4 the observed behaviour of the three kinds of piles is compared at two different rates of settlement. Their response is seen to be drastically different. The smooth piles failed at low stress in a brittle manner, loosing all their strength after a displacement of about 2 mm. The shaft resistance of corrugated piles continued to climb up to about 2 MPa, reaching its peak at about 5 mm, and decreasing slowly towards the residual, which after 12 mm displacement remained still at 50% of the peak strength. The tapered piles showed typically a small first peak at about 0.6 mm displacement, indicating the loss of adhesion as in a smooth pile test, but after this, their resistance continued to rise steadily without any sign of strength loss.

From a practical point of view, these test results show a clear-advantage of using corrugated piles in permafrost, rather than smooth piles, which follows also from the Alyeska Pipeline experience (Thomas and Luscher, 1980; Luscher et al, 1983). The reason is that corrugated piles show generally a non-brittle behaviour, using most of the natural soil shear strength. With a proper margin of

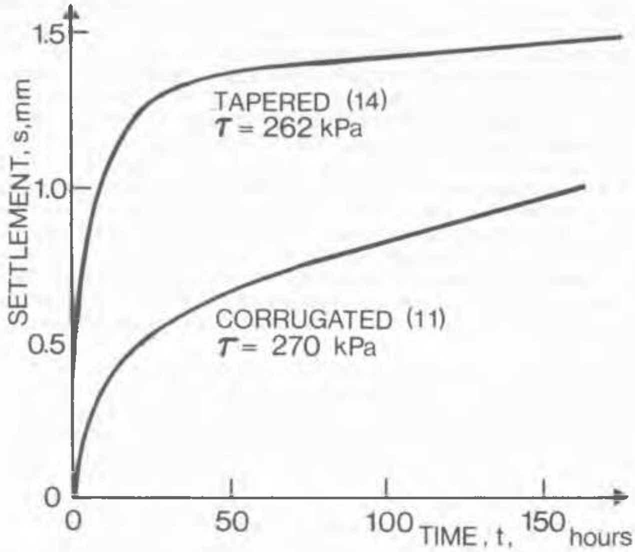

Fig. 1. Creep curves of a corrugated and a tapered pile under similar test conditions.

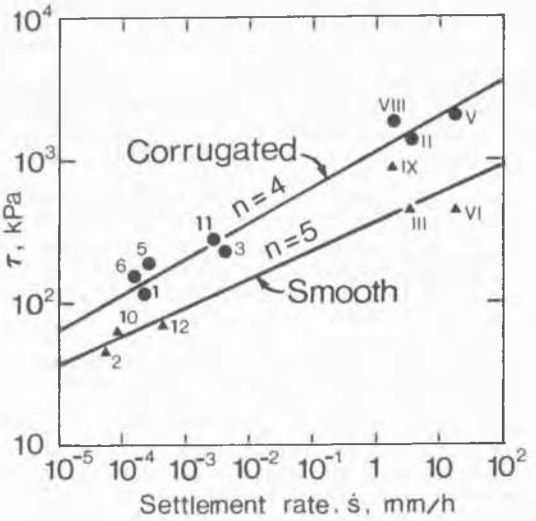

Fig. 3. Minimum creep rates vs. applied shear stress for smooth and corrugated piles.

