III.3. PILE FOUNDATIONS IN DISCONTINUOUS PERMAFROST AREAS

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The design and construction of foundations in areas of discontinuous permafrost requires a good understanding of frozen soil mechanics. Although similar foundation problems exist in the continuous permafrost zone where man or nature has caused thawing of the permafrost, the problems of the discontinuous permafrost zone are normally considerably more difficult. Any change, of short or long duration, in the delicate thermal balance and adjacent to a foundation on frozen soil produces corresponding changes in the mechanical and physical properties of the supporting medium. In all permafrost areas, heat flow from the structure to the underlying permafrost is a fundamental consideration. Degradation of permafrost containing segregated ice will produce settlements, usually differential, of the structure. Degradation may occur not only from building heat, but from other heat sources due to disturbance of the surrounding surface cover, solar radiation, drainage, underground utilities and groundwater flow.

The design of foundations partially or completely on permafrost consisting entirely of clean non-frost-susceptible compact sand and gravel deposits, free from segregated ice, may be the same as in non-frost areas (Reference 1). Sands or gravels with segregated ice which are susceptible to settlement on thawing, may be made suitable by prethawing and consolidating (Reference 2). Competent bedrock, or sands and gravels, are considered ideal for foundations and should be sought whenever possible; conventional foundation design approaches may be used in such cases. When, however, such sites are not available and the foundation soils are fine-grained silts or clays and contain ice inclusions, the problem becomes far more complex. In such silts and clays the advantages of pile foundations for safely transferring the loads to depths unaffected by settlement or thermal regime changes are well known. This paper presents some of the design and construction considerations associated with pile foundations in discontinuous permafrost areas.

SITE CONDITIONS

In areas of discontinuous permafrost, especially detailed and carefully executed site investigations should be conducted. Many of the foundation problems can be avoided by site selection based on aerial and ground reconnaissance. Air photos may be advantageously used to identify soil types and their boundaries (References 3 and 4). Air photos can aid substantially in eliminating undesirable sites and in suggesting possible usable sites. The shifting of a site a few hundred feet may place the structure entirely on or off permafrost, thus greatly simplifying the foundation problem.

Adequate borings are essential to establish the conditions which exist at the proposed foundation site. The conditions at depth may be completely different from those at or near the surface. When a relatively shallow layer of undesirable material exists over clean sands or gravels the design may be greatly simplified by excavating the poor material and backfilling with acceptable non-frost-susceptible material. Conversely, an apparently acceptable layer of sand or gravel may be underlain by silts or clays with excessive ice that would lead to settlement as thawing progressed.

To determine the actual extent of the foundation material, borings should be at appropriate intervals and to depths of at least the width of the structure. If the presence of isolated permafrost bodies is suspected, borings should be at especially close intervals. Where changes in soil types, ice distribution, ground water, frozen and unfrozen sections are encountered, sufficient additional borings should be required to delineate clearly any factors affecting the foundation design. Whenever possible (and in some cases it is essential) undisturbed soil samples should be obtained, rather than those from augering. Rotary core drilling will always be feasible in saturated frozen materials. Drive sampling will recover partially disturbed samples in frozen fine-grained soils down to about 25°F.

The samples should be classified and described in accordance with the frozen soil classification system (References 5, 6 and 7). Selected or continuous samples should be preserved in their frozen state for laboratory testing at a temperature as near as possible to that in place. The principal laboratory data required include dry unit weight, water and organic content, grain size distribution and in some cases degree of frost susceptibility, consolidation charcteristics, thermal conductivities (frozen and unfrozen), and shear properties, preferably in creep. From these tests and analyses, theoretical computations of the depths of freezing and thawing can be made. Using the computed depths of thaw, the results of thaw consolidation tests can be used to predict settlements. Preliminary estimates of the amount of settlement which will occur during the thawing of permafrost can be based on estimated changes in dry unit weight, and hence the volume of the soil, from its original frozen state. If continuous frozen samples are obtained, the effect of all the included ice (whether in lenses or not) can be accounted for in this way.

Ideally, explorations should precede the actual foundation design by at least one year, or more, if possible. Such a lead time is required to process soil samples and accumulate other necessary information such as: climatological records, design and performance information for other structures in the area or similar areas, ground temperatures, availability and costs of construction material, labor, etc. Thermocouple or other temperature sensing equipment should be placed in appropriate bore holes to record the ground temperatures at 2 to 3 ft vertical intervals prior to construction to ascertain the maximum ground temperatures. Preferably one or more of the assemblies should be in perimeter exploration holes, just outside the construction area, to serve as control assemblies during and after construction.

Ground temperatures which do not vary with time are particularly important in the design of pile foundations in permafrost because the potential strength of the permafrost is temperature dependent (References 8 and 9). Ground temperature observations, after a suitable period for dissipation of heat introduced by drilling, are recommended to establish the magnitude of annual temperature extremes at various depths. Precipitation, cloud cover, snow cover, and daily air temperatures when available are used to establish the influence of these factors on the ground temperatures recorded and the variation of the period from the previous means or averages observed by the Weather Bureau. Ground temperature isotherms superimposed on the soil logs give the thermal regime of the ground (Reference 10). A thorough appreciation of the thermal regime before, during and after construction is essential. To base the design of a pile foundation on only a measurement of the depth of the summer thaw is insufficient.

