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Parameswaran, V. R.

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by V. R. Parameswaran

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Creep of model piles in frozen soil

V. R. PARAMESWARAN
Geotechnical Section, Division of Building Research, National Research Council of Canada, Ottawa, Ont., Canada K1A 0R6

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Preliminary results of laboratory studies on the behaviour of model piles of wood, concrete, and steel, having different surface finishes, embedded in artificially frozen sand and subject to constant loads, are reported in this paper. The displacement rate $\dot{\gamma}$, was found to vary with the applied stress τ_f, according to a steady state creep equation of the type $\dot{\gamma} \sim (\tau_f)^n$, as found for other viscoelastic materials such as metals at high temperature. The value of the exponent n varied between 6.7 and 9.1 for various types of piles.

The results agreed favourably with field data obtained from pile pull-out tests carried out at Gillam and Thompson, Manitoba. The value of n from Gillam and Thompson tests, 8.05 and 7.5, respectively, were within the range of the present experimental values. Comparison with pile pull-out tests carried out in ice, and theoretical predictions for ice showed that the value of the exponent n was much larger for frozen soil than for ice. The bearing capacity at a particular displacement rate was also found to be almost an order of magnitude larger in frozen soil than in ice.

On rapporte ici les résultats préliminaires d'études de laboratoire sur le comportement de pieux modèles en bois, en béton et en acier, ayant des finis de surface différents, enfouis dans un sable gelé artificiellement et soumis à des charges constantes. On a établi que la vitesse de déplacement $\dot{\gamma}$ varie avec la contrainte appliquée τ_f suivant une loi de fluage en régime permanent du type $\dot{\gamma} \sim (\tau_f)^n$, semblable à celle valable dans d'autres matériaux viscoélastiques tels que les métaux à haute température. La valeur de l'exposant n varie de 6.7 à 9.1 pour différents types de pieux.

Les résultats concordent bien avec les observations de chantier faites au cours d'essais d'arrachement de pieux à Gillam et Thompson, Manitoba. Les valeurs de n déduites des essais de Gillam et Thompson, soit 8.05 et 7.5 respectivement, sont à l'intérieur du domaine des valeurs expérimentales de la présente étude. La comparaison avec des essais d'arrachement de pieux dans la glace et les modèles théoriques pour la glace ont montré que la valeur de l'exposant n est beaucoup plus grande dans le sable gelé que dans la glace. On a également établi que la capacité portante à une vitesse de déplacement donnée est pratiquement un ordre de grandeur plus élevé dans le sable gelé que dans la glace.

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Introduction

In a previous paper (Parameswaran 1978) laboratory measurements of the adfreeze strength between frozen sand and model piles, under constant rates of displacement were described. The tests were of short duration and the adfreeze strength at a particular rate of displacement of pile was calculated from the peak load prior to failure.

In practice, a pile foundation supporting a structure is subjected to constant loads, rather than constant rates of displacement. The process of deformation under such conditions is creep occurring in the soil at and adjacent to the interface between the pile and the soil. For the design of pile foundations in permafrost, it is important to know the total settlement that will take place during the service life of the structure and hence the relationship between the rate of settlement of the pile and the superimposed load must be known. Ideally, the quantity to be measured is the rate of settlement due to a constant load left on the pile for a long period of time in the field under controlled conditions, so that the rate of settlement eventually becomes constant. This is not possible, however, due to seasonal fluctuations in ground conditions and temperature. Several authors (Johnston and Ladanyi 1972, 1974; Tsytovich and Sumgin 1959) have reported creep tests of short duration conducted in the field. Crory (1966), Crory and Reed (1965), and Sanger (1969) have described the results of long-term pile loading tests in permafrost areas in Alaska. More recently, Nixon and McRoberts (1976) have reviewed the pile load test data available in the literature and have proposed a design approach for pile foundations in ice and ice-rich permafrost, based on a secondary (or steady state) creep law for ice.

Although the creep of frozen soils has been studied by several authors under various conditions of temperature and pressure (Sayles 1968; Sayles and Haines 1974; Chamberlain *et al.* 1972; Andersland and Akili 1967; Andersland and AlNouri 1970),

