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DESIGN OF FOUNDATIONS IN PERMAFROST

BY

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CHAPTER I

INTRODUCTION

The relatively recent growth in military and scientific facilities in polar regions and the industrial exploitation of Alaska and northern Canada, has necessitated a better understanding of the physical environment of cold regions. With this increased activity and occupation of cold regions also comes the need for better engineered structures. Foundations on permafrost, or perennially frozen ground, present special problems and demands in design and construction.

Design of foundations in areas of seasonal frost where the material below the frost line remains unfrozen present certain problems. However, by taking structural loads to depths below the frost line, potential problems from heave and lateral thrust are generally removed.

Permafrost itself is actually a fairly good material with high compressive strength. However, it does tend to creep under load, and cyclical freezing and thawing are of particular concern. Freezing and thawing have dramatic effects on the soil properties upon which the stability of a structure depends. The magnitude of these effects depend not only on the type of soil and its water content, but also on environmental factors such as weather, ground cover, snow, and the thermal properties of subsurface materials. Design of foundations on permafrost must consider heave and

subsidence, and differential movement under a structure, as well as the impact of a transient change in the conditions which affect the soil stability. One of these transient changes in conditions is caused by heat transfer from the building itself to the frozen soil. The effect of a heated 40' x 100' building may extend 50' below the surface.[1]

Occasionally a site may be determined to not be susceptible to freezing and thawing. In such cases, with few exceptions, foundations may be designed more or less according to standards used in temperate zones. However, these cases without freezing and thaw susceptibility are very rare and other methods must be employed. One alternative is to alter the site to make it stable. The other, more common alternative is to recognize the situation as it is and to design accordingly.

This paper will examine permafrost and its properties and characteristics as related to foundation design and construction, and the principles and practices currently being used in site investigation, foundation selection, and design, construction, and maintenance of foundations on permafrost.

CHAPTER II

PERMAFROST

Background

The term "permafrost" is defined as a condition existing below ground surface in which the temperature in the material remains below 32 degrees F (32F) continuously. (The U.S. Army specifies 2 years [2], and the U.S. Navy specifies 1 year [3] as the minimum length of time for which the material must remain frozen in order to be considered permafrost.) The texture, water content, and geological character of the subsurface are not considered by this definition. A definition with respect to temperature alone is not considered adequate, as often times a situation exists whereby a frozen state is not obtained even though the temperature has remained below freezing for several years. Therefore, permafrost is further defined such that if pore water is present, a sufficiently high percentage is frozen to cement the mineral and organic particles. [4]

Permafrost is generally divided geographically into continuous or discontinuous zones. A continuous zone exists where permafrost is continuous laterally under the ground surface to its lower boundary. There may be some widely separated thawed portions within a continuous zone. These thawed areas are called taliks, and will be discussed in a subsequent definition.

Discontinuous permafrost exists where the unfrozen

areas become the rule rather than the exception. In these areas the ground temperature is usually close enough to freezing such that local conditions such as water flow, variations in ground cover or insulation, and the thermal properties of the subsurface materials have a significant impact on the continuity of the frozen ground. The thawed areas may occur laterally or vertically to break up the continuity of the permafrost.

Figure 1 [5] shows the extent of continuous and discontinuous permafrost in North America. As noted, Figure 1 was derived from many sources, all of which are in basic agreement regarding the extent of continuous permafrost. There tends to be less agreement, however, regarding the extent of discontinuous permafrost areas.

In the Southern Hemisphere, there is very little permafrost except that found under the Antarctic Continent. The dry, coarse grained, loose deposits of soil which are prevalent appear to have precluded the formation of ice masses. [6]

It is generally agreed that permafrost originated during the Pleistocene period of glaciation which lasted from 1,000 to 10,000 years ago. It has been noted that in some areas the thickness of permafrost is growing thinner. The only known areas where new permafrost is forming is under recent arctic alluvial deposits. Except for the active zone, permafrost seems to be in a state of equilibrium with its present climate.

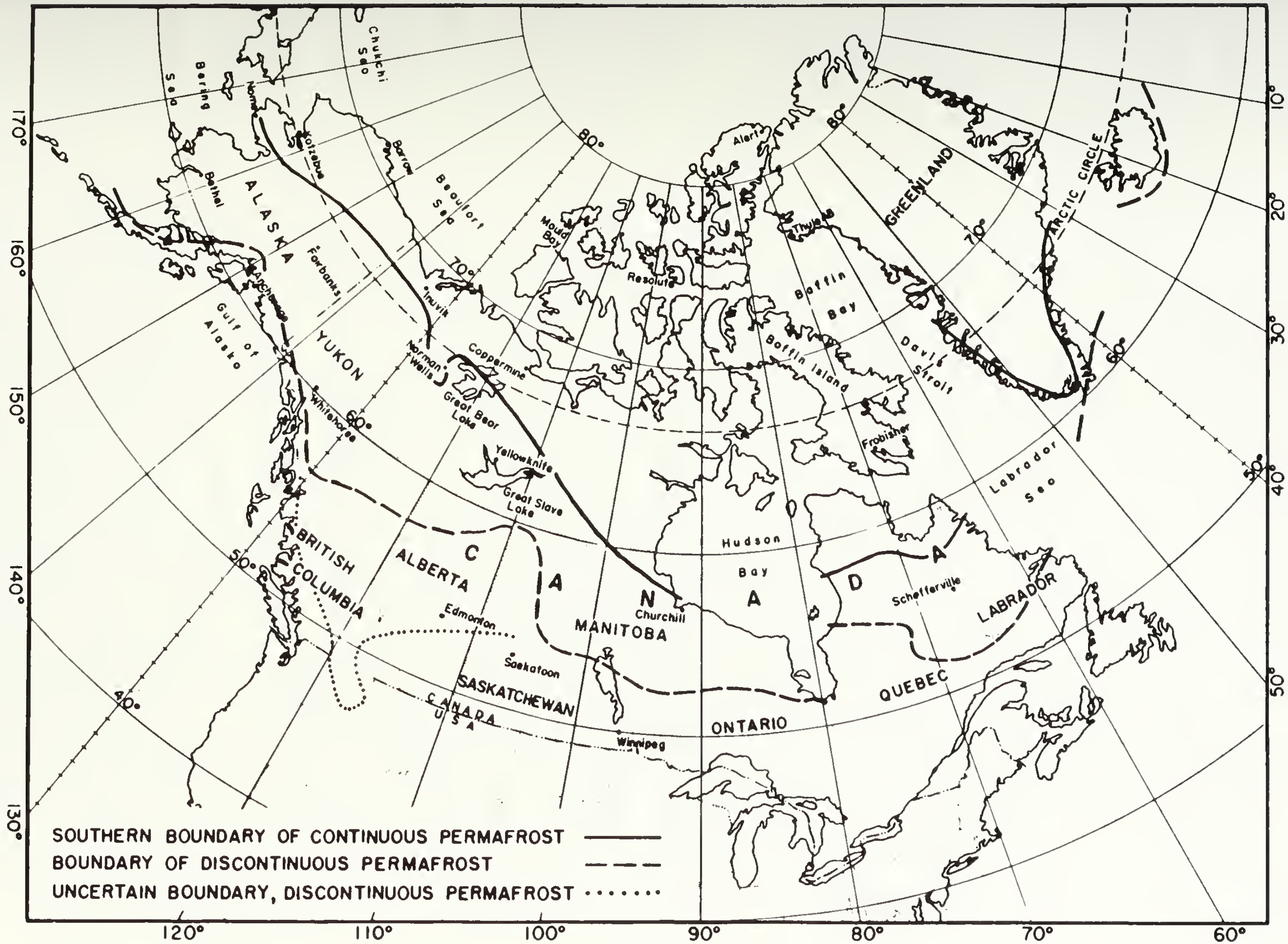


Figure 1 Distribution of permafrost in North America.
(compiled from many sources)

It was once thought that permafrost forms where the mean annual temperature is below 32F. However, examination of Figure 1 indicates that the permafrost line is generally closer to the 23F isotherm than to the 32F isotherm. An actual site investigation is required to determine if permafrost exists at a given location.

Dry frost is dry-frozen ground in which the water content is negligible and there is no visible ice. It is generally found in well-drained sands and gravels above the flood plain elevation. Since there is little or no frozen water cementing particles together, there will be little or no loss of volume or strength if the material thaws.

Wet-frozen permafrost, in which a varying amount of water content exists, is sometimes classified on the basis of pore space and ice volume. Supersaturated material has visible ice, and the volume of ice (V_i) is greater than the volume of the voids (V_v). Similarly, the material is said to be saturated when $V_i = V_v$, and undersaturated when there are open pores and V_i is less than V_v. [7] The ice may occur as crystals, lenses, sheets, layers, blocks, masses, wedges, vertical or horizontal veins, or coating films. If a large portion of the ice is in layers, wedges, or masses, a large volume of water results from melting, leading to loss of shear strength and large settlements as the water drains away. This is a "detrimentally-frozen" situation, and is generally encountered in fine-grained soils.

The above definition of permafrost includes both dry-frozen and wet-frozen ground. Of course for dry-frozen ground, otherwise called dry frost, with little or no pore water, temperature is the only criterion of concern. In wet-frozen ground varying percentages of unfrozen water will exist at temperatures less than 32F. One hundred percent freezing is not required in order that the material be classified as permafrost. The definition requires only that enough pore water to cement the grains together be in a continuously frozen state.

The strength of the frozen soil depends on its temperature. A rise in temperature will reduce the cementing action of the ice resulting in a loss of strength. The extent of the loss of strength is a function of the proportion of ice, and the difference between the existing temperature and 32F. Even at temperatures below -22F a significant amount of water may not be frozen. The strength of permafrost in the temperature range from 14F to 32F, and particularly from 23F to 32F, is lowered appreciably as the temperature increases because of the increase in the percent of unfrozen water and because of the decrease in the strength of the ice itself. [8]

Depending on the degree of ice cementation, permafrost may be classified as solid, plastic, or free-flowing. In solid permafrost the materials are firmly cemented into a solid mass. In plastic permafrost the particles are incompletely cemented together since a good portion of the

water is unfrozen. Plastic permafrost is generally encountered in cohesive soils at temperatures close to their thawing temperature. When the moisture content is insignificant in coarse grained materials and sand, a free-flowing state may exist. In this state the soils are easily worked and may be readily compacted.

A talik, meaning "thawed material," is defined as an unfrozen layer between the active seasonal surface zone and the permafrost below. It may develop where removal of the insulating cover has changed the thermal balance to allow summer thawing of the permafrost below the depth of winter frost penetration. It could also occur in the permafrost below moving water or in well-drained coarse materials in higher elevations. A talik may become an aquifer and maintain its unfrozen state, or it may periodically refreeze. The term does imply, however, that the thawed ground was frozen at some time.

The active zone, or annual frost zone, is the zone in which seasonal freeze or thaw occurs. Seasonal freezing may penetrate vertically to the top of the permafrost, or there may be a talik between the active zone and the permafrost. In areas of only seasonal frost, the soil below the active zone remains unfrozen. The depth of seasonal freeze and thaw is affected by many environmental factors such as vegetation, soil, moisture, exposure, temperature, and snow cover. The critical, dynamic forces causing frost heave and general loss of stability occur within the active zone.

Physical Characteristics

As generally implied in previous sections of this report, frozen soils are comprised of four components:

1. solids (s) - the mineral particles of the soil
2. ice (i) - viscous-plastic, frozen water
3. water (w) - unfrozen
4. air (a) - gas

When considering the characteristics and properties of permafrost, most of the basic principles and terminology used in the soil mechanics of temperate regions apply. However, many of the properties of frozen soil are expressed in terms of the properties of that soil in its thawed state. The following is a brief introduction to the basic terms and characteristics of permafrost as they apply to frozen soil and their significance in regards to foundation engineering.