INSTALLATION OF PILES

The pile installation method is of primary concern in permafrost having mean annual temperatures greater than 28°F. Each method has its merits, in terms of unit costs and resulting capacities, and with respect to disturbance of the permafrost and the mode of freezeback. Currently employed installation methods are: steam thawing, dry augering, and driving (References 9 and 10).

The majority of pile foundations constructed for military facilities in Alaska have been installed in dry augered holes. Truck mounted augers have been used efficiently in silts or clays and some sands. The efficiency of augering, unlike steam thawing, is not dependent on the ice or organic content of the soil. Holes less than 24 inches in diameter can be augered at a rate of about 1 ft per minute, approximately the same as in steaming. In sands or soils containing cobbles or boulders, augering may be difficult or impossible. In such cases, churn drilling or localized pre-thawing with steam have been used to aid in preparation of the pile hole.

Steaming is seldom used in Alaska today. Normally steam is used only in colder permafrost which can safely absorb the heat introduced. In warm permafrost the melted ice from the originally frozen soil, augmented by the steam condensate, contains a large amount of heat and freezeback may take months or even several years. The steaming of holes for piles in cold permafrost should only be done by experienced operators, who know the effects of using their equipment in various soil and ice conditions. While the steaming method introduces a considerable amount of sensible heat into the permafrost during the preparation of the hole, soil-water slurries placed at temperatures just above freezing introduce far less. Essentially all of the frictional heat developed in dry augering is removed with the auger cuttings.

The least disturbance to the thermal regime is realized when the piles are installed by conventional or modified driving methods. Conventional driving has been advantageously employed in both the discontinuous permafrost at Bethel (Reference 11), and the continuous permafrost at Fairbanks and Kotzebue, Alaska. In addition to the speed of installation, the distinct advantage of driving, over the use of steam or slurry, is that essentially no latent heat is involved, and freezeback occurs within minutes or a few hours, after driving. Less perfect surface bond is attained on driven piles, but this is readily compensated for by additional pile embedment area. High energy double acting or diesel hammers are recommended.

FREEZEBACK

The observed natural freezeback of more than 100 piles, of different types, has been correlated with theoretical heat transfer equations and has produced a method of calculating the time required for natural freezeback. Since detailed discussions of the freezeback of piles in permafrost have been previously reported (References 12 and 13), only the application of the freezeback principles as applied to discontinuous permafrost will be discussed herein.

Typical freezeback curves for slurried H-piles and pipe piles in augered holes, as observed by means of thermocouples, are shown in Figure 1. The heat to be absorbed by the permafrost is the heat produced by the installation method. Because the sensible heat of the backfill material is negligible in contrast to the amount of latent heat involved, the amount of heat per foot of pile length which must be removed is computed by the equations shown in Figure 2. Knowing the thermal properties (diffusivity, conductivity, and heat capacity) of the permafrost, one can compute the time required for freezeback by the use of the general equation shown in Figure 3. The solution to the natural freezeback problem was adapted by Leung (Reference 15) and Lee (Reference 16) from Carslaw and Jaeger (Reference 14).

Because the permafrost temperatures within the normal depth of pile embedment are variable, it is often easier to prepare a specific solution for the freezeback time required, as shown by the plot in Figure 4. This plot dramatically illustrates the prolonged freezeback time required for installations with large Q values, or in relatively warm permafrost. For instance, a 12 inch timber pile with a slurry backfill having a water content of 30% in an 18 inch diameter hole would require the removal by the permafrost of approximately 4000 BTU's per foot of pile. In 28°F permafrost this would require, as shown in Figure 4, approximately 6 days to freezeback; at 30°F it would require 16 days. Further calculations would show that in 31°F permafrost it would require 41 days; at 31.5°F, 97 days; and at 31.8°F, 302 days for freezeback.

Assuming the volume of slurry per foot of pile to be at a minimum, the temperature and water content of the slurry and temperature of the permafrost are the only parameters which can be varied to effect a rapid freezeback. In the continuous permafrost zone the time of installation can be made to coincide with the minimum ground temperatures experienced in late winter or early spring. When piles are installed during this period the large temperature differential between the slurry and the surrounding extremely cold permafrost may be utilized to effect a rapid freezeback. In discontinuous permafrost areas where the annual frost penetration may be very shallow or possibly not even penetrate a residual thaw layer above the permafrost, there is no optimum installation period. Present practice, irrespective of permafrost temperature, is to mix and place the soil-water slurry at the lowest practicable water content., yet fully saturated, at temperatures between 32 and 45°F. The difference in volumetric latent heat of high water content silt- and low water content sand-water slurries is obtained through the use of the equation:

$$Q = Lw\gamma_d$$

where

Q = slurry heat, BTU's/cu.ft.

L = latent heat, 144 BTU/lb of water

w = water content, expressed as decimal

γ_d = dry unit weight, pcf

<u>Q(silt</u> for $w = 70\%$, $\gamma_d = 58$ pcf

$Q = 144(0.70)(58)$

$Q = 5850$ BTU's/cu. ft.

<u>Q(sand)</u> for $w = 10\%$, $\gamma_d = 133$ pcf

$Q = 144\ (0.10)(133)$

$Q = 1915$ BTU's/cu.ft.