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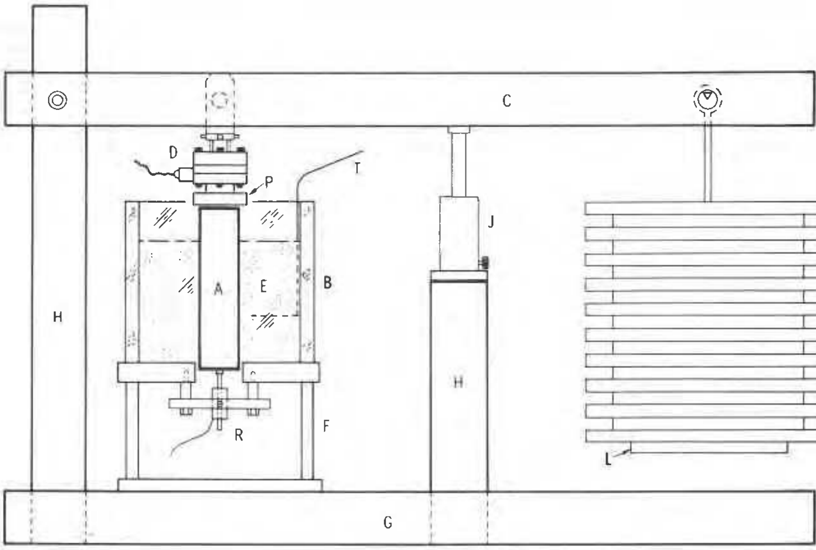

FIG. 1. Schematic diagram of the model pile creep test apparatus: A, pile; B, plexiglas box; C, loading arm (1:4 ratio); D, BLH (Baldwin–Lima–Hamilton) load cell; E, frozen sand; F, legs; G and H, rectangular steel tubing, 127 mm × 76.2 mm × 12.7 mm wall thickness; J, hydraulic jack; L, loading platform; P, loading platen; R, LVDT (linear variable differential transformer); T, thermocouple; W, weights.

there has been no systematic laboratory investigation undertaken to study the behaviour of pile foundations in frozen soil. An experimental programme is under way, therefore, in the permafrost laboratory of the Division of Building Research, National Research Council of Canada, to study the effects of various factors, such as temperature, type of pile and its surface properties, and type and characteristics of the soil, on the behaviour of pile foundations under short- and long-term loads. Some preliminary results from these laboratory studies in which model piles of wood, steel, and concrete, in frozen sand and a naturally occurring silty sand and subjected to constant loads, are presented in this paper. These results are also compared with the data available in the literature on field behaviour of piles and with theoretical predictions.

Experimental Method

The piles used in the study were B.C. fir, plain and creosoted, steel cylindrical pipes and H-sections, and concrete. The sand used was Ottawa sand (ASTM specification C-109, passing sieve No. 30 and retained on sieve No. 100) mixed with 14% by weight of water, which when compacted around the pile, had an optimum density of about 1700 kg m^{-3}, as determined by a standard Proctor test. Methods of preparing the piles and of moulding the Ottawa sand around them in the test box were the same as those described by Parameswaran (1978).

The constant-load creep apparatus used for the tests is shown in Figs. 1 and 2. The two horizontal legs, G, and the vertical members, H, were made from rectangular steel tubing, 127 mm × 76.2 mm × 12.7 mm wall thickness. The loading arm, C, was made from steel plates, 101.6 mm wide and 12.7 mm thick. After mounting the box containing the pile and frozen sand on the creep frame, the loading arm, C, and the platen, P, were adjusted so that the arm

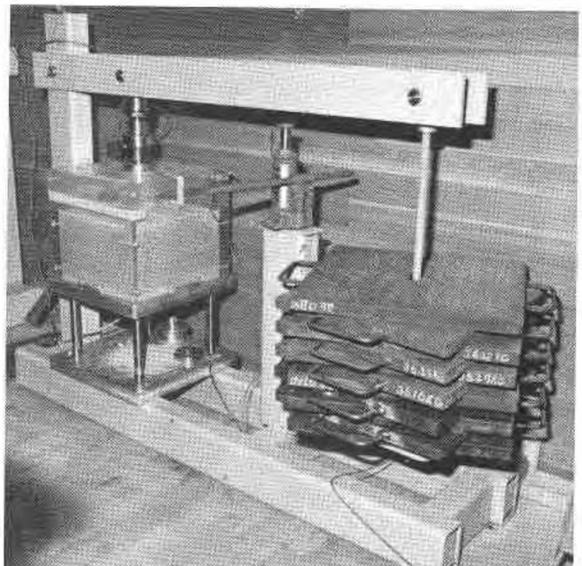

FIG. 2. Experimental set-up of a model pile creep test.