The structural density of frozen soil (D_s) in its natural state is defined as the weight of the solid particles (W_s) per unit volume (V) of the natural frozen soil. That is:

$$D_s = W_s/V \qquad [9]$$

This figure is used in determining the thermal properties of the soil, the relative compression in thawing of the soil, and in other computations. The structural density of the material per unit volume of thawed soil (D_{st}) must also be known for these purposes.

The relative compression of the soil thawed under a given pressure without allowing lateral expansion is:

$$C_r = (H - H_t)/H$$

where H and H_t are the heights of the specimen in its frozen and thawed conditions respectively.

Considering that the weight of the solid particles does not change after thawing, then:

$$D_s H = D_{st} H_t \qquad [10]$$

or $H_t = D_s H / D_{st}$

By substituting this value of H_t into into the above equation for C_r:

$$C_r = (D_{st} - D_s)/D_{st}$$

Relative subsidence (S_r) is defined as the relative compression as obtained from the above equations for C_r if H_t or D_t is found using a pressure of 36 lb/in^3 (1 kg/cm^3) in thawing. Soils can then be classified by their relative subsidence as follows:

non-subsident ...... when $S_r \leq 0.03$

subsident ...... when $0.03 \leq S_r \leq 0.1$

severely subsident ...... when $S_r > 0.1$ [11]

Another measure of density that is particularly useful with fine-grained soils is the density index (I_d) which is defined:

$$I_d=[(D_s-D_{st,min})/(D_{st,max}-D_{st,min})](D_{st,max}/D_s)$$

where $D_{st,max}$ and $D_{st,min}$ are the maximum and minimum densities respectively in the thawed state that can be attained from the specimen. [12]

The degree of looseness (R) of the soil is characterized by:

$$R=[(D_{st,max}-D_s)/(D_{st,max}-D_{st,min})](D_{st,min}/D_s)$$

Soils are considered solid when $R \leq 0.33$; of average density when $0.33<R\leq 0.67$; and very loose when $R>1$. [13] It is this degree of looseness which indicates the compaction potential of the frozen soil.

The percent pore space (PPS) is the volume of voids over the total volume, or:

$$PPS = V_v/V = 1 - D_s/D$$

It should be noted that the volume of voids includes air, water and ice.

As can be seen from the above equations, the frozen state of the material is of no real significance except in the cementing bond provided by the ice. However, only a part of the moisture which is present freezes.

The extent to which the ground water freezes, or its water phase composition, is dependent not only on temperature, but also on salinity, particle size and gradation, and mineralogical composition of the soil. Smaller particle size and denser soil results in more "securely bound" water. Very securely bound water never freezes, even at very low temperatures, and this portion of bound water is part of the solid particles. In less dense soils the water is more "loosely bound," and depending on the soil type, the water partially freezes at temperatures from 32F, and almost entirely freezes at -4F. [14]

The total amount of frozen and unfrozen water is determined by its moisture content (w), which is the ratio of the weight of the water and ice to the weight of the soil:

$$w = W_{(w+i)}/W_s$$

Moisture content and structural density are interrelated by:

$$D_s = D/(1+w) \quad [15]$$

The relative soil iciness (i) is expressed as the ratio of the weight of ice to the total weight of the water and ice per unit volume:

$$i = W_i/W_{(w+i)} \quad [16]$$

The weight of ice per unit volume is used in determining the settlement of a foundation as a result of thawing. For a given sample, the volumetric weight of ice can be calculated simply as the product of iciness, moisture content, and structural density, iwD_s. [17]

Mechanical Properties

The forces of adhesion are generated by the cementation bonds between the ice crystals and the particles of the mineral structure. Adhesion is produced by films of unfrozen water which envelop the soil particles and the ice crystals. Since the amount of ice and its properties are dependent upon temperature, any change in temperature also alters the adhesive qualities of the permafrost and its strength.

Under pressure, a plastic flow occurs in the ice. The ice tends to melt when under pressure at the points of contact between the ice and the soil particles. When the flow of this water occurs, soil particles are displaced causing creep deformations in the permafrost, and long term strength reductions.

Depending on the degree of stress developed, the creep deformations may become damped (extinguished) with time, or they may be undamped. The lowest stress at which undamped creep occurs is called the extreme long term frozen soil strength. No matter how long the duration of the load application, there will be no soil failure if the stresses are less than the extreme long term strength. [18]

The strength of frozen soil also depends on moisture content. When the amount of water in the solid and liquid phases is raised to approximately fill the pore spaces the strength of the soil increases. At higher moisture contents the separation of solid particles decreases the structural

density of the material and lowers its long term strength.

The distribution and dimensions of ice inclusions have a significant effect on the mechanical properties of permafrost. Continuous ice inclusions improve compressibility much more than do discontinuous ice inclusions. But, soil with thin, frequent ice inclusions result in more settling than do soils with thicker but more infrequent inclusions. [19]

Heaving results when the volume of a soil increases due to the volumetric expansion of water when it turns to ice. About a 9% increase in the volume of water results with freezing. The extent of heaving depends largely on the composition and gradation of the solid particles. The most pronounced heaving will occur in fine grained materials. In rock debris, gravel, and coarse sands with unequal granular distribution, heaving will be considerably less.

Ice does not thaw out uniformly in soil. First the cement ice thaws out, then the ice inclusions. In passing from the solid to the liquid state the water volume decreases by about 9%.

The bearing capacity of thawing soils decreases drastically because the cement ice bonds are destroyed and the shear strength consequently decreases. Internal friction angle ϕ of the thawing soil approaches that of unfrozen soils. Cohesion (c), however, is materially reduced below its value for soil not subject to freezing and thawing. In practice, cohesion is practically zero for many materials. [20]

Thermal Regime

The mechanical properties of frozen soils are very much dependent on temperature. Therefore, the temperature within permafrost and its gradient are very significant in the design of foundations. The thermal regime of permafrost is dependent upon past and present external, surface and internal factors. External factors include air temperature, humidity, wind, precipitation, and exposure. On the surface such things as vegetation, snow and ice cover, and stationary or slowly moving water provide insulation from external factors. Internal factors such as type and composition of soil, water content and flow, conductivity, and density affect the temperature gradient within the permafrost.

Permafrost is subjected to heat influx from the air above, from the earth below, and in some cases from subsurface water movement. The balance of the heat loss or gain between these sources causes the rise or fall of ground temperatures, and the possible freezing or thawing within the permafrost zone. Freezing and thawing indexes are simplified, convenient, but somewhat imprecise measures of this balance, which reflect primarily the external climatic conditions. The freezing index is the sum of degree days below 32F for the year. The thawing index is the sum of degree days above 32F for the year. Both indexes are taken over a continuous period, that is through freezing and thawing seasons. [21] This is significant due to a 3 to 6

month lag between the heat flow and the seasonal ground temperatures. The degree day index should not be used independently since there are so many other factors involved in the thermal balance. Additionally, heat transfer and phase changes enter into the development of permafrost. They depend on heat capacity, volumetric latent heat, and soil conductivity which depends on the kind of soil, structural density, and water content.

The soil itself has both a direct and an indirect effect on the thermal balance and the depth of permafrost. Directly, the heat flow is controlled by the soil's thermal properties, specifically soil type, moisture content and structural density. Indirectly, the soil effects the thermal balance by its drainage properties and its ability to support insulating vegetation.

Vegetation is an important environmental factor contributing to the presence or absence of permafrost by providing insulation to the frozen ground from warmer air. It impedes surface drainage, thereby reducing the melting effect of running water.

The permafrost itself, however, has an effect on vegetation that might otherwise be permitted by climatic and soil conditions. Like bedrock, it is a physical barrier to the downward growth of roots. Instead the root system must spread laterally, and therefore deep rooted vegetation is limited. Also, its low temperature retards the growth of vegetation.

Large bodies of water tend to insulate the earth from the low temperatures of winter while using the warmer summer temperatures to raise the ground temperature. Consequently, there is almost never permafrost under large bodies of water, or if there is it is very deep below the surface.

Moving water, whether surface or subsurface, melts permafrost. Consequently, the depth to permafrost will generally be less on flat terrains and greater on steeper runoff slopes.

The depth of permafrost, and the depth at which the seasonal temperature fluctuations are not felt is of great importance in foundation design. This depth at which the seasonal temperature fluctations are extinguished is called the zero annual temperature range. [22] This value is exceptionally important in determining the bearing capacity of permafrost and the depth of perennial thaw. Since the temperature at this depth does not change, the zero annual temperature range can be determined during the field investigation by measurement. It can also be computed as will be shown in the foundation design chapter of this report.

Another important consideration regarding the thermal regime of permafrost is the effect that construction and human activity will have. Soil temperatures may change due to such things as snow removal, vegetation removal, regrading of terrain, additional shading from buildings, and changes in surface water drainage patterns.

CHAPTER III

BASIC DESIGN CONSIDERATIONS

Soil Conditions

As discussed previously, construction changes the natural temperature regime of frozen soils, and consequently, their structural properties. One of the chief design considerations must be the state of the soil in which the foundation will lay. One of the following criterion will apply:

1. seasonally frozen - thawing in summer and freezing in winter.

2. frozen - kept frozen during construction and for the duration of the life of the installation.

3. thawing - composed of both thawed and frozen soils which gradually thaw during the life of the installation.

4. thawed - kept thawed for the life of the installation.

5. freezing - where soils which are thawed under natural conditions gradually freeze during the life of the installation.

Foundation designs consider bearing capacity and settlement. The problems of design with respect to bearing capacity are to limit the plastic deformation to ensure stability. With respect to the latter, the problem is to limit the settlement such that tolerable limits are not exceeded. Soil heaving properties determine whether the

foundation may be laid in the active zone, that is if the foundation is governed by criterion number 1. In the other cases the heaving properties only make it necessary to make sound decisions about other design considerations.

The extreme load bearing capacity of frozen soil is higher than for thawed soil. In some cases it may be economical to artificially freeze the soil to keep it in this state. However, settlement, not bearing capacity is generally the governing limitation for foundation design. The subsidence of soils is therefore of practical significance in foundation design. The greatest degree of subsidence occurs when the soils thaw. By comparing the degree of subsidence expected to the limits imposed by the structural requirements of the installation, it can be determined whether thawing can be tolerated.

Permafrost will thaw when sufficient heat is transferred to it from the underside of a structure. The rate of thaw depends on the temperature of the building, floor insulation, air space or ventilation provisions, and the soil properties of the active zone and the permafrost. Insulation retards and reduces heat flow from the building, but it does not eliminate it. Air space ventilation or artificial refrigeration can be used to remove the heat. However, such a system must be carefully designed to maintain the thermal regime of the permafrost.

Ground temperatures influence the bearing capacity of the frozen soil and also its creep rates. In areas of

discontinuous permafrost the annual mean ground temperature is very close to the freezing point. As such, difficult engineering problems can be encountered due to the low strength and high creep rates of the soils. In areas of continuous permafrost it is important to maintain the relative stability of the thermal regime. Permafrost will thaw if a long duration increase in surface heat occurs. Additionally, natural thawing can occur from below if the permafrost is responding to a prior colder climatic condition.

The magnitude of seasonal frost heaving is dependent upon several factors including rate and duration of frost penetration, soil type, surcharge, and moisture content. Clean GP, GW, SP, and SW gravels and sands with a negligible percentage of materials smaller than 0.02 mm are relatively non-heaving. If saturated, it is possible to experience a small amount of heaving or freezing, however, and it should not be assumed that heaving or thawing can be neglected without investigation.