In 28°F permafrost the sand-water slurry would freezeback in 2 to 3 days, the silt-water slurry in 10 to 11 days; in 31.5°F permafrost, the difference in freezeback time would be important, 16 days for the sand-, 131 days for the silt-water slurry. Thus a careful selection of the backfill material and a proper field control of the water content can effect a substantial reduction of the heat to be absorbed by the permafrost and the time required for freezeback.

The preceding general and specific solutions for the natural freezeback of piles assume the slurried pile to be a finite cylindrical heat source inside a semi-infinite medium, with a suddenly applied constant temperature source (32°F) which dissipates heat only in a radial direction. The actual heat path is always toward colder permafrost than exists at the depth considered, as shown in Figure 5. The approximate heat paths during summer and winter are shown in the figure. The "effective" temperature of the permafrost at any depth can be approximated over the time and distance of the heat path to account for the vertical heat flow, if accurate initial ground temperatures are known. In continuous permafrost the vertical gradient may be important, but in discontinuous permafrost the gradient may be insignificant. While the freezeback time may be reduced by vertical heat flow, the greatest increase in freezeback time is caused by the proximity of adjacent piles.

The effect of pile spacing on the overall rise in permafrost temperatures caused by slurry heat is shown in Figure 6. The relationship between spacing and temperature rise is based on the "method of mixtures" as shown by the equations in the figure. When the rise in permafrost temperature (ΔT) in Figure 6 exceeds the difference between the freezing point and the initial permafrost temperature ($32-T_p$), the permafrost will freeze only that amount of the slurry water necessary to raise the permafrost temperature to the freezing point. The remaining latent heat of the slurry will not freeze until the surrounding permafrost has been made colder.

In the case of the silt- and sand-water slurries previously described, it is obvious from Figure 6 that a pile spacing of greater than 12 ft would be required for the 2000 BTU's per lineal foot of the sand-water slurry at a permafrost temperature of 31.0°F. The silt-water-slurry (6000 BTU's) would theoretically raise the permafrost temperature 1°F even at a 15 foot spacing. If the initial permafrost temperature were 31.5°F this would mean a final temperature of 0.5°F above freezing. This, of course, is impossible. In reality, only about 3000 of the 6000 BTU's per foot would be removed and the slurry immediately adjacent to the pile would remain thawed. The permafrost temperature would rise to just 32°F.

When the heat capacity of the permafrost is insufficient to effect freezeback of slurried or steamed piles within an established construction schedule, or because of close pile spacing, the piles may be installed by driving, or artificial refrigeration may be employed. Pipes or tubing can be quickly attached to the piles prior to installation and connected to a portable refrigeration system (Figures 7 and 8). Normally, positive artificial freezeback can be achieved in less than 2 days by careful control of the slurry temperature, water content, and a specific time limitation between augering, pile placement and the start of refrigeration. The cost of such refrigeration is often offset by the savings resulting from continuous construction. The refrigeration pipes, purposely designed to remain in place, are available for use throughout the life of the structure in the event further refrigeration is required.

No factor of safety is incorporated in the freezeback equations discussed. Thermocouple assemblies, or other temperature-indicating devices, should be installed to verify the theoretical freezeback before the piles are actually loaded.

BEARING CAPACITY

The bearing capacity of piles in discontinuous permafrost may be achieved solely by adfreeze strength, provided sufficient pile surface area is available at the low strength values of the adfreeze bond at temperatures which are only slightly below the freezing point (Reference 9). If the permafrost is relatively thin, the piles may be designed to act, in part at least, as friction piles, similar to conventional designs in thawed soils. If a firm bearing stratum or bedrock is within economical reach, the bearing capacity can be augmented or solely derived by point bearing, as in the temperate zone. The contribution of point bearing in unconsolidated soils containing ice is normally disregarded when the load is being carried by adfreeze in the permafrost.

Negative skin friction should be considered if the soils above the permafrost are unconsolidated or additional thawing of the permafrost will produce settlement in the soil surrounding the upper regions of the pile. Negative skin friction can be a significant problem if unconsolidated soils are surcharged by gravel fills.

SUMMARY

This paper has outlined only a few of the problems associated with foundations in discontinuous permafrost areas. Although a considerable amount of information has been published, most recently in reports of the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) through the First Canadian Conference on Permafrost - April 1962, through the International Conference on Permafrost - November 1963, and through revised Army foundation design manuals which will soon be available, many aspects are still unknown or unpublished. In addition to further research, field reports of construction and performance are needed.

The efficient design and construction of foundations on permafrost require a thorough understanding of the conditions to be experienced before, during, and after construction. In addition to its being good practice for obtaining design information and for monitoring effects during construction, instrumentation of the foundation with temperature sensing devices and vertical control points may be essential to detect long term changes in the thermal regime and verify the stability of the foundation.

Careful consideration of the heat introduced by the pile installation method and of the potential heat capacity of the permafrost to absorb this heat at different ground temperatures and pile spacing, is essential in design. Theoretical freezeback time for piles in permafrost can be advantageously employed for construction scheduling. When the heat capacity of the permafrost is insufficient for freezeback within an established construction schedule, artificial refrigeration may be used. Artificial refrigeration may be used intermittently or continuously to establish or retain a desired thermal regime, with an adequate adfreeze or bearing strength. At the low strength of permafrost in discontinuous permafrost areas, where permafrost temperatures are only slightly below freezing, the design, construction and maintenance requires a thorough understanding of the factors influencing foundation stability.