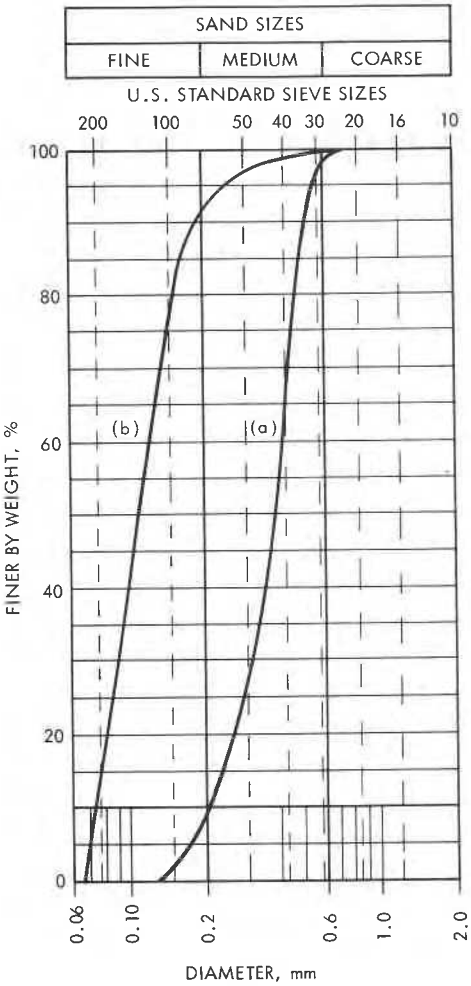

FIG. 3. Grain size distribution: (*a*) Ottawa sand, (*b*) silty sand from Norman Wells, N.W.T.

was horizontal and the platen was just above but not touching the pile, A. A hydraulic jack, J, was used to raise the arm. Weights made of 25.4 mm thick mild steel plates, each weighing about 36 kg and having dimensions 457.2 mm × 406.4 mm, with a slot cut to facilitate loading, were placed on the loading platform, L. After placing the necessary number of weights, the arm, C, was lowered by the hydraulic jack, J, so that the platen, P, rested on the pile. The weight, W, placed on the loading pan was enhanced by a factor of 4 at the pile due to the ratio arm, C. The load on the pile and the displacement of the pile were continuously monitored using a calibrated load cell, D, and a LVDT, R, both connected to a chart recorder.

In addition to the tests in frozen Ottawa sand, a few tests were carried out in a naturally occurring

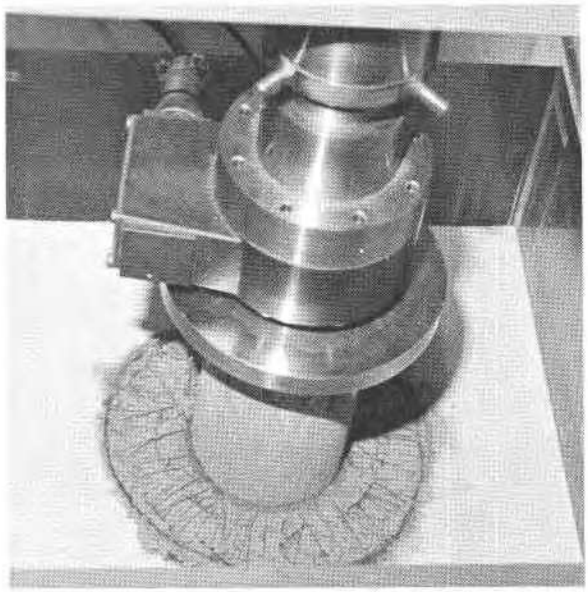

FIG. 4. Frozen silty sand around a pile, showing fine ice lenses.

silty sand obtained from Norman Wells, Northwest Territories. Figure 3 shows the grain size distribution of both the Norman Wells silty sand and the Ottawa fine sand.

The method of freezing the pile in the Norman Wells silty sand was as follows. Ottawa sand, mixed with 14% by weight of water, was placed and compacted around a plexiglas tube, 152.4 mm outside diameter and wall thickness of 12.7 mm, placed centrally inside the plexiglas moulding box shown in Fig. 1. This was done in five layers, each 38.1 mm thick. The box, B, with the sand and the plexiglas tube was placed in a cold room at −6°C for 4 days to allow the sand to freeze completely. The tube was then removed by warming the inner surface with hot water bags. A 76.2 mm diameter pile was then placed in the centre of the box and a thick slurry made of the dried Norman Wells silty sand mixed thoroughly with 20% by weight of water, was placed and compacted in the space between the pile and the frozen sand. This was done only inside the cold room. After compacting the slurry, the box assembly consisting of the pile, the slurry, and the sand was left inside the cold room for 3 more days for complete freezing. Thin ice lenses distributed radially around the pile were formed during freezing of the silty sand. This pattern of lenses is shown in Fig. 4.

All creep tests were carried out in a cold room maintained at −6 ± 0.2°C. Some of the tests were of short duration, i.e., less than 150 h, others ran for periods of up to 1800 h.

