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Publisher's version / Version de l'éditeur:

Foundations in permafrost and seasonal frost: proceedings of a session, pp. 16-28, 1985

National Research Council Canada
Conseil national de recherches Canada

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Reprinted from
Proceedings of a Session, "Foundations
in Permafrost and Seasonal Frost", p. 16 - 28
ASCE Spring Convention
Denver, CO, April 29 - May 2, 1985.

DBR Paper No. 1289
Division of Building Research

Price $1.25 OTTAWA NRCC 24534

Canada

RÉSUMÉ

Cette communication analyse les résultats de plusieurs essais de fluage à long terme sur des pieux dans les gélisols. L'auteur y constate que c'est le fluage à vitesse décroissante qui prédomine dans la plupart des cas. Pour une conception rationnelle des fondations sur pieux dans les zones de pergélisol, cette caractéristique doit être prise en considération.

Reprinted from Proceedings of a Session
Sponsored by TCRR Council, ASCE/Spring Convention,
Denver, CO, April 29, 1985

ATTENUATING CREEP OF PILES IN FROZEN SOILS

V.R. Parameswaran*

ABSTRACT

Results of several long-term creep tests of piles in frozen soils are analyzed; the primary or attenuating creep is predominant in most cases. For effective design of pile foundations in permafrost areas, this aspect should be taken into consideration.

INTRODUCTION

Pile foundations are used extensively in permafrost areas to isolate buildings from the frozen ground by providing an air space between the two, thereby preserving the frozen state of the ground. Timber piles are most commonly used, since they are usually readily available and are relatively simple to install compared to other foundations. For special purposes, steel and concrete piles are also used in some instances.

A pile foundation bears the superimposed loads (static due to the weight of the structure, and dynamic due to moving loads and vibrating machinery within the structure) by two mechanisms: adfreeze bond between the pile and frozen soil, and end bearing. In ice-rich soils, end bearing is usually neglected and design of pile foundations is based mainly on the static adfreeze strength. The effect of dynamic loads is also neglected; it is probably compensated for by the factor of safety usually incorporated in the design.

The soil adjacent to the pile in frozen ground is subjected to a constant mean stress and undergoes long-term creep. Design of pile foundations is based on an allowable settlement for a structure during its life, from which an average allowable settlement rate can be calculated. The objective of any good design is to arrive at a value of allowable stress that can be borne by the pile foundation, without exceeding the total allowable settlement during the life span of the structure. Several methods have been used to calculate such an allowable design stress.

Adfreeze strength from short-term tests

A few quick tests are carried out in the laboratory on piles frozen into different soils, by loading them at different rates, and determining the peak adfreeze strength values (15). By plotting the peak adfreeze strength versus the displacement rate and extrapolating

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the curves back to the desired (allowable) settlement rate for a structure, an allowable peak adfreeze strength is obtained. Previously this value was used together with a suitable factor of safety (9,24). Black and Thomas (3) described prototype pile tests of 72 hours' duration in permafrost soils, and calculated an ultimate bearing strength as that stress at which the test pile attained a displacement of 0.5 mm (0.02 in) in the last third of the test period. This corresponds to a displacement rate of 3.47×10^{-4} mm/min (8.2×10^{-4} in/hr), which is much larger than the usual allowed settlement rate of about 25.4 mm (1 in) during the life of a structure. If the anticipated life of a structure is 25 years, the allowable settlement rate will be about 2×10^{-6} mm/min (4.72×10^{-6} in/hr). [For calculations based on such allowable settlements see (1,14,22)].

The danger in this method of extrapolation of data from short-term high-rate tests is that the allowable stress obtained by extrapolation to the desired settlement rate is always much higher than the stress that the pile foundation can withstand under long-term constant load creep conditions. Thus, the allowable stress is overestimated by this technique and this could lead to premature failure. This was shown by comparing the results from long- and short-term tests carried out in the laboratory (17). However, in winter when the ground temperature of the active layer is much colder than that of the perennially frozen ground underneath, the piles have a much higher bearing capacity than the allowable design values.

Long-term creep tests

A logical alternative to the previous method is constant load creep tests carried out in the laboratory or in the field for sufficiently long periods that a rate comparable to the desired settlement rate in the field is obtained. From a plot of stress versus steady-state creep rate, the allowable stress for a desired settlement rate is obtained.

This technique is normally used by designers, but the design is based entirely on the secondary or steady-state creep rate. The instantaneous creep or settlement and the primary stage, where the rate of settlement decreases with time, are neglected. These are assumed to be very small compared to the total creep in the steady-state regime (11,13), based on the assumption that long-term pile behaviour under static load is analogous to the behaviour of viscoelastic materials (10). However, the primary creep regime continues for a considerable period of time, especially under low loads. This has also been observed in field situations (3,12,20,23).

