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Field Tests of Deep Power-installed Screw Anchors in Permafrost[1]

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A test program was conducted in northern Manitoba to evaluate the creep behavior and load capacity of 8-, 10-, and 15-in. (20.3-, 25.4-, and 38.1-cm) diameter power-installed screw anchors embedded in permafrost (frozen, stratified silts and clays containing ice at about 31.5 °F (−0.3 °C)). The test results show that the anchors behaved under uplift loads, in a manner that was essentially very similar to that exhibited by deep footings of the same size. No failure planes (slip surfaces) were observed around the plates but a deformed zone above the plates was clearly visible in the varved soil when the anchors were excavated after testing. A method of analysis which allows the test data to be used directly for design, based on secondary creep rates and allowable displacements, is described.

Un programme d'essais, destiné à étudier le fluage sous charge et la capacité portante d'ancrages hélicoïdaux de 8, 10 et 15 po. (20.3, 25.4 et 38.1 cm) de diamètre vissés dans le pergélisol, fut effectué dans le Nord du Manitoba. Le pergélisol du site choisi pour ces essais, consistait en couches intermittentes d'argile et de silt et avait une température d'environ 31.5 °F (−0.3 °C). Les résultats d'essais montrent que, sous l'action des charges d'arrachage, le comportement de ces plaques d'ancrage ressemble fortement à celui des fondations circulaires profondes. En effet, lorsque les ancrages furent excavés après les essais, on a constaté une absence complète de toute surface de rupture au voisinage des plaques, mais la plus grande partie de déformation semblait plutôt être concentrée dans une zone sphérique au-dessus des plaques, clairement visible dans le sol stratifié. En utilisant les données sur le taux de fluage secondaire tirées de ces essais, et en adoptant le critère d'un déplacement admissible, on a réussi à mettre les résultats d'essais sous une forme analytique, convenable pour la conception de tels ancrages et permettant une comparaison de leur comportement avec d'autres types d'ancrages établis dans le pergélisol.

Introduction

The design and installation of anchors in permafrost for various types of structures, including guyed towers and buried pipelines present difficult engineering problems. The behavior of frozen soil in resisting uplift forces and the displacements associated with the long-term creep of frozen soil under load are of particular interest. Although experimental information on the behavior of anchors under uplift forces in unfrozen soils has been reported in a number of papers (Adams and Hayes 1967; Bhatnagar 1969; Hanna and Carr 1971; Adams and Klym 1972) comparable information for frozen soils is still very scarce. The Division of Building Research of the National Research Council of Canada undertook, therefore, a field study of anchors in permafrost to evaluate their creep behavior and load capacities.

Two types of anchors were installed early in 1967 at two permafrost test sites located at Thompson and Gillam, Manitoba (Fig. 1). The results of investigations conducted on grouted rod anchors have been reported (Johnston and Ladanyi 1972). This paper describes the installation and testing procedures and presents the results of the test programs conducted on

[1]Presented at the 26th Canadian Geotechnical Conference, Toronto, Ontario. October 18–19, 1973.

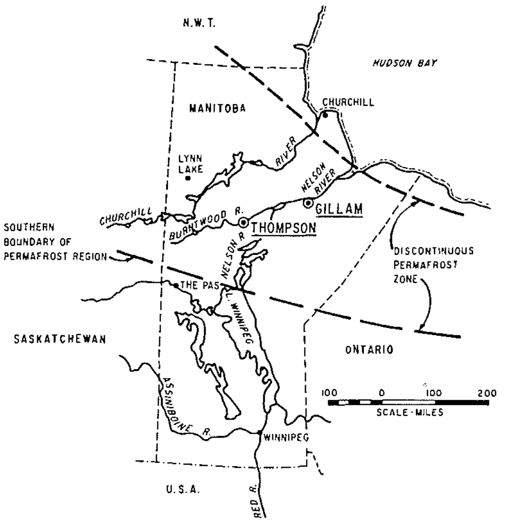

FIG. 1. Location of test sites, Thompson and Gillam, Manitoba.

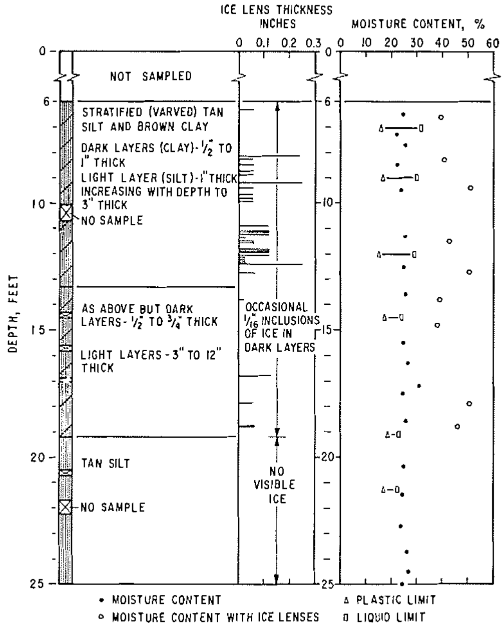

FIG. 2. Soil conditions, Hole TH-1, anchor test site Thompson, Manitoba.

power-installed screw anchors at the Thompson site.