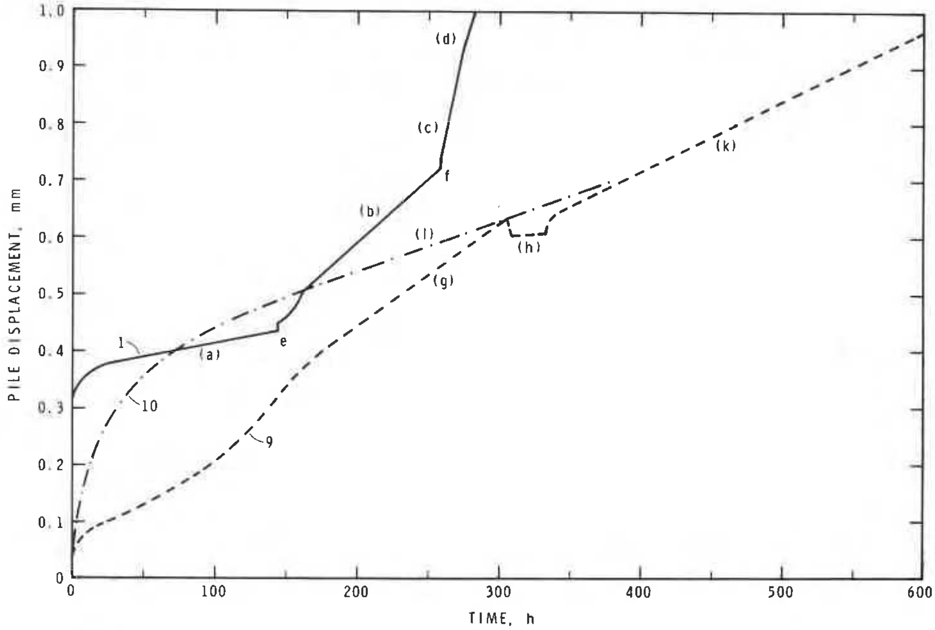

FIG. 5. Creep curves for two concrete piles (1 and 9) and a steel H-section pile (10).

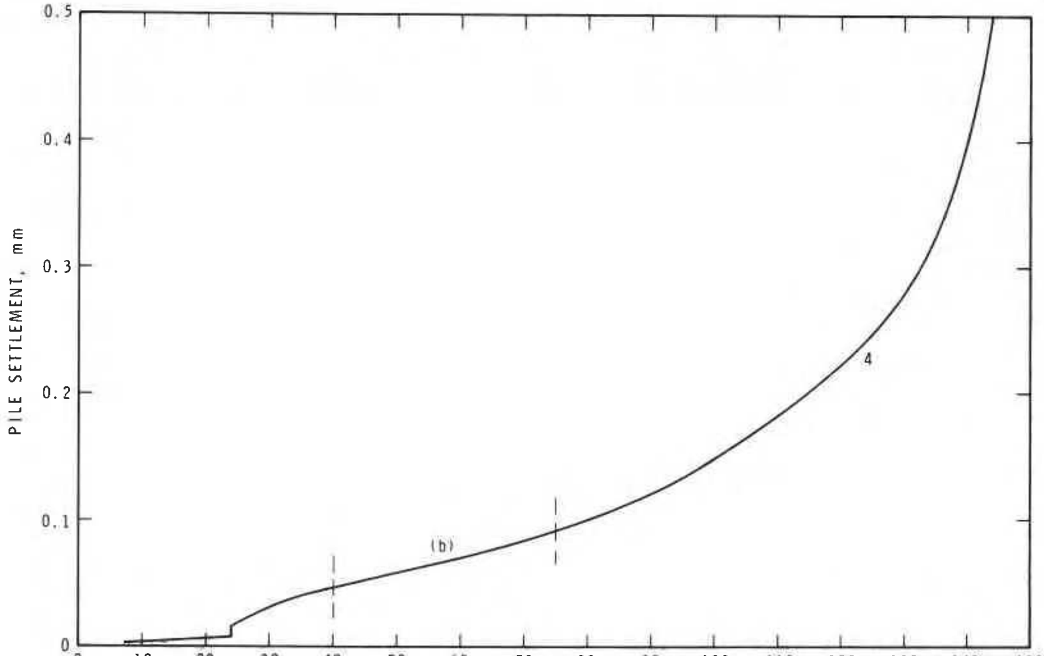

FIG. 6. Creep curve for a creosoted B.C. fir pile, (4).

Test Results

Figures 5–8 show typical creep curves obtained for different piles. Table 1 gives details of the loads applied, stress on the pile cross section (pile shaft stress), stress along the pile–soil interface, and creep rate in the steady state regions.