Failure time

In the method suggested by Vyalov (25), the difficulties of the two previous methods are somewhat eliminated by considering a "failure time" defined as the time required for the onset of tertiary or accelerating creep at the end of the steady-state regime. For frozen soils at a particular temperature this time (t_f) is a function of stress (σ):

$$\ell n\ (\frac{t_f}{B}) = \beta/\sigma \qquad (1)$$

with B and β characteristic constants for a material. Thus, from a plot of $\ell n(t_f)$ vs $1/\sigma$ for various temperatures, the stress corresponding to a desired life span can be obtained (see also Sayles, 21). Vyalov's method thus takes into account the instantaneous displacement, and the primary and secondary creep regimes; hence, it could be the most suitable model to predict the design life under a particular load or vice versa. However, the empirical parameters B and β have to be determined from many tests carried out in different soils.

It would be convenient to have an analytical model to predict the total creep behaviour of the soil at the pile/soil interface. However, simple creep laws as applied to pure materials such as metals or ice cannot be applied to frozen soil for the following reasons: (a) the nonhomogeneous nature of frozen soil and the nonuniform distribution of ice lenses in the material, and (b) various processes such as viscoelastic behaviour of ice, dislocation plasticity within each ice grain, grain boundary viscous flow or rigid body rotation of grains, diffusion through the unfrozen water in the soil, and intergranular soil friction. [For a general review of the physico-mechanical processes occurring in frozen soils, see (2)].

LABORATORY STUDIES ON THE BEHAVIOUR OF PILES IN FROZEN SOILS

The experimental apparatus and procedure used to measure the displacement of piles in frozen soils in the laboratory under static and dynamic loads are given by Parameswaran (16,18).

Typical creep curves show the displacement of various piles in different frozen soils (figures 1(a) to 3(a)). In all these figures, the primary or decelerating creep regime extends over more than half the period required for the onset of tertiary creep or failure. In some cases, the piles were entirely in the primary creep regime without ever attaining a steady-state regime.

Figures 1(b) to 3(b) show the variation of displacement rates with time, corresponding to the creep curves in figures 1(a) to 3(a). Again, primary creep is dominant for very long times, as indicated by the continuous decrease in creep rates during the test period. The abrupt peaks seen in some of these curves correspond to an increase in the static load on the pile, hence, an increase in the stress at the pile/soil interface. These are also indicated on the creep curves.

For various piles tested in the laboratory, the total displacement (including instantaneous) in the primary creep region was much larger than that in the secondary creep region, especially for wood piles. Similar behaviour of the dominant primary creep regime for piles in frozen soils was observed by previous workers, from pile load tests carried out in the field (3,12,20,23).

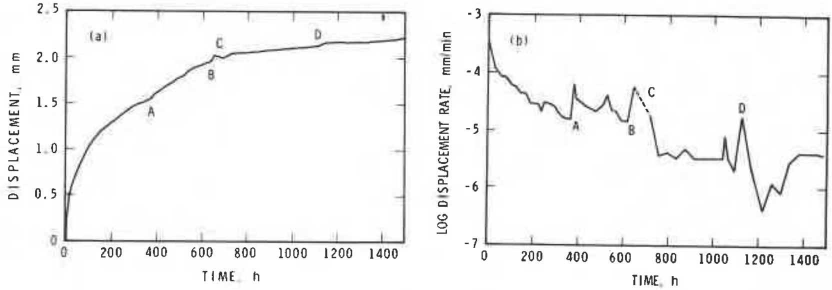

Figure 1(a) Creep curve showing displacement with time for an uncoated B.C. fir pile in frozen sand (average grain size: 0.2-0.6 mm (0.008-0.02 in); moisture content: 14% by weight of dry sand; T = -2°C (28.4°F). Stress (τ) at pile/soil interface was 0.238 MPa (34.52 psi), except in region A to B of the curve, where τ was 0.27 MPa (39.16 psi). At C and D, temperature fluctuations occurred in the cold room.

(b) Variation of displacement rate with time for creep curve in Figure 1(a).