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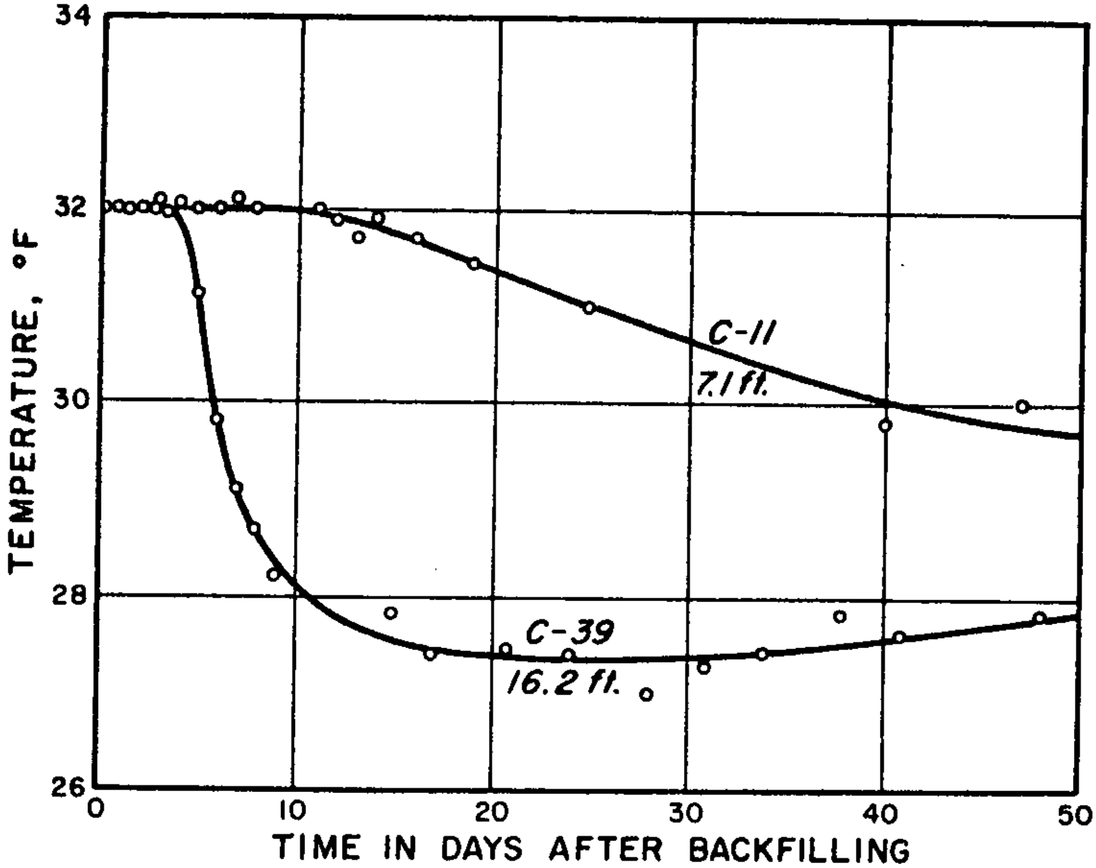


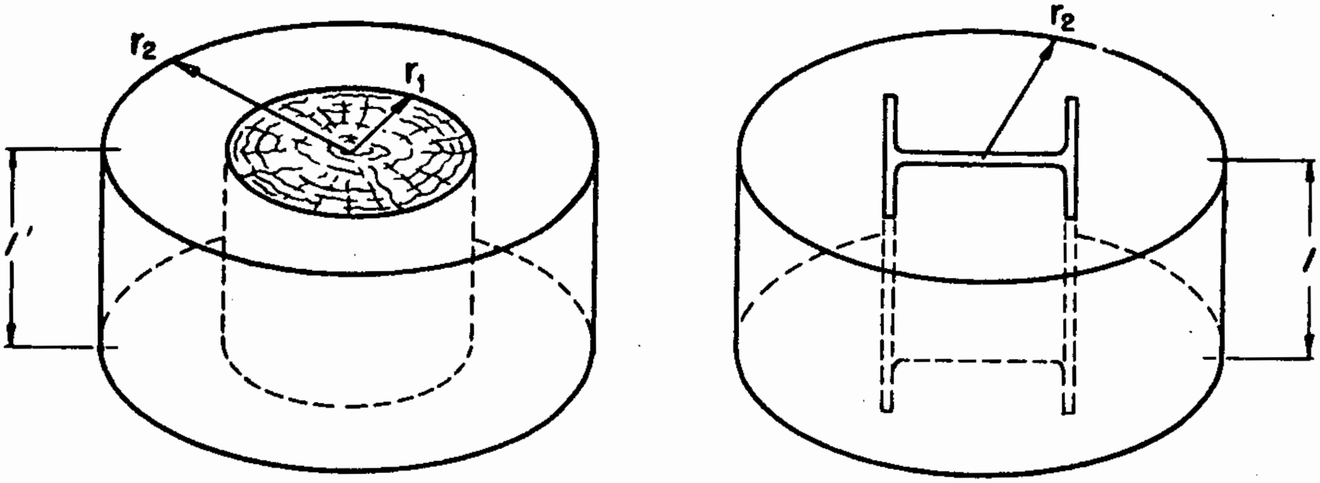
Pile No.	Size	Hole Dia.	Date Installed
C-11	10 BP 42	18 - inch	25 April 1957
C-39	8" pipe	14 - inch	17 April 1957

Both piles installed in dry augered holes and backfilled with silt-water slurry.

