FOUNDATIONS OF STRUCTURES IN COLD REGIONS

Frederick J. Sanger

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COLD REGIONS
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FOUNDATIONS OF STRUCTURES
IN COLD REGIONS

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PREFACE

Much of the data used in the preparation of this monograph is from the work of engineers in the Military Construction Investigation program conducted for the U.S. Army Corps of Engineers, directed by Mr. K.A. Linell, Chief of the Experimental Engineering Division at the Cold Regions Research and Engineering Laboratory (CRREL). The interpretations and opinions, however, are those of the author, and not necessarily of CRREL, because research in foundations of structures in cold regions is continuing, and observed phenomena may be interpreted in various ways. CRREL reports are continually being published and the Laboratory has a great amount of additional material derived from about twenty years of testing, mainly in the field, available as Internal Reports, Technical Notes, etc.

The author is a Civil Engineer, serving as Special Assistant to the Chief of the Experimental Engineering Division of CRREL. He planned the Monograph Series, of which he is Editor.

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EDITOR'S FOREWORD

Cold Regions Science and Engineering consists of a series of monographs written by specialists to summarize existing knowledge and provide selected references on the cold regions, defined here as those areas of the earth where operational difficulties due to freezing temperatures may occur.

Sections of the work are being published as they become ready, not necessarily in numerical order but fitting into this plan, which may be amended as the work proceeds:

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F.J. SANGER
FOUNDATION OF STRUCTURES IN COLD REGIONS

by

Frederick J. Sanger

INTRODUCTION

Foundations in an area where the surface of the ground undergoes temperature cycles that pass through the freezing point of water require special care because of the freezing and thawing of water-substance. The phase change may profoundly affect the soil properties on which the stability of a structure depends: the most obvious effects are a gain in shear strength and a volume increase on freezing, and a loss of shear strength and subsidence on thawing; but important thermal changes also occur. The magnitude of the frost effects depends upon the type of soil, its water content and dry unit weight, and upon environmental conditions such as the weather, surface cover, snow, and the way that water exists in the ground. Heave and subsidence are almost never uniform and differential movements could make a structure unusable in a year or two. Design of a structure founded on compressible soil requires study of a transient soil deformation; design of a structure on permafrost requires study of a transient temperature disturbance, perhaps as well as soil deformation. In all cases, the designer must investigate the transient and ultimate steady-state conditions. The latent heat of fusion introduces a discontinuity of the thermal boundary conditions, complicating the mathematics of heat transfer and usually demanding a solution by numerical techniques.

The freeze-thaw cycling affects a certain depth of soil, called here the active zone,* which varies from zero to 20 ft; but the combination of soil and environment required for extreme depths is quite uncommon and 10 ft is a more realistic maximum depth of active zone to be expected. Further, the top surface of permanently thawed or permanently frozen ground at a particular site is often quite irregular, a situation that may cause problems in subdrainage in a permafrost area. Occasionally the frozen layer of soil does not reach the permafrost; the unfrozen layer between is known as talik. Sometimes the annual thaw does not reach the permafrost table for a season or two, leaving a temporary, frozen layer termed pereletok. The active zone (layer) and the intermediate layer together constitute the suprapermfrost, a word rarely used, however.

The thickness of the active zone is important. In a seasonal frost area, frost should be taken into account if the penetration is more than 6 in. Any disturbance of existing conditions may modify the depth of the active zone. The replacement of turf by concrete, or bituminous, paving causes an increase in the depth of the active zone; the replacement of a frost-susceptible soil by a non-frost-susceptible soil also tends to increase the depth, largely because of a lower water content and hence a reduced latent heat of fusion.

In an area of seasonal frost the design of the foundation is little affected by frost although special problems do arise. Structure loads are taken to depths below the active zone; appropriate backfill materials obviate possible trouble from heave, and lateral thrust from frost-susceptible soils. In permafrost, however, it is usually impossible to carry loads to permanently unfrozen

*Other terms are active layer, annual frost zone, annual thaw zone and zone of annual freezing and thawing.
ground so that generally the foundations go to permafrost, a good material for support if maintained in its frozen condition. Unfortunately, most buildings are heated and maintaining the frozen state of the soil becomes a major problem. Some soils are quite stable whether thawed or frozen and present no problem, but such ideal conditions are extremely rare. On a continental scale the cold regions gradually change from an area of seasonal frost to one of seasonal thaw (permafrost region) and the intermediate areas (with sporadic and then discontinuous permafrost) give added difficulties since at one point there may be no permafrost and 100 ft away permafrost may be thick enough to be used as a good support. For example, at an Alaskan site, the moving of a structure 75 ft from its planned position placed it on good permafrost instead of on a part-thawed, part-frozen area. This points up the necessity for a detailed site exploration before the final placement of a structure.

It should not be inferred that foundations on permafrost are always more difficult than those on unfrozen soils. Permafrost near Thule, North Greenland, was a better foundation material than coarse unfrozen gravel at Clear, Alaska, for the Ballistic Missile Early Warning System (BM EW S), because of its higher Young's modulus, required for a high spring constant to control vibrations transmitted through the structure from a heavy moving radar installation. However, the necessity for careful study of temperatures, and their control, made design, construction, and maintenance added problems at Thule.

One big difference between major foundations on permanently thawed ground and those on permanently frozen ground is that the temperature of the latter must be checked, usually with thermocouple strings.

Another point of difference is that a heated basement is desirable in seasonal frost areas but undesirable in permafrost. Any excavation in permafrost is avoided as much as possible because temperature control around a foundation is difficult and a pit excavation is often quite difficult and costly.

![Figure 1. Thaw progression beneath a heated building on frozen soils, Vorkuta, USSR.](image-url)
The flexibility of a structure becomes important in the choice of foundation type. Special care is mandatory in the design of foundations in permafrost. Differential settlements of a foot or more, caused by unplanned thawing of permafrost under buildings, have been observed (Fig. 1). The design of foundations in areas of deep frost penetration is adequately covered in reference 57.

**PRINCIPLES OF FOUNDATIONS DESIGN IN PERMAFROST**

As in conventional foundations engineering, the selection of a foundation type for construction in permafrost is made from structure and site data but the general principles can be outlined. Reference 58 should be consulted for general background material in arctic and subarctic construction.

A design usually consists of two parts: rheological and thermal. The first deals with the effects of stress upon frozen soils; these effects are strongly temperature dependent. The second deals with heat flow and temperature analysis and, usually, control. No important structure can be designed without at least a study of both parts.

**Conventional practice as used in non-permafrost areas**

If the permafrost consists of sound rock or of compact granular soils which are not affected by frost action and if ground-water conditions are favorable, it may be feasible and proper to ignore frost effects and proceed conventionally, knowing that any thawing beneath the structure can do no harm. These ideal conditions are somewhat rare but have occurred in Alaska, e.g. near Fairbanks.

**Modification of site conditions to an ideal state**

It is sometimes possible by simple (but usually expensive) means to modify the soil-and-water condition to the ideal state by excavation and backfill, or by thawing followed by compaction if necessary. These procedures are uncommon but have been used. Reference 53 describes an Alaskan project where 30,000 ft$^2$ of loose gravel was prethawed by steam-points to a depth of 30 ft and then compacted by shocks from explosions. It is unlikely that this thaw-blast technique would now be used unless no other method could be used at a particular site: logistics plays a large part in arctic engineering, and movement of constructional materials and heavy equipment might well decide ways and means. Thawing is very costly and would not normally be considered for a frozen island more than, say, 40 ft across and 20 ft thick; but special circumstances, such as those for the Alaskan power plant, affect procedures. A Russian criterion is to prethaw if the thickness of the layer is less than 60% of the computed thaw penetration in 10 years.

**Permitting thaw after construction**

If conditions are not ideal but the cost of maintaining the pre-existing ground temperatures is prohibitive, or for other reasons, this technique may be used.

The procedure of allowing a long-continued thaw-consolidation, however, is quite risky because estimates of settlement under thaw are so uncertain.

The main cause of difficulty with this technique is differential settlement. Uniform soil conditions, uniform surface and surrounding-soil temperatures, and uniform loading, all together and for a period of several years, are too much to expect. Special building construction, including slip joints, has been used in the USSR with variable, often unsatisfactory, results.

Reference 35 is a news report from the USSR on the use of electric heating by alternating current, followed by electro-osmosis by direct current, for the thaw-consolidation of a permafrozen fine-grained soil at Vorkuta under a three-story house. Energy consumption was about 19 kwhr/ya$^3$.
of soil for thawing and 15 kwhr/yard$^3$ for drainage. On the same site nearby the amounts were 37 and 10 kwhr/yard$^3$ respectively.

Research in thaw-consolidation is quite active in the USSR, where cities have been built on permafrost, building materials are scarce, and the cost of special foundations is high.

The thaw-consolidation procedure is generally avoided but sometimes there is no choice. Construction of dams on permafrost is especially difficult if the dams cannot be taken to sound rock or to non-frost-susceptible soils (which may need thawing and grouting to make a cutoff) because the reservoir and seepage cause temperature changes that are difficult to control. Special precautions are essential in design, construction, and maintenance to preserve stability and to control seepage.

**Maintaining permafrost in its frozen state**

Usually conditions are not ideal and pretreatment is uneconomical or undesirable (thawing of silt full of ice lenses is not good practice!).

The design principle is to keep the ground frozen, preferably at the same temperatures as those existing before construction. The ground has reached a stable thermal condition beneath the permafrost table and any artificial disturbance will be costly. Whether the volume fluctuations in the active zone can be allowed to affect the structure will depend upon the type of structure and its purpose. The best way to maintain the ground temperatures must be decided for each project. Usually a simple ventilation system using the cold air of winter to refreeze a buffer layer of non-frost-susceptible soil that is allowed to thaw during the summer and refreeze in winter suffices. Occasionally, however, a more positive refrigeration system permitting no thawing whatever is required.

**Foundations on rock**

A foundation on rock demands special care. There have been several unfortunate experiences in Alaska due to assumptions that the rock was ice free. Figure 2 illustrates a building founded on rock that was assumed to be sound.

*Figure 2. Effects of thaw beneath a heated building founded on frozen rock. USSR.*
FOUNDATIONS OF STRUCTURES IN COLD REGIONS

The top few feet of frozen rock under a soil mantle is usually a zone of extensive fracturing in which ice may comprise from 30 to 100% of the total volume. Shales are especially likely to be shattered in this zone, which is treated as frozen soil for foundations.

Concrete should not be poured in or on frozen rock without careful study. Precast sections (with or without poststressing) on a leveling pad of damp sand are recommended.

TYPES OF FOUNDATIONS

Foundations commonly used are: surface groundsills or surface footings, isolated buried footings, surface rafts, and wood or steel piles (rarely precast reinforced concrete and never cast-in-place piles). Precast reinforced concrete piles, round, square, I-shaped, etc., are preferred in the USSR. Small concrete pads can be cast in place if cast on thick gravel pads with favorable permafrost conditions. Large subsurface rafts and continuous footings are rarely used. Temperature is controlled by ventilation (air space or air ducts) or by mechanical refrigeration. Figures 3-19 illustrate typical foundations on permafrost. References 21 and 37 give good records of foundations in the Canadian Arctic.

General considerations

Most foundations in permafrost are designed to maintain the frozen state of the supporting soil, thus ensuring that the properties of that soil will remain constant throughout the life of the structure unless they are affected by external factors. It is not safe to assume that conditions will always remain unchanged, however, even if the foundation itself is well designed and constructed; this implies that careful observation and maintenance are essential for important structures on permafrost.

As examples, a differential settlement of several inches was caused by the diversion of groundwater flow from its accustomed path to the soil beneath a hangar apron, where the permafrost then degraded. At another place, an odd settlement at the corner of an important communications building was found to be caused by drops of warm water dripping occasionally from a heating conduit entering the building a few feet above ground level. In these examples the settlements were obvious. In another hangar, a dangerous situation was detected by thermocouples that indicated undesirable ground temperatures before any settlement was observable. Here the well-designed cooling ducts responsible for freezing back the pad under the hangar were being blocked by slow accumulation of ice and windborne silt that restricted the airflow, thus reducing the cooling effect. Each summer the 32°F isotherm moved a little lower in the pad, approaching the natural subgrade, a very frost-susceptible till with much segregated ice. Correction by cleaning out the ducts was costly but effective: for $100,000, a $3-million building was saved by the use of the few feet of thermocouple wire.

A further example is an important power station that began to show serious differential settlements. Investigation showed that, despite a well-designed freezeback system, the underlying permafrost was being degraded by hot waste water carelessly thrown outside the building. Other settlement and building cracking have been caused by the closing in of air spaces (because of cold floors or utility problems, which could better be solved in another way).

The kind of foundation to be adopted in given circumstances depends upon many factors, but mostly on: 1) the type of structure and its purpose; 2) the available materials and labor; 3) the site conditions -- ground temperatures, soil types, ice content and condition, etc.; and 4) the cost.
SITE CONDITIONS

Annual Frost Zone I (P-GM) 5 ft (G-M) Mean Annual Temp: 12°F
Permafrost: Silty, sandy GRAVEL Temp Range: -40°F to +60°F
(Temp: 12°F to 32°F) Annual Precipitation: 4 inches
Mean Thawing Index: 700 deg days
Mean Freezing Index: 8000 deg days

Figure 3. Foundation for men's barracks, Thule, Greenland.

SITE CONDITIONS

Annual Frost Zone III (P-GM) 5 ft (G-M) Mean Annual Temp: 12°F
Permafrost: Silty, sandy GRAVEL Temp Range: -40°F to +60°F
(Temp: 12°F to 32°F) Annual Precipitation: 4 inches
Mean Thawing Index: 700 deg days
Mean Freezing Index: 8000 deg days

Figure 4. Foundation for men's club, Thule, Greenland.
No permanent structure should be finally located until all local conditions have been thoroughly analyzed. This is true of all constructional engineering; far too often a structure is designed and located before the foundation conditions have been studied. This is almost criminal negligence in permafrost engineering.

**Structural considerations**

Important structural considerations are: live loads, flexibility, floor elevation with respect to surroundings, useful life, question of whether structure is heated or not or has periods of inactivation, heat production in the structure, and special functional requirements (e.g., pits for Nike systems, rigidity for Ballistic Missile Early Warning Systems and other warning and communications structures).

Important structures in cold regions are very likely to be steel-framed, with reinforced-concrete floors, special panel sidings and roofs, and light partitions; to be as fireproof as possible; and to require a minimum of on-site fabrication. Precast concrete buildings have been used in Greenland (the sections were shipped from Denmark). These structures are heavy and reasonably efficient, despite past problems with joints. The very heavy buildings now being erected in the USSR look unnecessarily rigid, but their thick concrete walls reduce the need for insulation, which is apparently in short supply or very costly.
Foundations of structures in cold regions

Figure 6. Pan-slab foundation for warehouse showing plenum chamber and ventilation duct, Sondrestrom, Greenland.
Figure 7. Pan-slab foundation for garage, Fairbanks, Alaska.

Figure 8. Pan-slab foundation, Thule, Greenland.
NOTES

1. THE DIKE RETAINED WATER, ADDED TO GRAVEL DURING COMPACTION, THE FILL WAS THEN ALLOWED TO FREEZE IN LAYERS TO ABOUT 1 FT BELOW THE LEVEL OF THE CONCRETE BASE.

2. THE REFRIGERATION COILS WERE LAID IN A 6-IN. CONCRETE LEVELING COURSE, UNDER THE 8-FT CONCRETE BASE. THE REFRIGERATION SYSTEM CONTROLLED TEMPERATURES DURING CONCRETE PLACEMENT TO PERMIT CURING WITHOUT THAWING OF THE FROZEN FILL. THE REFRIGERATION ALSO KEEPS THE GRAVEL FILL ALWAYS BELOW 26°F, THUS ENSURING ADEQUATE YOUNG'S MODULI UNDER THE BASE.

Figure 9. Refrigerated base slab for radar structure, Thule, Greenland.

Figure 10. Wood-pile foundation for small residence, Fairbanks, Alaska.

SITE CONDITIONS
Annual Frost Zone: SILT
(Approx thickness: 5 ft.)
Permafrost: SILT; Ice Lenses
(Temp: 28°F to 32°F)
Mean Thawing Index: 3300 deg. days
Mean Freezing Index: 5700 deg. days
Mean Annual Temp: 26°F
Temp. Range: -55°F to +90°F
Annual Precipitation: 11 inches
(Includes 48 inches of snow)
SITE CONDITIONS

Sporadic Permafrost
Annual Frost Zone: 8 ft ±
Permafrost: Silty CLAY with layers of fine SAND
Mean Thawing Index: 2910 deg. days
Mean Freezing Index: 5490 deg. days
Mean Annual Temp.: 25°F

Figure 11. Wood-pile foundation for heating building, Hay River, NWT, Canada.57

SITE CONDITIONS

Annual Frost Zone: SILT
(Thickness: 5 ft. ±)
Permafrost: SILT, Ice Lenses
(Temp. 28°F to 32°F)
Mean Thawing Index: 3300 deg. days
Mean Freezing Index: 5700 deg. days
Mean Annual Temp.: 26°F
Temp. Range: -55°F to +90°F
Annual Precipitation: 11 inches
(Includes 48 in. of snow)
Low Wind

Figure 12. Steel-pipe-pile foundation for utility building, Fairbanks, Alaska.
Figure 13. Steel-pile foundation for utility building, showing sunshade, Bethel, Alaska.

Live loads vary from a minimum building standard of 40 lb/ft² to several hundred lb/ft² in warehouses, and vehicle and aircraft wheel loads of tens of kips. Dynamic loads may require special consideration in military structures; wind loads are often severe in polar regions but in some places are remarkably low. Snow must be allowed for but snowfall is often quite low in permafrost areas and snow loads are not usually as severe as in many places in the temperate regions.

A flexible building with small live loads might be founded on groundsills (Fig. 3) or footings, on a gravel pad (Fig. 4,5) or on piles. A rigid building with very large live loads generally requires a pan-slab or ventilated slab-on-pad foundation (Fig. 6, 7, 8) because piles are usually lightly loaded and well-spaced and the framing with a simple airspace then becomes heavy and costly. A lightly loaded, flexible building with a desired useful life of 3 to 5 yr could be placed on a minimum foundation, without full protection from thawing under it; a temporary flexible building like a Jamesway, Shelterwell, or Quonset, but requires no foundation in most permafrost areas. For a light, flexible building the probable deflections can be easily corrected by wedges and shims. Such a building is placed on a high spot if possible (for best drainage conditions) and silts are best avoided ("slud" is a nuisance wherever it is).

Temporary buildings are customarily placed on a leveled patch of ground but more permanent buildings, such as barracks, dwellings, and light office buildings, are usually placed on non-frost-susceptible pads of gravel with groundsills or footings and an airspace to restrict heat loss into
the ground. Small (e.g., 4-in. pipe) piles may be more economical in special cases where gravel is scarce and difficult to get to the site.

Warehouses, garages, workshops, powerhouses, and hangars are rarely placed on anything but ventilated concrete slabs on gravel pads; this gives full protection against permafrost degradation.

Of special importance are the foundations of the White Alice (microwave transmission) stations and the large radar structures at Thule (Fig. 9). Both demanded exceptional control of rotational and vertical settlements. White Alice structures were comparatively lightly loaded and pile foundations have been very successful. Some of the radar foundations were very heavily loaded and required large rigid slabs of reinforced concrete on specially prepared and refrigerated bases, and dynamic loads demanded special considerations.