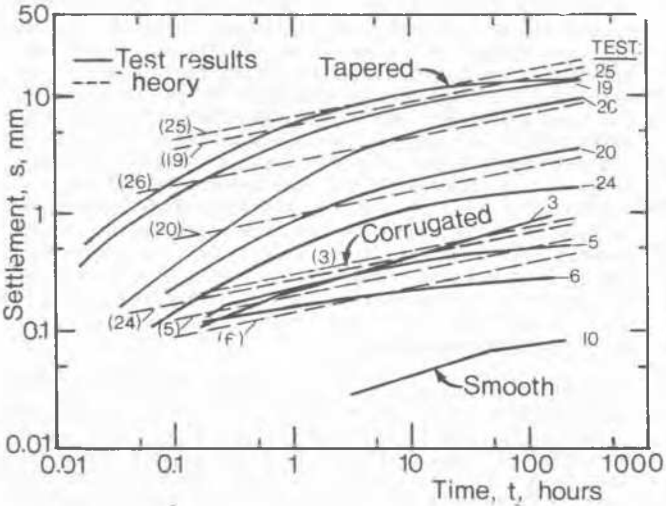

Fig. 2. Typical creep curves of smooth (10) corrugated (19-25), and tapered (3-6) piles, compared with the theory.

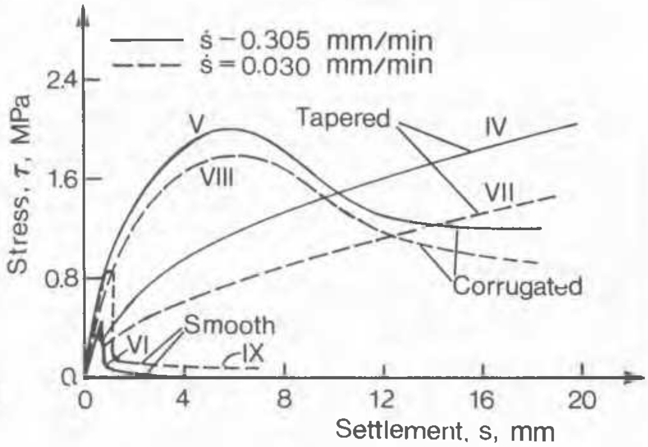

Fig. 4. Results of settlement-rate-controlled tests with three types of piles and at two different rates.

safety of at least 2, these piles will neither show excessive settlement, nor go into tertiary creep. The tapered piles are clearly the safest of the three, not requiring any safety factor against failure. However, they need 3 to 4 times larger displacements for attaining the same resistance as corrugated piles at the same rate. In other words, they may become economical only at large displacements, or if preloaded.

THEORY

Under an increase in stress, containing a substantial deviatoric component, an ice-saturated frozen soil will show an instantaneous response, both elastic and plastic, followed by creep, sometimes combined with consolidation. The creep is initially of a primary type, followed by a short steady-state portion, eventually accelerating towards failure, under favorable boundary conditions. It is usually assumed in frozen soil mechanics that total strain ε, resulting from a deviatoric stress increment, is composed of an instantaneous strain, ε_{inst} and a delayed strain, ε_{creep}

$$\varepsilon = \varepsilon_{inst} + \varepsilon_{creep} \tag{1}$$

Because of the relatively high rigidity of an ice-saturated frozen soil at short term loads, the instanteneous portion of the strain is much smaller than the creep portion, which keeps increasing with time. As this paper is mainly concerned with assessing the long-term behaviour of piles, attention will be concentrated only on creep strains. Besides, for an instantaneous elastic-plastic frozen soil response, the pile design methods developed in unfrozen soil mechanics remain valid.

A convenient form of a creep law for frozen soil, proposed by Ladanyi (1972), and extended to include the primary creep by Ladanyi and Johnston (1974), can be written as

$$\varepsilon_e = (\sigma_e/\sigma_{c\theta})^n \; A \; F_t \tag{2}$$

where subscript e denotes the von Mises equivalent stress and strain, $n \geq 1$ is the creep exponent for stress, and F_t is a time function. In this paper the power law of time $F_t = t^b$ with $b \leq 1$ was adopted as the time function, in which case: $A = (\dot{\varepsilon}_c/b)^b$, and $\sigma_{c\theta}$ denotes the reference stress at the arbitrary reference strain rate $\dot{\varepsilon}_c$, obtained from a test made at temperature θ (θ is the number of degrees C below 0^oC). In the two last-mentioned papers, it is also shown how $\sigma_{c\theta}$ can be made to incorporate the effects of temperature and normal pressure on creep. This power-law type of time function has been adopted in this paper.