FIGURE 1

TYPICAL FREEZING CURVES FOR SILT SLURRY

BR 3459-10



$$Q = \pi L (r_2^2 - r_1^2) w \gamma d \quad \text{or} \quad Q = L(\pi r_2^2 - A) w \gamma d$$

where L = Latent heat of water

r_2 = Radius of hole

r_1 = Radius of pile

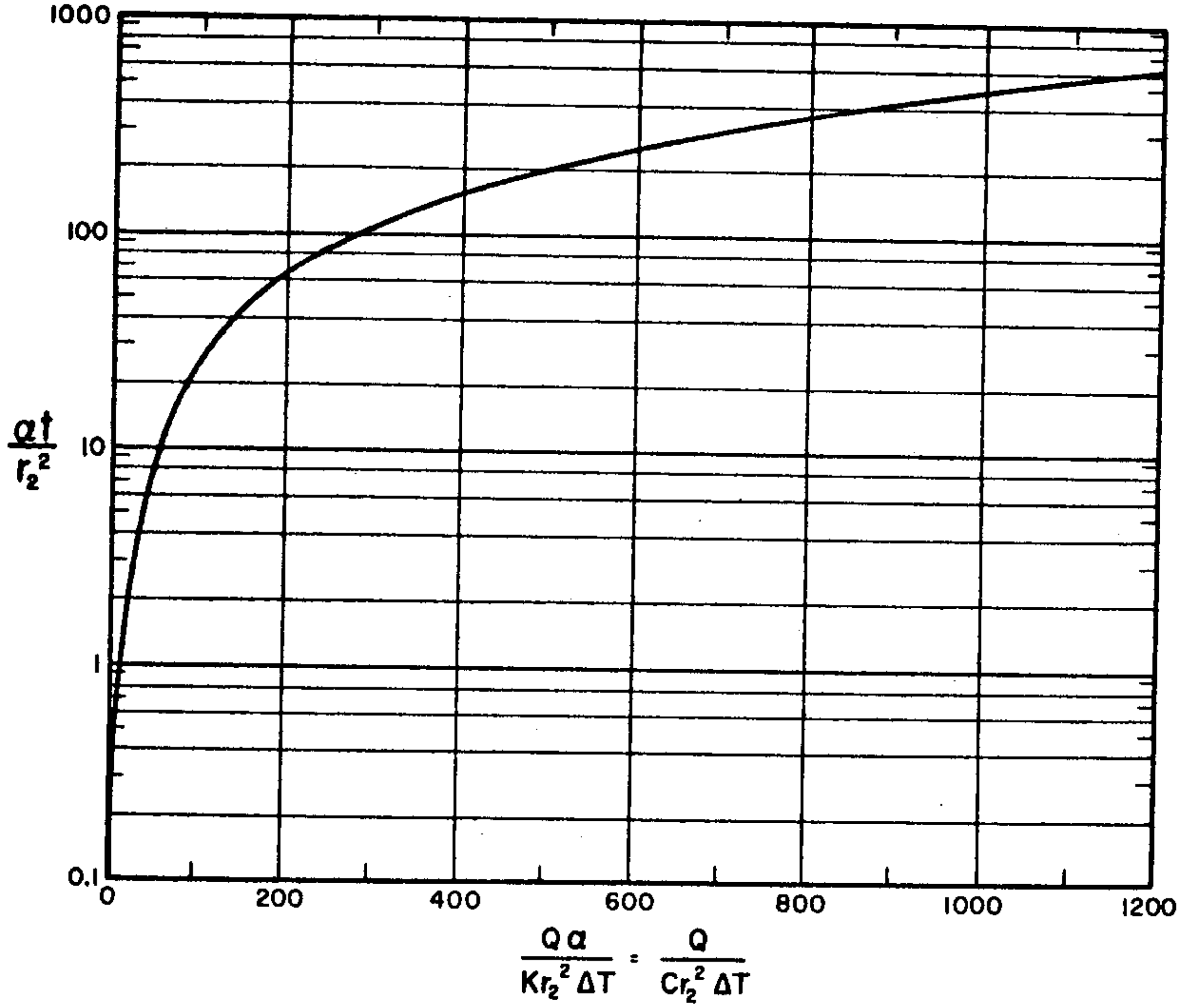
A = Cross sectional area of H-pile

w = Water content, percent dry weight

γd = Dry unit weight of slurry

FIGURE 2 LATENT HEAT OF SLURRY BACKFILL

BR 3459-11



where t = Freezeback time, hrs.