In Fig. 5, curve 1 shows the creep of a concrete pile in frozen sand. Steady state creep was observed in two regions, (a) and (b), prior to and after an increase in load point e. The load was further increased at point f and again, in regions (c) and (d), steady state creep was observed. Curve 9 shows the creep curve

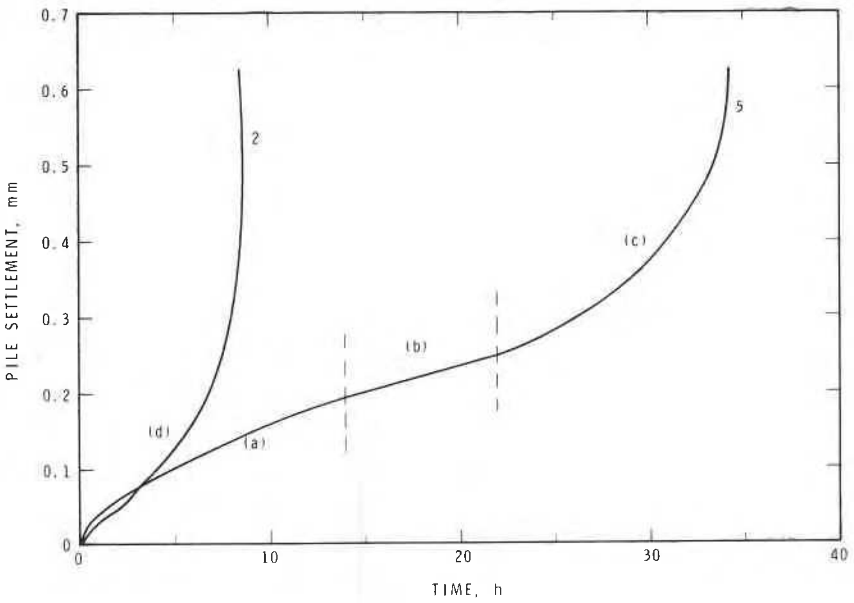

FIG. 7. Creep curves for two painted steel piles.

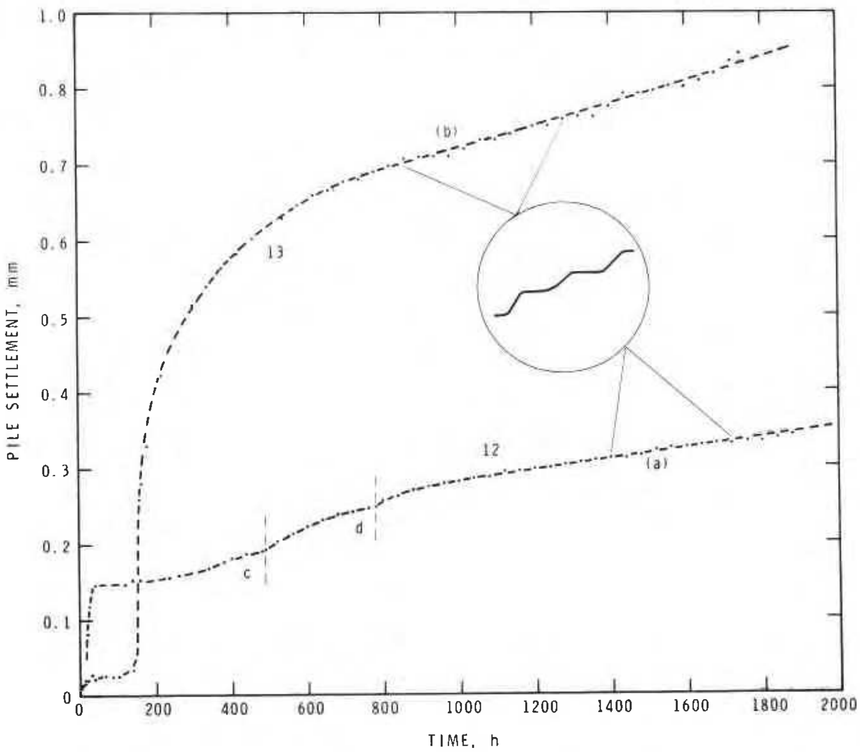

FIG. 8. Creep curves for an uncoated B.C. fir pile in frozen Norman Wells silt (12) and for a creosoted B.C. fir pile in sand (13).

for another concrete pile under a higher load. Although the initial displacement was smaller, the first steady state region, (g), shows a higher rate than for (a). In region (h), the temperature of the cold room was lowered from −6 to −10°C, and the curve shows a significant decrease in creep rate. After approximately 24 h at −10°C, the cold room was brought back to −6°C, and the creep rate increased