Straight-shafted piles

As shown by Johnston and Ladanyi (1972), as long as there is no slip between the pile and the soil, the pile settlement s due to shear stresses only is equal to the cumulative shear deformation of the frozen soil at the contact with the pile. For a soil obeying the creep law of Eq. (2) with $F_t = t^b$, the pile settlement is given by

$$s = 3^{(n+1)/2}\left[D_m/2(n-1)\right]\,(\tau/\sigma_{c\theta})^n\,(\dot{\varepsilon}_c t/b)^b \tag{3}$$

where s is the settlement, D_m is the average pile diameter, τ is the average shear stress at the pile-soil interface, and the other parameters are defined as in Fig. 2. The range of validity of Eq. (3) depends on the roughness of the pile surface. For very rough grouted-rod anchors, described by Johnston and Ladanyi (1972), the slip started after a displacement of 2 to 3 cm. In this study, the slip occurred for smooth piles at 0.4 to 0.8 mm and for the corrugated ones at about 5 mm.

In the present study, the piles had no end resistance. In reality, the end resistance will be mobilized proportionally to the pile settlement and settlement rate, as described in Ladanyi and Johnston (1974) and Ladanyi (1983).

Tapered piles

When a tapered pile, as in Fig. 5, is pushed into the soil, its shaft resistance originates from the mobilized adhesion and friction along the shaft-soil interface, similarly as in the case of a straight-shafted pile. The main aspect by which it differs from the latter is that the lateral normal stress is not only a function of the state of stress in the ground but it also depends on the pile displacement. This stress provides a resistance to pile penetration, even if the pile is perfectly smooth. As the pile settles, it enlarges the hole in the ground, which mobilizes the lateral earth pressure proportionally to the ratio of the lateral displacement over the initial hole radius. As this ratio, at any given pile settlement, increases with depth, the lateral soil reaction will follow the same trend, as observed also in unfrozen soils (Bakholdin and Igon'kin, 1978).

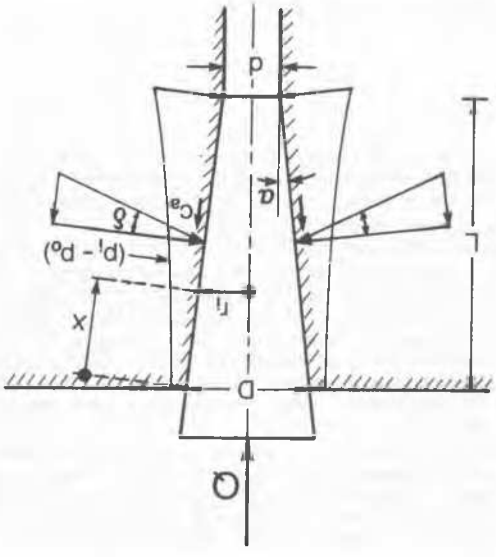

Fig. 5. Notation used in the theory of tapered piles.

In the following, a solution of this problem is proposed, taking into account the particular creep properties of frozen soils. Using the notations of Fig. 5, the total load Q that can be applied to a tapered pile with no end-resistance can be expressed as

$$Q = 2\pi\,(I + II + III) \tag{4}$$

where I is due to the adhesion, c_a:

$$I = \int_0^{L/\cos\alpha} c_a \cos\alpha \; r_i dx \tag{5}$$

II is due to the friction resulting from the friction coefficient $\tan\delta$ and the natural total ground stress p_o, assumed to act normally to the shaft surface:

$$II = \int_0^{L/\cos\alpha} p_o \;(\sin\alpha + \tan\delta\,\cos\alpha) r_i dx \tag{6}$$

Considering that

$$r_i = D/2 - x\sin\alpha \tag{7}$$

and assuming for the average values of p_o, c_a and δ to be constant along the shaft, these two integrals can be easily evaluated, to become

$$I = c_a\,L\,(D + d)/4 \tag{8}$$

and

$$II = p_o(\tan\alpha + \tan\delta)(D + d)/4 \tag{9}$$

The third integral has the form

$$III = \int_0^{L/\cos\alpha} (p_i - p_o)(\sin\alpha + \tan\delta\;\cos\alpha) r_i dx \tag{10}$$

where $(p_i - p_o)$ is the net frozen ground reaction mobilized by the hole expansion.