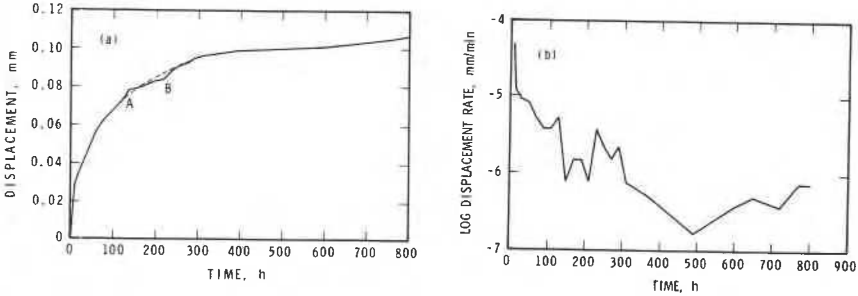

Figure 2(a) Creep curve for creosoted B.C. fir pile in silty soil from Northwest Territories (moisture content: 20%; T = -2.5°C (27.5°F); τ = 0.1903 MPa (27.60 psi); temperature fluctuations at A and B).

(b) Variation of displacement rate with time for creep curve in figure 2(a).

Discussion

Analogous to the total creep strain of a material, the total displacement ($\Delta \ell$) of a loaded pile in frozen soil can be represented by an equation:

$$\Delta\ell = \Delta\ell_0 + \Delta\ell_1 + \Delta\ell_2 + \Delta\ell_3 \qquad (2)$$

where the four terms on the right side represent the instantaneous displacement, and the displacements in the primary, secondary and tertiary creep regions, respectively. In this discussion only the primary or attenuating creep behaviour of piles in frozen soils will be considered.

Several equations relating the strain (ε) and time (t) have been used to describe the primary creep behaviour of viscoelastic materials such as metals, rocks, concrete, and ceramics (see Pomeroy, 15, for a review). Some of these equations are given below:

Power law: $$\varepsilon = At^n \qquad (3)$$

Logarithmic law: $$\varepsilon = B_1 + B_2 \ln(t) \qquad (4)$$

Hyperbolic law: $$\varepsilon = \frac{t}{at + b} \qquad (5)$$

Exponential law: $$\varepsilon = C[1 - \exp.(-Dt)] \qquad (6)$$

In these equations the constants A, n, B_1, B_2, a, b, C and D are characteristic of the material.

The data from the present pile creep tests in frozen soils were fitted to equations (3) to (6), with the strain (ε) replaced by the pile displacement ($\Delta\ell$), using a standard programme available with a desk top computer. Besides these four equations, a polynomial of the type:

$$\Delta\ell = \sum_{k=0}^{5} A_k t^k \qquad (7)$$

was also fitted to the data.

Once a curve fit was selected, the regression values were calculated. The quality of fit achieved by regression was given by the value of the 'coefficient of determination' r^2, given by:

$$r^2 = \frac{[n\sum(\Delta\ell \cdot t) - \sum\Delta\ell \cdot \sum t]^2}{[n\sum t^2 - (\sum t)^2]\,[n\sum(\Delta\ell)^2 - (\sum\Delta\ell)^2]} \qquad (8)$$

where r is the sample correlation coefficient (for details see Crow et al., 6). Theoretically, the closer the value of r^2 is to 1, the better the fit.

Values of r^2 (the coefficient of determination) obtained by fitting the data from several tests to the five equations (3 to 7) showed that, although the polynomial equation gave the highest value of r^2 for most of the tests, there was a tendency for the polynomial curve to oscillate about the actual creep curve; hence, it was not considered a good fit. For most of the tests, the power law (equation 3) fit quite well and for some tests the hyperbolic law (equation 5) fit well. In all cases, the equations fit the

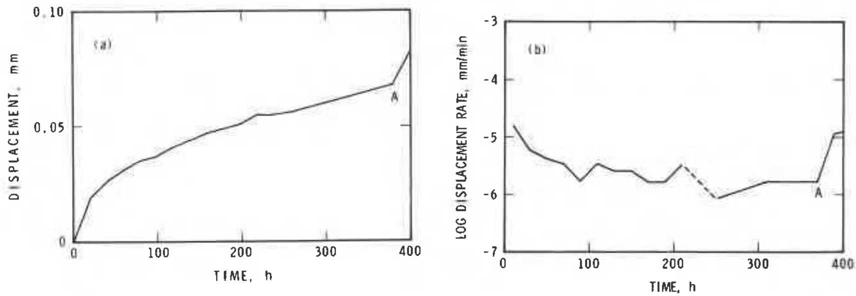

Figure 3(a) Creep curve for uncoated B.C. fir pile in clay from Thompson, Manitoba (moisture content: 50% by weight of dry soil; T = -2.5°C (27.5°F); τ = 0.0474 MPa (6.87 psi). At A the displacement rate increased due to superimposed dynamic loads.)