Test Site Conditions

Thompson lies within the discontinuous permafrost zone. Stratified sediments (varved clays) of low to medium plasticity occur at the test site to a depth of about 19 ft (6 m). The thickness of the silt layers varied from 1 to 12 in. (2.5 to 30.5 cm) and of the clay layers from 0.5 to 1 in. (1.3 to 2.5 cm). The varved clay was underlain by a dense silt. Ice lenses were mainly horizontal and varied in thickness from hairline to about 0.5 in. (1.3 cm) in the top 13 ft (4 m). Very little ice was noted between 13 and 19 ft (4 and 6 m) and none was visible in the silt between 19 and 30 ft (6 and 9 m). Soil and ice conditions are shown in Fig. 2.

Ground temperatures at the test site, from about 5 to 30 ft (1.5 to 9 m), were between 31.5 and 31.8 °F (−0.3 and −0.1 °C); *i.e.* they were essentially isothermal throughout the year. The original depth of thaw (active layer) of 3–4 ft (1–1.2 m) increased to about 8 ft (2.4 m) over a 3-year period due to clearing of vegetation and moss cover and as a result of testing activity.

Further information on soil and permafrost conditions at the Thompson site is given in a paper by Johnston and Ladanyi 1972.

Installation of Screw Anchors

A total of 12 screw anchors were successfully installed at the Thompson site. Four specially fabricated anchors with round hubs, two of which had single and two had double helix screws (TS series, Fig. 3) were installed in February 1967 to depths of from 10 to 15 ft (3.1 to 4.6 m). Eight, commercial-type single helix screw anchors with square hubs were installed in March 1967 to depths of from 8.5 to 13 ft (2.6 to 4.0 m). Details of the screw anchors reported here are given in Fig. 4. Single lengths of 1⅝-in. (4.1-cm) diameter round steel rods were used for the TS anchors. The rods for the other anchors were 1-in. (2.5-cm) diameter round steel bars coupled together in 7-ft (2.1-m) lengths. All rods were attached to the anchor hubs by threaded connections. Helix diameters varied from 8 to 15 in. (20 to 38 cm).

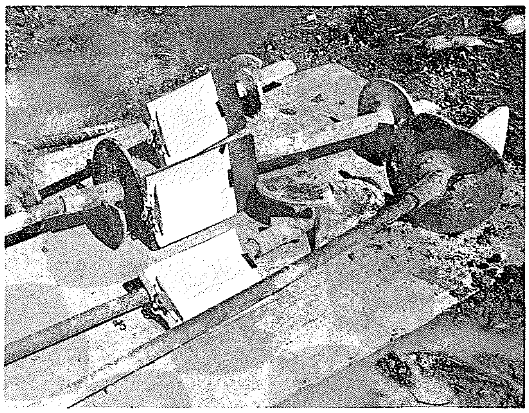

FIG. 3. Power-installed screw anchors (TS series), after testing.

The anchors were spaced at least 10 ft (3 m) apart

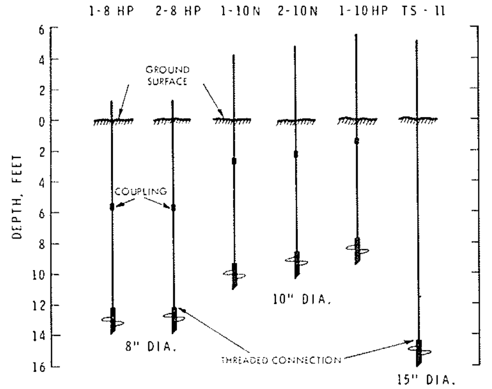

FIG. 4. Power-installed screw anchors, Thompson, Manitoba.

within an area 50 × 50 ft (15.1 × 15.1 m) in size at the test site. All anchors were installed vertically using a special 'wrench' attached to the drive motor or kelly bar of a truck-mounted, hydraulically-operated, radial-arm, power digger. This machine could exert a maximum torque of 5000–8000 ft-lb (690–1100 kg-m). The wrench (a long hollow tube) was slipped over the anchor rod so that it engaged either the back edge of the helix plate of the TS anchors (Fig. 5), or the square hub of the other anchors. In all cases, an attempt was made to 'screw' the helix into the ground, by applying a suitable downward pressure and rotational speed, to keep disturbance of the ground above the plate to a minimum. The 2–3 ft (60–90 cm) thick, hard frozen seasonal frost layer was extremely difficult to penetrate and was usually churned up before the helix would 'bite' and screw into the frozen ground. The anchors were advanced with difficulty even in the underlying warmer temperature frozen materials. Similar problems were experienced in penetrating the dense silty material which occurred at the greater depths. On some occasions insufficient torque could be generated by the power digger and the machine stalled or the anchor could not be installed to the desired depth. Several anchors were broken and lost during installation; failure occurred because the helix buckled or the weld connecting the helix to the hub fractured. The time taken to successfully install an anchor to about 10 ft (3.3 m) varied from about 30 min to 1 h.

When the wrench was removed after installing an anchor, a void remained between the anchor rod and the wall of the hole made by the wrench. In most cases, an attempt was made to fill this void by pouring loose fill insulation down the hole.