K = Conductivity of permafrost, BTU/hr ft °F

C = Volumetric heat capacity of permafrost, BTU/ft °F

α = Diffusivity of permafrost, ft²/hr = $\frac{K}{C}$

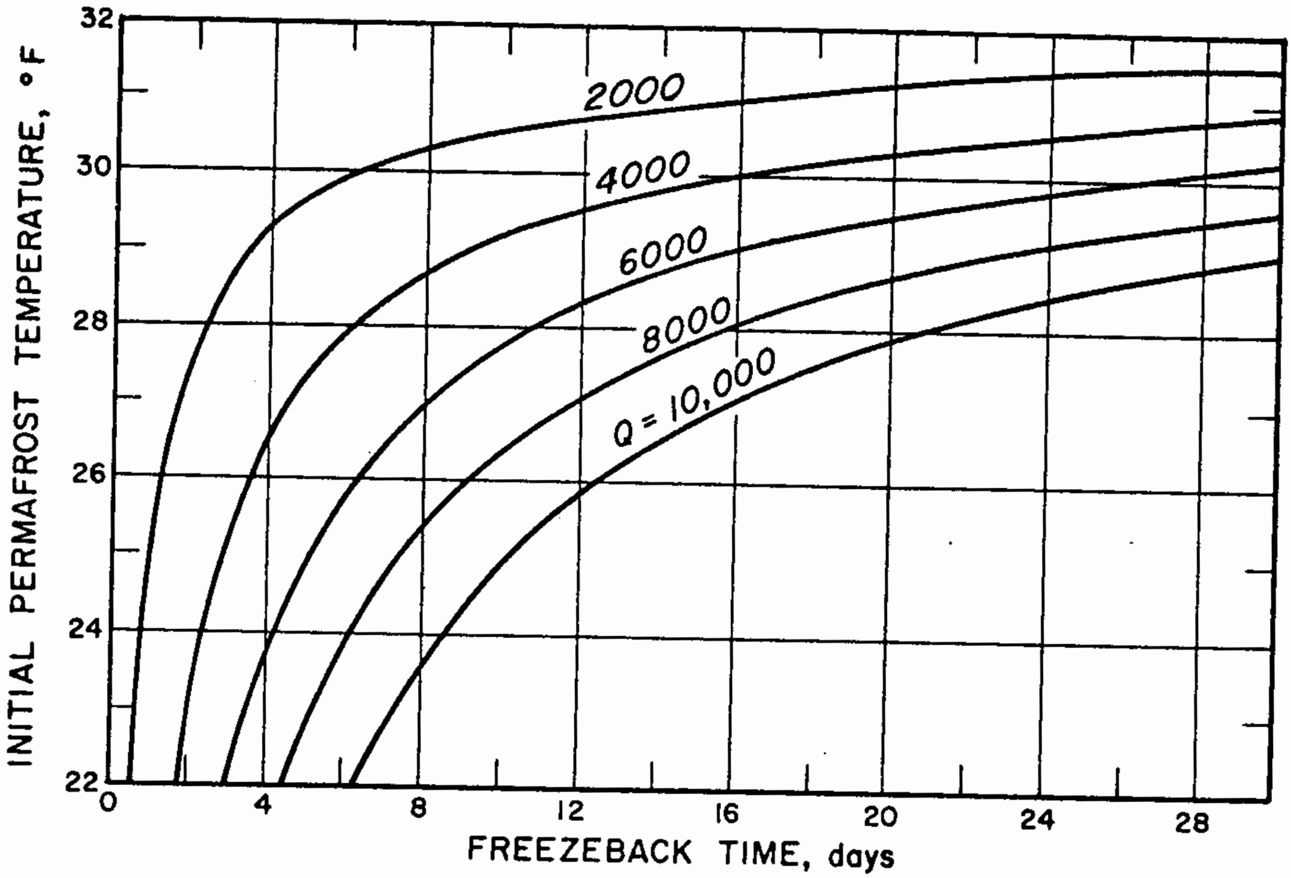
Q = Latent heat of slurry per foot of pile length, BTU/ft.

ΔT = Initial temperature of permafrost, expressed as number of °F below freezing ($32° - T_p$)

r_2 = Radius of pile hole, ft.

FIGURE 3 GENERAL SOLUTION OF SLURRY FREEZEBACK

BR 3459-12



where $C = 30$ BTU/ft^3 °F

$K = 1.4$ BTU/ft hr °F

$r = 0.75$ ft

Q = Latent heat of slurry per ft of pile length, BTU/ft.