TABLE 1. Results of the constant load tests on piles

Material	Effective load on the pile (kN)	Pile shaft stress (MPa)	Stress along the pile soil interface (MPa)	Creep Rate (mm min^{-1})	Total creep at end of test (h)	(mm)	Remarks
Concrete (Fig. 5, curve 1), $T = -6$°C							
Region (a)	11.60	2.54	0.254	7.98×10^{-6}			
(b)	14.47	3.17	0.318	3.788×10^{-5}			
(c)	17.35	3.80	0.38	2.17×10^{-4}			
(d)	17.35	3.80	0.38	1.614×10^{-4}	284.5	1.032	Pile did not fail
Concrete (Fig. 5, curve 9)							
Region (g) $T = -6$°C	17.90	3.926	0.393	3.25×10^{-5}			
(h) $T = -10$°C	17.90	3.926	0.393	4.26×10^{-6}			
(k) $T = -6$°C	17.90	3.926	0.393	2.04×10^{-5}	550	0.895	Pile did not fail
Steel H-section (Fig. 5, curve 10), $T = -6$°C							
Region (l)	25.47	11.03	0.303	1.417×10^{-5}	380	0.698	Pile did not fail
Natural (uncoated) B.C. fir, $T = -6$°C							
	19.77	4.33	0.433	1.667×10^{-5}			
	26.10	5.723	0.572	3.71×10^{-4}	360	1.94	Pile failed
Creosoted B.C. fir (Fig. 6, curve 4) $T = -6.3$°C							
Region (b)	14.47	3.173	0.317	2.37×10^{-4}	146.5	0.60	Pile failed
Painted steel (Fig. 7, curve 2), $T = -6.1$°C							
Region (d)	13.93	3.054	0.305	4.792×10^{-4}	8.53	0.56	Pile failed
Painted steel (Fig. 7, curve 5), $T = -6.2$°C							
Region (b)	11.09	2.43	0.243	1.04×10^{-4}	34.25	0.614	Pile failed
Uncoated B.C. fir in Norman Wells silt (Fig. 8, curve 12), $T = -6$°C							
Region (a)	32.78	7.19	0.72	1.12×10^{-6}	1880	0.34	Pile did not fail
Creosoted B.C. fir in frozen sand (Fig. 8, curve 13), $T = -6$°C							
Region (b)	14.00	3.069	0.307	2.417×10^{-6}	1745	0.845	Pile did not fail

as shown in region (k). Curve 10 is for a steel H-section pile in frozen sand. After a primary creep stage of about 120 h steady state was attained in region (l) and continued until the end of the test. Figure 6 shows the creep curve for a creosoted B.C. fir pile in sand. After a short secondary stage, (b), the creep rate for the creosoted pile accelerated and led to failure of the pile by shear at the pile–sand interface.

Two creep curves for a painted steel pipe pile are shown in Fig. 7. The load on pile 2 was much higher than on pile 5. Both these tests were of short duration. Curve 5 exhibits a primary stage, (a), where the initial creep rate decreases to a steady value, a short secondary stage, (b), of about 8 h duration, and a tertiary stage, (c), where the creep accelerates until failure of the pile occurred after approximately 34 h.

Figure 8 shows the creep curves from two typical long-term tests extending to about 1800 h. Curve 12 shows the behaviour of an uncoated B.C. fir pile in the Norman Wells silty sand and curve 13 is for a creosoted B.C. fir pile in Ottawa sand. The creosoted pile shows a larger rate and larger displacement for a smaller superimposed load compared with the uncoated pile. Both these curves show that beyond about 800 h, although creep was progressing, there was no real steady state creep rate. The creep seemed to progress in steps, as shown by the inset in Fig. 8. Approximate overall creep rates in the regions (a) and (b) were calculated from the slopes of the dashed lines. Points c and d on 12 indicate the times when the load on the uncoated B.C. fir pile in Norman Wells silt was increased.

Discussion

The steady state rates of pile settlement calculated from Figs. 5–8 for various types of piles are plotted against the applied stress at the pile–soil interface in Fig. 9. It can be seen that the stress required to maintain a certain strain rate is least for painted steel (line (a), Fig. 9) and highest for natural B.C. fir (line (b)). This is consistent with the earlier observation (Parameswaran 1978) that at a particular displacement rate the adfreeze strength was highest for natural B.C. fir, least for painted steel and creosoted B.C. fir, and that concrete piles had adfreeze strengths between the two.

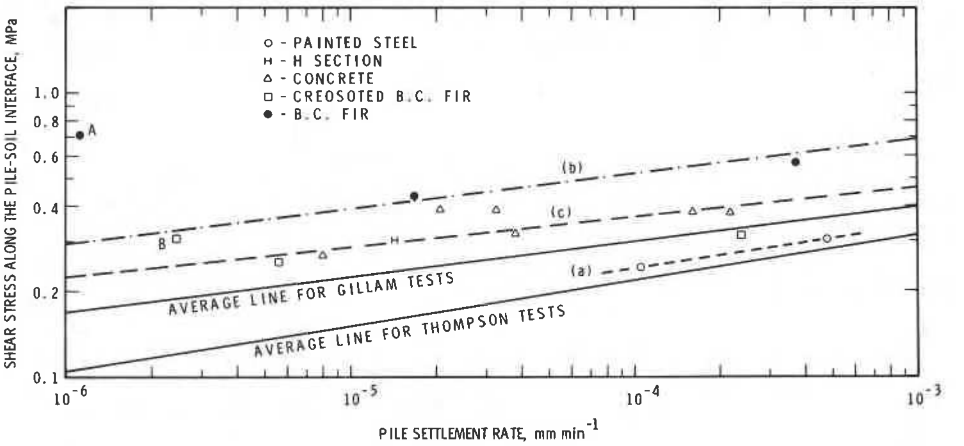

FIG. 9. Variation of pile settlement rate with shear stress along the pile–soil interface.