Test Equipment, Procedures, and Results

Most of the anchors were tested during May and June 1970; two were tested in November 1969. Loads were applied to the anchors using 30- or 100-ton (27- or 90-t) center-hole hydraulic jacks with 6 or 3 in. (15 or 7.5 cm) travel, respectively. The jacks were slipped over the anchor rod onto a specially designed steel tripod test frame. The uplift load was applied to a heavy nut or steel clamp on the top of the anchor rod and the downward thrust taken by timbers on the ground surface which supported the legs of the test frame. Cantilever beams (10:1 lever arm) loaded with concrete blocks were used occasionally to apply constant loads. The test arrangement for both methods is shown in Fig. 6.

To maintain constant loads for several hours or days, the hydraulic system was pressurized by an air-actuated pump equipped with an accumulator to dampen pressure fluctuations in the system. A hand pump was also used to apply pressure to the jacks when small loads were to be applied for short periods of time.

Stainless steel rules were securely attached to the anchor rods and movements of the anchors under test were measured using an

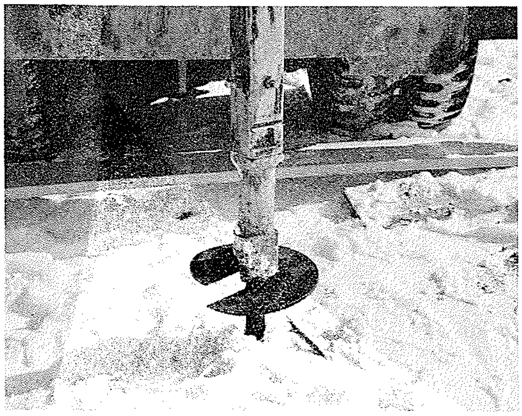

FIG. 5. Installation of single helix anchor.

FIG. 6. Anchors under load at Thompson.

engineer's level. All movements were referenced to a bench mark (another stable anchor rod) to which a steel rule had also been attached.

All anchors were stage-loaded, that is, loads were applied in increments of 2 or 5 kips (0.9 or 2.3 t), each load increment being maintained until a constant rate of strain was obtained.

The time period during which a load increment was applied varied from one anchor to another but ranged from about 1 to 18 h. The total time taken to complete testing of an anchor also varied, therefore, and ranged from about 5 h to 80 h. Most tests were completed in less than 40 h. Two screw anchors (not reported here) were tested by applying an initial load of 15 or 20 kips (6.7 or 9 t) for 2 months and then stage-loaded to failure over a 24- to 36-h period.

During some tests, the jack travel was used up because of settlement of the test frame and bearing timbers and because anchor movements increased at the high loads. When this occurred the load was released, the jack cylinder returned to its initial position, and the load reapplied as quickly as possible. 'Recycling' the jack usually took from about 5 to 15 min.

Immediately after the test program had been completed, trenches were dug adjacent to all anchors so they could be examined *in situ* and salvaged. A number of the anchors were damaged or broken when struck by the back-hoe during excavation of the trench or when they were being extracted from the ground (Fig. 3). No failure planes (slip surfaces) were observed around the plates but a deformed zone above the plates could be clearly distinguished in the frozen varved clay (Fig. 7). In some cases, a cone of compacted material adhered to the plate when it was removed (Fig. 8). The voids observed beneath the plates corresponded closely with the displacement measured during the tests. No upward movement or cracking of the ground surface was observed during the tests. Some disturbance of the column of frozen soil between the ground surface and the plate had occurred during installation of the anchors in a few cases but was not evident in others. The deformed zone immediately above the plate extended beyond the disturbed cylinder of soil.

The results of stage-load testing of six single helix screw anchors which are representative (three diameters and several depths) of those installed are presented as displacement–time curves in Figs. 9, 10, and 11. Pertinent details of the anchors and tests are given in Table 1. The results of tests on the two double helix anchors are not included at this time because the effect of interaction between the plates is not clear and this complicates the analysis.

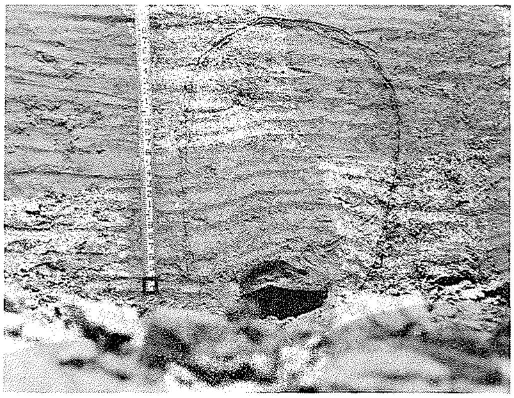

FIG. 7. Deformation of frozen varved soil above plate anchor, Thompson.