(b) Variation of displacement rate with time for creep curve in figure 3(a).

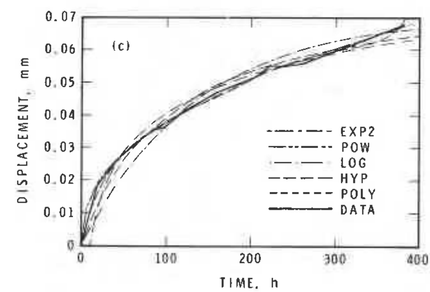

Figure 3(c) Curve fitting of the data of figure 3(a) using equations (3) to (7). Thick line shows actual data.

observations reasonably well, the maximum deviations being less than ±5%. The values of various constants used in equations (3) to (6) obtained from the test data are given in Table I.

Figure 3(c) shows the result of the creep test presented in figure 3(a), and the best fit of the primary creep equations (3 to 7).

Figures 4 and 5(a) show two typical long-term creep tests with load increments at intervals, and figure 5(b) shows the displacement rates as a function of time corresponding to 5(a). The piles are under primary or attenuating creep regime throughout the duration of the tests. The spikes (A', B', C') in 5(b) correspond to the load

TABLE I. Values of r^2 and the constants in equations (3) to (6) for various tests

Pile and soil type	Test No.	Mean stress at pile/soil interface MPa (psi)	Value of r^2 for the power law (Eq. 3)	Values of the constants in equations (3) to (6)							
				A	n	B_1	B_2	a	b	C	D
Uncoated BC fir in frozen sand (See fig. 1a)	46	0.238 (34.52)	0.982	0.203	0.342	-0.759	0.407	0.414	63.16	2.213	0.0035
Steel H-section in frozen sand (-6°C) (21.2°F)	10	0.303 (43.95)	0.984	0.073	0.386	-0.100	0.123	1.385	69.29	0.699	0.010
Creosoted BC fir in silty soil (See fig. 2a)	27	0.190 (27.56)	0.952	0.017	0.293	-0.019	0.019	8.833	544	0.108	0.0073
Uncoated BC fir in sand	44	0.300 (43.51)	0.994	0.058	0.51	-0.425	0.236	0.824	70.58	1.001	0.010
Concrete in sand	48	0.182 (26.40)	0.973	0.011	0.632	-1.12	0.290	0.539	565	1.408	0.001
Concrete in sand	55	0.178 (25.82)	0.998	0.070	0.504	-0.153	0.198	0.809	42.74	1.231	0.010
Uncoated steel pipe in sand (-6°C) (21.2°F)	62	0.366 (53.08)	0.994	0.045	0.335	-0.044	0.057	3.218	116	0.287	0.015
Uncoated BC fir in Thompson clay (See fig. 3a)	89	0.047 (6.82)	0.998	0.005	0.428	-0.037	0.017	12.22	1293	0.071	0.0074
Dense tropical wood in Thompson clay (-2.5°C) (27.5°F)	92	0.047 (6.82)	0.960	0.002	0.560	-0.040	0.016	11.55	1969	0.073	0.0048

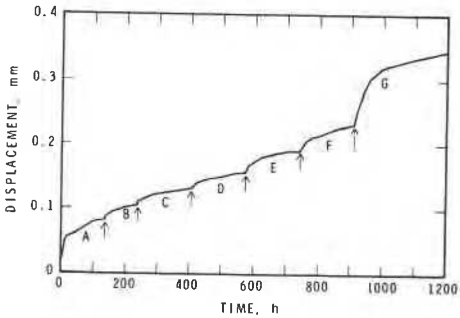

Figure 4 Result of long-term creep test of uncoated B.C. fir pile in frozen sand at -2.5°C (27.5°F), with load increments at points shown by arrows. Stresses in different regions shown in Table II.

increments. Equations (3) to (7) were again fitted to the data along the segments of the creep curves for different loads, by shifting the coordinate axes to the point of the load increments. The power law equation gave the best values of r^2. (The polynomial was again excluded because of oscillations about the measured creep.) The values of the exponent (n) as well as the stress corresponding to each segment of the creep curves in figures 4 and 5(a) are given in Table II.

Since the power law (equation 3) fit the primary creep region well, the values of the exponent (n) obtained for various tests were plotted against stress (figure 6), to determine any stress dependence of this parameter. The scatter in the data is large, especially in the low stress region, and the exponent (n) does not seem to depend on the shear stress (τ) at the pile/soil interface. An average value of n was about 0.46, with a standard deviation of ±0.125.