Points A and B in Fig. 9 are those calculated for the two long-term tests (about 1800 h) shown in Fig. 8. Point A corresponds to the average steady state creep rate of the B.C. fir pile in Norman Wells silty sand. This point lies above the line for B.C. fir in frozen sand. Point B in Fig. 9, although for a creosoted B.C. fir pile, shows a higher stress for a lower creep rate compared with the other points for creosoted B.C. fir piles. This is because the pile used in this test had a much smaller quantity of creosote than the piles used in the other tests, i.e., 45 kg m^{-3} compared with an average creosote content of about 100 kg m^{-3}. This shows that the higher the amount of creosote impregnated into the wood, the lower the load bearing capacity. Hence, for practical purposes the amount of creosote introduced into the surface of that portion of the pile to be embedded in permafrost should be kept to a minimum—just enough to prevent decay.

Curve 9 in Fig. 5 shows the effect of lowering the temperature during a test. The creep rate was reduced from 3.25×10^{-5} mm/min in region (g) to 4.26×10^{-6} mm/min in region (h), by a reduction in temperature from −6 to −10°C. Later, when the temperature was increased to −6°C the creep rate also increased in region (k), but only to 2.05×10^{-5} mm/min, a value slightly smaller than the original value observed in region (g). This could probably be explained by the reduction in the amount of unfrozen water in the frozen sand, as it was cooled from −6 to −10°C and warmed again back to −6°C. Williams (1963) has observed, by calorimetric measurements, that the unfrozen water content in clays as a function of temperature has, because of hysteresis, somewhat different values depending on whether the soil was freezing or thawing. In sands, even though the amount of unfrozen water and the hysteresis of phase change are much smaller than in clays, these could still exert an influence on the long-term strength of frozen sands.

Comparison with Field Data

In this section the data obtained in the present series of laboratory experiments are compared with field data obtained on the performance of piles in permafrost areas in northern Canada.

The three straight lines (a), (b), and (c) in Fig. 9, corresponding to painted steel, uncoated B.C. fir, and concrete, respectively, show that the steady state creep rate $\dot{\gamma}$ can be related to the applied shear stress τ_f, at the pile–soil interface, by an empirical equation of the type:

[1] $$\tau_f \sim (\dot{\gamma})^m$$

or

[2] $$\dot{\gamma} \sim (\tau_f)^n$$

where $n = 1/m$.

Equation [2] is analogous to the simple power creep law proposed for viscoelastic materials such as metals at high temperatures and frozen soils (Nadai 1963; Vialov 1965). The exponent n is characteristic of the material and is a function of temperature. The values of n calculated from the slopes of lines (a), (b), and (c) in Fig. 9 are 6.7 for painted steel, 8.1 for natural B.C. fir, and 9.1 for concrete.

Johnston and Ladanyi (1972) have applied an equation of the type [2] to analyse field data obtained from pile pull-out tests conducted at Gillam and Thompson, Manitoba. The value of n calculated for the Gillam tests was 8.05 and for the Thompson tests, 7.5. These values were for rod anchors grouted with cement and compare favourably with the values obtained from the present laboratory measurements.

