An artificial permafrost test plot

Un plot d'essai artificiel de permafrost

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SYNOPSIS: Many of the overseas construction projects under-taken by Finnish firms take place in areas with permafrost. In order to increase our experiences of building in such areas, an artificially frozen test plot has been set up in Oulu, northern Finland. The construction work began in 1985, and research is due to reach completion in 1989. The aim is to generate information to enable new foundation techniques to be developed for use in permafrost areas and also in areas with a seasonal frost layer in the ground (depth range 1 - 4 m). The test plot has been used to experiment with different methods of working with frozen soil, to study the freezing process by means of temperature measurements and to make comparative calculations of the advance of the frost in the ground using the finite element method.

Both piles cast in situ and piles driven into the ground have been tested, the former involving the filling of holes drilled into the frozen soil with antifreeze concrete. Both short piles inserted into a given soil horizon and longer ones passing through several horizons have been used. The present paper reports on changes in the strength of these piles and of the bond between the pile and the soil in the course of loading experiments. A total of 22 piles were inserted in the test plot and subjected to loading at ages of one month, six months and one year.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE TEST PLOT

The location of the test plot is shown in **Fig.1.**

Figure 1. The location of the test plot

The area around it is even and of an average height of +6.50 m a.s.l. The granulated sedimentary bedrock of the area is covered by several tens of metres of loose deposits comprising silt, clay and non-cohesive material. The surface soil consists of 0.5...0.7 m of post-Littorina silt and sand of glaciofluvial origin, underlain by a 1 m horizon of dark grey or black silty clay deposited as a symmict in a saline environment during the Littorina phase of the Baltic Sea. This horizon contains over 2% humus and some iron sulphide.

Below the sulphide horizon is a layer of approx. 0.8 m of grey clay laid down under freshwater conditions during the Ancylus Lake phase of the Baltic, while beneath this, from the +3.80 level downwards in the test plot, are several metres of varved reddish-grey Yoldia silt containing some fine silt (borings extended down to the -3 m level, i.e. 10 m from the surface).

The various horizons can be regarded as normally consolidated in geotechnical terms. The unit weight of the surface horizon is 20 kN/m^3, that of the sulphide clay 16 kN/m^3, that of the grey clay 17.5 kN/m^3 and that of the reddish silt 19 kN/m^3, the corresponding mean water content figures being 25%, 70%, 57% and 40%. The organic matter content of the sulphide clay was approximately 2.1% and that of the other horizons slightly over 1%. The low electrical conductivity values indicate that the soil is of low salinity.

The geotechnical properties of the test plot are set out in Fig. 2.

The groundwater table prior to freezing was approximately at the +6.3 m level.

2. FREEZING OF THE TEST PLOT

The test plot was frozen by means of a grid of horizontal freezing pipes (ø 32 mm, total length 290 m) installed in three trenches (Figs. 3 and 4).

The two outer-most pipes were installed first, and these were used to freeze the ground in order to obtain sufficient stability for the central trench. The coolant used was glycol, 40% by weight, which was pumped into the grid at a

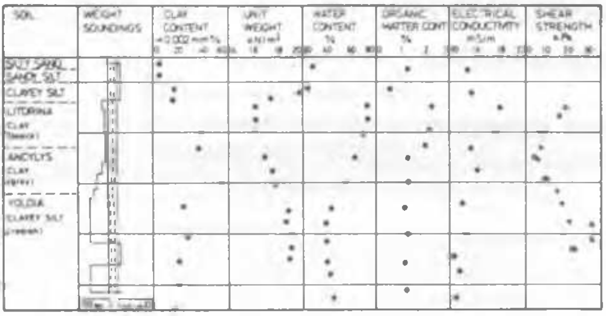


Figure 2. Soil properties of the test plot (Kujala 1988)

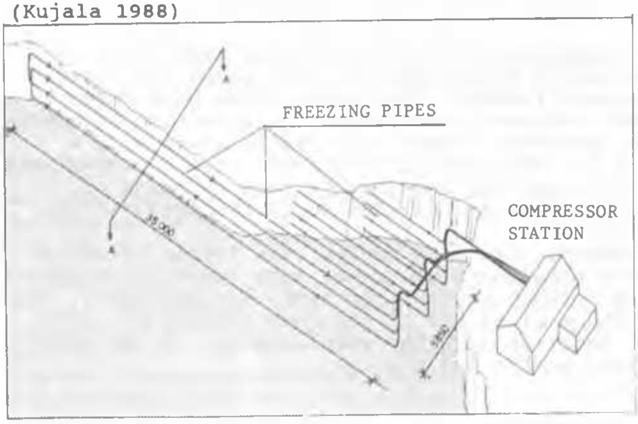


Figure 3. Freezing system of the test plot.