As shown in some earlier papers, (Ladanyi and Johnston, 1973), Ladanyi, 1976), for a frozen soil obeying the creep law of Eq. (2), this reaction is related to the radial hole expansion u_i by

$$p_i - p_o = C(u_i/r_i)^{1/n} \tag{11}$$

where

$$C = \sigma_{c\theta}(n/\sqrt{3})\left[(2/\sqrt{3})(b/\dot{\varepsilon}_c)^b\right]^{1/n} t^{-b/n} \tag{12}$$

Substituting Eq. (11) into Eq. (10) and taking into account that r_i is given by Eq. (7), while u_i is related to the pile settlement s by

$$u_i = s\tan\alpha \tag{13}$$

Eq. (10) can be integrated to give:

$$III = C\,\frac{1 + \tan\delta/\tan\alpha}{2 - 1/n}\left(\frac{D}{2}\right)^2\left(\frac{1 - d/D}{L}\right)^{1/n} \cdot \left[1 - (d/D)^{2-1/n}\right] s^{1/n} \tag{14}$$

Observing Eq. (12), it will be seen that the portion of the load Q due to hole extension

$$III = \bar{Q} = \frac{Q}{2\pi} - (I + II) \qquad (15)$$

is proportional to $(s/t^b)^{1/n}$, or inversely, that the pile settlement s is proportional to $(\bar{Q}^n t^b)$. On the other hand, expressing from Eq. (14):

$$s = K \bar{Q}^n t^b \qquad (16)$$

where K contains all the other terms in Eqs. (12) and (14), one finds for the settlement rate

$$\dot{s} = ds/dt = b K \bar{Q}^n t^{b-1} \qquad (17)$$

from which the time can be eliminated to give

$$\bar{Q} = (\dot{s}/b)^{b/n} s^{(1-b)/n} K^{-1/n} \qquad (18)$$

Eq. (18) shows that, for a constant settlement rate, the value of $\bar{Q}$ is proportional to the settlement s to the power of (1-b)/n.

It is interesting to note that, when the same considerations are made in connection with a tapered pile embedded in an elastic, laterally infinite, medium, Eq. (11) becomes

$$p_i - p_o = 2G u_i/r_i \qquad (19)$$

where $G = E/2(1+\nu)$ is the shear modulus of the medium. One gets then instead of Eq. (14)

$$III_{el} = \bar{Q}_{el} = G\frac{(D - d)^2}{2L}(1 + \frac{\tan\delta}{\tan\alpha})s_{el} \qquad (20)$$

from which the instanteneous settlement s_{el} of a tapered pile due to hole expansion can be evaluated.

Note that Eq. (4) with all the three sources of shaft resistance included, covers all the cases from a straight-shafted cylindrical pile to a conical pile with d = 0.

COMPARISON WITH OBSERVATIONS

When creep curves obtained from a set of step-loaded creep tests approximately linearize in a log time-log displacement plot, the preceding theoretical considerations show that their slope should give the value of the exponent b, while the value of the exponent n can be obtained from the displacement vs applied stress relationship at any given time, similarly as shown by Ladanyi and Johnston (1973) for pressuremeter creep tests. In addition, when a load-settlement curve at a constant penetration rate is plotted in a log-log plot, according to Eq. (18), its slope should give the value of the ratio (1-b)/n.