In the classical power law creep equation proposed by Andrade, and applied to creep of metals, rocks, etc. (19), the value of the exponent (n) is 0.33. For ice also, Glen (7) found a value of n = 0.33 to fit his creep data. Several other workers (4,5,8) found, however, that a value of n ≈ 0.5 gave a better fit to the primary creep for granular ice, columnar grained ice and for single crystals of ice oriented for nonbasal glide. The value of 0.46 observed from the present creep data on frozen soils is close to this latter value, which indicates that the creep of ice rich frozen soil at temperatures close to the melting point of ice is governed essentially by the creep of ice.

The scatter of the results shown in figure 6 and the wide variability of the values of other constants shown in Tables I and II

TABLE II. Values of stress and the power law exponent (n) for various segments of creep curves in figures 4 and 5(a)

	Segment of curve					
	4(A)	4(B)	4(C)	4(D)	4(E)	4(F)
Stress (τ) at pile/soil interface MPa (psi)	0.523 (75.86)	0.604 (87.60)	0.644 (93.41)	0.682 (98.92)	0.722 (104.72)	0.762 (110.52)
n	0.20	0.37	0.44	0.50	0.40	0.47

	Segment of curve				
	4(G)	5(A)	5(B)	5(C)	5(D)
Stress (τ) at pile/soil interface MPa (psi)	0.802 (116.32)	0.121 (17.55)	0.152 (22.05)	0.181 (26.25)	0.243 (35.24)
n	0.47	0.41	0.61	0.65	0.61

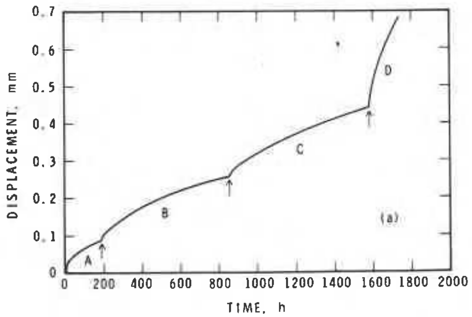

Figure 5(a) Long-term creep curve for uncoated B.C. fir pile in Thompson clay at -2.5°C (27.5°F). Arrows indicate load increments. Stresses in regions A to D shown in Table II.

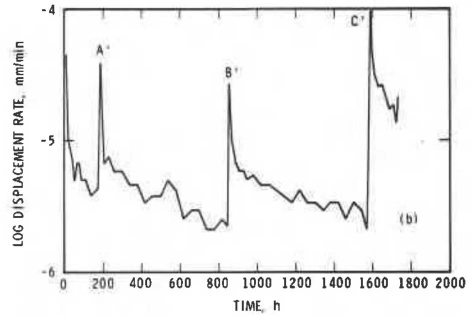

Figure 5(b) Variation of displacement rate with time for creep curve in figure 5(a).

point out the variability of parameters in nonhomogeneous frozen soils. It appears that a general creep equation cannot be obtained that will predict with accuracy the behaviour of pile foundations in frozen soils. Each test gives its own characteristic values; thus large safety factors become imperative in designing foundations in frozen ground. In spite of the scatter and variability in the values of the creep parameters, the results presented here show that primary or attenuating creep is the dominant regime to be considered in the design of pile foundations in frozen ground for the long-term support of structures.

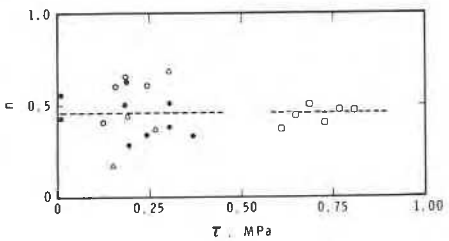

Figure 6 Variation of power law exponent (n) with stress.
(○, □, △) data from three different tests with step loading
(●) individual constant load creep tests

Conclusions

Long-term creep tests carried out in the laboratory using different piles embedded in various frozen soils showed that, for stresses in the range 0-1 MPa at the pile/soil interface, the dominant regime is primary or attenuating creep, where the displacement rate decreases continuously with time. Different primary creep equations proposed to describe the attenuating creep of viscoelastic materials, such as power law, logarithmic, hyperbolic, exponential, and polynomial equations, were fitted to the data from the present pile creep tests. Although no unique parameters could be derived by curve fitting, the power law equation closely fit most of the observations. The average value of the power law exponent (n) obtained was 0.46.

For proper design of pile foundations in permafrost areas, the primary creep regime has to be considered in detail, and not the steady-state regime only.

Acknowledgements

The author wishes to express his sincere thanks to Colin Hubbs for conducting the experiments, and to Douglas Bright for computation and plotting. This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

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