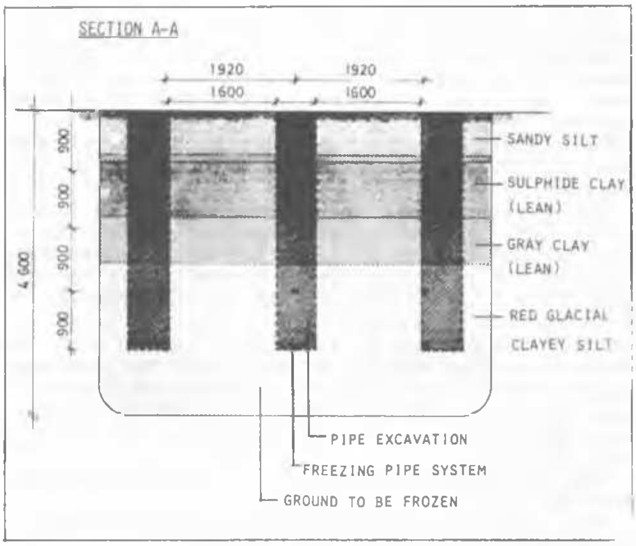


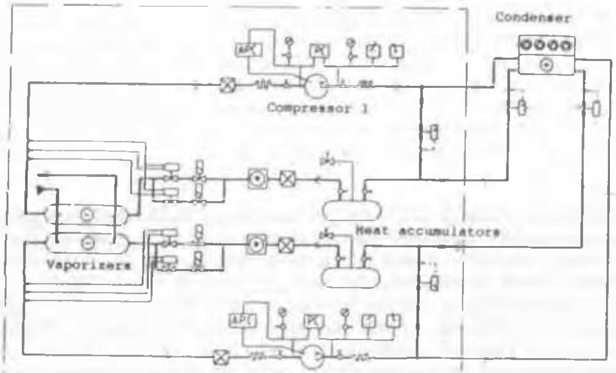
Figure 4. Section of the test plot.

mean temperature of -12.5°C and circulated at a rate of 1.3 l/sec. The temperature of the glycol rose to 2...3°C while in the grid.

The temperatures of the outgoing and incoming liquid at the compressor and the electricity consumption of the compressor were monitored throughout in order to calculate the energy required for freezing.

The plot was kept free of snow in the winter of 1985...1986 in order to speed up the freezing process and insulated in summer 1986 to prevent thawing. Freezing tests were commenced in September 1986, and the target temperature was achieved in spring 1987, after which the changeover was made to a temperature maintenance regime.

The total power of the compressors, 2 x 15 kW, was used for freezing between September 1986 and May 1987 (Fig. 5), the subsequent maintenance regime requiring a power of 7.5 kW.



Compressor
-cooling capacity 2 * 15 kW,
pover demand 1.2 kW
Vaporizer (planned)
-solution in - 5°C
-solution out -10°C

Figure 5. Scheme of the equipment in the compressor station.

The total electricity consumed in freezing the approx. 920 m^3 of soil between 15.9.1986 and 4.6.1987 was 50 000 kWh, and that used in maintaining the plot in a frozen condition over the 16.5 months between 4.6.1987 and 20.10.1988 has been 84 500 kWh.

Total frost heave in the plot to date has been 600 mm. The freezing of the plot reached the stationary heave phase in May 1987, when the 0 isotherm reached a level of +2.2 m. The temperature during the maintenance stage has varied from -6.5°C to -2°C. The heave rate was 1.2 mm/d during the transient phase and has been 0.7 mm/d during the stationary phase. The system employed to measure soil temperatures in the plot is depicted in Fig. 6.

Before freezing of the plot, temperature distributions were calculated for the soil prior to freezing and 1/3 year after freezing, employing the known geotechnical parameters, meteorological data for the area and a model for the freezing of soil.

The advance of the frost in the ground was then monitored in the field and the observed values compared with the calculated ones as shown in Figs. 7 and 8.

The considerable deviations of the measured values from the predicted ones suggest that there is room for further refinement of the model.