The water content of a soil has a substantial effect on the depth of freeze or thaw which will occur at a given surface freeze or thaw index. An increase in moisture tends to reduce the penetration by increasing the latent heat and specific heat capacity.

In the active zone, if a freezing soil has no access to free water other than that contained in its voids, the resulting heave is limited. If water can be easily drawn to

the plane of freezing from an underlying aquifer, the resulting heave can be critical. A water table within 5 feet of the plane of freezing can result in significant heave. However, lowering the water table to a great depth will not guarantee the elimination of heave. The remaining water in the voids of most soils is not easily drained by gravity, and will be available to migrate to the plain of freezing. [23] One factor to consider is that in areas of permafrost the supply of water from below the freezing zone may be cutoff by the presence of underlying impermeable permafrost.

It has been demonstrated both experimentally and in the field that the rate of heave is decreased by increasing the loading on the plane of freezing, and that heave can be restrained if sufficient pressure is applied. In foundation design the heave reducing effect of the load may be considered and taken advantage of to reduce the amount of seasonal heaving. In an experimant conducted in Fairbanks, Alaska it was shown that maximum seasonal frost heave was reduced by 40% with only a 2 psi surcharge and by 80% with an 8 psi surcharge. [24]

For engineering purposes it is important to know whether significant settlement is likely should thawing occur. As a general rule, if the ice present will produce more water upon melting than the void of the soil can retain, then the material is thaw-unstable. If all melted

water can be absorbed by the soil voids without significant settlements, then the soil can be considered thaw stable.

Materials of Construction

The materials to be used for construction must be carefully considered. Foundations of structures in areas of deep seasonal frost and permafrost may tend to experience larger and less predictable stresses than in temperate regions due to thermal expansion and contraction, as well as those stresses induced by the freezing and thawing of the soil. Consideration should be given to logistics, supply, cost, and problems of field fabrication and construction. This information should be determined early in the design as it will influence the foundation type.

Wood

Wood is a satisfactory and widely used material because of availability, low thermal conductivity, adequate durability, flexibility, and the fact that it is relatively unaffected by low temperatures. A principle use of wood is in piling. However, wood piles cannot be hammer driven into into permafrost. Spread footings are frequently made of wood, and heavy timbers are used to distribute structural loads onto pilings or other principle supporting members. Timber is also used as an insulating material under concrete footings. It may be assumed that timber embedded in permafrost will not deteriorate. However, at the ground line, deterioration may be rapid. Creosote treatment is of some value. The moisture content of wood tends to allow for

added strength in cold temperatures over that in warmer temperatures. However, failure of wood in a frozen state is often sudden with no warning.

Metal

The main problem with the use of metal in cold regions is that of cold embrittlement fracture. As temperatures decrease, the hardness, yield strength, modulus of elasticity, and endurance limit of most metals increase. However, many metals will shatter or fracture when subjected to stresses that would be allowable at normal temperatures. The cold embrittlement of materials used should be known and the effects of the possible temperature extremes examined. Tests at Fairbanks, Alaska proved that steel piles in permafrost will not corrode. [25]

Concrete

Concrete is a useful material in cold regions. Problems do occur when it is placed on or near permafrost. It must be protected against freezing for the first 48 hours and for such time as may be needed to meet minimum strength and durability criteria. Beyond this critical point concrete hardens very slowly and below freezing there is almost no strength increase or hardness. [26]

Regulated-set cement, developed by the Portland Cement Association, contains an ingredient which provides very high early strength with the liberation of large quantities of heat as a by-product of the chemical reaction. Tests indicate that concrete containing regulated-set cement will

gain strength even when unprotected when placed in temperatures as low as -10 C. [27] This product does not, however, have extensive field use.

Freezing and thawing cycles in completely cured good quality air-entrained concrete are not generally harmful. However, if the concrete is below standard, or if especially unfavorable conditions exist at the time of the freeze-thaw cycles, the effects could be serious. For example, heaving may produce unacceptable tensile stresses. Therefore, concrete is more favorable in areas which remain frozen.

Masonry

Concrete block or brick masonry is generally avoided in cold regions because of its inability to withstand differential movement.

CHAPTER IV

SITE INVESTIGATION

The site data needed for the design of foundations is similar to that required in temperate regions, with the addition of certain requirements imposed by the special climate conditions. The site information needed includes climate, both general and local, geography, including ground cover, subsurface conditions, thermal regime, hydrology and drainage, transportation facilities and access. Adequate attention to these conditions during the site selection phase can greatly reduce the cost and potential foundation problems. It would minimize design, construction, and maintenance costs a great deal if sites could be selected on deep, free draining, non-frost-susceptible granular materials. Obviously, this cannot always be the case. Careful collection of data will provide a reliable picture of the natural conditions existing at a specific site. This data can then be used for considerations of the design options and the performance of design analyses.

Climate

Climatic data can provide both direct and indirect information to be considered in foundation design. Temperature is the most important data obtained. Freeze and thaw indexes are essential direct information used to determine the depth of freeze and thaw. Precipitation provides indirect data to be used to indicate working

conditions, and also direct data of surface and subsurface drainage conditions. Snowfall frequency, amounts and intensity provide both direct and indirect data. In addition to an effect on working conditions, the snow may impose severe structural loads on the foundation. Additionally, foundation loading conditions are determined to a great extent by wind direction and velocity. If the weight of a footing, or its depth must be increased to resist uplift forces, significant revisions in the ultimate design could result. Combined effects of severe climatic conditions should also be considered.

Thermal Regime

Ground temperatures have been recorded for many locations in North America particularly. This data can be used for preliminary estimates of ground temperature at a new site. However, it is very important to obtain accurate information for a particular site. This information is used in conjunction with air temperature records and other detailed data to determine the thermal regime of the site.

Physiography and Geology

Information on the physiography and geology of the general area including bedrock exposures, glacial landforms, and alluvial deposits can be used to interpret the detailed results of the exploration of the site itself. Information on surface cover conditons can be used for estimating the extent of change in the thermal regime which may be caused by construction.

Soil Materials

The most important information obtained in the site investigation is the description and classification of the soil material itself, including the frozen soil classification. Frozen soil may have very high compressive strengths making subsurface exploration difficult. Superficial exploration, however, may lead to serious problems in the future. Explorations should normally extend to a depth equal to the least width of the foundation, unless ice-free bedrock is encountered at a shallower depth.

Deep core drilling, using refrigerated drilling fluid to prevent melting of the ice provides nearly completely undisturbed samples for a wide range of materials including frozen bedrock. In fine-grained soils at temperatures above 20F drive sampling is feasible and is generally satisfactory at much less expense, time, and effort, although samples may be somewhat disturbed.

Density and Moisture Content

The dry densities and natural moisture contents of both frozen and unfrozen samples should be determined. The bulk densities of typical frozen samples should first be obtained, and then melted and the dry solids and the moisture content, as a percent of dry weight, should be obtained. These values should be plotted vs. depth. Knowing the general range of moisture contents to be encountered, and from the number and thicknesses of ice layers, potential settlement problems due to thawing can be

identified. Quantitative analysis in accordance with the foundation design section of this report can be accomplished to determine the amount of settlement which will occur.

The maximum heave that will occur as a result of the freezing in the voids of a non-frost-susceptible soil (no ice segregation) may be computed by the following formula:

$$h = 0.144 w D_d H \times 10^{-4} \quad [28]$$

where:

h = heave, ft.

w = moisture content, % of dry weight of soil

D_d = dry density, lb/ft^3

H = thickness of deposit, ft.

The amount of heave will be negligible if it is sufficiently well drained so that the excess volume of water corresponding to the expansion of water upon freezing is released. However, because natural soils may not be completely non-frost-susceptible and because drainage below the plane of freezing may not be effective, freezing of upper layers of soil may have made the upper layers to be less dense than underlying materials. This effect may extend as deep as 30 feet. [29]

Thaw Consolidation and Settlement

The amount of settlement which will occur upon thawing of the soil can be readily estimated by measuring the amounts of ice visible in core samples. In cases where the ice is relatively uniformly distributed throughout the soil

rather than as segregated layers, where clear ice is visible but there may still be direct contact between solid particles, or where swelling rather than reduction in volume may occur, thaw consolidation tests should be performed on undisturbed samples. These tests are similar to conventional compression tests, except that the sample is subjected to an initial compressive stress, usually 1 psi, and allowed to thaw and consolidate until primary consolidation is complete. This demonstrates the consolidation which is attributable to the ice content of the specimen. Successive increments of load are then applied in accordance with conventional consolidometer test methods to develop plots of pressure versus void ratio or pressure versus settlement strain to determine the stress which can be expected in the foundation at that level. Using this information, a table similar to the one shown in Figure 2 can be developed for individual locations. By developing a series of these tables for the entire foundation under a structure, the distribution of settlement across the foundation can be determined.

Ground Water Records

Ground water levels should be recorded routinely during the subsurface exploration. In saturated silty soils, which are common in areas of permafrost, the ground water table in the active zone may drop rapidly during the freezing season and disappear well before annual freezing reaches permafrost level. However, in fine grained frost-susceptible soils

with a 95 % or higher degree of saturation, frost heave may continue almost up to the time freeze-up is complete. [30]

Frost Susceptibility

The generally accepted dividing line between soils susceptible and not susceptible to ice segregation is 3% of the material by weight being less than 0.02 mm. Although this is a rough measure, and does not necessarily mean zero frost susceptibility, it does indicate a level of which is acceptably small for most engineering purposes. For any specific soil the exact limiting criteria may be above or below this value.

Figure 2

Example of Settlement Estimate

Depth (ft)	Average pressure (psi)	Strain (in/in)	Settlement of layer (in.)	Cumulative settlement (in.)
10-15	11.6	0.064	3.84	3.84
15-20	13.6	0.036	2.16	6.00
20-25	15.8	0.0425	2.55	8.55
25-30	17.8	0.124	7.44	15.99
30-35	19.6	0.088	5.28	21.27
35-40	21.2	0.071	4.26	25.53

CHAPTER V

SELECTION OF FOUNDATION TYPE

Foundation Selection Factors

The engineering design is developed with regard to many factors including site data, general and environmental criteria, cost restraints, knowledge of state-of-the-art, and specific facility requirements such as the design life, maintenance requirements and capabilities, heat flux from the building, accessibility, reliability of its power source, and intended use and loading conditions. Some of the more critical specific considerations are discussed in the following paragraphs.

Design Life

The design life of a permanent structure is normally assumed to be 25 or 30 years. [31] However, construction in cold regions tends to vary more, in that the design life could range from less than 5 years, perhaps for an oil field facility, to up to a normal design life of 30 years. For this report, permanent construction is defined as that which incorporates the type and quality of materials, and details of construction suitable to serve a specific purpose over a life expectancy of 30 years with normal maintenance. Temporary construction is that which incorporates materials, and details of construction suitable to provide minimum accommodations at low first cost to serve a specific purpose for a short period of time, 5 years or less, in which the

degree of maintenance is not a primary consideration. [32] For design lives in between 5 and 30 years, engineering judgement should used.

Maintenance Requirements and Capabilities

Maintenance requirements in cold regions can range from periodic shimming and releveling at 5 to 10 year intervals, to daily and weekly inspection and maintenance of refrigeration equipment. The costs, and capabilities and availability of maintenance personnel over the design of the facility must be considered during design.

Accessibility

It must be determined whether equipment is available and the site is accessible to various types of construction equipment, especially drilling equipment capable of pre-drilling for piling.

Reliable Power Source

If artificial refrigeration or mechanical ventilation systems are to be used, a reliable power source must be available. The consequences of a power failure must also be evaluated.

Sensitivity to Settlement

The use of the building must be considered with respect to its sensitivity to settlement. If critical equipment is to be housed in a structure, and this equipment cannot tolerate even minute settlement, this fact must be recognized and considered in the selection of the type of foundation to be used.