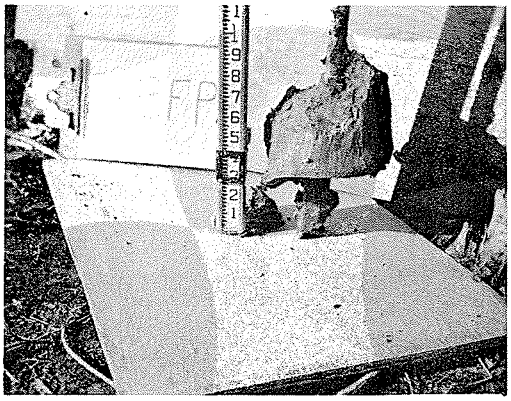

FIG. 8. Cone of compacted frozen soil above plate anchor, Thompson.

Discussion of Results

Ground surface observations made during the tests, and visual examinations of the anchors exposed in the trenches excavated after testing was completed, showed that the disturbed (during installation) column of soil above the anchors did not visibly affect their behavior. In fact, while the disturbed column could be detected after excavation, there was no indication that it was vertically displaced during the tests and the soil deformation was

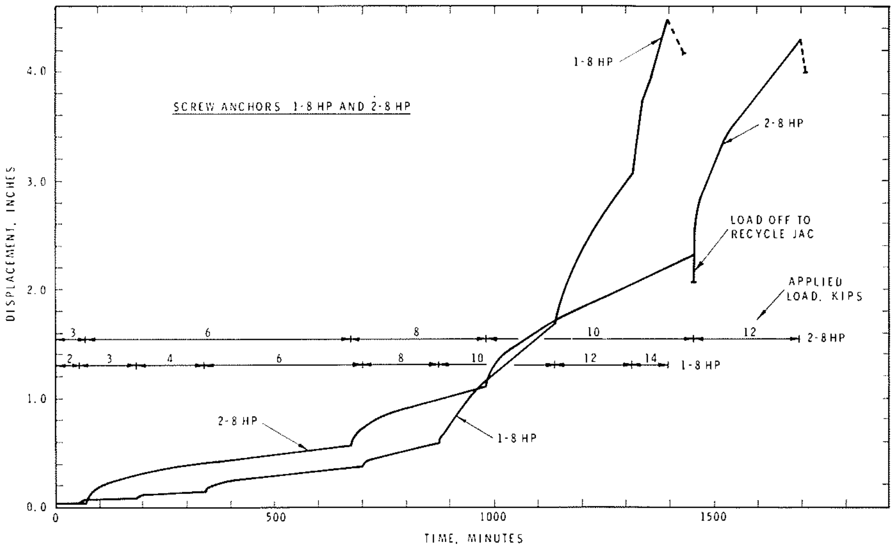

FIG. 9. Stage-load tests of screw anchors, Thompson.

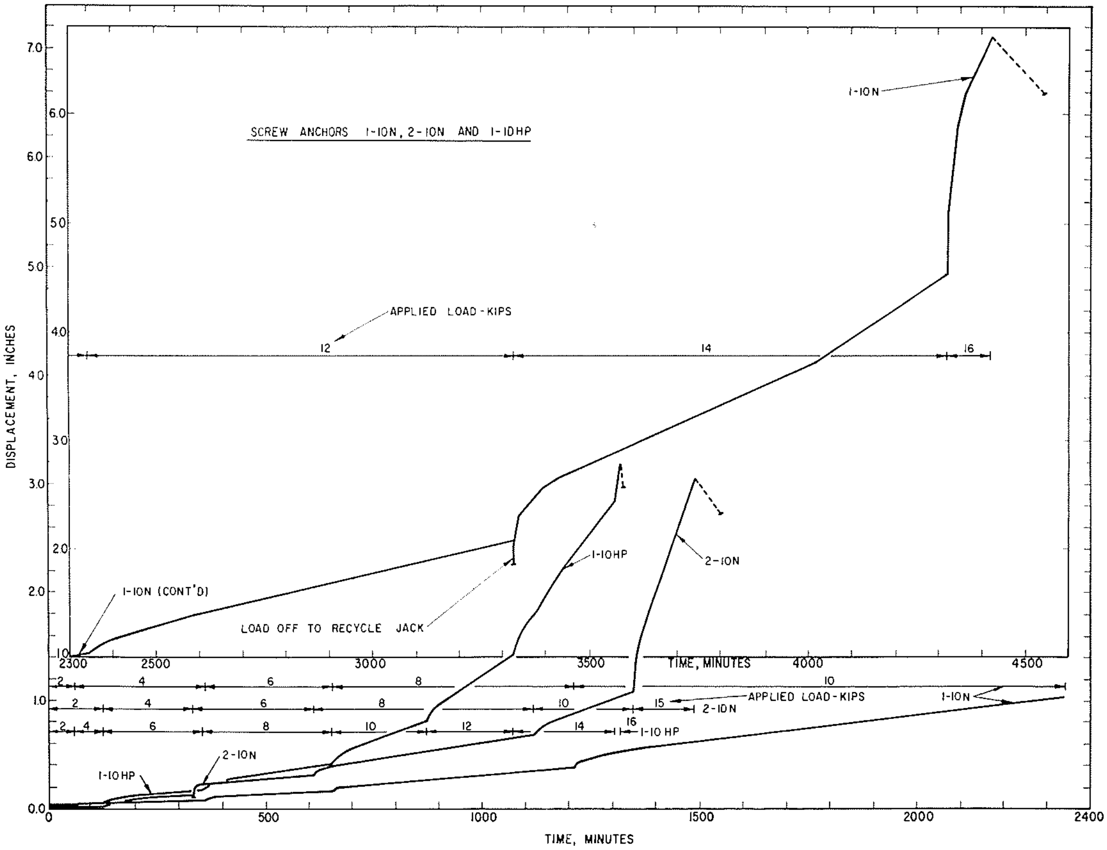

FIG. 10. Stage-load tests of screw anchors, Thompson.