3. INSERTION AND CASTING OF THE PILES

3.1. Concrete piles

Holes of diameter 220 mm were drilled in the

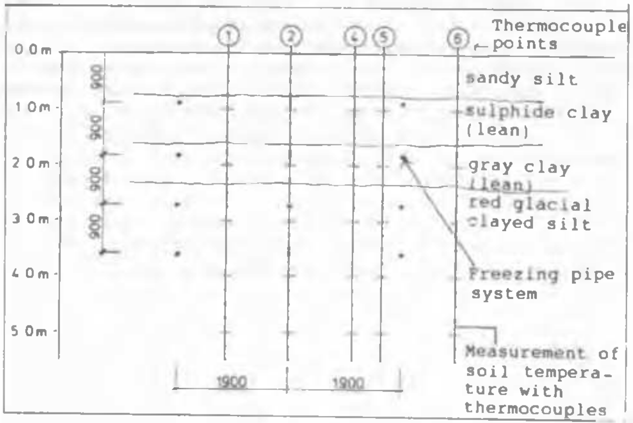
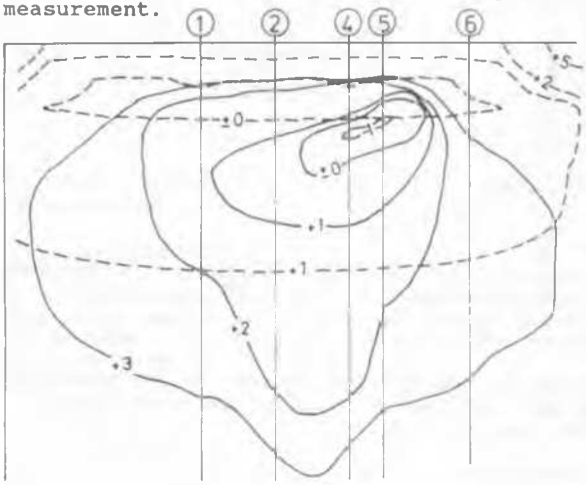


Figure 6. Arrangement of the soil temperature measurement.



— — — Calculated distribution of soil temperature at the beginning of freezing

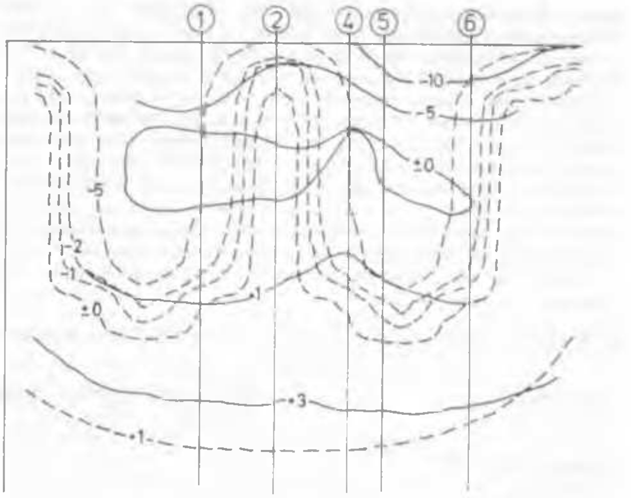
——— The distribution of soil temperature at the beginning when freezing started 15.09.1986 (measured)

Figure 7. Calculated and measured distribution of temperature in the plot before freezing started.

frozen soil using an Auger borer to depths which allowed the piles to be constructed in the different geological horizons distinguished in section 1 above. The full-length piles extended down to a depth of 4 m. The boring rate was 0.02...0.05 m/min. Drilling with the Auger borer was complicated by the thawing of the soil around the hole due to friction.

The reinforcement for the short piles subjected to upward loading consisted of steel plates placed beneath the piles with steel rings welded onto them, as depicted in Fig. 9.

Two types of concrete were used for casting the piles, antifreeze concrete and a conventional concrete which was surrounded by heating wires for the initial hardening period. The principal component of the antifreeze admixture was Sodium nitrite ($NaNO_2$). The compositions of the mixes, calculated from the mean air content and densities of the constituents, are given in Table 1.



— — — Calculated distribution of soil temperature after 4 months' freezing

——— Measured distribution of soil temperature after 4 months' freezing

Figure 8. Calculated and measured distribution of temperature in the plot after 4 months' freezing.

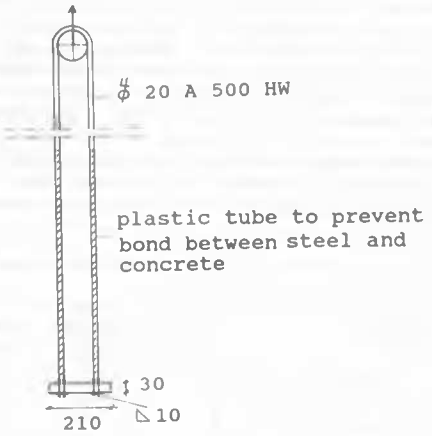


Figure 9. Reinforcement of the short piles for loading in an upward direction.