Bridges over streams are rare in permafrost areas but will become increasingly common as the arctic and subarctic regions are developed. Usually the foundation can be placed on bedrock or on non-frost-susceptible material. The main structural considerations are a minimum number of supports and protection of the piers and abutments from ice pressures. Each bridge is a special problem (as it is where frost is not a factor). Heave may be important and demand special attention, but river icings and ice jams are the major considerations. Solid ice may build up beneath the bridge deck and be long in thawing because of its mass and shading from the sun. Thaw in the river bed may be progressive, so that pier-piles may be in unfrozen soil. Piles must be strongly braced by tie-beams to minimize the effects of differential settlements (see ref 7).

Ground-floor elevation may influence the type of foundation. A hangar floor with deep airspace on a pad may not go well with apron and taxiway on a fill because of the gradients of the ramp.
Steps are commonly used with barracks and office buildings, and ramps with workshops and garages. Care is essential to prevent jamming of doors in a heated dwelling with a heaving ground outside. Doors should open inward in this case. Commonly, steps and a landing are provided for barracks, etc.; doors may then open outward, providing a desirable means for evacuation of the building in the event of fire. Ground floors of buildings where free water may fall must be watertight and positively drained. If possible, heat-producing “spots” should be distributed throughout a building and raised well above the ground floor.

Construction materials

Availability of construction materials is a consideration in all civil engineering but becomes a crucial matter in permafrost engineering. Every effort is made to use locally obtainable materials, but in the deep Arctic the choice is very limited and shipment of imported material is difficult and costly. In foundations, the properties of most construction materials at low temperatures are not important because the temperatures are not low enough to have significant effects on the materials.
Figure 16. Ducted foundation at Station Nord, N.E. Greenland (Lat. 84°N).\footnote{1}

Figure 17. Large-diameter cooling ducts, generating station, Elgen, USSR.\footnote{12}
Figure 18. Design of floor cooling system with large-diameter ducts for a maintenance building, northern Greenland.

Figure 19. Combined airspace and large cooling ducts, USSR.
ordinarily used. Availability of non-frost-susceptible soils and rock is a major consideration; standing timber points to a pile foundation as a possible economical solution. For a permanent structure in fine-grained permafrost in a tundra area devoid of gravel, sand, or rock, it might be advisable to import and place piles rather than to haul large quantities of gravel or crushed rock over hundreds of miles.

Wood has many characteristics that make it an attractive material for construction in cold regions: availability, low heat conductivity, low thermal expansion and contraction, adequate durability especially when treated (creosoted), flexibility, unimpaired properties at low temperatures, low weight, nailability, lack of heat of hydration, usability with poorly skilled labor, and low cost. In permanent construction its chief use is for piles (Fig. 10, 11), but a great deal of it has been used for groundsills, pads, posts, footings, cribs and mats, and as insulating layers under concrete footings. Combined with rock or boulders, timber makes a good bridge abutment. The most common useful tree in the subarctic is spruce, sometimes yielding piles to 30 ft in length (long enough for most purposes) within a short haul. Wood piles cannot be hammer-driven into permafrost, however, and in some jobs this may rule them out in favor of steel, especially if time is short. Timber groundsills and cribs are often of untreated wood but piles are usually creosoted. Untreated groundsills have shown remarkable durability in very dry arctic areas; but all constructional timber, especially piles above permafrost, should be pretreated. Twelve-inch widths and 2 to 12-in. thicknesses are used in construction.

Steel is used mainly in the form of piles; 4- to 8-in. piles (Fig. 12) and 8- to 14-in. BP sections (Fig. 13) are commonly used for military structures in Alaska. CRREL tests at Fairbanks proved that steel piles are undamaged by corrosion.

Concrete is a popular foundation material, in cold regions as well as elsewhere. Ambient temperatures are usually no problem but the heat of hydration may cause difficulties with the thawing of ice if the concrete is poured on or near permafrost as, for example, in deep footings (Fig. 14, 15). The curing temperature of concrete must be maintained if strength gain is to be at a reasonable rate; but this has a serious effect in thawing the permafrost, which should be maintained below its melting point and preferably at its normal temperature. Occasionally a quite elaborate system of refrigeration coils is required to control temperature gradients. Not only does thawed ground lose strength and subside but thawed ice presents a drainage problem for which well points and special ditches have had to be used on occasion. 35

High early-strength (not special aluminous) cement is recommended for low-temperature concreting. Reference 59 shows how HES concrete at 75F was successfully poured against rock containing ice at 25F. The success of this operation was undoubtedly due to a high rate of heat generation in the hydrating cement in a comparatively large mass of concrete.

The results of other projects in which concrete was placed against frozen soils show that concrete of a thickness of 15 in. or more will probably cure satisfactorily.

There is scope for the use of precast concrete, now much used for superstructures but rarely used for foundations. Post-tensioned blocks are also a possibility not yet exploited. Additives such as calcium chloride to lower the freezing point of water have been used in the USSR where "cold concrete" has been mixed and poured at temperatures well below 32F. CRREL has experimented with "cold concrete" but no recommendations regarding its use in construction can yet be made. Precast reinforced concrete piles have been used in the USSR in situations where U.S. practice would be to use steel; they have been tested in Alaska (by ACFEL while investigating skin friction) but they have not been used very much in construction. For special local conditions,

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*Concrete made with salt solutions that freeze at temperatures well below 32F and thus allow curing at low ambient temperatures.
precast (and possibly prestressed) reinforced concrete piles could make the best foundation, but they cannot compete with wood or steel in ordinary construction, in North America. In the sea around Alaska, which is infested with borers harmful to wood, RC piles could be useful.

Insulating materials are important in cold regions, where control of heat flow is essential. Closed-cell foamed plastics, especially foamed glass, are valuable insulators. It must be remembered that insulators lose their value if they absorb moisture. Cellular glass has been used extensively but organic foams like polystyrene and polyurethane have recently been introduced.

Vegetation and peat are useful ground-surface insulators, but are rarely permissible around structures because of fire risk.

Site conditions

The site information required for the design of foundations in cold regions is similar to that desirable in temperate regions where frost is negligible but must include important additions made necessary by thermal factors.

Climatic data are required for temperature studies and the estimate of thawing and freezing indexes or heat input for the ground surface. These data may not exist for the site and quite often have to be arrived at by averaging weather records from the nearest stations, using latitude, elevation, nearness to large bodies of water, and similarity of aspect with regard to slope, the sun, and exposure. As an example, the author required the pertinent weather data at a place of latitude 56N, longitude 96W, and elevation 600 ft, in rolling country near a lake. The three nearest stations having similar ground conditions and near lakes were:

<table>
<thead>
<tr>
<th>Station</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Elevation (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54</td>
<td>98</td>
<td>720</td>
</tr>
<tr>
<td>2</td>
<td>57</td>
<td>92</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>54</td>
<td>101</td>
<td>890</td>
</tr>
<tr>
<td>means</td>
<td>55</td>
<td>97</td>
<td>537</td>
</tr>
</tbody>
</table>

(Latitude and elevation are important.)

The tabulated weather data from the three stations were then averaged to give an estimate of the probable weather pattern at the site: mean annual temperature, air freezing index, air thawing index, average windspeed during freezing and thawing seasons, cloudiness, precipitation, and water temperatures in the lakes. Before computations proceeded, the whole pattern was checked against patterns of several well known locations to ensure that the data were natural and reasonable; then the difficult step of estimating ground indexes from air indexes could be made with some confidence. Observations based on a few years of record later showed an error of almost one degree in mean annual temperature but less than 2% in other estimates; and computations on permafrost degradation proved to be perfectly adequate.

It is regrettable that precise methods cannot yet be used to derive ground temperatures or to compute heat flow without a major research effort at the site, but as more and more experience is gained, and micrometeorological data are collected and analyzed, estimates will gradually become more reliable, and better techniques will be evolved.* This will take a long time in view of the large areas and few observers involved.

*See CRSE Monograph II-A1, Heat exchange at the ground surface, by R. F. Scott.
When the general site has been selected, often with the help of airphoto analysis, the precise location of a structure demands a detailed study, mainly by borings, in order to avoid large masses of ice in the form of wedges or of irregular blocks that may have a plan area bigger than that of the structure. Wedges 2 ft thick at the top and 20 ft deep and blocks 40 ft across and 8 ft thick are not uncommon in the high Arctic.*

Winds and precipitation are often very important considerations. At a site in northern Greenland, wind gusts up to 185 mph at 200 ft had to be allowed for in foundation design. At an Alaskan site, the fact that a windspeed as high as 75 mph had never been recorded was critical in the modification of a foundation design in which the piles were extended to serve as columns: the unsupported length could be increased from the designed value, which had been based on a 100-mph windspeed.

Precipitation affects drainage which, as in all foundation work, is very important in cold regions. Surface water percolating through the active layer to permafrost may be a cause of serious degradation. Considerable damage has been caused by the fallacious belief that all precipitation in the Arctic is in the form of snow, even if the 1.95 in. of rain that fell at Thule on 26 July 1957 (a normal summer's rainfall in one day), a week after a 100-mph gale, was exceptional. Surface hydrology and drainage are related to precipitation characteristics.

Weather observers should include icing data in their records. The formation of ice on structures and cables is becoming increasingly important with modern electronic devices, which cannot tolerate ice or snow on reflecting or on transmitting surfaces, often several acres in area. The foundation design for 100-ft towers with their guy wires, transmission lines, and large antenna surfaces depends to a great extent upon icing, its amount, time of occurrence, and accompanying winds. Creep of anchors, however, is a difficult problem still under study.

Even with the data provided by a first-order weather station, the gaps are serious enough to make estimates necessary in important parameters.

Accurate topography showing geomorphology and vegetation** is always essential. Aerial reconnaissance and photography are often of great value but should be checked by ground investigators because photointerpretation effective in one area may not apply in another.**

Location of construction materials and a study of transportation facilities accompany the surface investigations. Airphotos are valuable in spotting good borrow areas and in planning access routes.

The geological and soils investigations are essential for the efficient design and construction of a foundation. The studies include: date of drilling, surface features, soils log, soil classification, ice classification, unit weight of representative samples (dry unit weight, water content and amount of unfrozen water, later), thickness of active layer (date is important here), ground temperatures, free subsurface water, thickness of frozen ground, and special tests.

Core drilling, perhaps to 100 ft, gives the best results, but geophysical methods combined with the use of test holes or pits may be useful. Some data, such as unit weight and ice content, are obtained from portions of the cores straight from the barrel and the cores are photographed. Other data may be obtained from tests made in the laboratory. Dry unit weight and water content are essential but unfortunately are often not provided, so that the thermal properties are somewhat approximate even when estimated from those properties. Actual measurements of conductivity, heat capacity, and cementation temperature (where for practical purposes all the water in the soil is

*Refer to CRSE Monograph I-A2, Permafrost (perennially frozen ground), by S.R. Stearns.
Foundations of Structures in Cold Regions

Frozen to ice) are unusual but may be desirable for some jobs. A typical boring in permafrost in northern Manitoba showed eight major ice lenses from ½ in. to 2½ in. thick, a total of 11 ½ in. of ice in a depth of 25 ft, and the estimation of values for design was quite difficult.

Thaw-consolidation is not always studied but must be considered if the soil contains segregated ice, and thawing is possible.

Pile tests to obtain data on such characteristics as driving, rate of freezeback of permafrost after pile emplacement, and settlement rate under steady loads are desirable if pile foundations are being considered, although good estimates can be made for preliminary designs and cost estimating. The method of pile emplacement often requires tests because relatively few foundations exist in the vast areas of permafrost territory, so little experience is available.

Unusual military projects could demand special investigations, e.g., Young's modulus (in place or in the laboratory), Poisson's ratio, geological parameters, wave propagation (including damping factors), creep deformation under repeated impulses, and electrical properties.

Many of the mechanical properties can be found by means of seismic investigations* at the site: small dynamite charges can be used to generate dilatational waves, and a hammer or a drill string to make shear waves. But temperature is so important in all the properties of frozen soil that a program of laboratory work must usually accompany the field testing. Electrical resistivity surveys are used to some extent in the USSR but rarely elsewhere.

A deep active zone with highly frost-susceptible soils and high permafrost temperature indicates that a pile foundation will probably be best. A shallow active zone with soils of low frost susceptibility and a low permafrost temperature points to pads. A frozen uniform silt at high temperature is favorable for piles; but bouldery till might preclude piles because of placement difficulties. There are no simple criteria here: experience and judgment are vital.

Footings on Permafrost

Footings and slabs on pads

The simplest foundation consists of a leveled ground surface, preferably on elevated ground, on which a wooden floor rests. With this foundation there is no temperature control, so that only the lightest, most flexible and temporary buildings can be supported so simply if the active zone is of frost-susceptible soils. On organic terrain a 12-in. layer of sand or gravel (if available) can be laid directly on the vegetation for the support of the building. Differential subsidence is almost certain but is easily corrected by shimming where necessary. Stripping of the surface is an error, and construction equipment must keep to the prepared roads, made by dumping gravel, or other fill, on the natural cover.

For one-story light barracks and similar buildings up to 40 ft across, a satisfactory foundation can be made of a pad of non-frost-susceptible (NFS) material about as thick as the natural active zone but not less than 4 ft. Wood sills and pedestals provide an airspace of 1 to 3 ft between pad and floor (Fig. 3). Roughly speaking, to prevent thaw of the subgrade (full protection) the pad should be about 1½ times the thickness of the undisturbed active zone, but the reduced thickness usually suffices in this type of building. Small differential vertical movements can easily be corrected by shims. This principle has worked excellently at Thule Air Base. Quite often, uniformity of the soils of the active zone, the smoothing-out effect of the NFS pad, and some

*CRSE II-A2a, Seismic exploration in cold regions, by H. Roethlisberger (in preparation).
building stiffness result in uniform movements of a few inches; which are not detrimental. If the pad is made thick enough, an airspace could entirely prevent subgrade thawing; the "active zone" is then an NFS layer not affected by annual thawing and freezing.

In all designs, the floor must be adequately insulated and airtight. A cold floor encourages personnel to close up the airspace, giving a serious risk of permafrost degradation from loss of cooling. Snow must not be permitted to plug the airspace; this is one reason for the recommendation of 3 ft as a minimum height from pad to floor. (Airspaces only 1 ft high are not desirable unless forced draft is provided or the climate is exceptionally severe and dry.)

While wood is commonly used for sills, precast concrete may be used for more permanent, fireproof construction. Footings of wood or of concrete, either precast or poured (Fig. 4, 5), are variants on sills, and are more economical in a large building. A simple and effective foundation can be made by casting, say, 3-ft concrete footings flush with, or a foot below, the surface of a pad with posts formed of concrete poured into empty fuel drums, usually in plentiful supply around an arctic camp.

The pad should extend at least 10 ft around a building with a small slope for runoff and then slope down to subgrade at not more than 1:1. It is placed and compacted by truck and bulldozer. A bearing pressure of about 3 tons/ft² has been used on pads 4 ft or more thick. A Russian standard, however, is 1 ton/ft² on a fill specified thus: $\frac{1}{4}$ 50% of sand size 0.25 mm and $\frac{1}{4}$ 10%, size 0.10 mm; this describes a very "clean" NFS material.

If there is any choice in NFS material, the finer soils are preferable to coarse ones as they hold more water, giving a higher latent heat; they are therefore superior in reducing rates of thawing and freezing. A maximum size of 3 in. is sometimes specified.* If vegetation will grow in the area, it is good practice to protect the surface of the pad with it. Living tundra mat may be laid on the edges as a valuable insulator but, owing to fire risk, this may be prohibited in some military installations. If thaw will penetrate the pad, a coarse fill material should be separated from a fine subgrade soil by a 6-in. sand layer; this is a good practice and is followed in laying base courses under pavements of airfields in areas of seasonal frost.

High live loads (as for hangars, warehouses, workshops, garages, and powerplants) may demand a raft support. This is not economical with a simple airspace and the ventilated pan-duct type of design becomes necessary. (See Fig. 6, 7, 8 and 16 through 19.)

It is important to note that a gravel pad of itself will not prevent ultimate degradation of permafrost under a heated building, although it can reduce differential settlement. Thawing of permafrost under 12 ft of gravel has caused differential settlement in two years.

A steel tower where the superstructure gives no shade from the sun presents special problems if the original vegetation cannot be restored around piles or pedestals: heat transfer down steel columns and piles must also be considered.

Creep behavior of frozen soil. The strength of a frozen soil depends on: the type of soil, dry unit weight, degree of ice saturation, and temperature. Strength increases with coarseness of the soil, mainly because of the frictional component but also because of unfrozen water phenomena in fine-grained soils. Strength increases with degree of ice saturation (up to 100%; i.e., the pores are filled with ice but there is no segregated ice). Ice itself has a very low bearing capacity, especially at high temperatures, because of poor creep qualities. Temperature is a very important

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*For example: Canadian Building Code for the North (1968).
parameter. Strength increases rapidly as temperature falls. A drop in temperature reduces ductility and at very low temperatures frozen ground becomes hard, strong and brittle, resembling concrete in those qualities. Rate of loading is very important also but in ordinary footings the creep resistance governs design since it is much less than the strength at high rates of loading.

When a cylinder of frozen soil is loaded axially in compression at constant load and temperature the deformation-time curve is very characteristic and in general shows creep behavior as in Curve 2, Figure 20 (a). At point F, in time \( t_f \) the curve starts an upswing, indicating the onset of uncontrolled flow. If many tests are made at the same temperature but at different stresses and the stress is plotted on a base of \( t_f \) (time to flow), the result is a curve, Figure 20 (b), for each temperature \( T \). This curve approaches an asymptote representing the stress at which flow would never occur at that temperature. Tension, shear tests, adfreeze bond tests on piles, and anchor tests all give the same type of curve. Vialov\(^2\) has made an extensive study of the phenomena to arrive at an empirical equation:

\[
\frac{\beta}{\sigma} = \log \frac{t_f + t_0}{B}
\]

and since \( t_0 \) is very small indeed, \( \beta/\sigma = \log(t_f/B) \). The form, \( \beta/\sigma = \log(\alpha t_f) \) is convenient in practice where \( \sigma \) is the stress causing flow in time \( t_f \). \( B, \beta \) and \( \alpha \) are constants for the soil, its conditions, and the temperature. A similar expression applies for shear parameters although the approximate angle of shearing resistance \( \phi \) is much less sensitive to time than is the cohesion, \( c \). Figure 20 (c) shows typical Mohr envelopes for various times at constant temperature.

The constants \( \beta \) and \( \alpha \) in the Vialov equation can be found by laboratory tests on four or more sets each of, say, four specimens, for each temperature. The logarithm of time and reciprocal of stress are plotted and the best straight line drawn through the points (Fig. 20 (d)). \( \beta \) and \( \alpha \) are then easily computed from slope, and \( 1/\sigma \) at time 1 hr as shown below.

**Example** (data by F.H. Sayles, CRREL; see Fig. 21, uniform medium sand at 31°F):

<table>
<thead>
<tr>
<th>Stress (psi)</th>
<th>Time to flow ( t_f ) (hr)</th>
<th>( \frac{1}{\sigma} ) (in.(^2)/kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>745</td>
<td>0.003</td>
<td>Not used</td>
</tr>
<tr>
<td>400</td>
<td>0.18</td>
<td>2.50</td>
</tr>
<tr>
<td>1.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>260</td>
<td>1.0</td>
<td>3.85</td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>53</td>
<td>6.67</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 21 shows \( 1/\sigma \) on a base of \( \log t_f \). The fitted straight line should be steep for conservative design and make use of as many points as possible; lines for different temperatures must be parallel. (These principles were used in drawing Fig. 21.) The tests are difficult and some points, particularly that for \( t_f \) equal to a few seconds, must be ignored because of experimental errors, etc. \( 1/\beta = \) slope of line = change in \( 1/\sigma \) per cycle = \( 2.25 \times 10^{-4} \text{in.}^2/\text{lb} \) and \( \beta = 445 \text{ psi} \).
Figure 20. Creep curves for frozen soils.