<u>Foundation Supporting Conditions Not Adversely Affected by Thaw</u>

Whenever possible, structures should be located on clean, granular, non-frost-susceptible materials or rock with no ground ice. When such conditions exist, the foundation design is similar to that for temperate regions. Seasonal frost heave and settlement are relatively small or negligible. However, in cases where the facility is critical and cannot tolerate minute settlement, or if the facility transmits significant vibratory stresses to the foundation, then it may be necessary to use one of the other techniques which will be discussed in following paragraphs.

<u>Foundation Supporting Conditions Adversely Affected by Thaw</u>

Any change in natural conditions which results in the warming of the ground, which in turn causes lowering of the permafrost table is known as degradation. Thawing of high ice content materials may produce volume reduction and settlement of overlying soil and structures. Consolidating soils may also have greatly reduced shear strength. Degradation subsidence in soils is almost always differential and consequently potentially damaging to structures. High ice content materials are fine grained soils and clays. In areas of fine textured soils, design should consider the following alternatives:

1. Maintain the thermal regime
2. Accept the changes in the thermal regime and foundation conditions which will occur and allow

for them in the design

3. Modify foundation conditions prior to construction

These alternatives are discussed in the following paragraphs. (See Figure 3)

<u>Maintain the Thermal Regime</u>

This design approach is applicable to areas of continuous and discontinuous permafrost. However, it will be apparent that it is much easier to maintain the thermal regime in areas of continuous permafrost than areas of discontinuous permafrost. The lower temperatures in the continuous permafrost areas allow a greater margin of safety and allow for a more liberal design.

It is possible to utilize the low temperatures of the freezing season to maintain the frozen conditions in fine grained soils by allowing for circulation of cold air through a foundation ventilation system. By cooling the upper layers sufficiently during the winter so that the materials which thawed during the previous summer are refrozen, progressive lowering of the permafrost table will be prevented, and there will be sufficient "cold storage" to maintain adequate safe soil support temperatures. In conjunction with this technique, seasonal frost heave and settlement can be controlled by placing a sufficient thickness of non-frost-susceptible granular soil. This added soil will retard the flow of heat from a building to the permafrost, and also decrease the depth of freezing and

thawing. Artificial refrigeration may also be used to maintain the thermal regime in some situations.

Accept the Changes in the Thermal Regime Caused by the Construction and the Facility

If small progressive thawing is expected, settlement of structures may be avoided by supporting them on piles frozen into the permafrost at a depth that is well below the anticipated level of degradation. Piles or caissons may also be used for end bearing on ice free rock or other firm underlying formation. End bearing piles are especially effective for structures where foundation ventilation systems are not practical.

Modification of Foundation Conditions Prior to Construction

One procedure under this alternative is to determine the depth of thawed or unfrozen conditions and to pre-thaw and pre-consolidate the soil within this region. Of course, prediction of the depth of the thaw caused by construction is very difficult and is a major disadvantage to using this alternative. If only a thin layer of fine grained material overlays an otherwise satisfactory granular material, this procedure may be more practical.

Another procedure under this alternative is to remove the fine grained frost-susceptible materials and replace them with granular non-frost-susceptible material. The cost of this alternative must be considered carefully along with the availablility of the granular material.

Figure 3 [33]

****

Design Alternatives

I. Foundation Supporting Conditions Not Adversely Affected by Thaw, Use Temperate Zone Approach

A. Use Temperate Zone Approach

II. Foundation Conditions Adversely Affected by Thaw

A. Existing Thermal Regime to be Maintained

1. Permanent Construction
   a. Piling
   b. Spread Footings
   c. Posts and Pads
   d. Ducted Foundation
   e. Artificial Refrigeration
   f. Rigid Structural Base

2. Temporary Construction
   a. Posts and Pads
   b. Sills
   c. Slabs or Rafts
   d. Piling

B. Acceptance of Thermal Regime Changes to be Caused by the Construction and Facility

1. Permanent Construction
   a. End Bearing Piles or Caissons, or Footings to Stable Stratum
   b. Rigid Structural Base (small structures)

2. Temporary Construction
   a. Piling
   b. Perimeter Sills

C. Modification of Foundation Conditions Prior to Construction (Permanent and Temporary Construction)

1. Pre-thaw and Pre-consolidation of Unfavorable Materials
2. Replacement of Unfavorable Materials

****

Design Depth of Freeze and Thaw Penetration

The design depth of seasonal frost penetration, not affected by heat from a structure, should be the maximum found by actual measurement during the site investigation, or by computation if not available. The computation is described in the following paragraphs.

The air freezing and thawing indexes (I) used should be selected on the basis of the life expectancy of the structure and its type. For permanent structures, the air indexes for the coldest year in 30 should be used. For temporary structures, or for structures more tolerant of some foundation movement, the indexes may be based on the coldest year in ten, or even on the mean freezing index.[34] Engineering judgement must be used. The "air index" is taken 5 feet off the ground, while a "surface index" refers to the ground surface. One conversion from ground to air index is to multiply the the air index by the appropriate n-factor from Figure 4. [35]

The volumetric specific heat (C, Btu/ft^3-F) for thawed and frozen soils are as follow:

$$C_t = D_d(c_s + w)$$

$$C_f = D_d(c_s + 0.5w)$$

where:

c_s = specific heat of the dry soil grains (0.17 Btu/lb-F near 32F)

D_d = dry unit weight soil

w = water content

(The specific heat of ice and water are 0.5 and 1.0 Btu/lb-F respectively.) [36]

Volumetric latent heat (L, Btu/Ft^3) is:

$L = D_d w L_s$

where L_s is latent heat of water (144 Btu/lb).[37]

Soils are conveniently divided into 3 groups for computing the coefficient of thermal conductivity (K): coarse grained, fine grained, and highly organic. The coefficient of thermal conductivity also depends on the dry unit weight, water content, and ice content. Figures 5 - 8 [38] can be used to obtain the appropriate value of K.

The depth of penetration of freezing and thawing (X) can be computed as:

$X = \lambda(48KI/L)^{(1/2)}$ [39]

where X is in feet, K in Btu/ft-hr-F, I in degree-days F, and λ is a dimensionless parameter, usually between 0.5 and 1.0, depending on the water content and local conditions. A value for λ can be obtained from Figure 9. When using Figure 9, a is the thermal ratio between 32F and the design surface temperature (T_s). The fusion parameter (u) is:

$u = T_s(C/L)$

Figure 4 [34]

****

n-Factors for Freeze and Thaw

Type of Surface (*)	Freezing	Thawing
Snow	1.0	-
Concrete	0.75	1.5
Bituminous pavement	0.7	1.6
Bare soil	0.7	1.4
Shaded surface	0.9	1.0
Turf	0.5	0.8
Tree covered	0.3	0.4

* Surface exposed directly to sun and/or air without any overlying dust, soil, snow or ice, except as otherwise noted, and with no building heat involved.

****

Estimation and Control of Thaw or Freeze Beneath Structures

Thaw Settlement

Thawing of uniformly distributed ground ice under a uniformly heated building proceeds rapidly near the center, and more slowly at the perimeter, tending to produce a bulb shaped thaw front and a dish shaped settlement surface. In a structure, interior footings tend to settle progressively in the same dish shape at about the same rate as the melting ice. However, a rigid foundation slab tends to develop a space under it for a time until an abrupt collapse may occur.

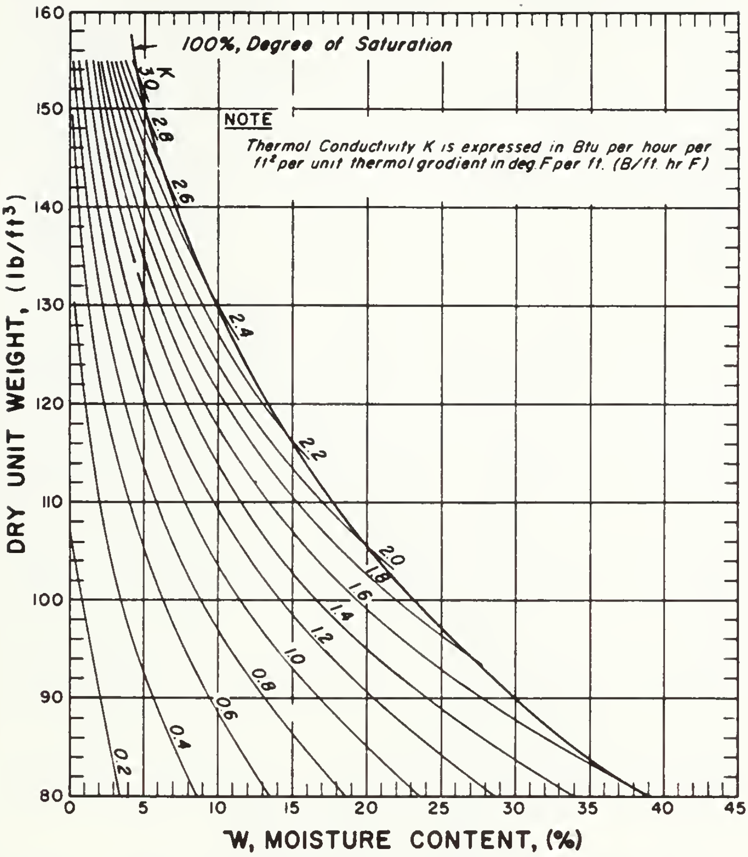


Figure 5 Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils—frozen.

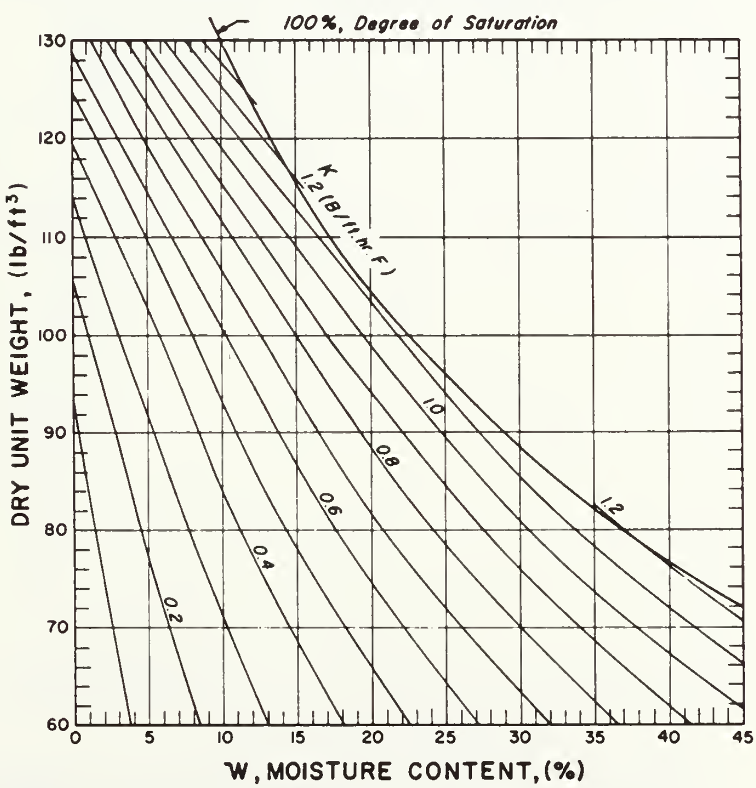


Figure 6 Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils—frozen.