FIGURE 4 SPECIFIC SOLUTION OF SLURRY FREEZEBACK

BR 3459-13

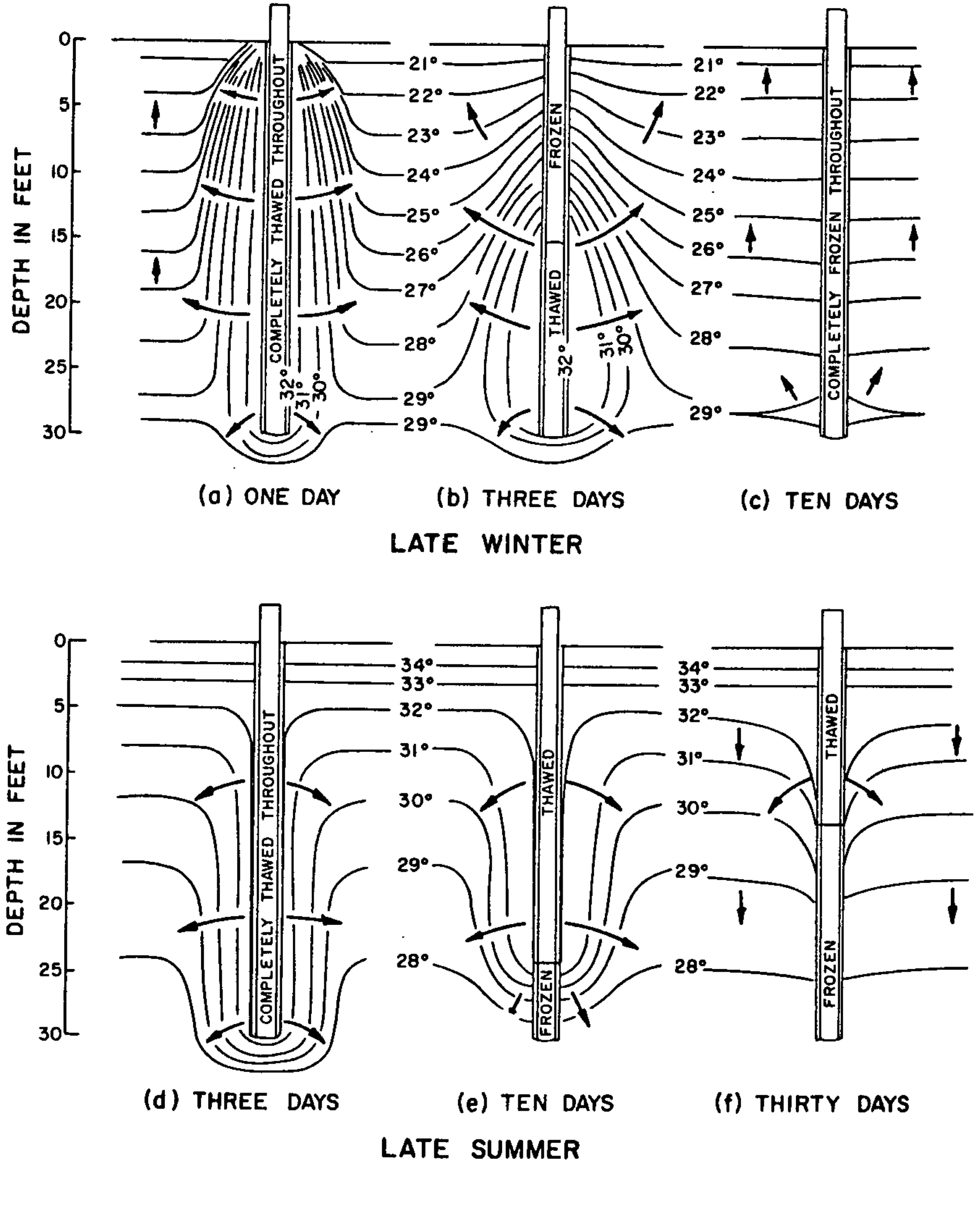
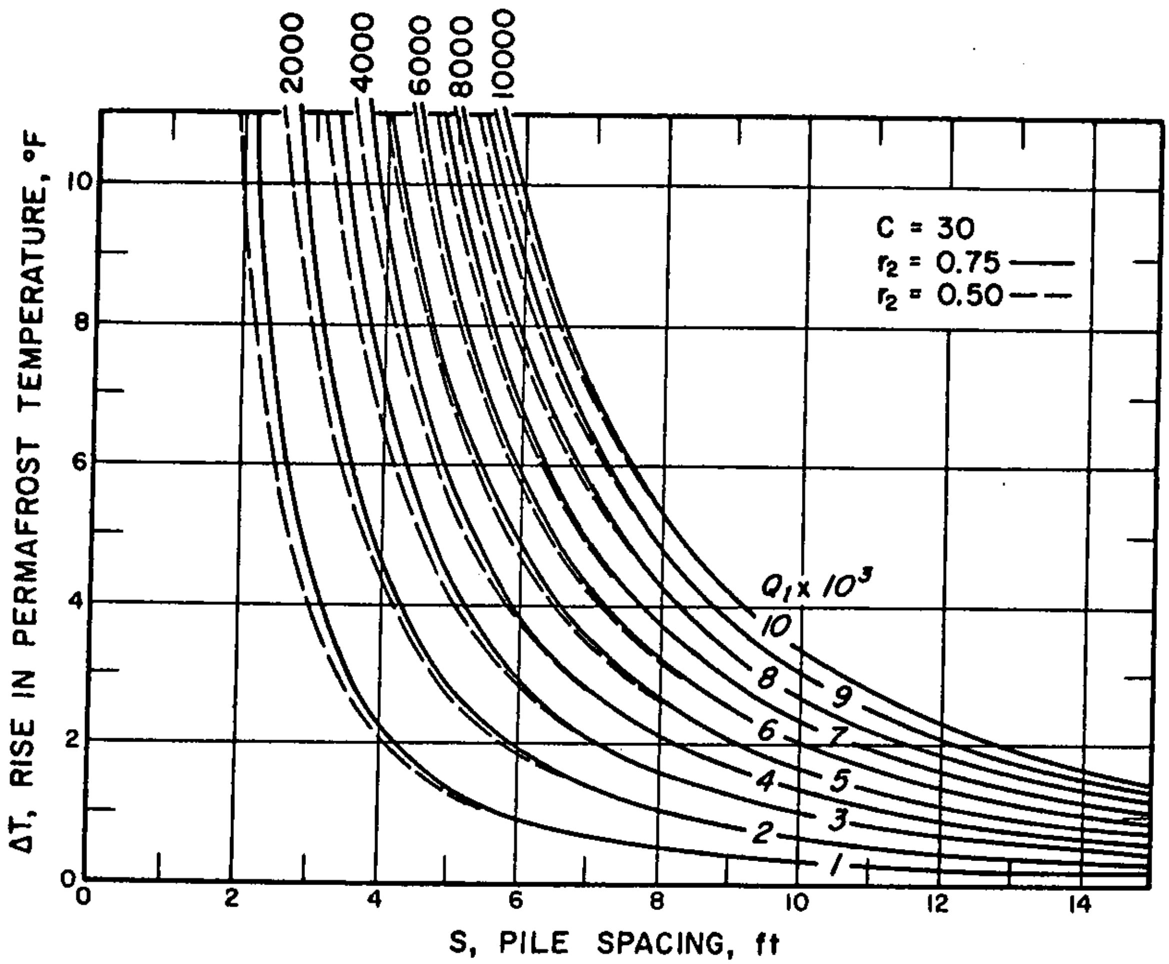