Figure 21. Time and reciprocal of stress plot for creep test.
The ordinate at $t_f = 1$ hr gives $\lg \sigma = \beta(1/\sigma) = 445 \times 2.71 \times 10^{-3} = 1.205$, and $\alpha = 16.0$. Hence, $\sigma = 445/(\lg 16 t_f)$ psi ($t_f$ in hr).

At $t_f = 50$ yr, $\sigma = 65$ psi or 4.7 tons/ft$^2$. Using the nomogram (Fig. 22), $\sigma_1 = 369$, $\sigma_{10} = 202$, and $\sigma = 65$ psi. (Multiply by 10 to use the nomogram conveniently.)

The rapid, conventional test gave 745 psi for failure - more than 11 times the 50-yr value.

**Footings directly on permafrost**

At present the procedure just discussed is not generally followed in design but it shows a rational approach that could be used.* Design could be based on an equivalent cohesion ($c$) of

*The author has used the method for finding design parameters for designing large artificially frozen soil-retaining structures with excellent results.
half the unconfined compressive strength, as computed (4.7 tons/ft²), and the assumption of purely cohesive soil. Since nothing is given about φ, the angle of shearing resistance, by the simple compression test this is probably the better method.

Alternatively, for this soil it would be safe to assume φ = 20° (it would be more than 30° in a natural deposit when unfrozen) and c then becomes 1.6 tons/ft²; design could then be based conservatively on c = 1.6 tons/ft², φ = 20°.

A variable ground temperature is best allowed for by using the lowest average parameters based on temperatures along the critical slip surface under the footing.

The factor of safety could be 2 or 3, depending on type of load. The determination of c and φ in a material that creeps is a major problem so far solved for only a few soils. The triaxial test will probably emerge as the best way of finding these parameters, although CRREL is also experimenting with torsion apparatus. From the very few envelopes published (time up to 24 hr), φ seems to be stable after a few hours but c continues to drop with time. Both parameters seem to follow the Vialov-type equation as do all soil strength parameters in tension, compression and shear, at least for 9 yr.*

Tsytovich* has developed a simple solution to the problem of measuring c and φ in his ball penetration test, which is essentially the Brinell hardness test on a large scale. By measuring ball penetration and time under constant load and using a simple formula, he arrives at an equivalent cohesion as a function of time, and then applies the usual design methods for a purely cohesive soil. Few tests have been made with the ball penetrometer outside the USSR so experience is very limited. Experiments by CRREL and others have been inconclusive, possibly because of the soil types used.

Most designs have been based on assumed bearing pressures, usually with technical success but of unknown economy. Failures have nearly always been attributable to lack of temperature control. By permitting the ground temperature to rise the soil loses strength even if it does not thaw. Hence in an important project care is taken to ensure that a safe temperature will never be exceeded; this may entail artificial refrigeration (see App. B).

**Empirical bearing pressures.** Examples of recent design bearing pressures are: 6 tons/ft² on a 16-ft by 24-ft footing on a dense bouldery till (GP) where the mean annual temperature was 12°F and the estimated highest soil temperature at the base of the footing was 25°F; 1½ tons/ft² on a 2-ft-square footing on uniform silty fine sand (SM), where the mean annual temperature was 27°F and the soil temperature at the footing depth was about 31°F. These design pressures were based solely upon engineering judgment and experience in the areas concerned; they are about one-half the Code values recommended in the USSR (Fig. 23) and have a factor of safety of about 6 based on test values. More research on footings on pads and on permafrost is desirable. Note that the proposed method is based on soil stress, not settlement. Experience shows that settlement is negligibly small if no thawing occurs. If differential settlement is possible, an adjustable bolted splice may be incorporated in the column near its base plate; shimming will take account of small movements until a bolt spacing is reached. This is also a useful remedial technique for unplanned settlement.

Figure 23** gives Russian Code (S.N.I.P. 91-60)* recommendations for footings on permafrost as of 1960. The values seem reasonable and are quite conservative compared with data listed by Tsytovich** and by Vialov.** The translation of the 1966 revision of the Code is being done by the National Research Council of Canada.

If creep-test data are not available, conventional compression test results can be used assuming that the long-term strength is about one-fifth of the value from the rapid test. (Many soils are one-third as strong.) Shear tests show somewhat larger ratios sometimes, especially for sands at high temperatures.

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*S.S. Vialov, in conversation with the author.
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Figure 23. Design bearing pressure and temperature, USSR standards, 1960.

**Depth of footing.** Apart from special local conditions the depth of footing is determined by the possibility of permafrost degradation caused by the disturbance of the original stable thermal regime. Normally the excavated soil will be replaced by NFS backfill (to minimize heave forces) and the valuable surface cover may be lost. Both factors lead to a deeper thaw penetration than with the soil in the undisturbed state; therefore, the footing should be set a few feet into the permafrost to make sure that there will be no detrimental effects from the inevitable degradation. There is a Russian technique for computing the most economical footing depth but it depends upon so much theory and assumption as to be of doubtful value in design.

**Pressure distribution.** As with unfrozen soils, the pressure distribution under a footing in permafrost depends upon footing rigidity and soil properties. It also varies with time; stresses in highly stressed regions are relaxed and the load is transferred to other areas of lower stress, thus tending to equalize the soil stresses. When temperature and pressure vary in frozen fine-grained soils, the viscosity of ice and the flow of unfrozen water introduce further uncertainties. The only reasonable design procedure is to assume a linear pressure distribution, as in unfrozen soils, unless the designer can ensure the conditions that justify a more theoretical treatment. Pressure is apparently greater near the edges than in the middle; this is a characteristic of stress distribution under rigid footings on cohesive soils.
Heave. In a properly designed and constructed footing on permafrost there will be no heave from vertical forces under the footing, but tangential adfreeze to the post or pedestal passing through the active zone has to be guarded against. Often this is obviated by using a coarse, dry NFS backfill. This is dealt with later under the general treatment of heave and foundations (p. 49).

Site tests for determining allowable pressure. At the present time (1968) site bearing tests are not made on permafrost. Although likely to be time-consuming and costly, such tests would give more confidence than the Russian ball hardness test does. If bearing tests in place are contemplated, reference 29 should be studied, but with caution because of the loading conditions: the load was applied on a reinforced concrete pad about 3 ft square by a jack getting its reaction from the soil in an undercut pit covered by a shed having an insulating layer on the ground around it. Such tests are very rare because of the large loads and small deformations associated with permafrost. (All field testing in a permafrost region is extremely difficult.) In the unusual circumstance of building upon permafrost that will thaw and consolidate under the structure, a simple field test is much easier (see ref. 19, p. 257). Thaw-consolidation is still more easily studied in a laboratory with a consolidometer; e.g., this is the method used by the Alaska District, U.S. Army Corps of Engineers.

In any test on frozen ground, its visco-elastic behavior must be remembered. A quick test with an increasing load is valueless and dangerous since the creep strength is much less than the strength found in rapid tests, and no correlation is possible.

Concrete footings on permafrost. The pouring of concrete on permafrost introduces two incompatibilities: the concrete poured must be warm enough to cure in a reasonable time without freezing and cold enough (as a rule) not to thaw the frozen soil. Concrete more than about 15 in. thick will not normally get too cold for curing because of its heat of hydration. The means taken to satisfy these requirements depends upon the size and importance of the footing, the kind of permafrost and its temperature. Timing is also an important consideration. Concrete is not laid directly upon the permafrost but upon an intermediate layer - an insulator, or a pad of NFS soil, or both. Early Russian footings were of wood and later of masonry placed on wood. A modern, attractive method is to use precast footings of reinforced concrete, thus sidestepping the problem of heat of hydration. The damp sand bed is still advisable to ensure good contact.

Ordinary cast-in-place concrete footings. A typical procedure is to excavate the site to the required depth in the autumn, just as the active zone begins to freeze. The excavation may cover the whole site or be in trenches (pits) about 4 ft wider than the footings. Its depth will depend upon the kind and depth of backfill, surface after construction, level of the building, and level of the permafrost table. Estimated and unplanned thaw during excavation must be remembered. The least possible area of permafrost should be exposed. Drainage sumps will probably be required and sometimes the site will have to be predrained.

A layer of damp sand or sandy gravel should be placed as quickly as may be necessary, tamped, and allowed to freeze in place. This may be from 1 to (exceptionally) 5 ft thick. If possible, a 4-in. layer of cellular glass, 12-in. layer of wood, or other satisfactory insulator should be placed before the concrete is poured. A layer of lean concrete has also been used to provide a bed for the concrete footing.11 The walls of the excavation may need lagging and a tent over the site may be worthwhile to keep out heat and precipitation.

The concrete materials must be warmed if necessary (by a simple cold-weather concreting technique) so that they are at not less than 40°F when placed. The concrete must be kept above 40°F for 3 days unless high-early-strength cement (type III) is used to shorten the curing period. Pedestal or post can then be added and the excavation covered and kept free from snow, but not backfilled; this allows the cold of winter to freeze back the disturbed soil area. Preferably the backfill is
placed and compacted the following spring but time does not always permit such an ideal schedule and backfill may have to be placed immediately after concreting. Work on the superstructure is best done in the summer but is often carried on during the winter. The Portland Cement Association publishes a useful pamphlet on concreting in cold weather and references 40 and 32 give good ideas on cold-weather construction. Reference 41 shows how, even with the greatest care, thaw became a problem, solved, with considerable difficulty, by the use of well-points.

**Special footings.** The 8-ft-thick, 55-ft-diam (octagonal) reinforced concrete footings for the tower of a radar structure on permafrost (Fig. 9) can be described as unusual because of the extreme precautions that were necessary. The scheme adopted was to build the footings on a specially prepared gravel pad, which was saturated, compacted, and frozen in layers to a height of 15 in. below the elevation of the base of the footing. A dry, 9-in. layer of heavily compacted, crushed traprock was then placed and allowed to freeze. In May, just before the spring thaw, a 6-in. layer of concrete was poured and kept at about 40F for 3 days during which time the HES cement ensured a reasonable strength. Embedded in the concrete was a pipe meshwork of a refrigeration system\(^\text{18}\) that controlled the temperature gradient when the main footing was poured and cured at normal temperatures under a heated tent. This procedure, planned by Haley and Aldrich, Consulting Engineers, was designed to prevent any thawing of the gravel pad. The refrigeration system has been kept operational and ensures a satisfactory temperature of the pad and the permafrost beneath it.

Artificial refrigeration has been used on several jobs, usually when nothing else could serve to maintain permafrost temperatures, but it is generally avoided. Deep excavations into permafrost, made only when essential for the functioning of the structures (e.g., a missile-launching facility) are quite difficult to ventilate and it is usually better to use heavy insulation and artificial refrigeration in such pits. Refrigeration has also been invaluable in saving vital buildings undergoing dangerous differential settlement caused by unforeseen heat sources or by inadequate design during the early years of permafrost engineering. Underpinning on permafrost is uncommon and very costly but is sometimes the best remedy for an unsatisfactory foundation. A very few footings have been placed on 12-in. gravel layers having refrigeration coils in them for permanent temperature control. This is quite exceptional and no information is available regarding the performance of the system or its cost of operation, but construction problems were known to be formidable.

**PILED FOUNDATIONS IN PERMAFROST**

Piled foundations are often used for major structures. The load is taken to a depth where small volume change or loss in shear strength occurs and an airspace ensures that the permafrost stays at its original temperature and supporting value. But very high floor and structure loads, bouldery soil, extreme transportation difficulties, or other factors may make piles impracticable. They are especially valuable in doubtful places where there is a great deal of segregated ice in silty soils and the permafrost temperature is just below 32F.

Settlement resulting from faulty emplacement, and heave caused by insufficient anchorage to resist uplift from adfreeze forces in the active zone, have occurred too often in the past, but a modern foundation should not suffer from these defects. Lack of appreciation of the viscous behavior of frozen soils in creeping at stresses well below the maximum adfreeze stress found by conventional rapid testing procedures led to serious underdesign only a few years ago. Because frozen ground seemed so hard, and the scanty published data did not warn against the creep effect, conventional safety (or load) factors were inadequate. The limiting bond stress is usually the limiting shear stress of the frozen soil; slip along the pile surface is probably rare in soils near ice saturation if the surface has not been painted or otherwise specially treated.
Time and method of emplacement are very important in piled foundations. Research in pile foundations is very active in CRREL but creep tests in permafrost areas with so many variables take many years and too few structures exist to give firm criteria for the sustained load condition in every kind of soil.\(^{29}\)

**Pile types**

Because wood piles have many desirable qualities they are very popular. In the USA, however, the trend in major structures is to steel piles, especially if driving is practicable. Few precast concrete piles have been used and cast-in-place concrete piles, whether in casings or not, cannot be used because of the heat input to the ground and chilling of the concrete. In Russia, precast reinforced concrete piles (up to 20-in. I shape) are preferred to steel piles, but precast concrete piles are more popular in Great Britain and Europe than in the USA for all construction.

Wood piles are usually from 6 to 10 in. in diameter at the top, 12 to 14 in. at the butt, and up to 35 ft long. Creosoted Douglas fir, southern pine or local timber, generally spruce, are used. Pressure-creosoted piles are usual but an excess of creosote must be avoided because of the possible reduction of bond stress by bleeding. A 12-lb empty-cell treatment is commonly specified by the Alaska District, Corps of Engineers, for Douglas fir piling in normal foundations (Alaskan coastal waters demand a heavy full-cell treatment of wood piles). If native timber is used, the preservative must be brushed on as well as possible, but only for the length that comes above the permafrost. Wood piles are commonly placed butt down to improve anchorage against heave forces arising in the freezing active zone.

Most piles are comparatively lightly loaded because bond stress (= "skin friction" in unfrozen soils) is small and limits the load. This gives wood an advantage over steel. Emplacement is usually easier with steel piles, hundreds of which have been satisfactorily driven by conventional piledrivers (diesel hammers are best). As driving techniques in low-temperature permafrost are improved, steel may be used more and more. At present, pipes and BP sections are used, but hollow square shapes might be used with advantage.

Getting heavy-construction equipment on the site is often an important factor. For smaller buildings 4-in. pipe piles have been successful, but wood piles may be competitive on cost and can be installed with comparatively light equipment in augered holes with an annular backfill of damp soil. Bond stress seems to vary inversely with pile diameter, giving small piles an advantage over large ones perhaps, but research is needed in this regard. Piles passing through thawing silts to point bearing on rock, etc. may fail from down-drag. Such a possibility demands special study.

**Design**

The major considerations are:

1. **Type and size of pile.** These are determined by local conditions: available materials, cost, labor, structure, etc.

2. **Length of embedment.** The pile must be embedded in, or anchored into, permafrost to transmit structure loads with a tolerable rate of settlement and to resist all heave forces coming from the active zone in winter. The length for anchorage depends mainly upon the soil properties but it should be equal to at least twice the active-zone thickness and preferably more. Untreated wood has a better bond than treated wood but since all wood piles should have preservative treatment, this is unimportant. When native timber is used and treated on the site, the portion embedded in permafrost is left peeled but untreated for economy and better bonding. The embedded surface of any pile should never be painted.
Figure 24 gives curves of average ultimate bond stress in creep; they are based on CRREL experience, checked against data from reference 52, but must be labeled "tentative." Values from these curves should be used with caution and results checked by field tests for final design. The distribution of stress is almost uniform along a pile at constant temperature but temperature may vary throughout its length. Bond also depends on relative strains between pile and soil. These factors result in grave uncertainties in practice, but the curves in Figure 24 give a starting point in field tests.

The critical time for piles is mid-winter when the permafrost is almost at its highest temperature and a frost-susceptible active zone is exerting its maximum uplift. The adfreeze bond in heave seems to be less than in downward loading although ref. 48 states that they are about equal. The problem of heave force is dealt with in section on Frost Heave Forces (p. 49).

Note that the curves of Figure 24 are for ice and for saturated soils. If the soils are less than about 90% saturated, these bond values are high. As the curves show, ice-saturated sands and fine sands have a better adfreeze bond than silts. The permafrost in which piles are normally used is saturated (and often supersaturated with ice lenses) so that the curves are usually applicable. Sometimes the ice lensing is severe and layers of pure ice several feet thick may be encountered. In these circumstances the bond stresses must be reduced (hence the curve for ice), but the factor of safety proposed covers most silts with ice lenses, giving about 1½ times the water content at 100% saturation. This is less important if slurried piles are used, but some reduction of load is advisable to minimize differential settlement.

Unsaturated coarse soils may give lower values in bond but point bearing will probably be operative instead.
FOUNDATIONS OF STRUCTURES IN COLD REGIONS

Figure 25. Predicted length of embedment of a pile in permafrost.

Very rarely, piles have been placed on rock in an open excavation and buried with coarse gravel and broken rock to develop full point bearing. In these cases, zero skin friction was assumed.

It is basically wrong to add full point-bearing resistance and full adfreeze strength in constructing piles in permafrost (just as it often is in piling in unfrozen ground). Sometimes it may be safe to use point-bearing resistance plus part of the bond resistance. Field tests should separate the two effects if point bearing seems worth considering. To develop full adfreeze bond, from 1/4 to 1/2 in. of relative displacement of pile and soil is required; point bearing on a coarse soil could easily permit only a small fraction of that movement and, hence, a reduced adfreeze strength. In some soils, notably supersaturated silts and clays, the displacement required to develop point bearing is more than can be taken in bond and the pile slips, breaking the bond before the point can pick up its load. The ideal state of compatible displacements for both effects to operate fully together is obviously very unlikely. Field creep tests on piles hint at the slow development of point resistance, where none was expected, as settlement proceeded.

Reference 4 (USSR Standards) states that point resistance is permissible with reinforced concrete piles but not with wood piles; it does not mention steel piles. (There is no record of steel piles having been used in the Soviet Union.)

Prediction of embedded length. In Figure 25, let curve T give the known highest permafrost temperatures in an ML soil with ice lenses. From Figure 24, curve S of maximum bond stress with
depth can be drawn. Integration by any simple method leads to curve C of maximum pile capacity, using the known pile perimeter* (8½ in. OD steel pipe pile = 2.25 ft perimeter):

Sustained load: 15 tons
Short duration peak load: 20 tons.

A factor of safety of 2 on sustained load and 3 on peak load gives a maximum of 45 tons (sustained).

A vertical through 45 tons intersects the capacity curve at an embedded length of 18½ ft; this is prima facie safe. Test piles would check this. Note that settlement is not considered here; however, it is most important and normally controls the design. Test piles are essential for estimating settlement (p. 47).

The use of an average temperature may lead to error. The average here is 30.3° F. From Figure 24, maximum bond stress = 0.87 ton/ft²; maximum capacity per foot = 2.26 × 0.87 = 1.97 tons; and embedment = 23 ft. If (max + min)/2 is used for average temperature, the mean = 30.8° F; T = 0.65 ton/ft²; capacity is 1.5 tons/ft²; and embedment = 30 ft.

These results may be astonishing. Only a few years ago engineers were using fantastically high bond stresses based on rapid pullout tests and load tests, usually with the load rising rapidly in steps; but experience has proved the wisdom of the change to these remarkably low values, often little more than the unfrozen skin friction. Conservative design stresses are very desirable because as of this time (1968) field experience is so limited.

Vibratory loads have not yet been adequately studied but it is reasonable to use higher factors of safety if the vibration is long and continuous.