Table 1. Compositions of the concrete mixes used to cast piles in situ.

Constituent	Antifreeze concrete	Conventional concrete
Portland cement (Lohja Rapid, P 40 /7), kg/m^3	410	412
Aggregate, kg/m^3	1714	1726
Water, l/m^3	184	186
Antifreeze admixture, % of cement	8.0	0
Plasticizer, Peramin F, % of cement	1.2	1.5
Air-entraining agent, Mischoel WR, % of cement	0.025	0.040
W	0.45	0.45
A/C	4.18	4.19

Attempts were made to reduce the temperature of the antifreeze concrete mixes as low as possible in order to minimize the heat introduced into the soil on casting. The aggregate, which had been stored at -10°C, the powdered anti-freeze admixture and the cement, at +0.2°C, were measured into the mixer first and mixed for 1 minute, after which the major part of the water was added, also at +0.2°C, with the air-entraining agent mixed into it. The plasticizer was added in a 50% solution half a minute after the water, and mixing was continued for another 3.5 minutes. Mixing thus lasted a total of 5 minutes. The mix was then allowed to stand in the mixer for 3.5 minutes before testing and the making of sample casts for the determination of compressive strength and frost resistance.

The critical characteristics of the mixes are listed in Table 2.

Table 2. Characteristics of the concrete mixes used to cast piles in situ.

Characteristic	Antifreeze concrete	Conventional concrete
Temperature, °C	+4	+18
Plasticity		
-slump, cm	11.5	13.3
-V-B time, sec	1.3	1.2
Aircontent, l/m^3	51	44
Density of mix, kg/m^3	2325	2352
Vibration limit, h.min	4.33	4.50

Three cubes of side 150 mm were produced alongside the casting of each pile for determination of 28 d compressive strength. Thus 42 such cubes of antifreeze concrete were obtained and 6 of the conventional concrete used for comparison. These sample cubes were stored at +20°C and a relative humidity of over 95% until the day preceding the test. Six cylinders of diameter 150 mm of antifreeze concrete and two of conventional concrete were produced for the determination of frost resistance. The critical characteristics of the hardened concrete are given in Table 3.

Table 3. Characteristics of the hardened concretes used for casting piles.

Characteristic	Antifreeze concrete	Conventional concrete
Compressive strength, MPa	35.0	49.5
Protective pore ratio p_r, %	0.30	0.26

The concrete was poured into a pile hole of the form described in Fig. 10 with the aid of a casting hopper, after the reinforcement element had first been placed in position. The concrete was then compacted in layers using an immersion vibrator. Each pile took approx. 30 minutes to cast.

Figure 10. Part of the wall of a hole drill into the frozen ground.

The shorter piles of 75 cm in length located in only one soil horizon were topped off by inserting a polystyrene insulation tube into the top of the hole. The longer piles, extending from the bottom of the drill hole to the ground surface, did not require any insulation.

The temperature changes brought about in the surrounding soil by the casting of a long pile in situ were studied by means of thermocouples installed at a depth of 2 m, as also was the drop in temperature in the concrete upon introduction into the permafrost environment. The results of these measurements are shown in Fig.11.

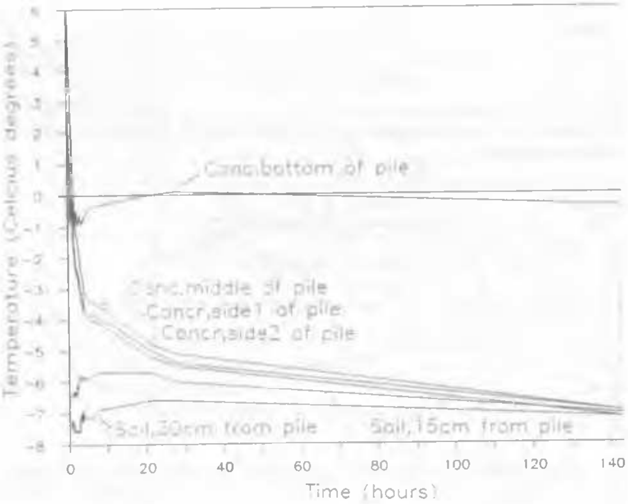


Figure 11. Temperature changes in the surrounding soil and the concrete of the pile itself after casting in situ.