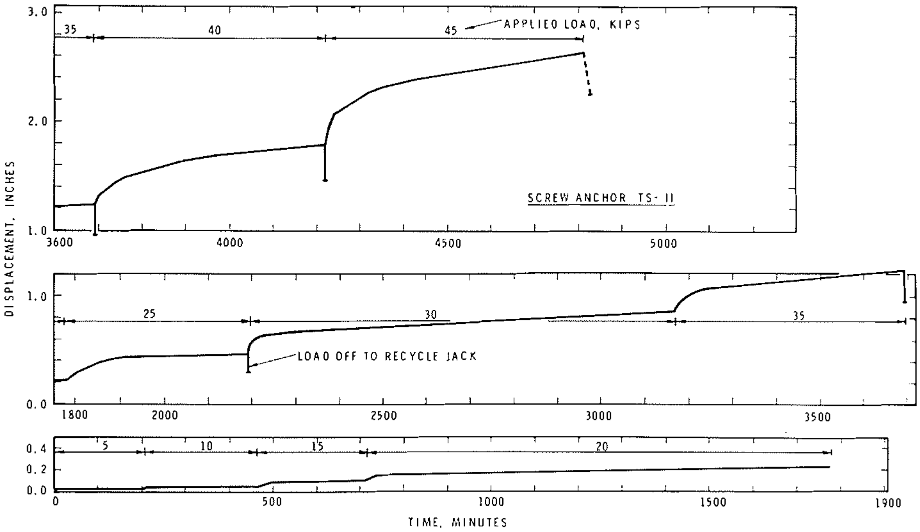

FIG. 11. Stage-load tests of screw anchors, Thompson.

TABLE 1. Values of parameters for displacement and creep rates for single helix power-installed screw anchors

Anchor* No.	Diameter B (in.)	Depth D (ft)	D/B	γD (p.s.i.)	For $F(q_{net})$ $s_c^{\,i}$ (in.)	C	$q_{as}^{\,i}$ (p.s.i.)	For $G(q_{net})$ $\dot{s}_c$ (in./min)	$\bar{C}$	q_{as} (p.s.i.)	$t_{50,2}$ (days)
1-8HP	8	13.0	19.5	11.3	10^{-2}	136.9	320	10^{-5}	206.0	323	3.2
2-8HP	8	12.8	19.2	11.1	10^{-2}	43.1	306	10^{-5}	80.5	277	7.5
1-10N	10	10.0	12.0	8.7	10^{-2}	15.3	216	10^{-5}	36.7	213	12.0
2-10N	10	9.3	11.2	8.1	10^{-2}	100.8	505	10^{-5}	57.7	200	6.8
1-10HP	10	9.0	10.8	7.8	10^{-2}	28.1	271	10^{-5}	92.0	210	4.7
TS-11	15	14.9	11.9	12.9	10^{-2}	37.3	328	10^{-5}	16.9	313	40.8

*'N' and 'HP' refer to the manufacturers identification code, 'TS' is D.B.R. anchor identification.

found to be distributed above the plate as if the column did not exist.

It should be noted, however, that the anchors were tested more than 3 years after they were installed and the mechanically disturbed soil had 'healed' naturally during that period so that the frozen material was again well bonded and had regained its strength to a considerable degree. One might speculate that refreezing of moisture resulting from thawing of the ground, caused by energy generated during drilling of the helix into frozen ground or by surface water percolating down from the surface along the anchor rod, would assist in rebonding the disturbed material. On the other hand, had these same anchors been tested shortly after they were installed, that is, before 'healing' could take place, then it is quite probable that the disturbed column of soil above the plate would influence the behavior of the anchor.

Behavior of Screw Anchors Under Load and Comparison with Grouted Rod Anchors

The single helix, power-installed screw anchors behaved under pull-out loads, in a manner that was essentially very similar to that exhibited by deep footings of the same size, that is, they showed very nonlinear load-displacement and load-displacement rate relationships and attained their ultimate bearing capacity in a hyperbolic manner.

It is known that, for deep circular footings and anchors installed in predrilled holes, the ultimate bearing capacity is attained only after a large displacement which is proportional to the footing diameter. For example, Bhatnagar

1969 reports values of prefailure displacements of up to 50% of the plate diameter in his pull-out tests on 3-in. (7.5-cm) diameter plate anchors in silty clay and Hanna and Carr, 1971 find displacements of more than 20% are necessary before the ultimate pull resistance of 1.5-in. (3.8-cm) diameter plate anchors in sand is attained.