At Barter Island, Alaska, a powerplant founded on 8-in. pipe piles in 1962 soon showed unexpected settlements, which were partly attributed to structural vibrations from the diesel sets. The plant had a frequency of about 575 Hz and the piles a natural frequency of about 475 Hz; these frequencies were too near resonance. The main cause of trouble, however, was the painting of the embedded pile surface, which drastically reduced the adfreeze bond. Underpinning by footings fastened to the piles at depth was necessary for remedial treatment.

Embedment to control heave must be considered but, in passing, we note that the embedment in the example is more than 10 ft, which experience has shown to be safe for piles in ML soil with temperatures very like those used in the example.

The actual bond stress varies with length, since it depends on relative strain between pile and ground, and pile stress (and hence strain) decreases with depth as the load is picked up along the pile length. However, with the ground temperature falling with depth the soil can deform in a favorable way to take up the load. Bond stress also varies inversely as pile diameter, but for structural sizes the effect is small.

Pile spacing. Permissible pile spacing depends on structural considerations, available piles, local soil conditions, available equipment and method of emplacement. Most of these are considerations with piles foundations anywhere in the world, but something additional can be said about the method of emplacement.

Good practice demands a minimum of disturbance of the existing thermal regime, but some heat input during pile emplacement cannot be avoided. The worst upset comes from steaming a hole, a procedure not generally permitted by the Corps of Engineers.

*The perimeter of an H section to be used for design is still uncertain, but about 75% of the total perimeter in contact with soil is about right according to CRREL test results.
Emplacement in an augered hole is a good method that introduces very little or no heat to be absorbed by the surrounding permafrost. Piles have been frozen back artificially within 2 days but usually the natural permafrost temperature is adequate for freezeback within 2 weeks. Any friction piling should have a minimum spacing of about 5 ft and, in permafrost, from 5 to 10 ft is recommended. A minimum can be estimated by simple heat-transfer principles. Suppose that a wood pile is installed in a pre-augered hole, then backfilled in the annulus with a cooled, damp mixture of soil and water that freezes and cements in the pile. The heat from the backfill warms up the surrounding soil; if the melting point is not reached, however, the soil will not be harmed since the heat can be dissipated in winter by conduction through the colder active zone and then by convection to the air.

Example:

**Given data**

<p>| | | |</p>
<table>
<thead>
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<th></th>
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<tr>
<td>Backfill placed at</td>
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<tr>
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<td></td>
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<tr>
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<td></td>
</tr>
<tr>
<td>Hole</td>
<td>18 in. diam</td>
<td></td>
</tr>
</tbody>
</table>

**Heat properties of soils:** (App. A)

Volumetric specific heat of backfill = 45 Btu/ft³°F
Volumetric latent heat of backfill = 4600 Btu/ft³°F
Volumetric specific heat of permafrost = 28 Btu/ft³°F
Area of annulus = 1 ft²

S = pile spacing, each pile having a cylinder of diam S around it for absorbing heat.
Heat lost by backfill = heat gained by permafrost.
Allow permafrost temperature to reach 31°F; then backfill cools by 9°F and permafrost warms by 4°F.

**Per foot of height:**

\[1 \times (4600 + 9 \times 45) = \left(\pi S² \cdot 4 \times 28\right)/4\]
and \(S = 7.5\) ft.

The computation is crude, and for time estimates more precise work is necessary. A spacing of 7 ft 6 in. would be a good value, however, to use as a minimum, and 8 to 10 ft would be better if acceptable structurally from desirable pile loads.

If possible, piles are placed at low permafrost temperatures to ensure rapid freezeback; they would probably be good for at least a ton per foot of embedment at warmest permafrost condition, sustained load condition, or, say, 20 lb/ft² of floor load per foot of embedment at a spacing of 10 ft. A 10-ft embedment (a recommended minimum) would allow a sustained load of 200 lb/ft², leading to pile loads of 10 tons, which are reasonable in wood, but structural requirements also affect the pile layout. These numbers merely give some idea of sizes. It should be noted in passing that the heat introduced by pile emplacement can have a radius of influence of 5 ft or more (Fig. 25).
Figure 26. Computed isochrones of temperature and radius for pile freezeback.

Pile freezing-in. It is important to know how long the refreezing of disturbed soil around a freshly placed pile will take. This may be more than a year if the pile is put in a steamed hole, and, even if it is placed in an augered hole, the time could be weeks, especially with permafrost near 32F. For this reason it is advisable to place piles in the spring when the permafrost temperature is at its lowest. Sometimes, especially after steaming, the piles are placed a year or more ahead of construction to give them adequate time for freezeback.

Driven piles go in most easily when the permafrost is at its highest temperature, and time of freezeback is not a problem here so the late autumn is a good time to place driven piles. The superstructure is added in the spring and summer, but materials are brought in during the winter (transport is often difficult over a thawed ground surface). If piles are placed during cold weather (air colder than the permafrost) and a hole is open a few hours before pile emplacement, some cold air flows down the hole and precools the soil a little. If they are placed when the air temperature is above ground temperatures, each hole must be backfilled immediately. If time is short and piles are placed in pre-bored holes, artificial refrigeration can be used to freeze back the slurry within a day or two or even a few hours. This method saves so much time that it has been used on some jobs in Alaska.14 56
On a few jobs, refrigeration coils have been installed on the piles to permit permafrost cooling to increase pile capacity, should this ever become necessary in the life of the structure. If pipes are attached to a driven steel bearing pile for this purpose, they should be symmetrically arranged and protected by light angles welded to the pile; otherwise driving will be unnecessarily difficult and the pile will almost certainly bend and twist in the permafrost. In a pile-driving test at Bethel, Alaska, in June 1960, a plain 8BP36 pile went to 34 ft at 22 blows/ft, whereas 18 ft away, where soil conditions were the same, a similar pile with refrigeration pipes took from 48 to 150 blows/ft when the pipes reached frozen soil. Freezing coils are either straight pipes, running parallel to the pile axis, or, for a non-driven pile, spiral tubing around the pile. The trend in Corps of Engineers' practice for placed wood piles is the spiral of copper tubing, used most recently where refrigeration was used.

In recent years liquid propane has been used as the refrigerant and coolant, whereas earlier systems used CaCl₂ brines as coolant, and ammonia or Freon as refrigerant. The refrigeration plant has changed radically from a large unit using commercially available make-do components to a modern, specially designed, very neat and compact propane plant of 1-ton capacity, without a secondary coolant and heat exchanger.

Cold air has been circulated by fan in a hollow pile for tests on freezeback but the indirect use of cold air via a refrigeration plant is preferable, in the author's opinion, unless many weeks are available for freezeback, because heat transfer is so inefficient with air.

Natural freezeback: The problem of the time required for natural freezeback has been solved in various ways. No solution is very precise because field observations show that soils do not behave as assumed in classical heat transfer analyses dealing with ideal media. The initial backfill temperature will be a little above 32°F, so the sensible heat will be very small and can be safely ignored. Vertical heat flow also can be safely ignored owing to the small vertical temperature gradients.

A simple but rather tedious solution is by numerical analysis of transient heat flow. The slurry is assumed to be at 32°F until all its latent heat has been dissipated, and then its temperature falls; freezeback is assumed complete when the slurry temperature has dropped 90% of the way from 32°F to ambient permafrost temperature. Modern computers quickly deal with finite-difference computations (or finite element procedures) and their use could be justified in important projects.

Figure 26 gives the result of a hand-computation by the author, using 45 time steps, showing temperature-radius curves at stated times of 0, 12, 24, 36, 48, 60 and 72 hr from the start. The temperature curve quickly flattens out, showing that the approximate method of estimating pile spacing has some merit. The added heat can escape only vertically when the temperature drops in winter but, in the meantime, over the foundation area, the permafrost temperature is roughly uniform at a particular depth only a week or so after pile installation.

Note the remarkable radius of influence of over 5 ft. It supports the view that piles should be widely spaced, with 10 ft as a good spacing. The thickness of slurry is somewhat excessive in this example. Modern practice has reduced both thickness and water content, thereby reducing input and hence freezeback time.

Figure 27 records many tests on freezeback of piles of wood and of steel (pipe and BP sections) slurried back in pre-augered holes in Fairbanks silt at CRREL's Alaska Field Station, Fairbanks. No satisfactory equation fits the data but they show that about two weeks is probably a safe maximum time to assume for piles to be frozen back under the conditions at the place and time. A typical freezeback curve is shown in Fig. 28.
The author has proposed a simple rule-of-thumb that gives results adequate for practical purposes, when the augered hole is about 18 in. in diameter:

"The rate of heat loss \( h \) during the first stage of freezing of the backfill at 32F is 2 Btu/ft²hr F; the second stage of cooling the frozen backfill 90% of the degrees of frost\(^*\) takes twice as long as the first stage."

Example:

Heat to be dissipated \( = 1130 \text{ Btu/ft of length.} \)

Area of augered hole = cooling surface for backfill: \( 4.4 \text{ ft}^2/\text{ft} \)

Original backfill temperature (assumed): 32F

Permafrost temperature: 28F.

Then

\[
 t_I = \frac{1130}{2 \times 4 \times 4.4} = 32 \text{ hr (to freeze slurry at 32F)}
\]

\[
 t_{II} = 2t_I = 64 \text{ hr (to cool slurry below 32F)}
\]

Total time = 96 hr when the slurry temperature is about \( (32 - 0.9 \times 4) \approx 28\frac{1}{2} \text{F} \). This checks well with a "finite difference" analysis.

\(^*\)"Degrees of frost" is a useful term commonly used in Europe to mean the number of degrees below the freezing point of water.
The parameter $h$ varies a little; the values for sands are higher than those for silts. It increases as hole diameter decreases and is proportional to the coefficient of thermal conductivity of the frozen soil, but variability in soils in a natural profile precludes high precision in $h$. For a 6-in.-diam hole, $h \ge 3$ Btu/ft²hr°F and in small pipes it may reach 5 Btu/ft²hr°F or more.

**Artificial freezeback**: Freezeback by refrigeration is used if the permafrost temperature is high and/or if the time required for natural freezeback is not allowed. Figure 29 compares the times for freezeback without and with refrigeration—a reduction from 15 days to 1 day.

The design of a refrigeration system is normally left to an experienced refrigeration engineer. The structural engineer's specification will require that the slurry temperature below a critical depth does not rise above the ambient temperature 24 hr after refrigeration has been turned off. Thermocouple strings are used for control, giving slurry temperatures at intervals of depth on about 5% of the piles, which are frozen in groups. Some typical computations are given in App. B.

References 15, 26 and 56 give details of refrigerated pile foundations, the first at Bethel, Alaska (1955-1956) and the second two at Kotzebue, Alaska (1955-1956). Reference 36 describes how the holes were augered.

At Bethel, with permafrost at about 30°F, the slurry was frozen to 25°F in about 2 days; at Kotzebue, with permafrost at about 25°F, freezeback was complete in 2 days. Calcium chloride brine was circulated as coolant.

In 1961 at Kotzebue, when liquid propane at a lower temperature was circulated and the area of the refrigerating coils was greater, freezeback took only 1 day (Fig. 29b). A fast freezeback is preferable in minimizing water migration to the pile surface where ice may form.

The saving in time by the use of refrigeration may be invaluable in a short working season. The standby safety measure of having piles that can be refrigerated to raise their bearing capacity may also be valuable and the artificial freezeback technique has much to commend it, but it is certainly expensive, and over-refrigeration must be guarded against.

**Heat flow down piles.** There is little practical information on heat flow down piles with consequent temporary loss of support (which is probably seasonal). Care is taken to shield steel piles from direct solar radiation or to reduce absorption by appropriately painting them. Individual shades are sometimes used on the upper column part of an isolated steel pile. A shade consists of a box, usually of aluminum alloy or aluminum-painted light-gauge steel. Care must be taken in the fastening to the column as wind damage has sometimes occurred. The air-space between box and column plus the reflective surface of the box is very efficient. Special devices have been used where considered advisable, but if any doubt exists, an extra 2 ft of embedment to cover the summer weakening would probably be satisfactory. A continued heat flow is most unlikely because the
length of pile exposed in the airspace effectively cools the pile down in winter. Wood, with its very low coefficient of thermal conductivity as compared with that of steel (about 1 to 250), presents no heat-conduction risk.

Computations based on heat flow down a bar are not very reliable. The recommendation of an added 2 ft of embedment is based on observations made by CRREL by probing around steel piles with a steel bar.

Heave force. The jacking effect of a freezing seasonal frost zone of frost-susceptible soil is well known and is guarded against by an adequate pile embedment in the permafrost. In the past, the reduction of embedment by isolating the heaving soil from the structure seemed an obvious economy, but when the low adfreeze bond stress and unreliability of point bearing became evident the increased length of embedment required for load became adequate for uplift, especially when aided by the load on the pile. The sleeves and other devices once proposed, therefore, are not now recommended except in special cases where anchorage is doubtful although bearing capacity is satisfactory.

The best method is that described under Bench Marks in Permafrost (p. 53) but tests are still being conducted at CRREL's Alaska Field Station for such structures as utility poles and lightly loaded posts where the negligible downward load makes deep embedment for load capacity unnecessary and hence uneconomical. Much trouble and expense have resulted from the jacking of

Figure 29. Freezeback of sand-silted piles, Kotzebue, Alaska, July 1961, curves of temperature-depth with time.
utility poles out of the ground during their first winter. (Lateral forces on utility poles, acting as cantilevers embedded in the active zone, have caused many failures also.) Ice formed in the cavity beneath a heaved pile prevents its recovery, as the heaved soil subsides with summer drainage. This reduces the adfreeze area, so that once a pile has heaved it will probably continue to do so every winter, until it is jacked out of the ground or toppled over earlier by lateral loads.

**Special anchorage in permafrost.** Research work on special anchorages has been largely confined to tests of devices used on utility poles for the reasons discussed. Pictures of devices
tested are sometimes seen; many are useless owing to the creep properties of frozen soil at points of stress concentration.

Solid horizontal crosspieces well fastened to a pole can be effective but putting the extra material of the crosspiece(s) into added length of pole to go deeper for proper anchorage is probably easier and cheaper than excavating the permafrost to install the crosspieces, plates, etc.

Wood piles are placed butt down to improve their anchorage and in this way may have an advantage over steel and concrete piles. Furthermore, the adfreeze bond is probably better with wood than with other constructional materials. Laboratory and field creep test results seem to be inconclusive on this point.

Cleats welded to steel piles have been tried with doubtful effectiveness.

**Pile installation in permafrost**

These methods of pile emplacement have been used:

1. Driving by conventional means (steel piles and R.C. piles).
2. Placement in a preformed hole a few inches greater in diameter than the pile and backfilling with slurry that freezes and seals in the pile (standard for wood piles). A recent (1965) Russian variant is to fill about one-third of an augered hole with slurry, tap the pile into it and then backfill to grade.12
3. Placement in an open excavation and backfilling with non-frost-susceptible soil around the pile.
4. Driving into a steamed or augered hole of slightly smaller diameter than the pile.
5. Placement by high-frequency vibration (USA CRREL, 1965, experimentally).

**Conventional driving.** If local conditions permit, simple pile-driving is the preferred method. As experience accumulates, it will become possible to predict with confidence whether conventional driving will be satisfactory and what hammer will be desirable, but at present only test driving at a new site can determine whether driving is feasible and the best way to do it. For example, a test was made in SM soil at 29-32°F, at Bethel, Alaska, in 1955. A 14BP73 pile was driven 35 ft by a Vulcan No. 1 hammer at an average of 60 blows/ft (maximum of 200). The compressor capacity was 600 ft³/min. Figure 30 gives some driving records.

Near this test pile, a similar pile was driven 50 ft with an average of 130 blows/ft for the last 15 ft (up to 260 blows/ft were required). As in unfrozen cohesive soils, it is important to minimize interruptions in driving. At this job, 500 blows were needed to restart a pile after a stop. The point of refusal is not easy to judge; at this job it was assumed if 1000 blows produced no measurable set.

In Vorkuta, USSR, 14-in. R.C. piles about 20 ft long were diesel driven in 20 min in a fine silty soil at about 28°F (29°F was thought to be the limit).

Some well documented pile-driving tests at Bethel in 1960 show how diesel hammers behave.6 Diesel hammers had been used in Alaska to drive 4-in. pipe piles in SM-ML soils at high temperature (31-32°F) but here the piles were 8BP36's. Figure 31 gives site data and Figure 32 gives the driving records for two of the piles: BR-1 and BR-3. The second pile had a ¼-in. refrigeration pipe protected by a 2½ × 2½ × ¾-in. angle along each side of the web; when these projections reached the permafrost in driving, the resistance to penetration increased markedly (Fig. 32). The hammer was a Delmag D12, a diesel driver with fixed leads on a crawler crane. Its specifications were:
Figure 30. 14BP73 driving tests, Bethel, Alaska, Nov 1955.

Overall weight 5290 lb
Piston weight 2750 lb
Anvil weight 754 lb
Energy per blow 22,600 ft-lb (manufacturer's figure)
Rate 46-50 blows/min (50 used most of the time).

For some short piles (8BP36) with 16-ft embedment in permafrost the last 10 ft took about 21 blows/ft and the pile was placed in 6 min. For 30 ft of penetration, 22 blows/ft was a good average.

Plumbing and spotting were excellent and the piles were undamaged by the heavy hammer. (Figures 29 and 31 show that steel piles can be driven in the favorable conditions of a deep uniform fine soil at high permafrost temperatures.)
It is vital to remember that refusal does not necessarily indicate a satisfactory job; if the active zone can freeze and the soil is likely to heave, depth of penetration in permafrost sufficient to resist uplift must be ensured or other measures taken to isolate the pile from the active zone.

In arctic conditions, diesel hammers have much in their favor. Water for steam generation may be in short supply, apart from problems with freezing temperatures, and problems with logistics because of heavy and bulky equipment and supplies. Fuel is very costly and must be used with the utmost efficiency. New techniques employing vibratory drivers or special impulse procedures are being studied and the outlook for the use of high-frequency vibrators in permafrost is promising. Heating of the permafrost must be reduced to a minimum and high-speed driving may put an undesirable amount of energy into the ground. The equipment is very heavy, and electric power demands are great so that the technique is somewhat limited.

If steam hammers are used, compressed air should be considered, and it is, in fact, often used.

Drophammers have been employed, usually to drive piles in preformed holes, but they are really no more than an expedient to be used when more efficient equipment is unavailable.

No recommendations regarding hammer size for driving a given pile in particular ground conditions can yet be made. Heavy hammers are necessary for the driving of steel piles; recent
Driven piles develop an excellent bond with the soil. It is probable that frictional heat caused by soil displacement melts a small amount of ice, which quickly refreezes to form the bond. According to field tests by CRREL, friction between pile and ground gives additional tangential resistance of about \( \frac{1}{2} \) of the adfreeze stress so that after pronounced slip (breaking of the ice bonds) the resistance to motion is about \( \frac{1}{4} \) of the maximum amount.

**Placement in preformed holes.**

Steam jetting: Although not normally used by the Corps of Engineers, the practice of steam jetting holes for pile placement has been so common (and often successful when applied with understanding and control) that discussion on it cannot be omitted. The results of an improper use of this method are so serious, however, that anyone proposing to use it is cautioned to do so with great care. Failure of an individual pile is serious but the more important failure of a whole group of piles is possible when steam-jetted piles are too closely spaced and a block of permafrost is thawed out by careless steaming. In Siberia, between 1952 and 1961, piles were not
used because of failures due to uncontrolled steaming. Steaming by "points" left in the ground for some hours prior to excavation of permafrost for trenches etc., is a valuable expedient.