3.2. Installation of a pre-cast reinforced concrete pile

The fusing of pile elements into permafrost soil by means of a slurry is a widely employed technique, and thus one pile was produced in the artificially frozen test plot by this method. The element used was a standard 250 x 250 mm reinforced concrete pile of total length 5.0 m, inserted down to desired level in a hole of diameter 350 mm drilled into the frozen soil using a Big Button Bit as pictured in Fig. 12.

Figure 12. The Big Button Bit, diameter 350 mm.

The drilling rate was 0.05 m/min, using air compressors. The drilling was highly successful, and proceeded uninterrupted, with no intervening withdrawals of the bit. The pre-cast pile was placed in position by tractor and the space between its outer sheath and the wall of the drill-hole was filled with slurry having a moisture content of 16% and a grain-size distribution of its sand as shown in Fig. 13.

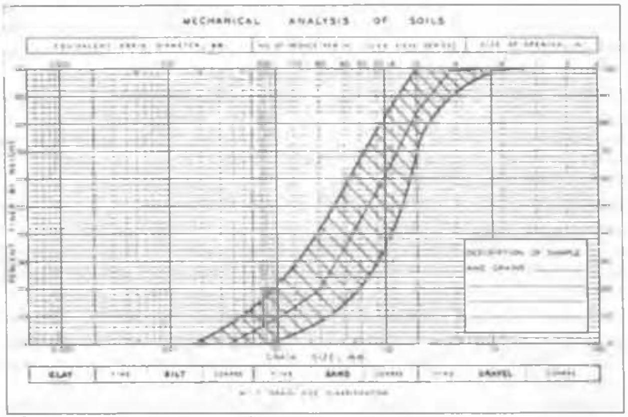


Figure 13. Grain-size distribution of the sand in the slurry.

Fixing of the pile by means of this slurry took 30 minutes. The rate of freezing of the slurry was monitored via thermocouples placed in the sheath of the pre-cast pile. The results are presented in Fig. 14.

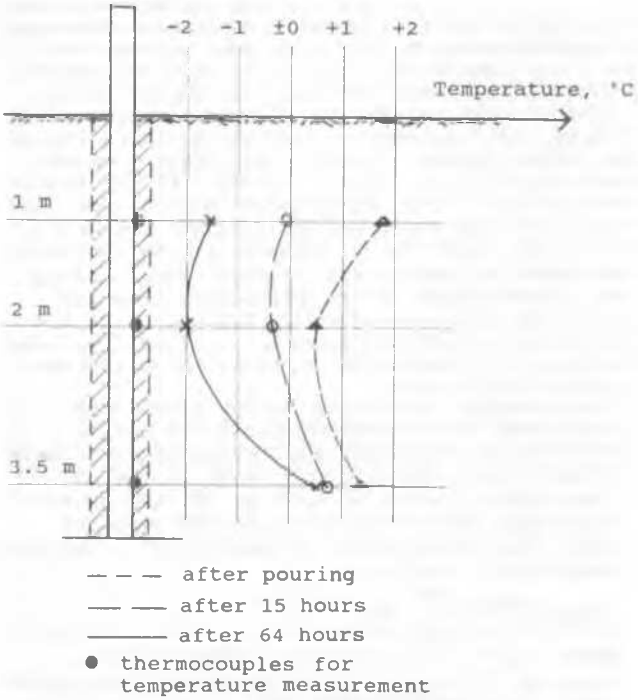


Figure 14. Rate of freezing of the slurry.

.3. Tubular steel piles

Two tubular steel piles of external diameter 219.1 mm and wall thickness 5.9 mm cast from X42 (API) steel with a yield strength of 290 N/mm^2 were also used in the tests. One of these was entirely uncoated, while the other had a polyethylene coating down to a depth of 1.75 m to prevent bonding with the active soil horizon. The piles were driven into pilot holes of diameter 115 mm. A Big Button Bit with air compressors was used at a drilling rate of 0.25...0.35 m/min.

This technique (Nottingham & Christopherson 1983) based on the temperature change achieved in the frozen soil by a small pilot hole and non-circulating hot water enables piles to be driven directly into permafrost. The pilot hole is a small drill-hole extending down at least as far as the intended depth of the tip of the pile. Applied correctly, this method disturbs the natural temperature regime in the soil very much less than do the drilling and fusion methods, the small amount of carefully controlled heat introduced into the soil around the pilot hole being dissipated relatively soon after the insertion of the pile and thus allowing refreezing to take place very rapidly. The water causes a temporary loosening of the soil particles followed by consolidation against the side of the pile, leading to a good frozen bond between the pile and the soil. Water-filled pilot holes allow the pile to be driven in with relative ease and improve the accuracy of this manoeuvre.