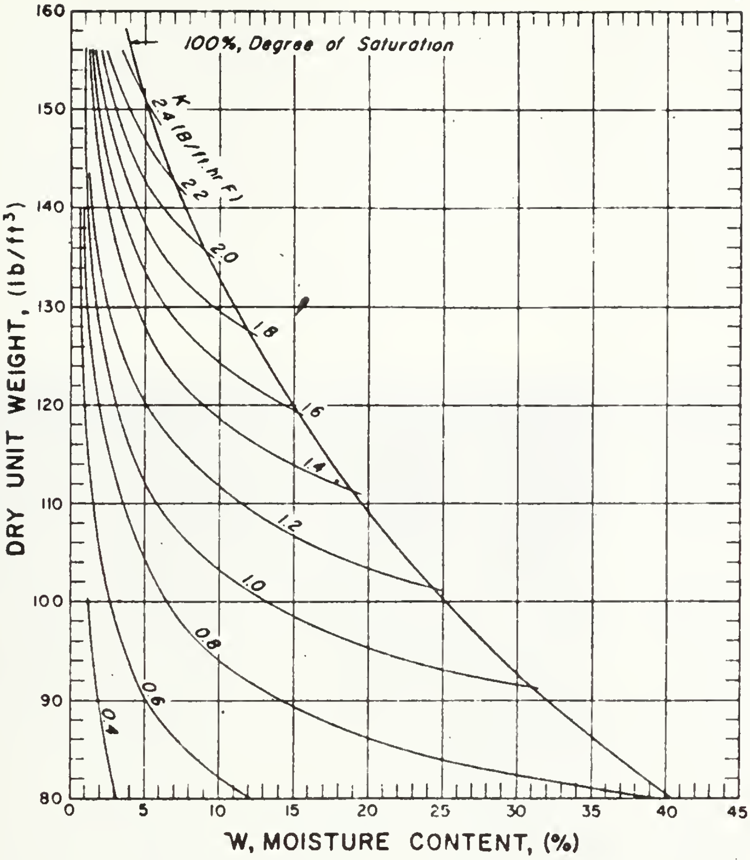


Figure 7 Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils—unfrozen.

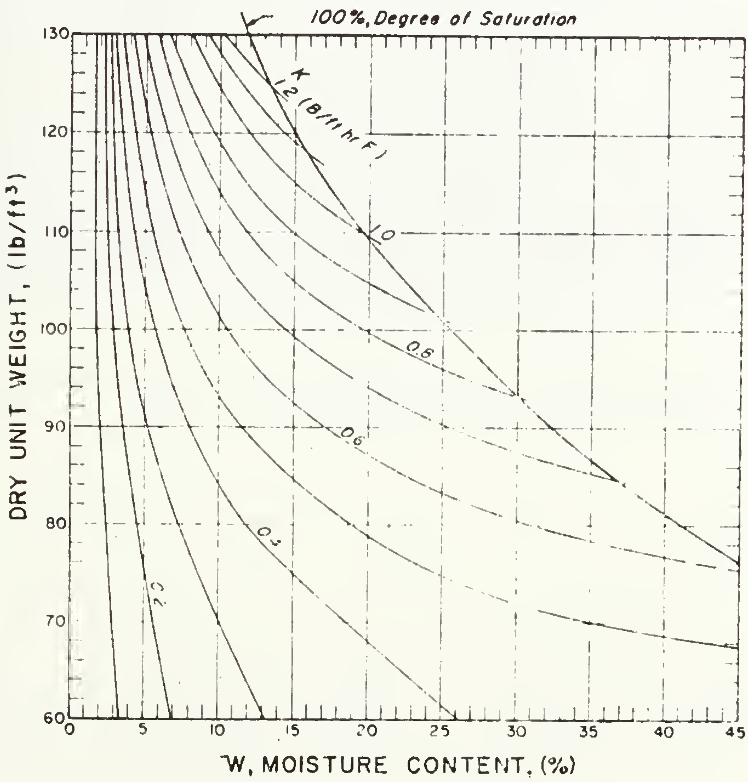


Figure 8 Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils—unfrozen.

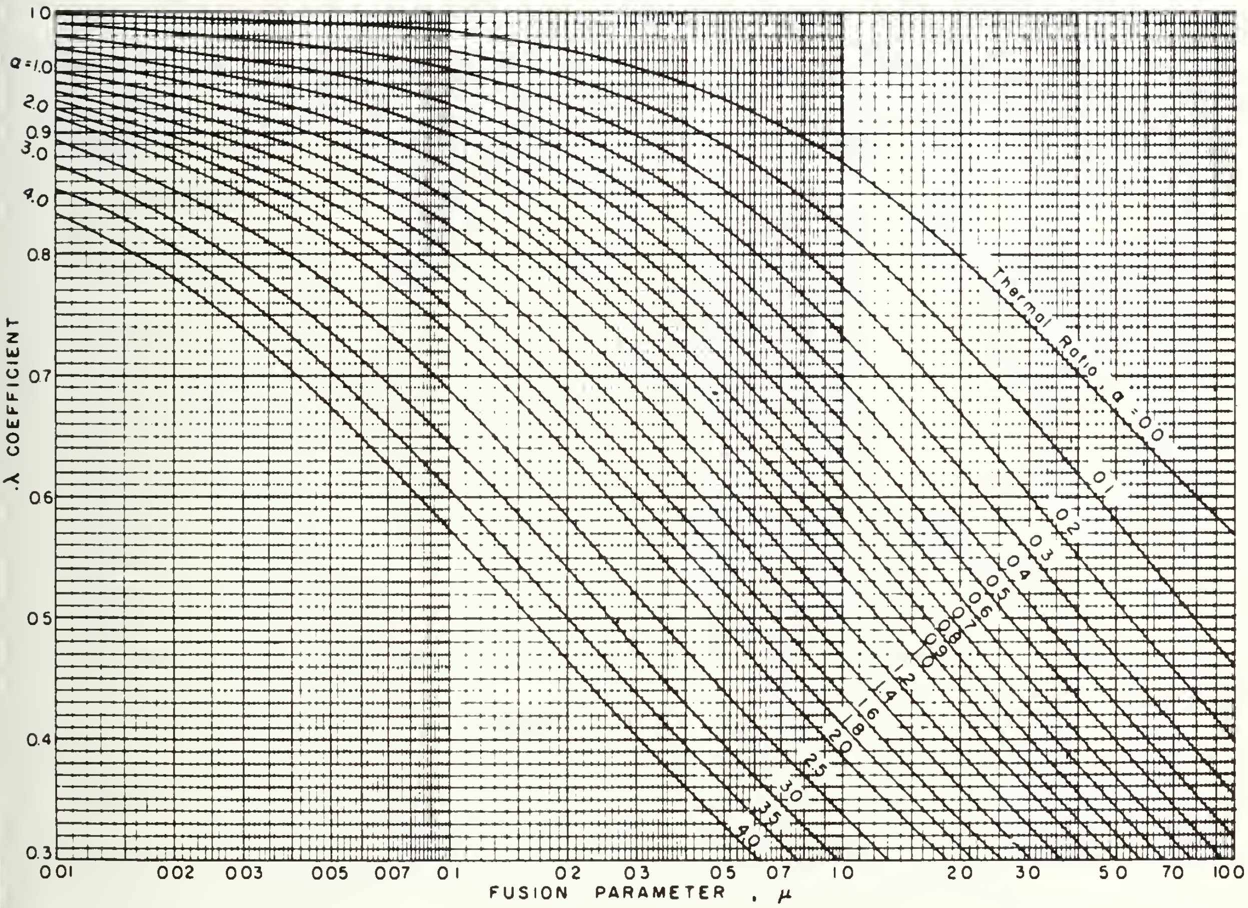


Figure 9 Thermal ratio, fusion parameter and lambda coefficient.

When the floor of a heated building is placed directly on the ground over permafrost, the depth of thaw can be determined by the same computations as described previously, except that the thaw index is based on the temperature difference between the building floor and 32F. [42]

Heave

When frost penetrates downward along the faces of footings, walls, or piles in contact with the soil, an adfreeze bond develops between the soil and the material of the foundation. If the soil freezes and heaves, the structure tends to be lifted with the layer of frozen soil because of this adhesion. As discussed previously however, the weight of the structure acts to restrict the heaving. The maximum upward force exerted on the structure is not limited by the uplift force which can be developed at the plane of freezing. It is actually a function of the adfreeze bond strength and the area of contact.

For practical purposes, a rough approximation of the heave forces on a given structure can be made by assuming that a 2 ton/sf upward pressure acts on the base of a pyramid formed by a 45 degree spread from the footing to the depth of frozen soil. For example, a 4'x6' base at a depth of 3 ft in soil which freezes to a depth of 5 ft, would have an 8'x10' area exposed to the heave force, giving an estimated force of 160 tons. [43] A comparison of the heave forces with the structural load and passive resistance forces will give an indication of the relative balance

between the forces. More precise quantitatve predictions require laboratory testing which are still not overly conclusive, and are generally not considered to be practical. [44]

Ventilated Foundations

The most widely used method of maintaining the thermal regime of the permafrost is by use of a ventilated foundation. The circulation of cold winter air is used to refreeze the upper layers of soil which were thawed during the summer. The ventilation may be open or ducted.

The simplest means of ventilation is by an airspace under the structure which allows the natural flow of air. As a general rule, the height of the airspace should normally be not less than one-tenth the smallest building dimension. However, it is recommended that the airspace be at least 2-3 feet high to ensure adequate ventilation.

Normally the depth of ground thawed under the airspace is less than the normal thickness of the active zone since the building shading effect lowers the thawing index. This shading effect may be taken to be about 15%. [45]

If a building is so large that adequate circulation will not be achieved through a simple airspace, or if at least 2 feet of airspace is not possible, or if heavy floor loads make provision of an open airspace difficult, a ducted foundation ventilation system may be used. A ducted system may utilize natural air circulation, or incorporate a forced draft system to increase the volume of air flow. Several

factors must be considered in the design of a ducted system. These factors include the following:

1. Damage to the duct may result from vertical movements. As such settlement and heave must be eliminated.

2. Accumulation of ground water or ice and snow must be prevented. The foundation should be sloped and elevated as necessary.

3. The system must be shut off in the spring when air temperatures reach a level such that continued circulation through the system would add to the warmer seasonal heat input. The human factor of forgetting to open or close dampers or stacks must also be considered.

4. Use of a mechanical and electrical blower system adds maintenance burdens and potential problems when failure occurs.

5. The construction must be airtight.

The most widely used and practical ducted foundation system consists of steel ducts approximately 20 inches wide and 12 inches high, spaced about 2 feet 8 inches on center. Associated with the ducts are a base slab of 4 or 6 inches, an 8 inch concrete wearing surface, a layer of cellular glass, or other insulation, and 6 inches of concrete between the insulation and the roof of the duct. Figure 10 shows a typical section.

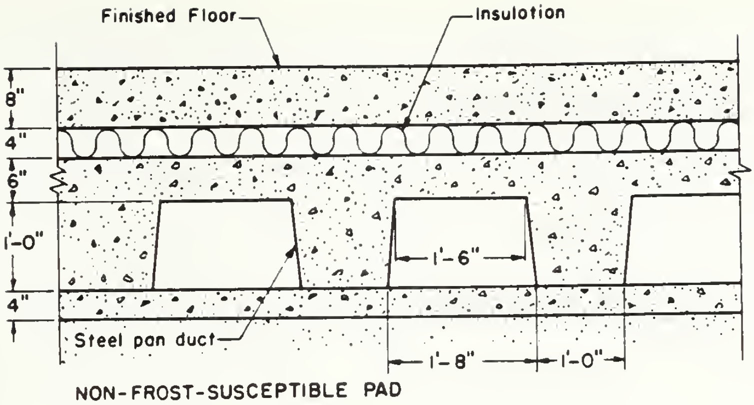


Figure 10 *Section of pan-duct floor.*

If the ducts are more than 20 feet long, special provisions for draft may be required. It is preferable to avoid a mechanical system if at all possible. To attain the necessary circulation, the stack height may be raised to increase the chimney effect. A general rule of thumb to determine the required height of the stack is:

$H = (1/140)\{L/[v_s(t+1)]\}^2(L + 250)$ [46]

where:

v_s = the air freezing index in degree days divided by the length of the freezing season in days

H = the stack height in feet

L = the length of the ducts in feet

t = the actual, or equivalent thickness of cellular glass insulation in inches based on thermal resistance.