References 37 and 38 give interesting information on the subject. Reference 34 presents more detail and shows that of 16,500 piles placed by steam-jetting only 20 piles failed. So small a proportion of failure could be important in military construction but for civilian housing, etc.; perhaps the risk is acceptable. On the other hand, reference 37 shows that 61% of piles foundations made by this method failed: 42% by settlement and 19% by heaving.

In 1952, the Alaska Field Station of CRREL (then the Fairbanks Permafrost Research Area of ACFEL) began a study of pile installation methods. Steam pipes 21 ft long, \( \frac{3}{4} \) in. in diameter, using steam at 80 psi and 220°F (at the jet), reached 20 ft in 10 to 20 min in a permafrozen silt at 28-32°F. Another 3 hr was required to produce a 12-in.-diam hole.

Progress in steaming is fast in soils of low ice content but water may have to be added if the soil is very dry. Large ice masses slow down the advance; boulders and fibrous organic remains are also troublesome. Boulders have been dealt with by a "springing" technique: the thawing of a "bulb" at the bottom of the hole (into which large stones can fall or be pushed by the pile when it is tapped into place). A soil of high ice content commonly becomes a soil suspension in which a wood pile floats when placed, and ballast may be required to hold the pile down. Ice lenses and organic layers of very high ice content tend to form ledges, so care is needed to ensure a full bore throughout. Rates of progress from 3 ft/hr in gravel to 50 ft/hr in silty clay have been recorded, using steam at 100 psi in "cold" permafrost to a depth of 20 ft.

The end of the steampipe may be left open, slightly crimped, or fitted with a special point to suit soil conditions. Russian techniques employ a pointed or chisel end with slots for the steam rather like the common well-points used in this country; the point is driven a short distance, halted for a time while steam is admitted, then driven a few more feet, steamed, etc. The pipe has a 25-lb driving head and a crossbar for handling and twisting.

The tendency is to use higher pressures but pressures of 80 to 120 psi have been used. From 80 psi up, the driving rate increases about 10% for every 15 psi steam pressure rise. Reference 16 describes a modern Russian technique using 150 psi.

Permafrost is coldest in the spring so that holes steamed then are likely to freeze back faster than holes steamed later in the year. At Inuvik holes steamed in spring took from 4 to 6 weeks to freeze back; those steamed in October took a slightly longer period. Autumn steaming is convenient in that the superstructure is most likely to be built in the summer construction season, but in some cases it would be advisable to steam in spring and build in the summer of the following year. Over-enthusiasm with a steam jet has provided foundation piles not frozen back a year after placement; however, some froze back in one day at the same site. Spacing is very important here.

If piles are steamed in the autumn and have not frozen back by the time a frost-susceptible active zone freezes, heave is probable during the first winter. If time permits, therefore, spring emplacement and a year's waiting period is safer if there is any doubt about efficient backfreezing.

The technique of backfilling around a pile with non-frost-susceptible soil in the active zone has been used. Sometimes it has been found advisable to pump out the water in a steam-thawed hole before placement of the pile to increase the rate of freezeback.

Artificial refrigeration is also a possibility to increase rate of freezeback but since the object of steaming is to save expense this technique is not particularly attractive.

Other thawing devices: Experiments in the USSR have proved the efficacy of thawing frozen ground by using hot water at 100 to 120°F in closed pipes. This suggests a system of thaw
pipes and headers in which circulating hot water (or other liquid) could thaw several holes at a time. This would be slow but under close control, whereas open-pipe thawing as presently practiced is not under good enough control.

Electrically heated “needles” have also been tried experimentally but have been used rarely, if at all, on jobs. They are mentioned in Russian publications.

The circulation of steam in a closed piping system is a possibility that gives control without the addition of water from condensation. Heat transfer in this system is slower, however, and the system has not been used to the author’s knowledge.

Dry-augering: The Corps of Engineers prefers dry-augering to drilling of holes for piles. Dry-augering introduces the minimum amount of heat and produces a hole of known size and shape. The Corps of Engineers has tried several drilling methods but dry-augering is the only one it now uses. The hole produced by this method is commonly larger than the pile but some 4-in. pipe piles have been placed in slightly smaller holes and then driven into place.

Not all soils are suitable for dry-augering although good results have been obtained even in coarse gravels by use of the “Alaskaug” bit (Fig. 33). This auger is power driven and has special replaceable carbide cutters (boron carbide is best for material hard to cut). It is made in many sizes of even inches from 12 to 24 in. At Kotzebue, holes up to 24 in. in diameter and 31 ft deep have been augered in permafrozen silt and ice at 25 to 31°F at an average rate of about 35 min for
a hole; the time required ranged between 15 and 50 min. Specifications used by the Alaska District, Corps of Engineers do not allow any heat or chemicals to be used with the auger. Casing may be used temporarily in the active zone. The hole must be kept dry and clean; surface drainage may be required to ensure this.

A sand slurry of the following grading is preferred for an annulus greater than \( \frac{1}{2} \) in. (for an annulus of \( \frac{1}{4} \) to \( \frac{1}{2} \) in., a silt slurry is recommended).

<table>
<thead>
<tr>
<th>Sieve size</th>
<th>% finer by wt</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{4} ) in.</td>
<td>100</td>
</tr>
<tr>
<td>#4</td>
<td>93-100</td>
</tr>
<tr>
<td>10</td>
<td>70-100</td>
</tr>
<tr>
<td>40</td>
<td>15-57</td>
</tr>
<tr>
<td>200</td>
<td>0-17</td>
</tr>
</tbody>
</table>

This material is mixed with a minimum of water at not more than 40°F and placed at a temperature between 35 and 45°F. No frozen material is permitted in the slurry. Each pile must be completed, from hole drilling to placement and backfilling, in 24 hr. The slurry is placed in 3- to 4-ft lifts around the pile and vibrated by a spud vibrator; the pile is then tapped to give good compaction and prevent voids along its surface. Excess water rising to the top must be carefully drained off. If the slump is about 6 in. deep, the material will flow adequately and provide a good backfill.

After being slurried, the pile is left to freeze back naturally or it may be artificially refrigerated; in this case, specifications require that the refrigeration must be started within 24 hr after the pile is slurried.

If the hole is accidentally drilled too deeply, it must be backfilled to the correct elevation with well tamped gravel. No undercutting of a hole is permitted.

**Artificial refrigeration of piles:** Two complete units must be supplied by the contractor to cover the risk of an interruption due to mechanical breakdown. A recent Alaska District, Corps of Engineers specification for a particular job stated that the refrigeration plant must be adequate to freeze back 18 piles in 18 days, or an individual pile in 16 hr.

Thermocouple strings are attached to some of the piles to indicate when freezeback is complete. The criterion is that the slurry temperature recorded below a 20-ft depth does not rise above a certain temperature within 24 hr after refrigeration has been turned off. This control temperature is determined by the ambient temperature of the permafrost before it is disturbed. Piles not equipped with thermocouples must be given the same amount of refrigeration as those under temperature observation. Propane under 15 to 35 psi pressure is the common refrigerant.

Careful records are kept of the freezing plant with observations at 4-hr intervals throughout the freezeback period. After completion, freeze pipes (or coils) along the piles are filled with (chilled) oil, SAE #10, to within 6 in. of the top and capped. Refrigeration can then be provided at any time in the life of the structure if the thermocouples, left in for periodical observation, show detrimental temperature rises. (Refer to App B for typical computations in artificial freezeback.)

**Special methods.**

**Open excavation and backfilling:** In only one major job (Thule Air Base) has the principle of placing piles in an open excavation been used. Here the active zone and upper permafrost consisted of bouldery outwash gravels too difficult for normal pile-placing procedures; yet large and heavy hangar doors had to be supported on rails with the minimum possible differential settlement.
The solution was to dig a deep excavation, place the piles and backfill with gravel. This may not have been the most economical method but the technique has resulted in no trouble for many years.

The support of a structure by pads on permafrost with posts extending above grade resembles a piled foundation with an expanded pile point, a shoe, for greater bearing area. Such shoes are sometimes placed in individual excavations and sometimes in trenches; they are not usually placed in large open excavations because the area of exposure of permafrost must be minimized.

Non-frost-susceptible backfill must be used to minimize heave forces and, if no volume change can be permitted, water should be excluded from the fill either by positive drainage or by encapsulation by a waterproof barrier.

**Modern vibration techniques (as of 1968):** Piles have been driven in cohesive soils by vibrators, vibrohammers and high-frequency vibrators; tests by CRREL have shown that steel piles can be driven by high-frequency vibrators in artificially frozen ground. However, extensive testing under natural conditions is required before recommendations can be made.

**Explosives and rocket techniques:** CRREL experiments have shown that placement of piles by using explosives and rockets is not yet practicable because of extreme energy requirements.

### Field tests on ‘friction’ piles

Pile tests in permafrost differ from those in unfrozen soil in the effects of temperature and time on the design load. Short-term tests with step loads rapidly applied give very unsafe bearing capacities; yet creep tests such as those made in the study of the rheology of frozen soils take too long a time, during which soil conditions vary.

**Ground temperature.** Temperature indicators, usually thermocouples, must be installed for the full depth of the pile at intervals not exceeding 5 ft for observations of the temperature field before and during the test. To check temperature data, probing with a rod or, if necessary, a drill, is necessary to measure the depth to frozen soil.

Tests should be made in the late fall or early winter when the permafrost is at its highest temperature and therefore has its lowest bond strength and least resistance to creep.

**Time.** The slow creep under tangential stress well below the maximum reached in a short-term test to failure means that ideally a steady test load should be left on long enough for the settlement to become constant. This may never occur. In frozen silt the load at which the pile remains stationary is probably too low to be of practical value and design must often be based on an acceptable rate of settlement.

The relationship between load and rate of settlement must be found through field tests after the length of the test piles has been estimated from laboratory creep data for the soil at the site (p. 21). The measurements of big loads and very small, slowly increasing settlements under varying air temperatures (which can normally be ignored) are particularly difficult in cold regions, even when all the instrumentation, supports and load are shaded from the sun, as they must be. The slowly varying ground temperature in a test that must be of long duration also introduces complications, since the soil properties are changing and several differing soil strata are common. Despite extreme precautions F.E. Canny (unpublished data) has experienced disconcerting diurnal air-temperature variations.

Figure 34 shows the results of a typical pile test. The test was conducted for a project at Goldstream Creek, near Fairbanks, Alaska. The load was applied in 10-ton increments at 24-hr
intervals and dial gages were read every few minutes. Figure 34a shows the relationships between settlement and time for these conditions. Figure 34b shows the individual rates of settlement on a base of load.

The procedure of extrapolating the semi-logarithmic plot is of doubtful validity but when combined with a factor of safety probably gives the best interpretation and application of the data. Under a 20-ton load, the maximum expected, a rate of settlement of about ¼ in./yr seems to be appropriate for the ground temperatures at the time of test. There are no data available for adjusting the rate of settlement for temperature changes. The permafrost temperature was about 31°F (August); the highest permafrost temperature was about 31½°F (November) and the lowest about 30°F (May). The load would not vary much throughout the year; the settlement rate would increase during the fall and diminish during the spring but on the average would not exceed ¼ in./yr. The structure was made rigid to even out possible differential settlements as much as possible.

Settlements have been continually recorded for future analysis of the structure and its foundations.

Reference 7 gives pile driving records and computations determined by the Engineering News Formula for possible comparison with other tests.

At present (1968), a pile foundation design is best based on previous job experience, with careful study and judgment to take account of variables, and checked by pile tests at the site. However, a comparison of data from test piles and laboratory experiments with performance records

*Data by F.E. Crory, Curve not cited report.
will gradually lead to a better preliminary design procedure although test piles will always be advisable for final design. For nearly all piles in permafrost emphasis must be on rate of settlement, not on load bearing.

FROST HEAVE AND FOUNDATIONS

No firm design data can yet be given regarding the force exerted by a heaving soil on a contiguous structural surface. Observations, test results and theories have been published but the only reliable way of estimating the heave forces is by field tests or by observations on structures at a site (or at different sites where conditions are known to match precisely). One reason so little is known about the phenomena of heave forces is that field tests are very difficult to make. It is not yet (1968) possible to predict surface heave displacement, with any confidence.

Vertical forces on a horizontal structural surface

In normal practice, footings are not placed in the active zone, but occasionally frost may penetrate beneath a slab or footing on heaveable soil in a seasonal frost area. By estimating the weight of blocks and buildings known to have been raised by heaving soil, computed values up to 8 tons/ft² have been found. Reference 23 proposes a reasonable procedure for computing normal heave forces on foundations. An upward pressure of 2 tons/ft² is assumed to act on the base of a pyramid formed by a 45° spread from the footing to the depth of frozen soil. Thus a 4-ft by 6-ft base at 3-ft depth in soil freezing to 5 ft would have an 8-ft by 10-ft area exposed to the heave force, giving an estimated force of 160 tons. The weight of the estimated pyramid of soil would deduct less than 10 tons, leaving, say, 150 tons upthrust or about 6½ tons/ft² on the footing area. This is well above the design pressure used on a frost-susceptible soil; therefore, the footing could be expected to heave.

Availability of water is important here and so many factors enter into this problem at a particular site that little improvement in precision seems possible. Migration of water to the freezing front, which is responsible for heave, is inhibited by pressure. The soil-water-ice system is extremely complex, and is an active topic in research. Theories exist but field data from controlled experiments are rare and inconclusive.

Tangential heave forces on a vertical structural surface

It is evident that the slow freezing of the soil limits the tangential stress to a yield shear stress of the soil but what that stress will be cannot be estimated very closely; test and field data are scanty and existing theories are not convincing. The stress will be between ½ and 2½ tons/ft²; it is roughly proportional to the degree of saturation of the soil, other factors being the same. The greatest heave force seems to be when the frost penetration is from ⅓ to ⅔ of the maximum depth reached during the winter. Figure 35 is recommended for use in the USSR. Reference 10 shows measurements of heave forces up to 13½ tons/ft of perimeter on piles in permafrost. It also quotes from 2 to 3½ tons/ft of perimeter for tests at Igarka, USSR. Corps of Engineers' tests at Fairbanks, Alaska, have given 6 tons/ft of perimeter on steel piles in organic silt. The heave force is temperature dependent, but is usually from 1 to 2 tons/ft². A small displacement significantly decreases the force.
Anchorage against heave in permafrost

Years of testing in Alaska indicate that a 10-ft embedment in permafrost is safe in critical places of warm permafrost and air temperature extremes. The old Tsytovich rule that embedment should equal twice the thickness of the normal active zone is usually satisfactory; however, three times the thickness is sometimes used. Again, experience and/or tests for a particular set of conditions are necessary for the design of an important structure.

Special anchorages for posts and piles are not usually worthwhile. Since frozen soil is a highly viscous material it flows faster at higher stresses so that small projections introducing high localized stresses cannot be expected to help much. Since larger projections demand more costly excavation, greater depth of embedment is an easier solution.

An increased dead load, if feasible from the point of view of bearing capacity, is a simple and inexpensive means of anchoring against frost heave forces. For this purpose, granular fill is almost as good as concrete and considerably cheaper; it has also been used as additional load for anchoring footings against high wind loads. Large concrete blocks have been cast on heaving columns in an attempt to prevent further uplift.

Anchors for guy wires, etc., in permafrost have given problems in the past and no single solution is possible because site conditions vary so much. Deadweight has been used because of these difficulties.

Frozen bedrock is often badly shattered and full of ice for a depth of several feet so that thawing produces chunks of rock in water. The best solution in rock, therefore, is to drill a hole and to grout in a wedge anchor bolt or bar with expanding mortar. A good cement that hardens at low temperatures without water, or heat, production is needed. In soil, the smallest possible hole should be augered. (Steamed holes, melted sulfur and molten lead seem to have no place in frozen ground engineering although they have been used in the past.) The anchor bars should be inclined at an angle to the pullout force. Pullout tests should be made at the site, and the time and temperature effects on the soil properties should be considered. These tests are creep tests, to determine the rates of anchor movement for a range of working loads, and should be made at the highest ground temperature (late autumn is the best time for testing).
Anchoring foundations in unfrozen soil

The foundation of an unheated structure in a region of seasonal frost will be exposed to heave forces if it is in contact with a frost-susceptible soil. Generally this is avoided by the use of a well-drained non-frost-susceptible backfill. If the frost penetration is small the structure load may be adequate to prevent structural heaving without the use of selected backfill but the stability must be carefully checked. The frost penetration will usually be more in the granular backfill than in the native finer-grained soil it replaces, roughly inversely as the square root of the weight of water per cubic foot (Fig. 43 and 44 can be used for an estimate).

If the base of a foundation is above the bottom of the frozen soil layer, the direct upward pressure must get its reaction from the ground below. Similarly the top surface of a footing below the frozen layer will give reaction for the heave force arising in the frozen soil above the footing.

Assuming 1 ton/ft² for tangential adfreeze stress and 2 tons/ft² for direct heave pressure acting downward on the footing, the net required area can be found. A factor of safety of at least 1½ is suggested, assuming that the full depth of freezing is effective. The tension between structure and foundation should not be overlooked.

Uplift forces on foundations indicate the desirability of using materials with appreciable tensile strength. Coursed masonry is particularly bad because the joints give convenient surfaces for maximum stresses so that the construction is readily torn apart. Poured concrete or precast concrete panels are better. Steel and reinforced concrete are also very good. Posts, pillars, and pedestals must be well fastened to shoes at their bases. The adfreeze bond between soils and the various construction materials has been studied and tests are still in progress, but results so far indicate small differences in the bond of the materials.

Figure 36 illustrates an interesting example of the anchoring of a pier foundation against the forces of frost heave by bolts drilled into bedrock. The foundation is at Kapuskasing, Ontario, Canada, in a seasonal frost area where frost penetration under the bare ground surface may reach 7 ft and the soil is highly frost-susceptible. Each foundation was designed for 200 tons of uplift from tangential frost action along the sides (the base of the pier was taken below the frost line) and great care was taken to reduce adfreeze between the concrete and soil by means of sloping sides and greased surfaces. Neither of these artifices can be relied upon to reduce adfreeze stress for many years. It is probable that heave forces just about balance the designed (and test-proved) anchor pullout resistance, so that structural live and dead loads provide the factor of safety against uplift of the pier.

Reduction of heave forces

Another approach to reducing heave forces, once popular but now rarely used, is to isolate the structure from the active zone, allowing the soil to slide up and down an immovable structural surface. Sloping and greasing, as mentioned above, have often been used, separately or together. Frost batters on walls, used for centuries, are of doubtful value. Tar paper also seems to have little permanent effect. A packing of a sand-bentonite mixture around the pile has been used but is not recommended.

Some years ago the isolation of piles by collars in the active zone was recommended for general practice. Although experience has shown that this is not the best way to control structural heave, the method is used for permanent bench marks (see p. 53) in permafrost; it has also been used for emergencies where piles have been placed deep enough for bearing but not deep enough for anchoring against heave from the freezing active zone.
In one case, in Alaska, the soil around "suspected" driven piles (steel BP sections) was excavated by hand to about 2 ft diam and to a depth sufficient to ensure an embedded length equal to at least twice the thickness of the predicted active zone below the sleeve, but with a maximum of two-thirds of the predicted active zone thickness. (Structural considerations in column design restricted the design in regard to heave forces.) A light (20 g) steel casing 18 in. in diameter (in halves) and flanged at the bottom was placed around the pile and bolted to form a pipe sleeve. The outside was greased and backfilled with sand and the special ACFEL grease (see Fig. 37) was placed between the pile and the pipe sleeve. The treatment was somewhat expensive but periodic leveling has proved its success.