The necessary diameter for the pilot hole can be determined for any size of pile by taking the diameter of the pile itself and subtracting 2d, where d is the desired distance of the isotherm from the edge of the pilot hole (in inches).

The present pilot hole thus gave a distance for the isotherm from the edge of the pilot hole of 2.08 inches. The melting time needed may be obtained from the formula

$$d = k(T)^{1/2}$$

where d = isotherm distance from the pilot hole edge in inches,
k = constant for various soil types and the isotherm desired, and
T = time in minutes.

For a -3°C isotherm in a silty soil, k will be approximately 0.3-0.5. A value of k = 0.4, for instance, gives a melting time of 27 minutes. The initial temperature of the pilot hole water does not seem to be critical, water at 15-27°C apparently being suitable for warmer permafrost (temp. > -5°...-3°C) and water at 65-100°C being necessary for cold permafrost.

The present tests showed that a tubular steel pile could be installed quite satisfactorily in permafrost at a temperature of -5.5°C using a bore hole of diameter 115 mm and depth 4 m filled with 20 litres of water at an initial temperature of +80°C for 17 minutes, until it had cooled to +4°C, after which it was removed and replaced with a further 80 litres of water at +80°C for the final 8 minutes before insertion of the pile.

The pile was inserted using a hydraulic pile-driver with a 4 tn weight and applying a maximum drop of only 5 cm. At times the pile went in merely by virtue of the weight resting on it. The pile-driving phase lasted 14 minutes.

4. DETERMINATION OF BEARING CAPACITY

Bearing capacities were determined for the piles inserted in the manners described above by means of long-term static test loadings. Principal emphasis was upon the piles cast in situ, the results for which were compared with those for the pre-cast and tubular steel piles.

The short piles cast in one geological horizon only were loaded in the manner depicted in Fig. 15 by exerting an upward pull on them for measurement of the adfreeze strength, i.e. the strength of the bond between the frozen soil and the pile.

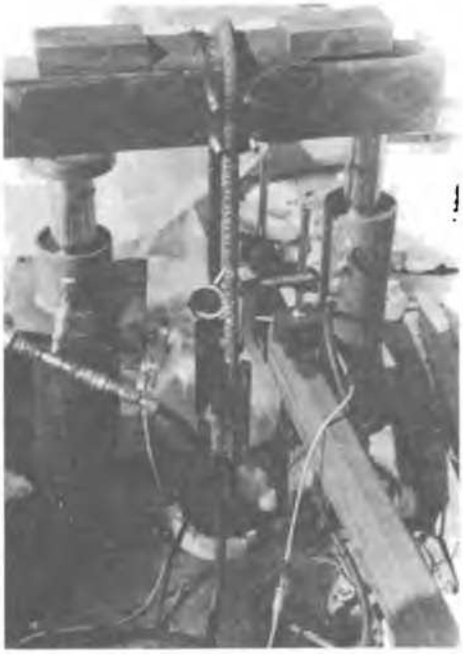


Figure 15. Arrangement for upward loading of the piles.

The long piles passing through several geological horizons were submitted to a downward loading, so that it is possible that the resistance against the tip of the pile may form an additional factor in determining the loading capacity. The arrangement for the counterbalance weight was as described in Fig. 16.

Figure 16. Counterbalance weight construction for loading of the long piles.

According to Crory (Crory 1982), there are no standard procedures for conducting pile tests in permafrost, although several methods have been suggested. The pile loading in many tests has become standardized at 44.5 kN per day. Pile tests can be particularly helpful when conducting on-site tests to define the differences in bearing capacity associated with different installation methods, since they can conveniently be used to demonstrate the performance of two adjacent piles, one driven into position and the other inserted in a slurried hole. Earlier pile tests were concerned with defining the loads which marked the transition to disproportionate settlement or complete failure, but current interest lies more in ascertaining the amount of settlement associated with each load imposed. Thus the point of interest in pile testing may be said to be the defining of long-term load capacity and settlement.