If the resulting stack height is higher than other conditions may allow practically, a greater thickness of insulation may be used. Insulation can reduce the quantity of heat flow, but it does not prevent it entirely. Thus, insulation should not be depended upon solely, to prevent thaw under a heated building. Also, care must be taken to ensure that the insulation remains dry.

Artificial Refrigeration

When the heat transfer requirements are known, a surface artificial refrigeration system can be designed using normal procedures for refrigeration systems. The "American Society of Heating, Refrigerating, and Air Conditioning Engineers' Guide and Data Book" is a recommended reference for design.

Granular Mats

A mat of non-frost-susceptible granular material placed at the start of construction serves to control the seasonal freeze and thaw effects, to provide a stable foundation support, and to provide a working platform for construction. When the mat is needed only for a construction work platform, and control of freeze and thaw is not required, it need only be thick enough to carry the loadings which may be applied during construction. Thickness can be determined using flexible pavement design curves.

When used as a foundation support, the mat should be of sufficient thickness to achieve any needed heave reduction through its surcharge effect. For temporary facilities, or

light flexible structures, complete frost control may not be required or economical. For facilities which require a foundation free from frost heave or settlement, the mat should be thick enough to keep the depth of penetration of freezing and thawing within the mat. The surface of the mat should extend outside the perimeter of the structure and any foundation members at least 5 feet before sloping down to ground surface.

Anchorage Against Heave

Special anchorages have been tried in the past for pile foundations, but found to not usually be worthwhile. An increased dead load, if bearing capacity will allow, is a simple means of anchoring against heave forces. Particularly, the granular mat is relatively inexpensive if material is available. Additionally, a general rule that embedment should be twice the thickness of the normal active zone is usually satisfactory. although three times is often used. [47]

CHAPTER VI

DESIGN OF FOOTINGS AND RAFTS

Allowable Design Stresses

The ultimate strength and deformation characteristics of a saturated frozen soil depend primarily on its temperature relative to 32F, and the duration of time that the soil will be subjected to a given stress. As temperature decreases, ultimate strength increases as has been discussed previously. Additionally, as the time that a stress must be resisted increases, the ultimate strength of the soil decreases.

Experiments have shown that the long-term shear strength of saturated frozen soil can be determined by what is known as the Vialov equation:

$$s_t = (B)/[\log(at_f)]$$

where: s_t = the constant stress level at which failure will occur at time t_f in psi

t_f = period of time after application of stress (s_t) that failure will occur in hours

B and a are soil constants that are temperature dependent. [48]

It should be noted that the Vialov equation is not valid for short periods of time.

When a cylinder of frozen soil is loaded axially in compression at constant temperature and load, and time is plotted vs deformation, at some time t_f, the curve will

start an upward swing, indicating the onset of uncontrolled flow. See Figure 11a. If many tests are conducted at the same temperature, but at different stresses, and the stress is plotted on a base of t_f, the result is a curve for each temperature. See Figure 11b. This curve asymptotically approaches the stress at which flow would never occur at that temperature.

The constants B and a in the Vialov equation can be found by lab tests on four or more sets each of four specimens, for each temperature. The logarithm of time and reciprocal of stress are plotted and the best straight line drawn through the points, as shown in Figure 11c. B and a are then computed from the slope, and $1/s_t$ at time 1 hour.

The above method is a rational approach that could be used, but generally is not used for design.

The load carrying capacity of frozen soil is essentially determined by its shear strength and within certain limits the relationship between shear strength and normal pressure, and may be written:

$$s = c + p\tan \phi$$

where: c = cohesion

p = total normal pressure stress

ϕ = angle of internal friction

One design approach is to assume a purely cohesive soil. This is a conservative approach since the internal friction term is assumed to be zero. Actually, internal friction can contribute substantial shear strength in some frozen soils,

such as unsaturated frozen soils. [49] However, the determination of the angle of internal friction requires tests for the specific in-situ conditions, whereas a value for the cohesion can be determined by relatively simple unconfined compression creep tests. Figure 12 shows Russian code recommendations for footings on permafrost as of 1960. The US Army Cold Regions Research and Engineering Lab considers these values to be reasonable, although quite conservative, especially compared to data listed by Russian researchers. [50]

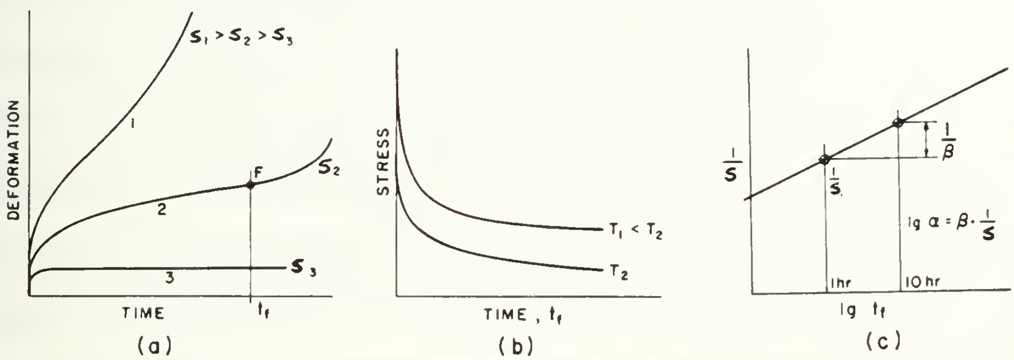


Figure 11 Creep curves for frozen soils.

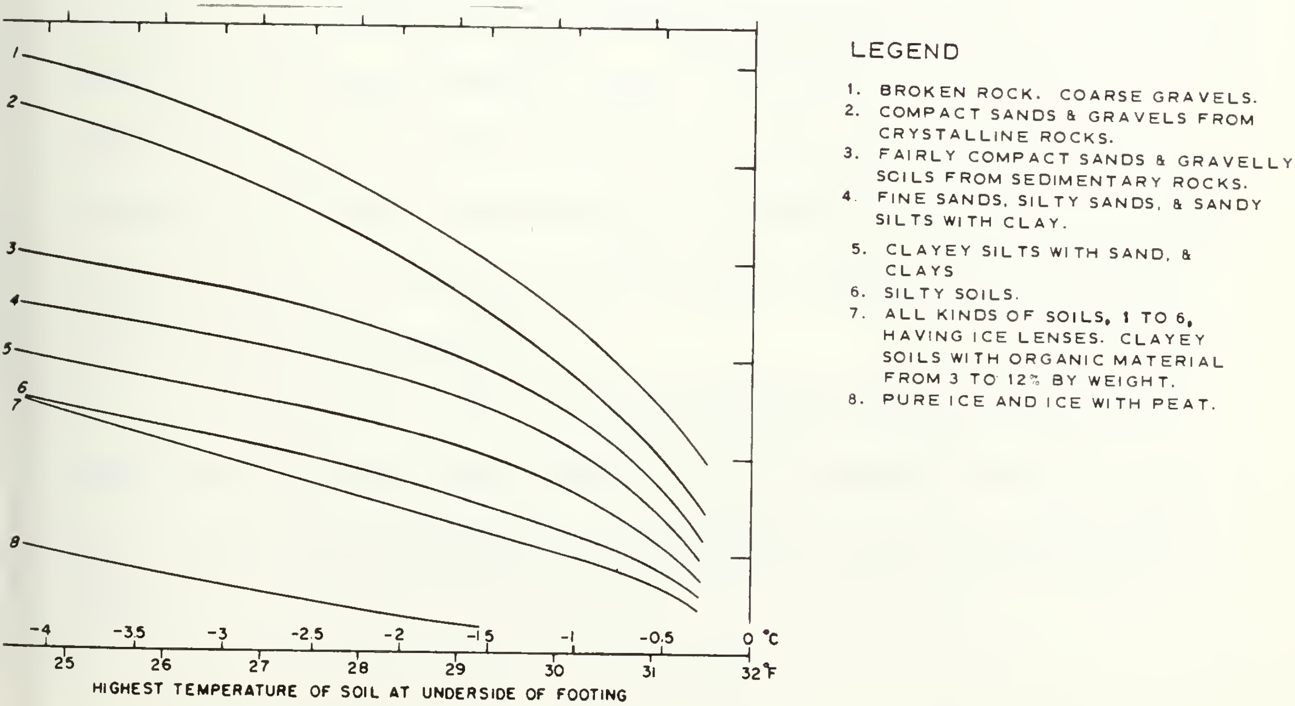


Figure 12 Design bearing pressure and temperature, USSR standards, 1960

Footings and Rafts

Footings and rafts share the common characteristic of developing load capacity through the bearing of a horizontal surface of adequate dimensions on the supporting soil stratum. Suitable foundations of these types should be safe with respect to bearing capacity failure, keep displacement, settlement, heaving, or creep deformations within acceptable limits, and be economical to construct and maintain.

Footings

A footing is an enlargement of a column or wall to distribute concentrated loads over a sufficient area so that allowable soil pressures will not be exceeded. With proper design, these types of foundations can be used on permafrost, although they may not be as economical as piles or other alternatives. Individual column footings are usually preferred over continuous footings. Footings on frost-susceptible permafrost soils may be placed on the surface of, or within a foundation gravel mat, or below the depth of maximim seasonal thaw. In the latter case, a granular layer of clean gravel or sand should be placed immediately below the footing to provide a suitable working and placement surface, and to reduce bearing pressures on the underlying material.

If soils are non-frost-susceptible clean, granular material, conventional temperate zone design practice may be used, and footings should be placed at a minimum depth of 4 feet.

For small structures with light floor loads, or temporary buildings, a variation of the continuous footing is the sill, or mud sill footing. These are generally structural members, such as timber, which rest directly on the soil and support the structure.

A shallow footing is one whose width is equal to or greater than the vertical distance between the surface of the ground and the base of the footing.

Rafts

Rafts, or mat foundations are combined footings which cover the entire area beneath a structure and support the walls and structural columns. They are generally used where heavy floor loads are required. Rafts are usually reinforced concrete slabs on gravel mats, with various means of insulation and ventilation. Differential movement is usually minimized in this type of foundation.

Steps for Design of Footings on Permafrost

1. Determine the required depth of the footing using the procedures discussed in Chapter V, and the appropriate information from Figure 13.

2. Determine the temperature distribution in the permafrost with respect to the base of the footing for the critical period of the year.

3. Determine the bearing capacity of the soil as discussed in previous paragraphs of this chapter.

4. Select a trial footing size and check the bearing capacity of the trial footing size for the critical period

of the year using Figure 13, and standard soil mechanics theory, and soil properties determined from step 3 above.

5. Adjust the footing size to obtain the desired factor of safety, usually 2.0 for dead load plus normal live load, and 1.5 for dead load plus maximum live load. [51]

6. Check settlement in accordance with Capter IV. If the estimated settlement will be excessive, enlarge the footing or revise the design to reduce settelment to an acceptable level.