On the other hand, it is also well known that comparatively much smaller displacements are required to mobilize the ultimate shear strength at the soil–pile interface, as, for example, on the lateral surface of grouted rod anchors. Bhatnagar 1969 found, for example, that, in silty clay, plate anchors of the same diameter and installed at the same depth as pile anchors required twice the displacement to mobilize the ultimate bearing capacity compared to the pile anchors. This means, therefore, that, even if the ultimate capacities of the two types of anchors, having the same diameter and at the same depth, are not very different, their capacities may differ much more when compared on the basis of similar displacements. For example, Bhatnagar 1969 found that when the displacement attained 10% of the plate and pile diameter, 85% of the ultimate load was mobilized in the case of the pile anchor and only 30% for the plate anchor.

Similar behavior was observed in the present investigations not only for the displacements but also the displacement rates. For all the screw anchors tested, the ultimate short-term pull load was relatively high, but, because of the large displacement rates which occurred under quite small pressures, the long-term allowable load that would guarantee that an allowable displacement would not be exceeded, was actually very small.

A comparison of the behavior of these screw anchors and the grouted rod anchors tested previously at the same site (Johnston and Ladanyi 1972), although interesting, is not easily made because of the different modes of failure. A comparison can be made using various assumptions as far as the depth, size, and the loading rate are concerned. A logical criterion to follow might be to compare the performance of the two types of anchors having the same diameter, at the same depth and at the same displacement or average displacement rate. Unfortunately, because the diameter of the rod anchors was only about 6 in. (15 cm) and that of the screw anchors varied from 8 to 15 in. (20 to 38 cm), the first condition cannot be met. Neglecting this condition, one can compare, for example, the short term capacities of two particular anchors installed at the same depth. Two such anchors are the screw anchor 1-1ON (Table 1) and the grouted rod anchor TG-3 reported previously (Johnston and Ladanyi 1972, Table 1). It will be seen that both anchors were installed to a depth of 10 ft (3 m). Although the 6 in. (15 cm) diameter rod anchor attained a maximum load of 38 tons (34.2 t) in about 37 min after about 2 in. (5 cm) displacement, the 10 in. (25 cm) diameter screw anchor attained only about 7.6 tons (6.8 t) load for the same time and displacement, that is, about 20% of the grouted rod capacity. With increasing times to failure, it was found that the difference very rapidly increases.

Interpretation of Test Results

A study of available experimental evidence on the behavior under uplift loads of footings and anchors embedded in unfrozen cohesive soils (Meyerhof and Adams 1968) has shown that when anchors are at shallow depths, the soil is mainly stressed in flexure and fails by tension cracking. At great depths, however, it was thought that flexing of the cohesive soil mass would be prevented by the weight of the overburden and that the resistance to uplift would be determined by the shear strength of the soil mass. For this case, the limiting uplift capacity of a plate anchor was found to be approximately determined by the bearing capacity formula for a deep footing.

As all of the screw anchors tested in this investigation were embedded deeply in frozen soil, which is, in fact, a highly cohesive material, their behavior was essentially similar to that of deep footings of the same size. The similarity can hold, obviously, only as long as the effect of the free ground surface remains negligible or equal in the two cases. Since the depth to diameter ratio of the anchors was greater than 10 and, in addition, the total displacements in the tests did not exceed about 50% of the plate diameter, the condition of similarity with deep footings was considered to be satisfied. In fact, as mentioned previously, when the anchors were excavated, there was no evidence that total uplift failure had reached the ground surface.

If this similarity is accepted, then the uplift capacity of a circular anchor plate embedded deeply in frozen soil is approximately given by the following expression (similar to Eq. [19] in Meyerhof and Adams 1968):

$$[1] \qquad Q_{\text{ult}} = (\pi B^2/4)q_{\text{ult}} + W$$

where:

W = weight of lifted soil mass and weight of anchor,
B = plate diameter, and
q_{ult} = the ultimate bearing capacity.

The ultimate bearing capacity can be expressed by the usual bearing capacity equation (by neglecting the soil weight term because of depth):

$$[2] \qquad q_{\text{ult}} = \gamma D N_{\text{q}} + c N_{\text{c}}$$

Ladanyi and Johnston 1974 have shown that the ultimate uplift capacity of the anchors can be predicted in the same manner provided the complete constitutive creep equation of the frozen soil has been independently determined by a laboratory or *in situ* investigation. It was found that not only the cohesion, c, in Eq. [2] is a function of time and temperature, but also that the bearing capacity factors N_{q} and N_{c} are affected by the nonlinearity of creep behavior. The equations and graphs for determining N_{q} and N_{c} are given in the above noted paper.

It is the intention here to describe a method which shows how field data obtained from creep tests of screw anchors can be put into analytical form and used for the design of anchors under the same site conditions but where different loading schedules and times under load may be involved.

As in creep testing of grouted rod anchors, in practice one is mainly concerned with the prediction of displacements in the secondary or steady-state creep stage. This is because the tertiary stage is usually considered to be beyond the point of creep failure, while the primary stage represents, for long time intervals, only a small portion of the total time. For long time intervals any point on the secondary creep line may, therefore, be given by the following equation:

$$[3] \qquad s(t) = s_i + \dot{s}t$$

where:

$s(t)$ = time-dependent displacement,
s_i = pseudo - instantaneous displacement (the intercept on the ordinate axis of the steady-state creep line at zero time, as defined in Johnston and Ladanyi 1972, Fig. 14)
$\dot{s} = \mathrm{d}s/\mathrm{d}t$ = steady-state displacement rate, and
t = time.