Ice is not the only material which creeps under stress, and both laboratory and pile tests confirm that frozen soils also exhibit creep stress, their creep rates increasing with temperature and the amount of ice present within the soil. In fine-textured soils, particularly clays and organic silts, the creep rate is also influenced by the amount of unfrozen water in what is normally frozen soil. This creep may be measured in pile tests by maintaining each incrementally applied load for three days or more. Should settlement on the third day be greater than 0.5 mm, loading should be continued for a further 10 days in an attempt to define whether the settlement rate will increase, decrease or remain constant. A rate of settlement greater than 0.5 mm per day should be regarded as indicating failure, as such a creep rate would normally produce an intolerable level of settlement. The adfreeze bond between the soil and the pile will usually fail at a gross settlement of less than 12.7 mm (Crory 1982).

Test loading of the piles in the artificially frozen plot was carried out by an adaptation of the above scheme, in which each load step was maintained for 24 hours at first. If the settlement at this stage was greater than 0.5 mm, the same load was continued for another two days, and if the rate still exceeded 0.5 mm per day it was given an additional 10 days. Test loading was discontinued if the settlement rate had still not declined after that time, or if the total settlement had exceeded 12.7 mm. The total duration of loading for most of the piles was approx. 1000 hours.

The bearing capacities of the piles were determined by the SNiP method (Fish 1982), yielding the experimental time-deflection curve presented on a logarithmic scale in Fig. 17.

The nominal bearing capacity is then determined from the intersection of the straight lines. The design bearing capacity Q^* or allowable design load N_d is

$$N_d = Q^* = \frac{k_t}{k_r k_s} P_n$$

where k_s = 1.1 is the soil safety factor,
k_r = 1.4 - 1.75 is the reliability safety coefficient, and
k_t = temperature correlation coefficient.

Test loading results for one short pile are detailed in Table 4 and the results in general are presented in graphic form in Figs. 18, 19, 20 and 21.

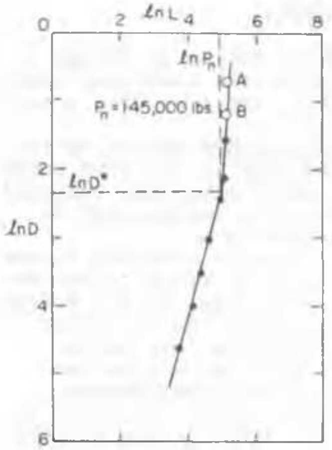


Figure 17. Determination of bearing capacity of the piles according to recommended Soviet practice. A and B are additional points.

Table 4. Characteristics of a pile used in the loading test and the surrounding soil, and nominal bearing capacity.

Size	ϕ 223 mm, L = 766 mm
Concrete	Antifreeze concrete
Layer	Lean, black sulphide clay
Age of pile on commencement of loading	12 months
Total duration of loading	407 hours
Soil temperature	- 3.1 °C
Ultimate upward load	220 kN
Adfreeze strength	410 kN/m^2
Compressive strength of concrete after loading	40 MPa (s=0.5 MPa)
Concrete density	2370 kg/m^3 (s=23 kg/m^3)

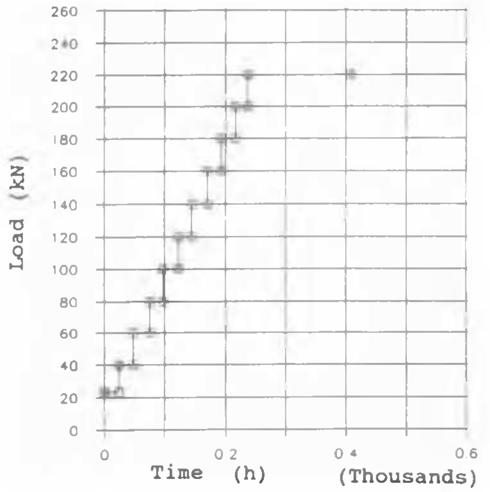


Figure 18. Time - load curve of a pile test.

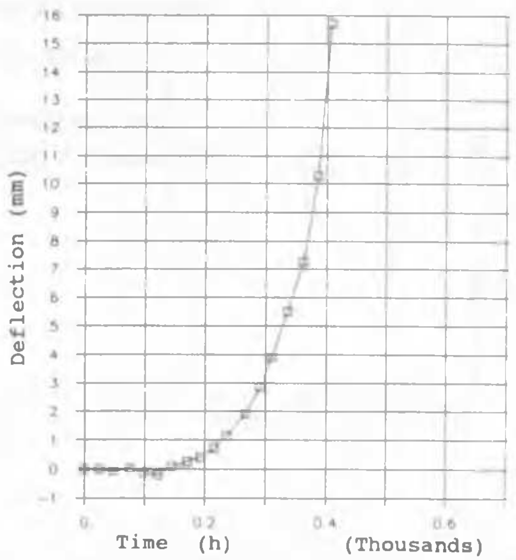


Figure 19. Time - deflection curve of a pile test.