Figure 13

a. Ultimate bearing capacity of shallow foundations

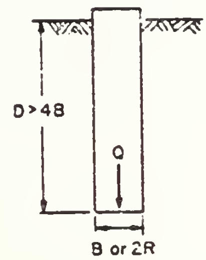
Strip footing:

$$q_u = Q/B = cN_c + Dd_fN_q + 0.5DBN_\gamma$$

Rectangular footing:

$$q_u = Q/(BL) = cN_c[1+0.3(B/L)] + Dd_fN_q + 0.4DBN_\gamma$$

b. Ultimate bearing capactiy of deep foundations



Strip loadings:

$$Q/L = cBN_c + Dd_fB + 2d_ff_s$$

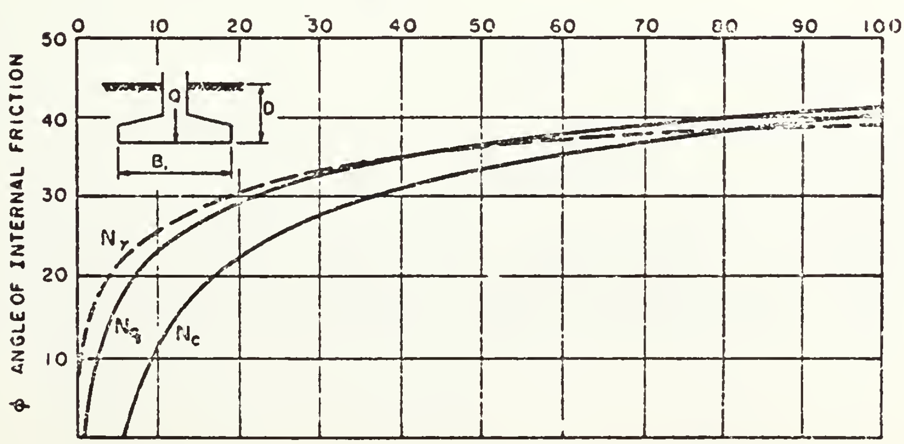
Square loading:

$$Q = cB^2N_c + Dd_fB^2 + 4Bd_ff_s$$

where:

Q = total load of the footing, lb.
q_u = ultimate bearing capactiy, psf.
c = allowable long term cohesion.
D = Unit weight of the soil.
L = Length of footing.
B = Width of footing.
d_f = depth of footing.
f_s = skin friction.
N_c, N_q, N_γ = dimensionless bearing capacity factors.

N_c, N_q, and N_γ, Bearing Capacity Factors



CHAPTER VII

PILING

Piling offers many advantages on frost-susceptible soils in areas of deep seasonal frost or permafrost with high ice content. Pile foundations can be constructed with minimum disturbance of the thermal regime, and limited degradation of the permafrost. Piling can also isolate the structure from the heaving and subsidence movements of the active zone by transferring the structural load to depths where soil supporting strength remains relatively stable throughout the life of the structure.

Pile Types

Timber

Timber piles offer many advantages. They are normally less expensive than other types, easy to handle, usually readily available in lengths from 30 to 70 feet, and are durable enough to last for unlimited periods of time as discussed previously. The material most often used is creosoted Douglas fir or southern pine. Excess creosote must be avoided because of the possibility of bleeding, with resulting loss of skin friction.

Steel

H-piles and pipe piles are the most commonly used steel piles. Normally, the average compressive stress should not exceed 9,000 psi. [52] Laboratory tests should be made to determine if the soil and water that the steel will be

exposed to will cause corrosion. If corrosion protection is required, a coal-tar preservative over a lead based primer is recommended. However, loss of skin friction should be considered if corrosion protection is applied. [53]

Concrete

When concrete piles are used, care must be taken to ensure that unacceptable tensile stresses due to heave forces are adequately resisted by reinforcing steel to prevent cracking of the concrete. Prestressed precast piles may be advantageous. Cast-in-place concrete piles are not generally used.

Thermal Piles

Thermal piles are used to carry structural loads, and to aid in maintaining the thermal regime by removing heat from the ground and dissipating it in the air above the surface.

In a two-phase system, a pipe pile is charged with propane, carbon dioxide, or some other evaporative material. Evaporation of this material by heat flow from the ground, and its condensation in the portion of the pile exposed to cold temperatures above the ground surface provides the heat transfer mechanism. Finned radiation surfaces are commonly used above ground. Condensation returns by gravity to the liquid reservoir at the lower end of the pile. In a single-phase system, the pile is filled with a non-freezing fluid. Heat is moved upward from the ground by a natural circulation induced by a density gradient of fluid resulting

from the temperature difference between the top and bottom of the pile. Both of these systems are self-initiating, and will cease when air temperatures become warmer than the temperature at the embedded end of the pile.

The rate of heat transfer (q) is expressed:

$$q = h_c A (T_v - T_a)$$

where:

q = rate of heat transfer, Btu/hr

h_c = surface transfer coefficient, Btu/ft^2 hrF

A = surface area of the pile exposed to air, ft^2

T_v = temperature of refrigerant vapor, F

T_a = ambient air temperature, F

A forced fluid circulation system could also be used. This type of system involves a pump to circulate the nonfreezing fluid through the pile and through a heat exchanger exposed to the atmosphere. The refrigeration may be used temporarily to aid in freezeback, or it may be permanent if design conditions require.

Pile Emplacement

Installation in Dry Augered Holes

Holes can be drilled in permafrost using conventional augers with special bits for frozen soil. Advantages include minimum effort and equipment, accurate positioning and alignment, and accurate control of hole size. Drilling is easily carried out in winter conditions when the frozen ground permits ready mobility, without problems of handling water or steam. However, this method is not likely to be

feasible in coarse or rocky material. Most commonly holes are oversized and a soil-water slurry placed in the excess space around the pile. The slurry can often transfer loads to the surrounding soil more effectively than the original in-situ soil.

If this procedure is used, certain considerations must be observed:

1. The soil material used in the slurry may be silt, gravelly sand, sand, or silty sand. It should not be gravel, unsaturated soil, clay, organic matter, or water alone. Concrete should also not be used.

2. The slurry should not exceed 40F maximum.

3. No more water than necessary to saturate the soil and produce a consistency of a 6 inch slump concrete should be used in the slurry.

4. The slurry may be placed by direct backfilling using a wheelbarrow or concrete bucket, or a tremie pipe. It is important that the slurry be vibrated to ensure that no bridging or voids are left along the pile.

5. Timber and closed ended pipe piles tend to float up when backfilled with a silt-slurry. It may be necessary to restrain or weight down the pile.

6. Backfilling may be accomplished prior to placing the pile. If so, it is more difficult to plumb and position the piles. However, if these are not critical factors, this is a much faster method.

7. In order to minimize the amount of heat introduced

into the ground, the hole size should be just large enough to efficiently place the pile and vibrate the slurry.

Installation in Bored Holes

Holes for piles may also be made by rotary drilling or drive coring under some conditions using various bits, and removing frozen materials with air or water. Procedures are otherwise the same as for dry augered holes. Use of water or air to remove cuttings may introduce unacceptable quantities of heat into the permafrost and must be controlled.

Installation by Driving

If local conditions permit, simple pile driving is the preferred method, although only steel piles can be driven with conventional methods. In cold temperatures diesel equipment is best suited for the task. Although the pile is heated by the driving action, the amount of heat introduced into the soil is usually negligible.

Installation by Steam or Water Thawing

Until the 1950's, piles were traditionally placed in permafrost by prethawing the ground before driving. However, too much heat is introduced into the ground, and this procedure is hardly ever used anymore.

Pile Rockets

Investigations by the US Army Cold Regions Research and Engineering Lab indicate that use of rockets for driving of piles is feasible. However, field testing is not yet complete. Regular use of this method may be in the future. [54]

Freezeback

Piles in permafrost attain their holding capacity after they are frozen solidly in place. Pile supporting capacity is dependent primarily on the strength of the adfreeze bond between the pile and the frozen soil. The strength of the bond is a function of temperature, and is the highest when temperatures are the lowest. Therefore, any transfer of heat to the foundation will decrease the capacity of the pile. Freezeback of slurry or thawed soil surrounding piles must be ensured before applying loads to the pile. Thus, it is necessary to determine the time required for freezeback, and consider this restraint in the construction schedule.

Natural Freezeback

Soil-water slurries placed in drilled or augered holes introduce heat which is conducted into the surrounding permafrost. The volumetric latent heat (L, Btu/ft^3) of the slurry is computed as follows:

$$L = L_s w D_d$$

where:

L_s = latent heat, 144 Btu/lb of water

w = water content

D_d = dry unit weight, lb/ft^3

The heat content of water, soil and pile can be computed if the water content and dry unit weight of the soil in the slurry are known, or can be determined by experiment. Slurry placed at temperatures from 32F to 40F, under normal conditions has a total heat content above 32F of less than

200 Btu/ft^3. This value is small enough to assume, and can be added to the latent heat computed above if applicable. [55]

The affect of excess water can be seen to have a pronounced impact on the amount of heat introduced into the permafrost. When the moisture content of the slurry is controlled, the slurry will retain relatively uniform characteristics after freezing. However, if there is excess water, consolidation of the solid particles in the slurry may result in separation of the excess water from the slurry.

Freezeback of slurry proceeds from the wall of the hole inward towards the pile. A layer of ice may form on the surface of the pile, particularly in fine grained frost-susceptible soils. Since the allowable bearing capacity of the pile depends on the strength of the adfreeze bond between the pile and the frozen soil, this layer of ice may be significant. If piles are placed in temperatures below freezing by the slurry method, it should be assumed that the layer of ice has formed. As such, the allowable bearing capacity of the pile should be based on the weaker of pure ice or consolidated slurry. It may be possible to avoid the formation of an ice layer on the pile during installation at low temperatures by use of a non-frost-susceptible slurry. However, as stated previously, "non-frost-susceptible" soils are not necessarily completely non-frost-susceptible. They are merely soils in which the

degree of frost susceptibiltiy is tolerable. This problem does not occur when piles are driven.

Figure 14 may be used to determine the time (t) in hours required for freezeback using the heat capacity (C) in Btu/ft^3-F and the coefficient of thermal conductivity (K) in Btu/ft-hr-F of the permafrost, the pile hole radius (r) in feet, and the latent heat of the slurry (L) per foot of pile length. T_s is the initial temperature of the frozen soil in degrees F. [56]

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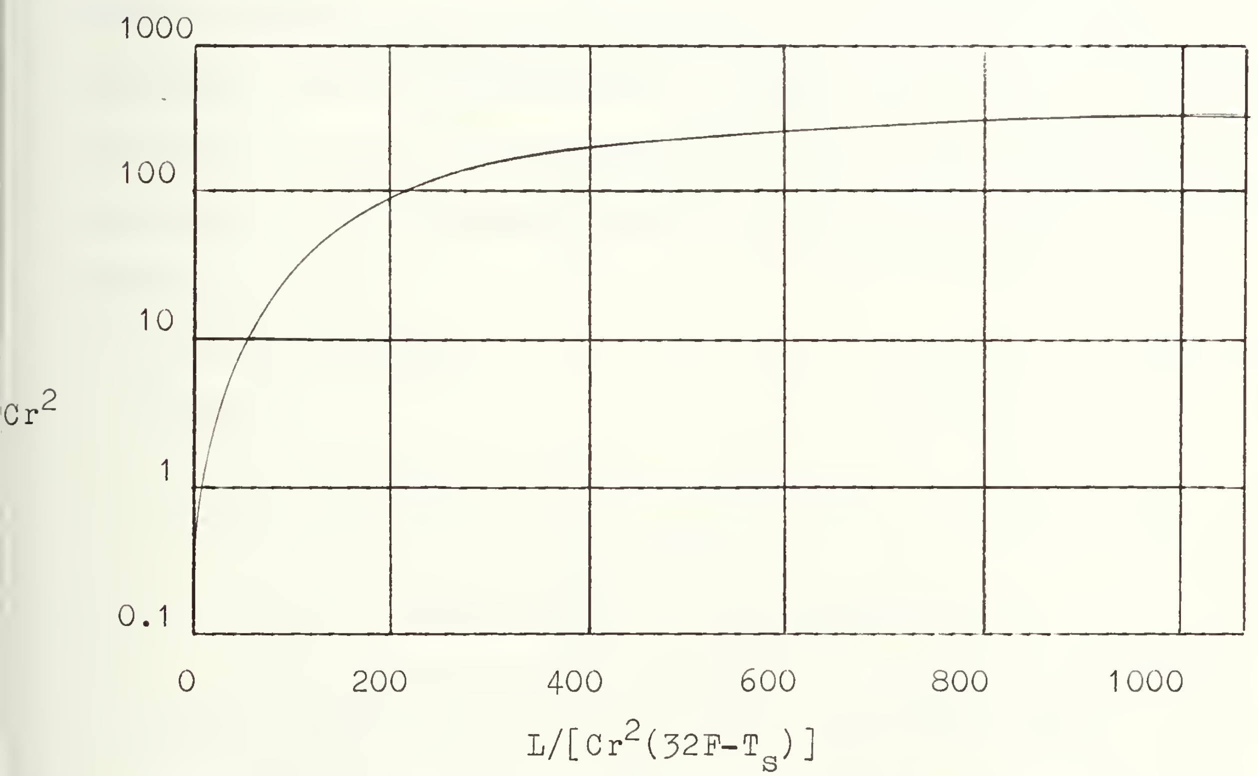
Figure 14