At a given frozen soil temperature, both s_i and $\dot{s}$ are functions of the applied net uplift pressure, q_{net},

$$[4] \qquad s_i = F(q_{\text{net}})$$

$$[5] \qquad \dot{s} = G(q_{\text{net}})$$

Once the functions F and G have been experimentally determined, *e.g.* from the anchor test data, the displacement of an anchor under a sustained load can be determined from Eq. [3]. The value of q_{net} is obtained by subtracting the overburden pressure γD from the pressure measured at the ground surface, *i.e.* from the uplift load Q,

$$[6] \qquad q_{\text{net}} = (Q - W)/[\pi(B^2/4)] = (Q - \gamma D)/[\pi(B^2/4)]$$

If the anchor is loaded by a series of increasing step loads, as shown in Figs. 9, 10, and 11, the displacement at a given time may be evaluated from

$$[7] \qquad s(t) = F(q_{\text{net}}) + \sum_0^t G(q_{\text{net}})\Delta t$$

The analytical form of the two functions, F and G in Eq. [7], can usually be determined by plotting the pseudo-instantaneous displacements and steady-state creep rates *versus* the applied stress. In the case of grouted rod anchors (Johnston and Ladanyi 1972) it was shown that the two relationships linearized when plotted in a log–log plot, thus permitting the two functions to be expressed by equations of a power-law type. This is understandable, because the rod anchors produced deformations very similar to those in an ordinary simple shear test.

As the deformations produced by the screw anchors were much more complex, similar to those around a deep footing, the form of the two functions was seen to be approximately hyperbolic, as is usual when deep footings penetrate into a soil mass (Perloff and Rahim 1966). In other words, the most convenient analytical form of the functions $F(q_{net})$ and $G(q_{net})$ was found to be:

$$[8] \quad F(q_{net}) = s_i = s_c^{\,i}\frac{Cq}{q_{as}^{\,i} - q} = s_c^{\,i}\frac{C(q_{net} + \gamma D)}{q_{as}^{\,i} - (q_{net} + \gamma D)}$$

$$[9] \quad G(q_{net}) = \dot{s} = \dot{s}_c\frac{\bar{C}q}{q_{as} - q} = \dot{s}_c\frac{\bar{C}(q_{net} + \gamma D)}{q_{as} - (q_{net} + \gamma D)}$$

In these two equations, $s_c^{\,i}$ and $\dot{s}_c$ are arbitrary constants. The four parameters, C, $q_{as}^{\,i}$, $\bar{C}$, and q_{as}, can be found by plotting the field data for pseudo-instantaneous displacement and creep rate for each creep stage in a plot originally shown by Kondner (1963), in which such hyperbolic data linearize. One such plot is shown in Fig. 12 for anchor 1-1ON.

Subsequent to selecting the arbitrary constants $s_c^{\,i} = 10^{-2}$ in. and $\dot{s}_c = 10^{-5}$ in./min, the following were plotted in Fig. 12: *on the abscissa*—the cumulative pseudo-instantaneous displacement, Σs_i over $s_c^{\,i}$, and the displacement rate, $\dot{s}$, over $\dot{s}_c$, respectively; *on the ordinate*—the values on the abscissa were divided by the total uplift pressure q, that is, $(\Sigma s_i/s_c^{\,i})/q$ and $(\dot{s}/\dot{s}_c)/q$, respectively. From the straight line obtained in this plot, *e.g.* for the rate line in Fig. 12, the values of the parameters are given by

$$[10] \qquad \bar{C} = A_0\, a/b$$

and

$$[11] \qquad q_{as} = a/b$$

In a similar manner C and $q_{as}^{\,i}$ are obtained from the displacement line. The values of the four parameters for the six single helix anchors tested at the Thompson site are given in Table 1.

When comparing the values of the parameters listed in Table 1 for the six anchors, it should be kept in mind that they are affected

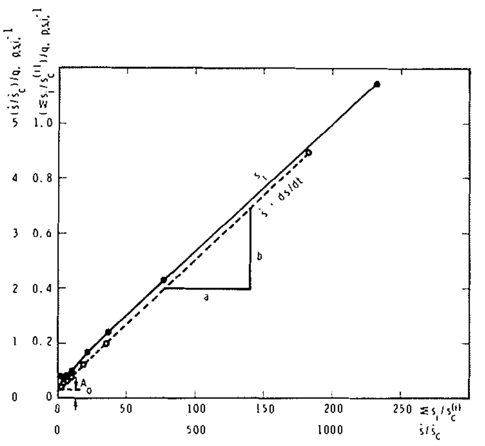

FIG. 12. Transformed plot of pseudo-instantaneous displacement and creep rate data for anchor 1-1ON.

by various factors, such as, anchor depth (the silt content increased with depth at the test site), anchor diameter, D/B ratio, soil temperature, and degree of disturbance during installation. It is, therefore, not surprising that the q_{as}, derived from the rate equation, was not quite the same for all six anchors, but showed a tendency to increase with increasing B, D, and D/B.