FIGURE 5 NATURAL FREEZEBACK OF PILES IN PERMAFROST

BR 3459-14



$Q_1 = Q_2$ (Heat loss = Heat gain)

$Q_2 = C(S^2 - \pi r_2^2)\Delta T$

where Q = Latent heat of slurry, BTU/ft.

C = Volumetric heat capacity of permafrost, BTU/ft³ °F

S = Pile spacing, ft.

r_2 = Radius of pile hole, ft.

ΔT = Rise in temperature of permafrost, °F.

FIGURE 6

INFLUENCE OF SLURRY ON SURROUNDING PERMAFROST

BR 3459-15

FIGURE 7

REFRIGERATION TUBING ON TIMBER PILES

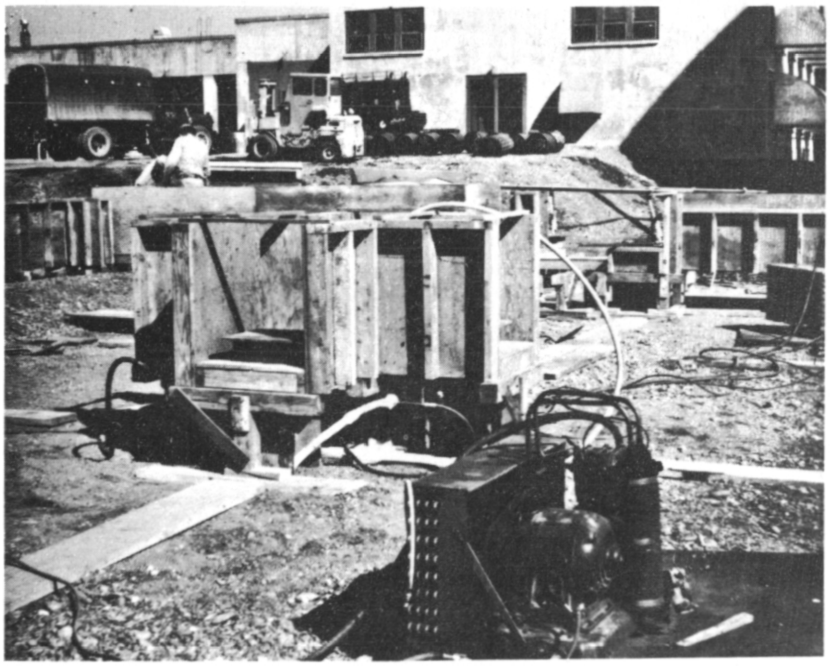


FIGURE 8

REFRIGERATION OF H-PILE GROUP

BR 3459-16

Discussion

G. Hollingshead asked what strength tests are performed on frozen soils. The author replied that earlier tests were of the conventional type such as rapid loading. More recently, tests similar to those carried out by Russian investigators have been employed. These include consolidation and shear strength under creep or long sustained loading. Long-term loads are also used in field studies. CRREL does not employ the earlier Russian ad-freezing strength data but their creep load data may be used. Both confined and unconfined specimens are used in testing with emphasis on confined specimens.

R.S. Taylor noted that the author referred to the use of copper tubing for circulating refrigerating fluids and he wondered whether plastic tubing was ever employed. Crory replied that CRREL used steel and plastic tubing at Hanover, N.H. It was found that plastic and polyethylene tubing became brittle at low temperatures.

In reply to an enquiry by G.S.H. Lock about Figure 1 showing freeze-back curves for piles, the author replied that these were obtained from field observations. The ground temperature remains at 32°F until the latent heat requirements are satisfied and then it decreases. G.S.H. Lock asked for more detailed information on the boundary conditions and whether there was a uniform heat flux or constant temperature. Crory replied that time is plotted versus temperature showing that the sensible heat is removed rapidly; the temperature is constant at 32°F. In reply to Lock's enquiry on whether this information can be used to analyze heat flow down the piles into the permafrost, the author replied that it was possible, the situation being similar to Long's thermopile.

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