Recent CRREL work at Fairbanks, Alaska, has proved the efficacy of simple sleeves on small lightly loaded pipe piles in silt but the sleeves tend to be jacked out of the ground with each annual cycle of freeze and thaw. In this example, a flange was used with success around the base of each sleeve to resist the jacking effect. Grease placed in a form of plastic sheeting could probably be used; and recent tests by CRREL at the Alaska Field Station, Fairbanks, show that an oil-wax mixture with a dry soil bulker is satisfactory without the sleeve. A 1:3 mix of grease to sand seems to be all right and reduces the amount of expensive grease needed.

Lowering the ground-water level reduces heave and heaving forces in many soils. A backfill of well-drained gravel in the active zone is an excellent way to isolate a structure from heaving soil. The ideal backfill consists of well-drained, smooth, rounded, uniform coarse gravel.

Heating a building is usually adequate to prevent freezing and heave uplift along the foundation walls. No lateral pressures due to heave outside an insulated wall have been reported.
FOUNDATIONS OF STRUCTURES IN COLD REGIONS

BENCH MARKS IN PERMAFROST AREAS

If bedrock is not available for a bench mark the permafrost may be used quite successfully if special care is taken.

Experience has shown that embedment of bench marks in permafrost even to three times the thickness of the active zone has not always resulted in a stable reference point. For that reason CRREL uses a bench mark isolated from active zone heave forces by a pipe sleeve containing a special thick grease.

Figure 37 shows a successful CRREL design which is a slight change from a bench mark that has had adequate stability for the past 15 or 16 yr, in a region of very frost-susceptible soils, weather extremes, and high permafrost temperatures with artesian water conditions – all combined into as bad a situation as possible. For a description of bench mark development see reference 55. Also consult reference 20 for recent Canadian practice. Similar bench marks are used in the USSR. The CRREL bench mark may seem rather elaborate but experience has proved the necessity for something of this kind. The oil-wax grease developed for the bench mark has proved to be ideal for a waterproof packing, which flows at very low temperatures and is stable and chemically inert.
Simpler bench marks have been used for short-term work but the absence of a holding-down load such as that helping to resist heave in a pile makes it necessary to use the more complicated design for any important job; the cost is relatively small for the reliability gained.

Bench marks preserve their value after the construction period since settlement observations must be taken on major structures for some years after construction. Structures on permafrost should not be built and forgotten—the effects of foundation movements are too serious. The first year of a structure’s life is critical but there is risk of trouble developing much later and level readings are not very difficult or costly. It is unfortunate that more data on existing structures are not available.

**THERMAL INTERACTION BETWEEN HEATED STRUCTURES AND PERMAFROST**

**General principles**

If the ground surface temperature is permanently above 32°F, as it usually is under a heated building, no practicable thickness of insulation will prevent ultimate heat flow into the permafrost and consequent degradation accompanied by a foundation failure. Heat flow from the building must be diverted from the ground but it is not usually necessary to divert all the heat flux all the time. A pad of non-frost-susceptible soil can serve as a buffer in permitting thawing and freezing through a designed thickness based upon the annual air temperature cycle. During the summer a certain depth of the pad thaws and in the winter the thawed portion refreezes. Careful design ensures that the natural ground under the pad never thaws.

The simplest way of preserving the permafrost is by an airspace under the structure through which the cold air of winter passes and freezes back the ground thawed during the summer. The depth of ground thawed under the airspace will be less than the thickness of the normal active zone unless the soil has been excavated and replaced by a backfill of different material; in the latter case, the depth of thaw can be estimated by the modified Berggren formula (App. A), based on the local thawing index.* The shading effect of the building will, of course, reduce the thawing index under the structure (about 15%) but some of the building’s edges will not be shaded. This is sometimes dealt with by adding sunshades formed of horizontal planking in steps without risers (Fig. 13) or an unheated overhang; either device shields the structure from the sun but does not obstruct air movement.

If a simple airspace is inadequate, forced circulation of air is necessary. The requisite draft is usually provided by manifolds and stacks. Fans have also been used but because they require powered mechanical equipment are avoided if possible. A fan system was used on one job, where permafrost degradation was causing serious building damage, but was inadequate, and as a last resort artificial refrigeration had to be used to maintain the ground in a frozen state.

While some form of airspace, with or without forced draft, is usually possible, occasionally a floor or wall of a heated building contacts the ground or ice where no temperature rise of the ground or ice is permissible; it may then be necessary to maintain the contiguous soil below its normal temperature. The only solution is to use refrigeration grids.

Artificial refrigeration is sometimes used in emergencies as the best way out of trouble. Examples are (1) a settling corner of a building caused by a subsurface flow of water developed

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*Thawing Index is the total accumulation of daily average temperatures measured above 32°F, summed for the summer season (see App A, ref 58(6) and 43).
over the permafrost after the building had been in use for some years; and (2) a reinforced concrete building in danger of collapse from differential settlements up to 2 ft caused by permafrost degradation due to heat flow from the building far in excess of that allowed in the design. In the latter case, the very costly underpinning and refrigeration was justified because the building was full of valuable communications equipment in an important system. For a different structure it might have been better to erect another building. (See App B for notes on refrigeration systems.)

Although most buildings can be dealt with either by simple airspaces or by ventilation ducts, special structures may require unusual treatment.

Airspaces are commonly up to 5 ft high, sometimes more. If they are smaller, keeping the space clear for efficient ventilation is difficult.

Removable skirting has sometimes been provided to keep the airspaces closed in summer and open in winter then, if the ground floor becomes cold and the airspace is closed by mistake a maintenance problem arises. Generally, however, there seems to be no need for skirting the airspace provided that the space is not allowed to become obstructed by snow, stores, or rubbish. The ground beneath should be graded to prevent ponding from rain, melting snow, or waste water.

Computation methods for estimating the required airspace height have been proposed but have doubtful value. If the height of the airspace is not less than one-tenth of the smaller building-width there should be no trouble. If an airspace less than 2 ft high is unavoidable, natural ventilation may be inadequate and some form of forced draft becomes advisable. Russian codes give not less than \( \frac{1}{3} \) m for widths up to 12 m, and not less than 1 m for widths of 20 m or more.

Ventilated pan-duct systems

Experience has shown that trapezoidal steel pan ducts 18-20 in. wide and 12 in. high with 12 in. of concrete between them (i.e., spacing of 2 ft 8 in.) are practical construction for the heaviest floorloads. Associated with the pan duct are a base slab of 4 in. or 6 in., an 8-in. concrete wearing course, a layer of cellular glass or other insulation, and 6 in. of concrete between the insulation and duct roof (Fig. 38). If the ducts are more than 20 ft long, special provisions for draft are necessary. The length of the ducts is determined by the building dimensions and use (wind direction may have some influence), so that the design consists of finding the best thickness of insulation and the required stack height, assuming that the "chimney effect" is to be used for forced draft. The importance of airtight construction in the ventilation system cannot be over-emphasized.

A rough rule of thumb devised by the author is:

\[
H = \frac{1}{140} \left( \frac{L}{v_{s}(L + 1)} \right)^2 (L + 250)
\]

where \( v_{s} \) is the air freezing index in degree-days divided by the length of the freezing season (= average daily degrees of frost during the freezing season)

- \( H \) is the stack height, ft
- \( L \) is the length of the ducts, ft
- \( t \) is the actual, or equivalent, thickness of cellular glass, in., based on thermal resistance.

With this rough formula, the thickness of insulation and the stack height can be estimated; the design consists of checking these dimensions. If another insulating material is used in place of
Figure 38. Section of pan-duct floor.

cellular glass, the equivalent thickness can be easily determined. The insulating layer will most probably be 4 or 6 in. thick. Owing to the high heat flow during summer when the ducts are closed and the non-frost-susceptible pad is being thawed, a smaller thickness will lead to an excessively thick pad.

Example of a design.

1. Building has 220-ft-long ducts of standard size.
2. Floor temperature varies, but 60°F may safely be assumed.
3. Floor section is as shown in Figure 38. The pad will be of dry gravel with $\gamma_d = 125$ lb/ft$^3$ (dry unit wt) and $w = 2\%$ (water content).
4. Mean annual temperature of the pad at outlet is about 32°F (reasonable assumption).
5. Surface heat transfer coefficient (including convection and radiation) $h_c = 1.0$ Btu/ft$^2$ hr°F. (This is based on field observations. It varies throughout the year but 1.0 is a good average value.)
6. Roughness of duct surface, $e = 0.001$. This number is difficult to predict but tests show that 0.001 is about right. (However, the effect of large variations on the friction factor $f$ is quite small.)
7. Each right-angled bend has the effect of adding 65 diameters to the length of a straight duct; inlet and outlet each adds 10 diameters and each duct has two bends, one inlet and one outlet, adding a total of 150 diameters.
8. An efficiency of 80% may be assumed for the stack; i.e., the design height will be 25% more than found theoretically.
9. Mean monthly air temperatures for the past 10 yr are known; from these the mean freezing index can be found (Alternatively, if the freezing indexes for the site are available from weather records the lowest may be selected.) The lowest freezing index is about 1000 less than the 10-yr mean for places having a mean freezing index between 3000 and 8000 degree-days.

The length of thawing (or freezing) season must be known: local weather data will give this. Approximate values are:

<table>
<thead>
<tr>
<th>Freezing index deg.-days</th>
<th>Length of freezing season days</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>160</td>
</tr>
<tr>
<td>5000</td>
<td>200</td>
</tr>
<tr>
<td>8000</td>
<td>260</td>
</tr>
</tbody>
</table>
In the example:

Minimum freezing index = 4000.
Length of freezing season = 215 days.

\[ v_s = \frac{4000}{215} = 18.6 \]

\[ H = \frac{1}{140} \left( \frac{220}{18.6 \times 5} \right)^2 (470) = 19 \text{ ft approximately.} \]

20 ft is assumed for check computations.

**Thickness of pad:** Ducts-closed condition. Assume the ducts are closed for the thawing season. Here they are closed for 150 days. Thawing index at floor surface:

\[ \theta = (60-32) \times 150 = 4200 \text{ deg-days.} \]

If \( R \) is the thermal resistance of the floor, depth of thaw penetration, \( X \), into the pad is determined by heat transfer theory (Modified Berggren equation - see App. A).

\[ X = KR \left( \sqrt{1 + \frac{45\theta}{KLR^2}} - 1 \right) \]

\( R \) is computed in the ordinary way assuming that the very-nearly-dead airspace is about equivalent in thermal resistance to concrete of the same thickness. \( K \) is the average coefficient of thermal conductivity found from curves of \( K \), \( y_d \), and \( w \) for granular soils (App. A).

\( L \), the volumetric latent heat, is computed from the weight of water in a cubic foot of gravel.

\[ L = 1.44 \nu \cdot y_d \text{ Btu/ft}^3 \]

here \( R = 13.5 \), \( K = 0.85 \), \( L = 450 \) and

\[ \frac{45}{KLR^2} + \frac{45 \times 4200}{0.85 \times 450 \times 13.5^2} = 2.71 \]

so that \( X = 13.5 \times 0.85 \times 0.92 = 10.6 \text{ ft.} \)

A 10-ft pad would be adequate to ensure negligible thaw penetration into the subgrade. (If the ducts were closed for \( D \) days, \( \theta = 28D \) and the computation would be similar.)

Latent heat per square foot for 10.6 ft depth = \( 10.6 \times 450 = 4770 \text{ Btu.} \)

Allow 10% for sensible heat.

Heat added to pad = 5240 Btu/ft².

This must be withdrawn by cold air ventilation when ducts are opened.

The duct will be open for the freezing season, here 215 days; therefore, the average rate of heat flow from pad = 1.0 Btu/ft² hr during the freezing season, based on floor area.

The average thawing index at the surface of the pad is found from the formula

\[ \theta = \frac{LX^2}{45K} = 1320 \text{ deg-days.} \]
This must be provided by cold air during the winter. The average freezing index = 1320 for 215 days, and the average surface temperature must be

\[
\frac{1320}{215} = 6.2 \text{ below } 32F
\]

\[= 25.8F.\]

The air at inlet has an average temperature below 32F of \(\frac{\text{freezing index}}{\text{days of freezing season}}\) = \(\frac{4000}{215} = 18.6\).

\[\therefore \text{Temp} = 13.4F.\]

Average temperature rise, assuming outlet air temperature about the same as that of the pad surface (checked by observation)

\[= 25.8 - 13.4 = 12.4F.\]

Heat flow from the floor during the winter season comes from the temperature drop between the floor surface and the duct air, divided by the thermal resistance between them.

Assuming 1.0 for surface heat transfer coefficient \(h_c\), the resistance is about 12 hr ft\(^2\)F/Btu.

\[\text{Temp drop} = 60 - 13.4 = 46.6\]

\[\text{Resistance} = 12\]

\[\therefore \text{Heat flow} = 3.9 \text{ Btu/ft}^2\text{hr.}\]

At outlet, temperature drop = 60 - 25.8 = 34.2 and heat flow = 34.2/12 = 2.9.

Then average rate of heat flow from floor = 3.4 Btu/ft\(^2\)hr.

Flow from pad = 1.0 Btu/ft\(^2\)hr and total heat flow = 4.4 Btu/ft\(^2\)hr. This is the amount of heat to be picked up by the cold air entering at 13.4 F and leaving at 25.8 F, as shown above.

Let \(V\) ft/min be air velocity in duct. Cross-sectional area = 1.58 ft\(^2\). Average air temperature \((13.4 + 25.8)/2 = 19.6\)F (say 20F).

Density = 0.0828 lb/ft\(^3\) (at 20F) and specific heat is 0.24 Btu/lb F.

Heat pickup = \((V \times 60)\) 1.58 \(\times 0.0828 \times 12.4 \times 0.24 = 23.3V\) Btu/hr.

Each duct covers 32-in. width.

Flow per duct = \(4.4 \times 220 \times \frac{32}{12}\)

\[= 2590 \text{ Btu/hr (220 ft long)}\]

\[V = 111\text{ ft/min, average.}\]

Check on duct system:

Assume 5 ft inlet pipes

220 ft duct itself (estimated)

20 ft stack height

245 ft.

(The stack height is not critical here).
$D$, equivalent diameter for friction computations $= \frac{4 \times \text{area}}{\text{perimeter}} = 1.22$ ft.

Bends, inlets and outlets @ $150D = 183$ ft.

Total length for friction drop $= 245 + 183 = 428$ ft.

Friction head $h_f = f \cdot \frac{L}{D} \cdot h_v$ where $h_v$ is velocity head.

$f$ depends on Reynolds No. and the ratio $\frac{e}{D} = \frac{0.001}{1.22} = 0.00082$.

Reynolds No., $N_R = \frac{VD}{\nu}$

where

$\nu$ is the kinematic viscosity

$V = 111 \times 60$ ft/hr

$D = 1.22$ ft

$\nu = 0.493$ ft$/hr$ at $20^\circ\text{F}$ (from standard tables).

$$N_R = \frac{111 \times 60 \times 1.22}{0.493} = 16,500.$$

$$f = 0.0055 \left[ 1 + \left( \frac{20,000 \times e}{D} + \frac{10^6 \sqrt{\nu}}{N_R} \right) \right]$$

$$= 0.0055 \left[ 1 + \left( 20,000 \times 0.00082 + \frac{10^6}{16,500} \right)^{\frac{1}{2}} \right]$$

$$= 0.026.$$

Then

$$h_f = 0.026 \times \frac{428}{1.22} \times h_v$$

$$= 9.1 h_v.$$

Total pressure $h_d = 10.1 h_v$.

A well known formula for $h_v$ is:

$$h_v = \left( \frac{V}{4000} \right)^2 \text{ in. of water}$$

($V$ in ft/min)
Then
\[ h_d = 10.1 \left( \frac{111}{4000} \right)^2 = 7.75 \times 10^{-3} \text{ in.} \]

This must be provided by the stack height.

Average outside air temperature during winter = 13.4°F
Average temperature inside stack = outlet temp = 25.8°F.

\[ H = \frac{5.2 h_d}{(1 - \frac{T_0}{T_c})} \quad \text{(a well known equation in heat power engineering)} \]

\[ h_d = 7.75 \times 10^{-3} \text{ in water} \]

\[ \bar{\rho} = 0.0828 \text{ lb/ft}^3 \text{ (from standard data)} \]

\[ T_0 = 460 + 13.4 = 473.4R \text{ (abs. F deg.)} \]

\[ T_c = 460 + 25.8 = 485.8R \]

\[ H = 16.3 \text{ ft and a 20-ft stack is indicated at 80\% efficiency.} \]

If 6 in. of insulation were used, \( H \approx \left[ (4 + 1)^2/(6 + 1)^2 \right] \times 20 \approx 10 \text{ ft}; \) i.e., the stack height is halved by increasing the thickness of insulation one half. It might be worthwhile to use more insulation and reduce the stack height, but 20 ft would probably suit the building quite well.

Completion of design: The ideal is to maintain a constant airspeed throughout the system. Areas of end chambers, inlets, and stacks are so arranged. Each duct is 1.58 ft\(^2\) in area; stacks about 3 ft 6 in. in diameter (area 9.6 ft\(^2\)) are used, giving 6 ducts per stack. End chambers (sometimes called plenum chambers, or simply plenums) are proportioned similarly.

The inlets and outlets should be cowled to take advantage of wind velocity head; measurements show that cowlings are directly advantageous. Since the design ignores wind, the cowlings provide a built-in safety factor.

At the inlet side of the building, there are much lower air temperatures in the ducts; therefore, there is less thaw in the pad and a lower pad surface temperature than at the outlet side of the building on which most of the design is based. Schemes for cooling each side of the building equally have been proposed but never used. These include duct insulation over part-lengths, airflow in opposite directions in adjacent ducts, and varying depths and branching of ducts.

The warming of outlet stacks by putting them inside the building (or by other means) has been suggested but not yet used. Another scheme discussed is to put the outlet stacks along the middle line of the building. This scheme looks promising if the function of the building permits it, and CRREL has experimented along these lines.

Mechanical ventilation: In a new design it is unlikely that fans would be used, but they could be valuable in an existing structure over degrading permafrost consisting of a fine-grained
soil with segregated ice. Since very little pressure is required, commercial blowers of low pressure and high discharge are needed. In this example the average airspeed is 111 ft/min.

\[
\text{area of duct: } 1.58 \text{ ft}^2 \\
\text{mean air temperature: } 20^\circ F \\
\text{Then air required } = 111 \times 1.58 = 175 \text{ CMF at } 20^\circ F \\
= 180 \text{ CMF at } 32^\circ F \text{ per duct.}
\]

Observations show that the maximum airspeed (during very cold weather) is about 3 times the mean, and the fan should be able to handle about 1000 CMF per duct to leave a safe margin.

Several ducts would be supplied by each blower; i.e., a pressure difference would be maintained between plenum chambers by several blowers. Spare fans and parts must be provided to allow for breakdown and maintenance.

**Artificial refrigeration by pipe grids (App B)**

The design of a surface refrigeration system follows normal procedures when the thermodynamic requirements are known. Each case must be studied individually, taking into account many local factors. In a simple building foundation, the heat flow computations are simpler for artificial refrigeration than for ventilation because floor temperature and ground temperature remain about constant, ensuring a steady state.

Reference 15 outlines a design procedure and describes an unusual refrigerated foundation. Reference 18 describes an interesting job where pits under a Nike structure had to penetrate pure ice, and no thaw whatever could be tolerated.

Control by thermocouples or other tell-tales is imperative in every foundation where permafrost degradation cannot be permitted.