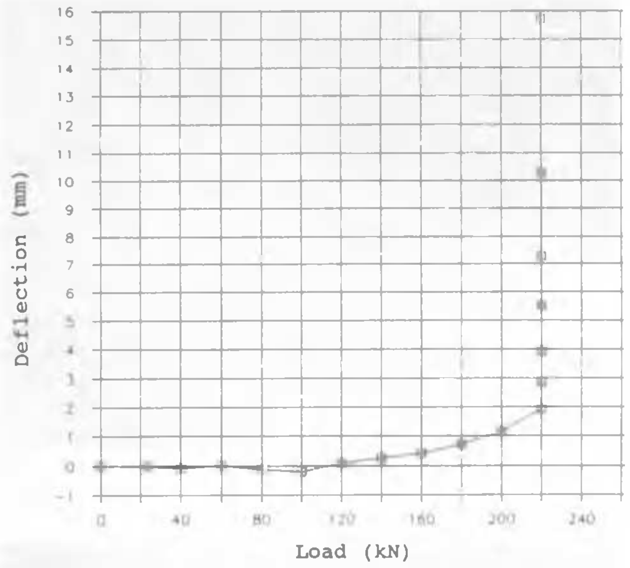


Figure 20. Load - deflection of a pile test.

Since according to the findings of Gaidaenko (Gaidaenko 1984), the adhesion caused by freezing of the soil to the pile is greater than the cohesion effect of the soil in a sideways direction, the movement observed will not have taken place at the surface of the pile but in the soil. In other words, the movement will not have taken place at the concrete/soil interface but within the soil itself, at a distance of 0.8 - 1.4 cm from the surface of the pile. In many cases, especially when carrying out such tests in silty clays, the ice lenses have melted and the pores filled with mortar, so that after completion of the test and removal of the soil a peculiar reticular cryogenic surface structure has been observed, which evidently increased the coarseness of the soil.

Upon completion of the present tests in the artificially frozen permafrost plot, the short piles were pulled out of the ground, their surfaces examined and the points determined at which the break had occurred when the bearing strength had finally been exceeded. Pile 101 A had a mean thickness of 2.8 mm of black clay on its surface, so that it must have moved into the layer of black clay to some extent, Fig. 22. The same reticular cryogenic surface structure was also observed here, Fig. 23.

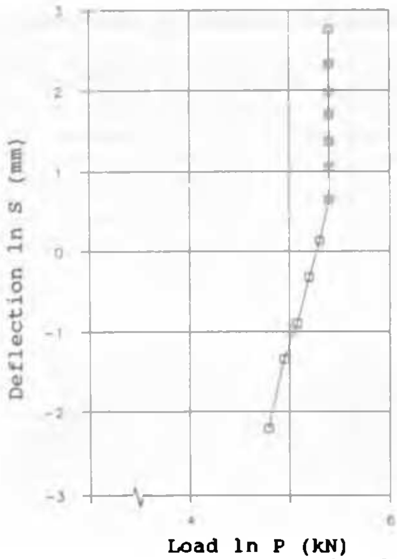


Figure 21. Load ln P - deflection ln S curve of a pile test.

Figure 22. The surface of a pile after the load test.

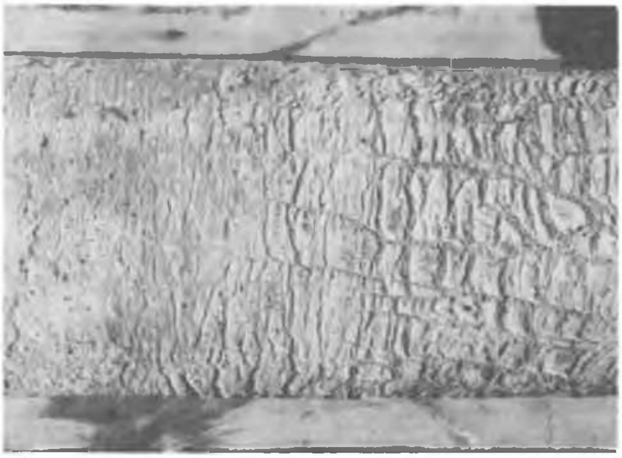


Figure 23. The reticular cryogenic structure observed on the surfaces of the piles.

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