Using all these available sources of information, it was found that the results of the performed creep tests with corrugated piles (Fig. 2) could fairly well be approximated by Eq. (3), provided one takes: n = 1.6, b = 0.2 and $\sigma_{c\theta}$ = 3500 kPa at $\dot{\varepsilon}_c = 10^{-5}\text{min}^{-1}$. These low values of n and b are close to those found by others for dense frozen sand (Weaver and Morgenstern, 1981b), and they fall within the limits established by Ladanyi and Eckardt (1983) on the basis of pressuremeter tests performed in the same frozen sand. However, as in this study, they also found that the creep curves did not linearize in the log-log plot but had an attenuating character, with the value of the exponent b decreasing with time and increasing with the applied shear stress. Clearly, in such a situation the assumption of a time function of the form t^b with b = const. is not the most appropriate one and other alternative forms should be considered in future.

This remark is still more valid for the creep curves in Fig. 2 obtained from the tests with tapered piles. In that case, the curvature is continuous, and a time function of t^b type can give only a crude approximation of the results. Attention was therefore concentrated only on the last portion of the curves, where their slope is about $b \approx 0.2$, and their distance corresponds to $n \approx 1.6$. These values imply that in the rate-controlled tests the load-settlement curves plotted in a log-log plot should have the slope of (1-b)/n = 0.500, which corresponds quite well with the experimental results.

However, in order to be able to use Eq. (14) for comparison, some additional information on adfreeze strength parameters c_a and $\tan\delta$ was needed. The information on the bond of frozen sand to the smooth aluminium surface was drawn from two sources (Guichaoua, 1984): First, from the results of loading tests on smooth piles, and second, from special torque tests performed with tapered piles. The maximum and residual bond strengths found from the former were, $c_{a,max}$ = 400-800 kPa and $c_{a,res}$ = 50 to 100 kPa, while, $c_{a,max}$ = 140 kPa and, $c_{a,res}$ = 30 - 50 kPa were found from the torque tests.

As for the friction coefficient $\tan\delta$ between the metal and the frozen sand, it is most likely that it was not much smaller than in an unfrozen sand of the same density. According to some published data (Yoshimi and Kishida, 1981), the friction coefficient for the sand against aluminium varies from about 0.26 to 0.50, depending on the roughness.

The values of the two bond parameters adopted in the calculation of the behaviour of tapered piles according to Eq. (14) were: c_a = 50 kPa and $\delta = 20^o$ ($\tan\delta$ = 0.364). With these basic data on creep and strength parameters, Eq. (14) was used for approximating the last portions of the observed creep curves. This was found to be successful only if the value of $\sigma_{c\theta}$ = 13 MPa was adopted in the calculation. This value of $\sigma_{c\theta}$, back calculated from the test results, is much higher than expected, and it is also over 3 times higher than that back calculated from corrugated pile tests. Two possible explanations are offered for this high $\sigma_{c\theta}$ value: First, in a dense sand, the value of $\sigma_{c\theta}$ is higher, because it contains a frictional term (Ladanyi, 1972, Eq. 81), and second, a freezing prestress might have been created around the piles due to the confined freezing of slurry (Ladanyi, 1982). The two effects would affect particularly the tapered piles, which rely on the soil reaction generated by hole expansion.

CONCLUSIONS

This experimental study and theoretical considerations lead to the following conclusions concerning the behaviour of three different types of friction piles, installed in frozen sand in slurried holes.

(1) Smooth, straight-shafted piles fail in a brittle manner at low shear stress and low settlement. They loose nearly all of their strength after the peak.
(2) Corrugated piles fail in a semi-brittle manner, retaining about 50% of their peak strength at large displacements. At any given penetration rate, their peak strength is from 2 to 3.5 times higher than that of smooth piles. They are the most efficient of the three.
(3) Smooth-tapered piles behave in a hardening manner, gaining strength continuously with increasing displacement. However, for attaining the same strength as corrugated piles, they need 3 to 4 times larger displacements. They are the safest of the three.
(4) Theoretical predictions according to two different

theories appear promising but further work is needed for finding a more appropriate time function in the creep equation for frozen sand in this kind of loading.

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