Where artificial refrigeration is used, the operators usually over-refrigerate "to make sure of it." This practice can be very costly and should be checked upon; the designer's recommendations should be followed, unless temperature observations indicate that a change is necessary. Generous factors of safety must be used in design; there are a few examples where refrigeration plants have had to be increased, or the systems modified, to provide an unexpected refrigeration load. In an important job (e.g., the BMEWS foundations) site tests are made on a pilot scale before a full system is installed.5

**Deep-duct cooling system**

In the first under-floor cooling systems, deeply buried CMP's, 1 ft in diameter, were used. They worked well, thermally, but 1 ft is too small a size and the maintenance is very costly in money and man-hours. The pan-duct system is now used almost universally for heavily loaded floors such as those in warehouses and hangars.4

If buried ducts are used, the computation is somewhat laborious but resembles that for under-floor ducts. The ducts should be big enough to enable a man to move through them for thorough inspection and easy cleaning. They must be efficiently protected from ground water and airborne dust, but even then they tend to collect ice and soil in a hard, restrictive coating so that they must be inspected frequently.

Figure 18 shows the author's design for a large vehicle-maintenance building in northern Greenland, designed in the same way as in the example.
Data. Test pits showed moraine soils having ice up to 65% by weight of dry soil so that thaw of the natural subgrade would lead to unacceptable, certainly nonuniform, subsidence. The normal pattern is a bowl because of the shape of the isothermal surfaces (Fig. 1).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean annual temperature</td>
<td>9°F</td>
</tr>
<tr>
<td>Thawing season</td>
<td>76 days, mean</td>
</tr>
<tr>
<td>Air thawing index</td>
<td>450 mean, 850 max</td>
</tr>
<tr>
<td>Freezing season</td>
<td>289 days, mean</td>
</tr>
<tr>
<td>Air freezing index</td>
<td>8800 mean, 9500 max</td>
</tr>
<tr>
<td>Floor</td>
<td>60 to 65°F all year</td>
</tr>
</tbody>
</table>

Considerations of cost and availability of gravel eliminated artificial insulation under the floor.

The sand-leveling course also serves as a filter, should the natural ground thaw. The design, however, showed that the thaw would probably not reach the sand layer each year before freezeback began.

In view of the planned short life of the building, the engineer decided against the very costly duct system in favor of an extra 4 ft of fill. The author's computations indicated a thaw penetration (at the middle of the building) to natural ground in 9 months and penetration below the original permafrost table in 15 months. Thereafter, thaw would progress at about 1 ft every 6 months and since thaw consolidation would occur an estimated settlement of about 6 in. a year was to be expected. The settlement would be uneven but in the form of a bowl which in time could affect the footings at the edge of the building and lead to serious damage as well as severe floor deformation, growing into feet of differential subsidence.

Thermocouple readings taken 10 months after the heated building was in use agreed well with the 9-month estimate. In 3½ yr, the floor had settled to a maximum of about 1½ ft in the middle. In 4 yr, the eaves were deformed, indicating differential settlement of the footings. For some time jacking and blocking-up had been necessary but at this time major remedial action had become essential. The added 4 ft of fill could only delay the inevitable, not prevent it.

Maintenance of cooling systems

All cooling systems must be carefully inspected and maintained operational before and during each winter. Thermocouples in the pad are essential to show operators how the thaw and freezeback progress; if the depth of thaw penetration continues to increase in successive years with about the same weather conditions, something is “going wrong.” If an examination shows that nothing is blocked or damaged, fans could be used to increase the air flow but a simpler remedy is to heighten or heat the outlet stacks. As a last resort, a refrigeration plant can be installed to ensure a safe (or even zero) thaw penetration of the pad. Other remedial treatment is to add insulation to the floor; this is almost impracticable with ducts but not so difficult in open airspaces that have proved inadequate, since the sheets of rigid insulation can be fastened underneath, or blanket insulation, held by plywood, is possible.

Monitoring of ground temperatures by thermocouples. CRREL practice is to make up strings of copper-constantan thermocouples in ¼ to 1-in. plastic tubing which is then filled with the ACFEL oil-wax mixture developed for non-heaving bench marks. Sensors are located at 2-ft intervals and readings are taken monthly with a portable potentiometer. The thermocouple strings are put in vertical or slanted drilled holes, 2 to 3 in. in diameter, usually between 20 and 40 ft deep, and backfilled with wet sand to ensure good thermal contact.

*Personal communication by E.F. Lobacz, CRREL, following his visit to the site.
The reference junction consists of an ice-water mixture in a vacuum bottle. Distilled water mixed with ice made from it is advisable for accurate measurements.

The spacing and location of thermocouple strings depend upon the local conditions.

For obtaining reliable data, careful attention must be paid to the cold reference and switching, known sources of error in practice. In a major installation the readout would be an electronic recorder but periodic checks with hand equipment also would be advisable.

THAW CONSOLIDATION AND SETTLEMENT

Compressibility of frozen soils

Frozen soils at low temperatures and low stresses well below the limiting creep stress have small elastic deformations, and formulas for bases on an elastic semi-infinite body are applicable the problems then are the values of Young’s modulus $E$, and Poisson’s ratio $\nu$, which enter into such formulas. $E$ rises with a fall in temperature ($E = a + b\theta$, where $a$ and $b$ depend on the soil, $\theta$ is degrees of frost, and rate of loading is constant); $\nu$ does not vary much. Published data based on laboratory tests at uncontrolled rates of loading and various sizes of specimens are of doubtful value but must sometimes be used. The recommended procedure is to make tests (preferably in the field) for any important job. Elastic constants determined by dynamic tests (seismic, etc.) are not valid for static conditions. Viscoelastic theory may have application. Fortunately, the compressibility of frozen soil is very rarely important.

Little has been done on the consolidation parameters but reference 49 tabulates some values for $m_p$, the coefficient (or modulus) of volume change, varying from $0.0002$ (fine sand at $-2^\circ C$) to $0.0231$ (silty sand at $-0.1^\circ C$) in square feet per ton. This parameter is very rarely needed in engineering practice; if needed, it must be found by soil tests made under controlled conditions in a laboratory.

Consolidation occurs by the flow of water (derived from thawing ice) as in the consolidation of unfrozen soils. Since the permeability of frozen soils is usually very small indeed, the rate of consolidation is also very small unless the temperature of the frozen soil approaches its melting point. The amount of unfrozen water in frozen soil depends upon pressure as well as temperature, so the process is very complex indeed, but unfrozen water is probably of minor importance in most permafrost.

Thaw settlement

In North American practice, thaw is not permitted under a foundation on permafrost unless a thorough study has convinced the designer that permafrost degradation will not result in detrimental settlement. A few major structures of this kind have been built in Alaska. Notable is the large hospital building for which soils and temperature data are given in Figure 39. As can be seen, the permafrost degradation beneath the building progresses at a steady rate, but the settlement is negligible. Exhaustive field and laboratory tests had proved that thawing and consolidation procedures had no measurable effect on soil dry densities so the permafrost was safely ignored. (At the thermal powerplant nearby a very elaborate treatment by steam thawing and shock by buried explosives was adopted with indeterminate results.)

Considerable damage has resulted from unplanned-for thaw consolidation of frost-unstable permafrost, and remedial action, if it is possible at all, is always very costly.

*See CRSE Monograph II-D1, Freezing process and mechanics of frozen ground, R.F. Scott.*
No satisfactory theory yet exists for predicting the rate of settlement due to thaw. The rate of thaw and the rate of consolidation are usually unequal and drainage conditions are difficult or impossible to predict. A good estimate of total settlement may often be made from the initial ice content and assumed final water content, based on void ratios and computations of ultimate depth of thaw (not an easy task, however, and one that should be left to specialists); the better way is the consolidometer test. Figure 40 shows a laboratory report on such a test made on ML soil, which is common in western Alaska. Test results show what total settlement could occur if thawing were permitted. This may save the construction of a costly foundation if tests prove negligible thaw consolidation. However, the consolidometer test gives no information on rates of settlement.

Research into the effects of thawing permafrost under a heated building is most active in the Soviet Union in an attempt to lower the cost of foundations by eliminating the provisions for underfloor cooling. Reference 51 (45 p.) is a very good study of the problem, both experimentally and theoretically.

It is very risky to erect a structure knowing it will deform and allowing for a permissible amount and rate of deformation as the building settles at a predetermined rate. Tables have been published* but the author doubts that they are used very often, if ever.

Thaw bowl under a heated building

Advanced theoretical studies have been made (e.g., ref. 25) to estimate the dimensions of the bowl of thawed ground caused by a heated building over permafrost, but the results do not generally justify the labor involved in what can only be an approximation. This simple procedure is proposed:

1. Compute the thaw penetration to be expected under an infinite area by the Modified Berggren equation* or other technique.

*Refer also to Appendix A.
2. Apply the curves and other parameters from Figure 41 (based on field observations) to find the approximate dimensions of the thaw bowl.

Straight-line extrapolation (up to a maximum of $k_c = 1.0$) is reasonable for buildings exceeding 100 ft in width: thus the value of $k_c$ for a 200-ft-square building would be about $0.78 + (0.78 - 0.65) = 0.91$. 

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**Figure 40.** Thaw-consolidation test report. (Courtesy of Mr. Erwin L. Long of U.S. Army Engineer District, Alaska.)
Reference 28 describes an ingenious new type of foundation invented by Erwin L. Long of U.S. Army Engineer District, Alaska, for special conditions where the conventional types of foundation are likely to be inadequate.

This device, called the Long Thermo-Pile, is patented; it comprises a closed steel tubular pile containing a quantity of refrigerant sufficient and suitable for use with the air, and ground, temperatures at the site. The top of the tube projects above the ground and is thus exposed to air temperatures. During winter, heat flow is upward from the soil surrounding the embedded part of the pile,
thus lowering the permafrost temperature several degrees below the normal value. During summer, the warmer air reverses the direction of heat flow and the soil temperature rises a little (to 2 or 3°F below the normal permafrost temperature). The next winter the cycle continues automatically. Propane is used in the Long Thermo-Pile; experimental piles in the USSR have been charged with kerosine. However other refrigerants may be as good as these or superior to them. Thus, artificial refrigeration is achieved without special plant or any moving parts.

The projecting part is finned to increase the cooling surface area and painted white to reduce radiative heat intake.

In a typical job, three 12-in.-diam Long Thermo-Valves were used to support each footing for the legs of a 300-ft steel tower. The conditions were difficult:

- Deep active zone
- Soils of clay, silt, sand, gravel and boulders
- Ice lenses up to 2 ft thick
- Natural cover to be stripped
- Ground temperatures not below 28°F, and 30 to 31°F at 6- to 22-ft depth.

Conventional foundations would have been pipe columns on deep footings, or 30- to 45-ft piles with artificial refrigeration for freezeback. The special foundation was accomplished with only 13 ft of embedment and 6 ft of projection in air, the latter assisting as column length above ground. The Thermo-Valves were slurred in augered holes and frozen in by a controlled escape of refrigerant which was then replenished. This is a quite good method of artificial freezeback of a pile but could cost more than conventional refrigeration on a large project.

Every September the foundation is inspected to ensure that the refrigerant is at the correct operating level as a safeguard against the possibility of a very slow gas leak undetected by the rigorous tests at the time of installation: winter cooling is then assured by the addition of refrigerant if necessary, through special valves provided.*

In the Long Thermo-Pile, heat transfer is by vaporization; in the Russian pile, by liquid convection.

ACFEL has proposed that tanks of water (ice) in a ventilated basement in permafrost would be useful as a heat sink in summer, frozen back in winter, but this idea has not been developed. The encapsulation of soil of high water content in a plastic membrane is another possible variant that so far has not been attempted.

**WALLS AND RETAINING STRUCTURES**

Walls and retaining structures are very rare indeed in subarctic and arctic regions in connection with structures because excavation is avoided as much as possible and when used is nearly always backfilled. In a very severe environment in frost active soils it is generally better to avoid a retaining wall by using a flat slope (at least 3 to 1) blanketed with about 2 ft of non-frost-susceptible soil, and a foot of fine soil and vegetation, or it may be feasible to allow the surface to slough and be cleaned up in the spring. Each case must be carefully studied and, if important, dealt with by specialists. Slope protection should be taken about 6 ft beyond the toe of the slope and ponding of water should not be permitted. If a retaining wall is essential, adequate thickness of non-frost-susceptible backfill must be used to protect against lateral heave.57

---

*This principle is sound but detailed observations at another site have revealed operating difficulties, mainly control of gas leaks, so frequent monitoring of installed thermopiles is vital. Research and study are still (1968) in progress.
For average conditions the Surface Freezing Index ($F_s$) may be taken as the Air Freezing Index ($F_A$) for design.

Example

$F_s = 1900$  Wall thickness: 4 ft.
Backfill: 130 lb/ft$^3$, 2.2%
Thickness required is 4.7 ft., say 4 ft. 6 in.
Add or deduct 6 in. for unusual conditions.

**PAVEMENTS & GROUND**
Compute $F_s$ from known surface conditions (n) and $F_A$:

$$F_s = nF_A$$

<table>
<thead>
<tr>
<th>Surface</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement</td>
<td>0.9</td>
</tr>
<tr>
<td>Ground</td>
<td>0.7</td>
</tr>
<tr>
<td>Bare</td>
<td></td>
</tr>
<tr>
<td>Turf</td>
<td></td>
</tr>
<tr>
<td>Turf plus 1 ft. avg. snow cover</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Frost may strike through a wall as well as into the ground: the effect on a frost-susceptible backfill is to force the wall outward with the risk of fracture or instability, apart from creating the unsightliness of a retaining wall leaning the wrong way.

Figure 42 shows what thickness of an available non-frost-susceptible backfill material would be required to prevent frost penetration into the frost-susceptible soil retained by the wall. It is

**Figure 42. Frost penetration through concrete into granular soils and till.**
based on the assumption of a sinusoidal temperature at the face of the wall to find the freezing index at its back; the Modified Berggren equation (App. A) is then applied to the backfill frost-penetration problem. The same curves may be used for penetration of frost or thawing through a concrete floor slab or thick concrete pavement.

For centuries it has been customary to put a frost batter on the back of a masonry retaining wall to reduce heave forces. This practice is of questionable value but 2 to 3 in./ft is the batter commonly used; it is more important to ensure smoothness at the joints. The correct procedure, of course, is to use drained non-frost-susceptible backfill.

DRAINAGE AROUND BUILDINGS

Considerable damage has arisen from apparently insignificant amounts of water coming into contact with the permafrost under and near buildings. Because of its high specific heat and mobility, surface water must be effectively drained away from buildings and waste water must never be discharged on the ground nearby. Snow should not be piled near a building because of the meltwater and airspaces must be kept clear of snow for the same reason as well as not to impede air circulation.

The surface of a gravel pad slopes gently to the top of the slope to shed water and as much of the pad as possible is protected by vegetation to minimize percolation.

Ditches may be troublesome but are sometimes unavoidable. Over permafrost they should be wide and shallow and lined with fine-grained soils to retain as much water as possible. Long ditches are inadvisable because they disturb the permafrost regime and are difficult to maintain and stable slopes are rarely achievable. French drains are not recommended; they quickly become clogged with ice and soil.

Special care must be taken in laying out ditches where ice wedges are known to exist, owing to the risk of thawing of the wedges under a shallow soil cover reduced by the depth of the ditch. Dangerous sink-holes that allow drainage water to percolate under structures where permafrost thaw is most undesirable may be formed.

Ditches should be not less than 30 ft from the toe of any fill. A reasonable slope is 0.003. A good profile is 2-ft width at bottom, side slopes 1 on 2, depth about 2 ft and top width of about 6 ft. To allow for spring flows, the ditch proper may be widened the required amount, again with a depth of 2 ft, and a 10% cross-slope on the widened bottom. Vegetation is a valuable cover but needs good maintenance. Ditches for roads are sometimes cut with a bulldozer but this technique is not the best around buildings.

DEPTH OF FOOTING IN AREAS OF SEASONAL FROST

The best guide for the depth of a footing is well-proven local practice but sometimes a site may be remote from other buildings and an estimate, based on local weather and ground conditions, is required.
These footing depths are used in New England by the Corps of Engineers.

<table>
<thead>
<tr>
<th>Location</th>
<th>Footing depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hartford, Conn.</td>
<td>3½</td>
</tr>
<tr>
<td>Otis AFB (Cape Cod), Mass.</td>
<td>3½</td>
</tr>
<tr>
<td>Providence, R.I.</td>
<td>3 to 4</td>
</tr>
<tr>
<td>Boston, Mass.</td>
<td>4</td>
</tr>
<tr>
<td>Portsmouth, N.H.</td>
<td>4</td>
</tr>
<tr>
<td>Westover, Mass.</td>
<td>4</td>
</tr>
<tr>
<td>Burlington, Vt.</td>
<td>5</td>
</tr>
<tr>
<td>Bangor, Maine</td>
<td>5½</td>
</tr>
<tr>
<td>Caribou, Maine</td>
<td>6½</td>
</tr>
</tbody>
</table>

**Figure 43.** Freezing index, surface conditions and penetration of freezing temperature, uniform soil.⁴³
If an unheated structure is to be built on frost-susceptible soils, it would be worthwhile to place the footings on a drained non-frost-susceptible compacted fill and to backfill with NFS material.

Serious damage may be caused by leaving unfinished throughout a winter a structure that is to be heated during operation; frost may penetrate below the footings, resulting in heave, which is nearly always differential. For example, such heaves of several inches have been observed with tills. If the structure cannot be heated during the winter, the following materials may be used for frost protection: hay, straw, sawdust or other insulating material (not gravel or other soil, unless of adequate depth as determined from local experience or computation). The flooding of a site has also been used to advantage.

Similarly, damage has been caused by using a building on permafrost for a heated storage when it had been designed for an unheated storage. Foundation conditions must be studied before such action.
CONCLUSION

Enough is now known for efficient design and construction of foundations in cold regions; yet failures with permafrost are not uncommon, probably because designers experienced in temperate regions do not appreciate the hazards of building on frozen soil, and disregard thermal effects. Users of buildings on permafrost must learn how to take care of them. In areas of seasonal frost, the worst foundation problems arise from leaving a partly finished structure unheated, or not frost-protected, on frost-heaving soil throughout the winter (gravel is a poor insulator).
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17. Deleted.


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48. __________ (1959) Personal communication.


56. __________ (in prep.) Freezeback control and pile testing, Kotzebue Air Force Station, Alaska. USA CRREL Technical Report 141.
LITERATURE CITED (Cont’d)


58. U.S. Army Technical Manuals on arctic and subarctic construction. TM 5-852-1 to 8 (various dates).
   1. General provisions
   2. Site selection and development
   3. Runway and road design
   4. Building foundations
   5. Utilities
   6. Calculation methods for determination of depths of freeze and thaw in soils
   7. Surface drainage design for airfields and heliports
   8. Terrain evaluation.


APPENDIX A: COMPUTATION OF DEPTH OF FREEZING OR THAWING*

Suppose the surface of a solid is at a temperature $T$ for $t$ days. The number of degree-days is measured by

$$\int_{0}^{t} (T - T_0) \, dt$$

where $T_0$ is a constant reference temperature. For computations in soil freezing and thawing, $T_0$ is usually 32°F, while $T$ is often constant.

With fluctuating daily temperatures, the mean of daily maximum and minimum temperature is used for the daily average (this may mean a considerable error over a short period and in some locations having peculiar weather patterns, but it is normally acceptable). The total degree-days is usually found by adding daily increments and plotting a mass curve of cumulative degree-days, but data precise enough for many computations may be found by plotting monthly averages and finding the area between $T_0$ and the temperature-time curve.