Artificial Freezeback

When ground temperatures are too high, or the amount of heat introduced into the permafrost is so high that natural freezeback cannot be accomplished within the allotted time for construction, artificial freezeback can be utilized. This normally is accomplished by circulating a refrigeration fluid through tubing attached to the pile. The fluids most commonly used are glycol, propane, and cold air.

Other Freezeback Considerations

The spacing of piles is normally based on structural requirements. However, consideration of pile spacing in the early stages of design may provide substantial savings. The distance between piles has an effect on whether natural freezeback can be satisfactorily accomplished. The relationship between pile spacing and the amount of heat introduced into the permafrost can be taken from the following equation:

$$Q = C(S^2 - A)T_{inc}$$

where:

S = the spacing of the piles, ft

A = the area of the hole, sf

T_{inc} = temperature rise of permafrost, F

Q = Latent heat of the slurry, Btu/ft of pile [57]

Since the natural freezing rate of slurried piles depends on the initial ground temperature, the months of February, March, April, and May provide the period of lowest ground temperatures. On the other hand, driving of piles

may be an advantage if conditions are such that the likelihood of natural freezeback is marginal.

Design Depth of Piles

Pile foundations must be designed for sufficient depth of embedment to support imposed loads in adfreeze bond, without unacceptable settlement, unless suitable end bearing on ice-free rock or other reliable strata can be obtained. Additionally, the piles must resist negative skin friction from consolidating or thawing soils, and provide sufficient anchorage and tensile strength to prevent upward displacement or structural failure to the pile from heave forces. The forces acting on a pile in permafrost in freezing and thawing seasons are shown in Figure 15.

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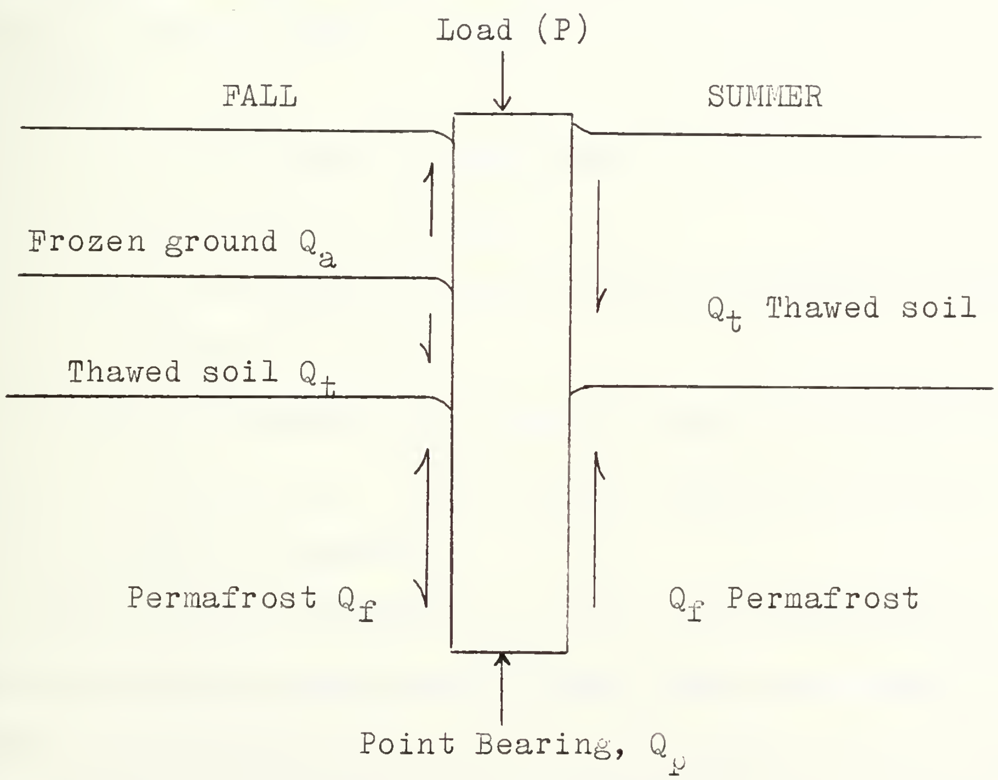

Figure 15

Friction Piles

During the spring and early summer months when the ground temperatures are at their lowest, the adfreeze bond is the strongest. Likewise, during the late summer and early winter, the adfreeze bond is at its lowest because of the warmer ground temperature. Unless the pile is sufficiently embedded during this freezing season, the pile will heave if the heaving force exceeds the combined weight of the pile and its load, the adfreeze bond strength, and any negative skin friction which may be present in the thawed zones. The bearing capacity of friction piles should be calculated for the ground conditions that exist during this critical time of the year. The allowable downward load on the pile will be:

$$Q_a = (Q_f \pm Q_t)/FS$$

where:

Q_a = allowable design load

FS = Factor of safety

Q_f = maximum load which may be developed in the permafrost zone

Q_t = maximum skin friction from the thawed soil on the pile. Under normal summer conditions this will be negative

The allowable loadings on a pile are determined by creep deformations which occur under continuous loads. Creep occurs in the permafrost zone at the contact points between the pile and the frozen soil. Non-elastic deformation

begins shortly after the application of the load. Immediately after a load is applied, compressive strain deformation is concentrated in the upper portions of the pile. As yield occurs in the permafrost and the adfreeze bond, more and more of the compressive strain is transferred down the pile until a point of equilibrium within the pile is achieved. The final distribution of load and strain along the pile is approximately triangular, with zero load carried at the point.

The effective unit values of the strength of adfreeze bond between the pile and the permafrost depend on many factors including soil type, moisture content, water chemistry, temperature, and the shape, length and material of the pile. Figure 16 [58] shows values of average sustained and average peak adfreeze bond strengths for frozen silt slurries in contact with steel pipe piles. These values were determined experimentally. Factors for adjusting the curves for different types of piling and slurry materials are also shown. Figure 16 applies only for soil temperatures down to 25F, for the warmest time of the year. Although H-piles have a larger surface area than equivalent circular piles, they do not develop a proportionately greater increase in load carrying capacity. [59]

When piles are driven into permafrost, the adfreeze bond is less complete than with slurried piles. Therefore, the allowable load should be reduced to 75% of that for a slurried pile. [60]

To determine the allowable load on a pile using the previously given equation, the maximum load developed in the permafrost zone can be taken as the product of the average sustainable adfreeze strength and the total surface area of the pile embedded in permafrost, and applying the appropriate correction factor. If the ground temperature is found to vary considerably with depth a more refined computation can be made using layers of various adfreeze strengths. The load carried, or added by skin friction in thawed zones is computed using conventional procedures for temperate regions.

The possibility that the foundation soil may have a lower shear strength than the adfreeze bond strength must be considered. If site investigation results indicate that this is a possibilty, the normal procedure is to intentionally oversize the hole by about 50%, and a select slurry backfill used. This will decrease the stress on the natural soil and permit the allowable strength of the slurry to control. The increased volume of slurry, however, will increase the freezeback time.

End Bearing Piles

When using the procedure for friction piles, the end bearing is assumed to be negligible. If a firm, reliable stratum is available within an economical depth, the bearing capacity can be augmented, or solely derived from point bearing using conventional procedures for temperate zones, except that a factor of safety against heaving may be required.

A satisfactory relationship against heaving is expressed in the following equation:

$$Q_a = (P+Q_f+Q_t)/FS$$

where:

Q_a = the frost heave force in the active zone

P = the structural load including the pile weight

To calculate Q_a, a value of 40 psi is assumed for the adfreeze bond strength of a steel pile in a silt slurry. This figure can be adjusted using the correction factors from Figure 16, and multiplied by the total surface area of the pile in the active zone to determine the frost heave force.

Factors of Safety

Recommended factors of safety for load determinations for friction piles are at least 2.5 for dead load plus normal live load, and 2.0 for dead load plus maximum live load. Factors of safety against heave are recommended to be 4.0. [61]

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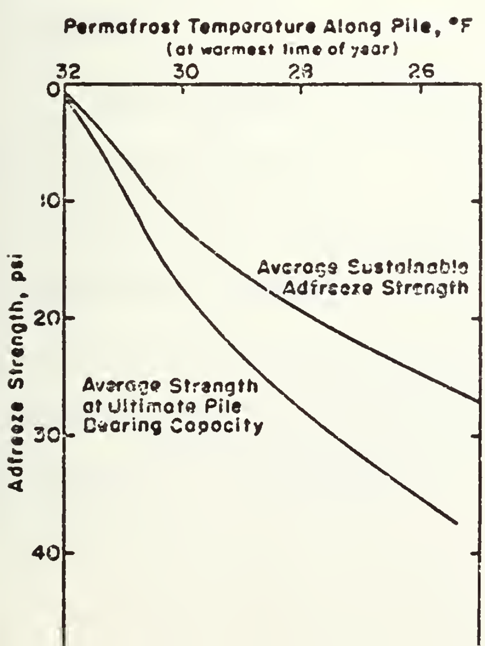

Figure 16

Correction factors for type of pile and slurry backfill (using steel in slurry of low-organic silt as 1.0)

Type of pile	Slurry soil: Silt	Slurry soil: Sand
Steel	1.0	1.5
Concrete	1.5	1.5
Wood, untreated or lightly creosoted	1.5	1.5
Wood, medium creosoted (no surface film)	1.0	1.5
Wood, coal tar-treated (heavily coated)	0.8	0.8

SUMMARY

Although most of the study of permafrost and its engineering applications is fairly recent, there is sufficient knowledge to allow for efficient design of foundations. In addition to the procedures used in temperate zones, foundations in permafrost require thorough understanding of conditions before, during, and after construction. Understanding the thermal regime and its balance between nature and the building are critical for a successful foundation design. Also, many other conditions not of as great a concern to the designer in temperate zones must be considered. These additional considerations include construction materials and procedures paricularly as impacted by logistics, costs, and effects of the environmental conditions. Additionally, maintenance requirements for the foundation over the life expectancy of the facility play a much more significant role in the life cycle cost of a structure in cold regions than in temperate regions.

This report presented basic concepts, considerations, and procedures for the design of foundations in permafrost. In many cases this design will probably only be of a preliminary nature, with refinement required according to the specific site conditions, the requirements and characteristics of the building, and particularly sound engineering judgement.

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