With the help of Eqs. [7]–[9], and by using the values of parameters given in Table 1, it is possible to extrapolate the measured information and predict the behavior of similar anchors under any given design loading conditions. One can, for example, predict the rate of displacement of a given anchor under the design load, or the time at which, under a given load, an allowable total displacement will be attained.

For example, if s_{all} is an allowable total displacement, then the time required to attain that displacement under a given net uplift pressure, q_{net}, will be according to Eqs. [3]–[5]:

$$[12] \qquad t_{req} = \frac{s_{all} - F(q_{net})}{G(q_{net})}$$

where $F(q_{net})$ and $G(q_{net})$ are given by Eqs. [8] and [9], respectively.

Taking anchor 1-1ON as an example, and using values from Table 1 we obtain:

$$F(q_{net}) = 0.153\frac{q}{216 - q}\text{(in.)}$$

and

$$G(q_{\text{net}}) = 0.529 \frac{q}{212.5 - q} \text{(in./day)}$$

from which, for example, one can find that under a step load of $q = 50$ p.s.i. (3.5 kg/cm^2) this particular anchor would reach an allowable displacement of 2 in. (5 cm) after

$$t_{\text{req}} = \frac{2.0 - 0.046}{0.163} = 12 \text{ days}$$

In the same manner, the times required for all the anchors to reach a 2 in. (5 cm) displacement under $q = 50$ p.s.i. (3.5 kg/cm^2) have been calculated and are given in Table 1. The time *versus* load line shown in Fig. 13 was calculated in the same way for anchor 1-1ON. The load Q, corresponding to a given time and selected allowable displacement, can then be picked from this plot.

In addition to the information just described, it should be noted that, in Fig. 11 of the paper by Ladanyi and Johnston 1974, a plot of the net pressure *versus* the resulting creep rate was shown for three of the anchors (1-1ON, 2-1ON and 1-1OHP). This plot can be used for quickly estimating the average creep rate for a given design load applied to the same type of anchor in similar ground conditions.

Conclusions

Several conclusions can be drawn from evidence obtained when a number of single-helix, power-installed screw anchors deeply embedded in frozen varved soil were creep tested at Thompson, Manitoba.

From the test results, it can be concluded

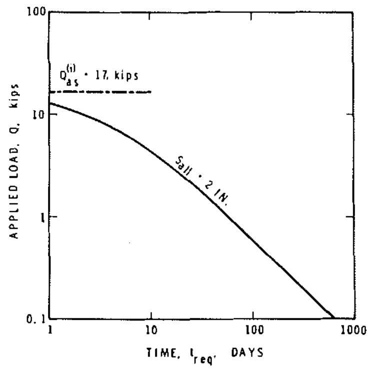

FIG. 13. Anchor 1-1ON. Load *versus* time relationship for an allowable total displacement of 2 in. (5.1 cm).

that the anchors behaved, under pull-out loads, in a manner that was essentially very similar to that exhibited by deep footings of the same size. One important aspect of this behavior, however, is that the anchors required relatively large displacements to attain their ultimate capacities. In addition, the tests showed that large displacement rates occurred under quite small pressures so that the anchors very quickly reached total displacements that might normally be allowed for structure anchors.

The relatively low pull-out capacities of the single helix anchors tested cannot be attributed directly to the disturbance of the soil column above the plates that might have occurred during installation. In fact, from a visual examination of the anchors in trenches excavated following testing, it was concluded that the disturbed soil column had completely refrozen during the 3 year period between installation and testing and it appeared to have had only a minor effect on anchor behavior. It should be noted that, had the anchors been tested shortly after they were installed, then the influence of the disturbed column of soil presumably would have been of more significance and perhaps would have resulted in even lower pull-out capacities.

In practice, groups of screw anchors might be considered to obtain the required pull-out capacity and limit the displacement. Whether or not multiple helix anchors would provide significantly increased capacities is pure speculation at this time and is a subject for further investigation. The ease with which screw anchors, either single or multiple helix, can be installed in frozen ground, the amount of soil disturbance caused during installation, and the time period between the time of installation and loading of the anchor (with regard to 'healing' of the disturbed material) are important factors to be considered.

The screw anchors reported here were installed with some difficulty in 'warm temperature', frozen, fine-grained soil-materials that might be considered ideal from an installation point of view. To improve installation of anchors in similar materials and other frozen soil types, and particularly when lower ground temperatures are experienced, the plates and rods need to be thicker and larger, the connections stronger, and the power of installation equipment considerably increased. Larger diameter plates could also be considered to increase the pull-out capacity; in this case much stronger anchors and more powerful installation machinery would be required.

As a general conclusion of the test programs carried out on both grouted rod and power-installed single helix anchors installed at the same permafrost site, it can be said that, in this particular type of frozen soil, grouted rod anchors are as easy to install and have several times higher pull capacities when compared with the screw anchors at the same depth, displacement, and time to failure. Notwithstanding the fact that mixing and placement of grout must be carefully controlled, it is concluded that for the site conditions encountered, the use of grouted rod anchors is very likely to be much more reliable, efficient, and economical than the use of power-installed screw anchors.

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