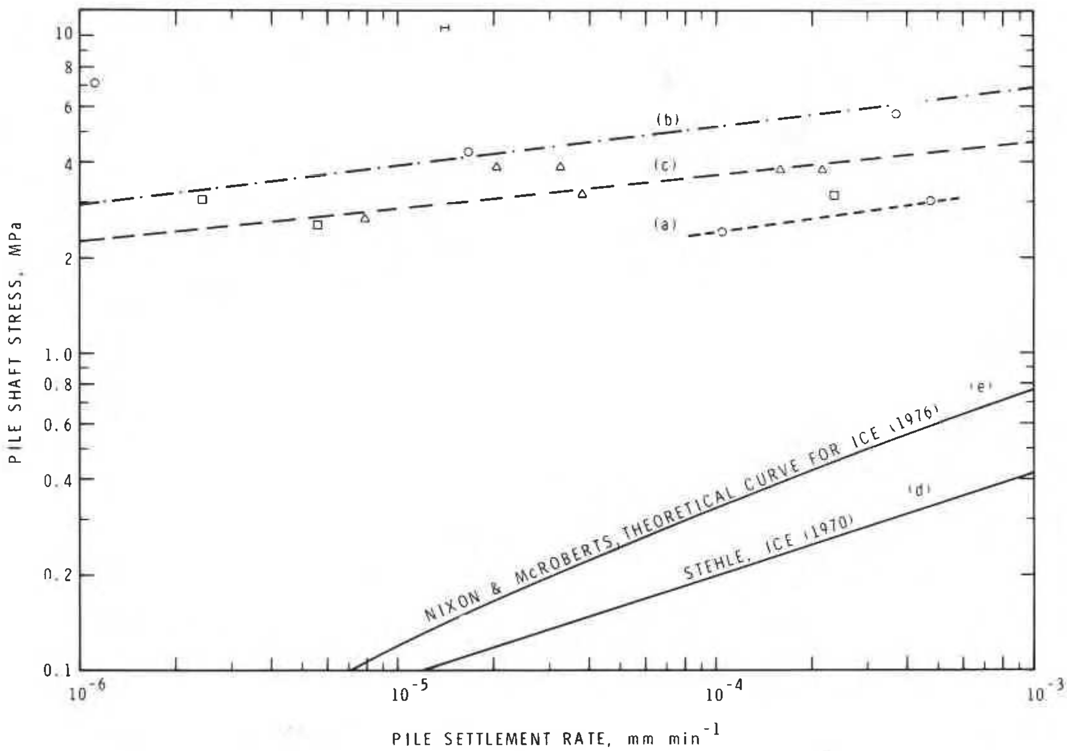

FIG. 10. Variation of pile settlement rate with pile shaft stress.

The average lines for the Gillam and Thompson tests are also shown in Fig. 9. The laboratory results lie close to these lines, although consistently above them. For a particular creep rate the stress values observed in the laboratory were higher than those observed in the field. The difference is mainly due to the lower temperature in the laboratory tests and to the difference in soil characteristics. The laboratory tests were carried out at −6°C in frozen sand of uniform grain size and composition, whereas the field tests were carried out in layered silty clay containing ice lenses, at temperatures around −2°C.

It is also worthwhile to compare the present experimental data with theoretical predictions based on the creep rate of ice. Figure 10 shows the data from the present series of experiments, lines (a), (b), and (c), and the data obtained by Stehle (1970*a*,*b*) from short-term pile tests in ice, line (d). Curve (e) is a theoretical prediction of the behaviour of a 304.8 mm diameter pile at −11°C in ice or ice-rich permafrost (Nixon and McRoberts 1976). According to their calculation, the curve (e) will be shifted upwards for a smaller diameter pile, but should be shifted downward parallel to itself, for a higher temperature. Figure 10 shows that the pile shaft stress for a particular rate of displacement of a pile is considerably higher in frozen sand with 14% ice distributed homogeneously than in ice or ice-rich permafrost. This is obviously due to the mobilization of soil grain friction in the frozen sand in addition to the adhesion of ice to the pile surface. The theoretical curve (e) was derived by taking into account the creep occurring in ice only. The slopes of the line (d) and (e) are also considerably different from those for piles in frozen sand. Whereas the exponents of the power law [2] relating the pile displacement rate to the stress were found to be 6.7, 8.1, and 9.1, respectively, for lines (a), (b), and (c), the value of the exponent calculated for line (d) for pile displacement rate in ice was 3.1. The theoretical curve (e) of Nixon and McRoberts was derived from an equation of the type

$$[3] \qquad \dot{u} = A(\tau_a)^{n_1} + B(\tau_a)^{n_2}$$

where $\dot{u}$ is the displacement rate, τ_a is the pile shaft stress, and A, B, n_1, and n_2 are constants for a particular temperature. The values of the exponents n_1 and n_2 obtained by them for various temperatures were

Temp. (°C)	n_1	n_2
0	1.34	4.0
−2	1.72	4.0
−5	1.92	4.0
−11	2.12	4.0

These values are considerably smaller than those observed for piles in frozen sand. This indicates that for a particular displacement rate the load bearing capacity of piles in frozen sand is larger than in ice.

Conclusion

The results of preliminary laboratory creep tests carried out on model piles in frozen sand at a temperature of $-6°C$ indicate that the rate of displacement of piles in frozen sand under constant loads has a power law creep rate dependence on the applied shear stress at the pile–soil interface, similar to high temperature steady state creep of viscoelastic materials. Calculated values for the exponent n agree favourably with those obtained from field tests on piles at Gillam and Thompson, Manitoba. On a log–log plot of stress vs. creep rate the laboratory results lie close to the average line for the Gillam and Thompson tests. The close agreement observed between the laboratory tests on model piles and field tests with larger piles provides encouraging support for further long-term laboratory tests.

Lowering the test temperature from -6 to $-10°C$ decreased the steady state creep rate of a pile in frozen sand by almost an order of magnitude.

Comparison of the present data with the data for piles in ice and with theoretical predictions using the creep law for ice showed that for a particular rate of displacement the stress required for piles in frozen sand of optimum density is considerably higher than for pure ice.

Acknowledgements

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