Freezing and thawing indexes

The total (negative) degree-days for a freezing period is called a freezing index. Similarly, the total (positive) degree-days for a thawing period is a thawing index; here the reference temperature ($T_0$) is almost always 32°F. For a varying temperature pattern the indexes are conveniently taken off the mass diagram. Absolute values of the indexes are commonly used in calculations. An "air index" is for a point 5 or 6 ft above the ground, while a "surface index" refers to the ground surface. The conversion from air- to surface-index is a continuing research topic. See reference 58(6) for further information.

Depth of penetration of the $T_0$ isotherm in ground containing water

A useful expression in ascertaining depth of penetration of the $T_0$ isotherm in material containing water is the Modified Berggren equation. This is based on a temperature step change at the surface of a semi-infinite slab, initially at constant temperature $v_0$ with respect to $T_0$, the phase-change temperature. If $v_s$ is the surface temperature $(T - T_0)$ suddenly applied and retained for a time $t$, $X$ the depth of the $T_0$ isotherm, $K$ the coefficient of thermal conductivity, and $L$ the volumetric latent heat of the medium, then

$$X = \lambda \sqrt{\frac{2Kv_s t}{L}}$$

where $\lambda$ is a dimensionless coefficient that is a complex function of $v_0$, $v_s$, $L$, and volumetric specific heat $C$. Analysis of ACFEL field data showed that in natural conditions, $v_s t$ is well represented by the freezing or thawing index $I$, and that the step-change theory can be applied with satisfactory precision.

*Excerpted from reference 43.
If \( y_d \) is dry unit weight of soil; \( w \) its water content, \% dry weight; \( f \) its ice content, \% dry weight; \( c_s \) the specific heat of dry soil grains (0.17 near 32°F); \( L_s \) the latent heat of fusion of water-ice (144 Btu/lb); 1.0 the specific heat of water; 0.5 the specific heat of ice; suffix \( u \) unfrozen condition; suffix \( f \) frozen condition—then

\[
C_u = y_d c_s + \frac{w}{100} \tag{2}
\]
\[
C_f = y_d c_s + \frac{0.5i}{100} + \frac{w - i}{100} \tag{3}
\]
\[
L = \frac{y_d}{100} L_s. \tag{4}
\]

For most computations all the water can be assumed to freeze at 32°F since tests, designs and analyses of field data have been based on this assumption. Not enough is yet known to allow with confidence for the phenomena of unfrozen water in soil at a temperature below the normal equilibrium value.

Then

\[
C_f = y_d c_s + \frac{0.5w}{100} \tag{5}
\]
\[
L = \frac{y_d}{100} L_s. \tag{6}
\]

\( K \) values are normally derived from Kersten’s curves (Fig. A1 to A4)\(^6\) although sometimes it is preferable to test the particular material.

Converting the parameters to consistent English units for soils,

\[
X = \lambda \sqrt{\frac{48KI}{L}}
\]

where \( X \) is in ft; \( K \) in Btu/ft hr °F \((K = \frac{(K_u + K_f)}{2})\) is adequate for most purposes; \( I \) in Fahrenheit degree-days; \( L \) in Btu/ft\(^2\); \( \lambda \) dimensionless, usually between 0.5 and 1.0, depending upon water content of the soil and local conditions.

**Coefficient of thermal conductivity \((K)\)**

Soils are conveniently divided into three groups for computing \( K \): coarse grained (high in quartz), fine grained (low in quartz and high in other minerals), and highly organic. \( K \) depends also on dry unit weight, water content, and ice content. The curves (Fig. A1 to A4) used are based on laboratory tests of soils and are labeled “unfrozen” and “frozen.” The amount of unfrozen water in frozen soil is immaterial if test temperatures approximate those in the problem (at much lower temperatures, only results of laboratory or field tests at proper temperatures are valid).
APPENDIX A

Figure A1. Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils—frozen.

Figure A2. Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils—frozen.

NOTE: Thermal Conductivity K is expressed in Btu per hour per ft$^2$ per unit thermal gradient in deg F per ft. (Btu/ft$^2$ hr F).
Figure A3. Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils—unfrozen.  

Figure A4. Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils—unfrozen.
Figure A5. Thermal ratio, fusion parameter and lambda coefficient.

The coefficient $\lambda$ is used in computations dealing with the 32F isotherm in ground. Curves of $\lambda$, extending slightly from the original of reference 60, are given for completeness (Fig. A5). Note that

$$ a = \frac{v_0}{v_s} \quad \text{and} \quad \mu = v_s \left( \frac{C}{L} \right). $$

Layered systems

It is easy to show that the surface degree-days required to move the 32F isotherm through a given layer $n$ is given by:

$$ l_n = \frac{L_n d_n}{24\lambda^2} \left[ \sum R + \frac{R_n}{2} \right] $$

where $\lambda$ is the weighted average for the layers down to, and including, layer $n$;

$\sum R$ is the sum of the thermal resistances of the layers above layer $n$; and

$R_n$ is the thermal resistance ($= d_n/K_n$) of layer.

Example: Boring log from a taxiway at Thule Air Base is given in Table AI. The mean annual temperature is 12F, surface thawing index $l$ is 1560 degree-days, and length of thawing season is 105 days. The problem is to estimate the depth of thaw expected. The solution is presented in Table AII.
**APPENDIX A**

Table AI. Field data for example of thaw problem.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Material</th>
<th>Ice* condition</th>
<th>Dry unit wt (lb/ft³)</th>
<th>Water content (%)</th>
<th>Layers for computation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Asphaltic Concrete (AC)</td>
<td></td>
<td></td>
<td></td>
<td>a</td>
</tr>
<tr>
<td>0.4</td>
<td>GW-GP</td>
<td>Nb</td>
<td>155</td>
<td>2.4</td>
<td>b</td>
</tr>
<tr>
<td>1</td>
<td>GP</td>
<td>Nf</td>
<td>157</td>
<td>1.8</td>
<td>b</td>
</tr>
<tr>
<td>3</td>
<td>GW</td>
<td>Nb</td>
<td>151</td>
<td>3.2</td>
<td>c</td>
</tr>
<tr>
<td>4</td>
<td>GW</td>
<td>Vx-Vc</td>
<td>151</td>
<td>3.2</td>
<td>c</td>
</tr>
<tr>
<td>5</td>
<td>GP</td>
<td>Vx</td>
<td>152</td>
<td>2.1</td>
<td>d</td>
</tr>
<tr>
<td>6</td>
<td>SM</td>
<td>Nb</td>
<td>136</td>
<td>6.5</td>
<td>d</td>
</tr>
<tr>
<td>7</td>
<td>SM</td>
<td>Vx</td>
<td>144</td>
<td>4.6</td>
<td>e</td>
</tr>
<tr>
<td>8</td>
<td>SM-GM</td>
<td>Vx</td>
<td>143</td>
<td>4.6</td>
<td>e</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td>140</td>
<td>2.8</td>
<td></td>
</tr>
</tbody>
</table>

*CRREL classification system.

Table AI. Solution of a problem of a layered system (asphaltic concrete pavement over base course and subgrade).

<table>
<thead>
<tr>
<th>No.</th>
<th>Material</th>
<th>d</th>
<th>yd</th>
<th>w</th>
<th>K</th>
<th>C</th>
<th>L</th>
<th>Cd</th>
<th>Sd</th>
<th>Ld</th>
<th>μ</th>
<th>λ</th>
<th>R</th>
<th>ΣR</th>
<th>ΣR+μR</th>
<th>I</th>
<th>ΣI</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>AC</td>
<td>0.4</td>
<td>0.4</td>
<td>138</td>
<td>0.86</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.46</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b</td>
<td>GW-GP</td>
<td>1.6</td>
<td>2.0</td>
<td>156</td>
<td>2.1</td>
<td>2.1</td>
<td>30</td>
<td>470</td>
<td>48</td>
<td>59</td>
<td>20</td>
<td>751</td>
<td>751</td>
<td>376</td>
<td>1.15</td>
<td>0.46</td>
<td>0.21</td>
</tr>
<tr>
<td>c</td>
<td>GW-GP</td>
<td>3.0</td>
<td>5.0</td>
<td>151</td>
<td>2.6</td>
<td>2.0</td>
<td>30</td>
<td>610</td>
<td>90</td>
<td>149</td>
<td>30</td>
<td>1830</td>
<td>1581</td>
<td>517</td>
<td>0.86</td>
<td>0.51</td>
<td>0.28</td>
</tr>
<tr>
<td>d</td>
<td>SM</td>
<td>1.0</td>
<td>6.0</td>
<td>136</td>
<td>6.6</td>
<td>1.9</td>
<td>30</td>
<td>1270</td>
<td>30</td>
<td>179</td>
<td>30</td>
<td>3851</td>
<td>3851</td>
<td>648</td>
<td>0.70</td>
<td>0.53</td>
<td>0.29</td>
</tr>
<tr>
<td>e1</td>
<td>SM-SC</td>
<td>1.0</td>
<td>7.0</td>
<td>144</td>
<td>4.6</td>
<td>2.0</td>
<td>29</td>
<td>955</td>
<td>29</td>
<td>268</td>
<td>30</td>
<td>955</td>
<td>955</td>
<td>686</td>
<td>0.65</td>
<td>0.55</td>
<td>0.20</td>
</tr>
<tr>
<td>e2</td>
<td>SM-SC</td>
<td>0.75</td>
<td>6.75</td>
<td>144</td>
<td>4.6</td>
<td>2.0</td>
<td>29</td>
<td>956</td>
<td>21</td>
<td>200</td>
<td>30</td>
<td>718</td>
<td>678</td>
<td>678</td>
<td>0.65</td>
<td>0.53</td>
<td>0.20</td>
</tr>
<tr>
<td>e3</td>
<td>SM-SC</td>
<td>0.70</td>
<td>6.70</td>
<td>144</td>
<td>4.6</td>
<td>2.0</td>
<td>29</td>
<td>956</td>
<td>21</td>
<td>200</td>
<td>30</td>
<td>718</td>
<td>678</td>
<td>678</td>
<td>0.65</td>
<td>0.53</td>
<td>0.20</td>
</tr>
</tbody>
</table>

\[ I_0 = 32 - 12 = 20 \]
\[ I_0 = 1580 \]
\[ v_o = 14.9 \]
\[ α = \frac{v_o}{v_s} = 1.34 \]
\[ t_e = \frac{L_st}{24R} \left( ΣR + \frac{R_s}{ε} \right) \]
\[ t_e = \frac{(955)(0.70)(3.43)}{(24)(0.30)} = 340 \]
\[ t_e = \frac{(965)(1.0)(3.49)}{(24)(0.30)} = 464 \]

Total thaw penetration 6.70 ft. (Observed value: 6.5 ft)

*No water in AC; hence negligible deg days required to thaw the pavement.

\[ I_e = \frac{(610)(3.0)(1.97)}{(24)(0.30)} = 579 \]
\[ I_e = \frac{(610)(3.0)(1.97)}{(24)(0.30)} = 579 \]
\[ I_e = \frac{(855)(1.0)(2.98)}{(24)(0.30)} = 544 \]
\[ I_e = \frac{(855)(1.0)(2.98)}{(24)(0.30)} = 544 \]

Successive trials
APPENDIX B: REFRIGERATION SYSTEMS COMPUTATIONS

Pile installation - backfill freezeback

**Coolant:** Sodium chloride, calcium chloride or liquid propane.

**Piping:** Black iron pipe, ¼ in. to 3 in. in diameter; rubber hoses for connections.

**Temperatures:** Brine from 10 to -45°F for freezing soil to 25°F.


**Example:**

a. The average volume of slurry backfill for a group of piles is 31 ft³/pile. It will be placed at an average of 40°F and must be frozen to 25°F. An installation of 225,000 Btu/hr (∽18½ tons of refrigeration) is available. How long will it take to freeze back a group of 80 piles? Take C = 30 and L = 3000 (App. A) for the slurry.

Heat to be abstracted from slurry

\[
= 80 \times 31 \times (40 - 32) \times 30 + 3000 + (32 - 25) \times 30
\]

\[
= 80 \times 31 \times 3450 = 8,560,000 \text{ Btu.}
\]

Time required = 8,560,000/225,000 = 38 hr, assuming good design.

Allowing for losses - say 42 to 48 hr (allow 2 days).

b. What mixture of sodium chloride brine is recommended?

| Temperature of frozen soil | 25°F |
| Brine temperature 10°F less | 10°F (a common criterion) |
| Highest brine temperature | 15°F |

Allowable temperature rise in brine - about 5°F (a common criterion)

Lowest brine temperature 10°F.

The freezing temperature should be 10°F below the lowest operating temperature and hence zero°F. It must be remembered that the air temperature might fall below zero during the job and this temperature would then govern. Assuming that 0°F is not reached by air, standard references give a 21% sodium chloride brine with a specific gravity of 1.159 (=20°F Baumé). For lower temperatures, a calcium chloride brine or propane (if available) might be better. (Magnesium chloride and lithium chloride have been used in artificial ground freezing but their cost is prohibitive for ordinary cooling.)

c. The temperature rise is to be about 4°F. Compute the rate of circulation in gal/min. (Work on 225,000 Btu/hr.)

From tables, specific heat of brine = 0.8 and 1 gal weighs 9.65 lb.

Circulation = 225,000/(60 × 0.8 × 4 × 9.65) = 121 gal/min.

d. Each pile has one coil of ¼-in.-diam pipe. Compute the rate of circulation in the coils. (The actual pipe diameter is 0.824 in.)

The total flow is divided among 80 cooling coils. Flow per coil = 1.51 gal/min (7.5 gal = 1ft³) and \( V = (1.51)/(7.5) \times (144)/(0.53) = 55 \text{ ft/min or about 1 ft/sec.} \) (This is somewhat high for best performance in artificial freezing and a larger pipe would probably be more efficient).
e. The cooling coil consists of four 30-ft lengths of pipe along the surface of the pile. What is the pressure drop (psi) per pile? (The circulating pump must provide this pressure as well as velocity head and frictional loss in the mains, bends and connections.)

At 12°F, kinematic viscosity = 1/25,000 (from the cited handbook).

\[ N_R, \text{Reynolds No. } = 0.92 \times 0.824/12 \times 2500 = 1600 \text{ (laminar flow)} \]

\[ (v = 0.92 \text{ ft/sec and } D = 0.824 \text{ in.}) \]

For laminar flow, \( f = \frac{64}{N_R} = 0.040 \)

\[ h_f = \frac{0.040 \times 4 \times 30 \times 12 \times 0.92^2}{2g \times 0.824} = 0.92 \text{ ft of brine} \]

\[ = 0.92 \times 1.16 = 1.07 \text{ ft of water} \]

\[ = 0.5 \text{ psi} \]

Adding 20% for bends and connections: Pressure drop about 0.6 psi per pile.

f. What size headers should be used if the speed of flow is to be not more than 5 ft/sec?

Assume flow in a header is 121 gal/min. (Because the cooling coils are being fed along its length, the actual flow will be less.)

Pipe diameter = \( 12 \left(\frac{4}{\pi}\right) \times (121)/(7.5 \times 60 \times 5) = 3.16 \text{ in.} \)

3-in. diameter would do well. (Actual speed 5.3 ft/sec).

g. The two headers have a total length of 1000 ft (including allowances for bends and fittings). Compute the pressure drop along them.

\[ N_R = 5.3 \times 0.256 \times 25,000 = 34,000 \text{ (turbulent flow)} \]

From ‘‘Guide,’’

\[ e = 0.00015 \]

\[ \frac{e}{D} = \frac{0.00015}{0.256} = 0.0006 \]

\[ f = 0.025 \]

Pressure loss = \( (0.025 \times 1000)/0.256 \times 5.3^2/2g = 42.6 \text{ ft of brine} \)

\[ = 50.0 \text{ ft water} = 22.0 \text{ psi} \]

This gives a total pressure drop in the piping of 23 psi or 54 ft of water.

h. What horsepower would be required for coolant circulation, allowing a 35-ft head loss in the brine cooler?

Total pressure drop = 54 + 35 = 89 ft

\[ 121 \text{ gal/min} = \frac{121}{449} = 0.27 \text{ ft}^3/\text{sec} = 16.8 \text{ lb/sec} \]
and

\[
power = \frac{16.8 \times 89}{550} = 2.72 \text{ say } 3 \text{ hp}
\]

\(1 \text{ hp} = 550 \text{ ft-lb/sec}\).

i. **Check on heat transfer in the cooling coils (coolant to pipe wall).**

The flow is laminar, for which:

\[
\frac{hD}{k} = 1.86 \left( \frac{VD^2cp}{kL} \right)^{1/3}
\]

where

- \(h\) = the film coefficient
- \(D\) = pipe diameter
- \(k\) = coefficient of heat conductivity of fluid
- \(c\) = specific heat of fluid
- \(\rho\) = density of fluid
- \(L\) = length of pipe

Here

\[
D = 0.824 \text{ in.} = 0.069 \text{ ft}
\]
\[
k = 0.25 \text{ Btu/ft hr F}
\]
\[
c = 0.8 \text{ Btu/lb F}
\]
\[
\rho = 72.4 \text{ lb/ft}^3
\]
\[
L = 120 \text{ ft}
\]
\[
V = 4000 \text{ ft/hr}
\]

and

\[
h = 0.25 \times 1.86 \left( \frac{4000 \times 0.069^2 \times 0.8 \times 72.4}{0.25 \times 120} \right)^{1/3}
\]

\[
= 22.4 \text{ Btu/ft}^2 \text{ hr F}
\]

area of pipe per foot length = 0.370 ft².

Therefore, heat transfer per pile = 120 \times 0.370 \times 24 = 1065 \text{ Btu/hr F}.

Heat abstracted per pile = 107,000 \text{ Btu in 48 hr}

\[\therefore \text{ rate is } 2230 \text{ Btu/hr}\]

and temperature drop = \(2230/1065 = 2.1\text{F (average per pile)}\).

(The design is based on a 4F drop for the system and is safe.)
j. Check on heat transfer outside the cooling coils (pipe to slurry).

Surface area of cooling pipes per pile (outside diameter = 1.05 in.)

\[ A = 120 \times \pi \times \frac{1.05}{12} = 33 \text{ ft}^2 \]

Average rate of heat flow = 2230 Btu/hr

\[ \text{Rate per ft}^2 = \frac{2230}{33} = 68 \text{ Btu/hr}. \]

Temperature difference most of time = 32 - 12 = 20°F (but varies from 30 to 11) and heat flow required per ft² per ft per degree temperature difference is about 68/20 \(\approx\) 3.4 Btu (which is well below an acceptable value for such a small pipe).

Comment on design.

The computations have been checked on a system found to be efficient; in another successful system, 1-in.-diam cooling pipes were used with a more satisfactory velocity of circulation. The number of piles per group is somewhat large and careful placing of locations of thermocouple strings is very important to ensure that all the piles are properly frozen back. The flow per pile must be controllable by valves owing to the temperature changes in the brine and variable pressure drops through the cooling coils.

The pile installation should be made in cold weather to minimize heat pickup and, in view of assumed parameters, site tests on individual piles are essential before the complete system is set up.

A small modern refrigeration plant of 1-ton capacity would accommodate about 4 piles at once, and the lower coolant temperature of -40°F would freeze back much more rapidly. Time is proportional to degrees of frost of coolant; so the time would be about 32/72 \(\times\) 38 hr = 17 hr, say 1 day. This is a common specification - to freeze back the pile in 24 hr.

Underfloor cooling. The design of a refrigeration system is still largely empirical because so few systems have been installed and field operation data are very scanty. The cost of installation is not excessive but the continual power requirement and maintenance are better avoided if possible. Automatic controls still require human supervision and a duplicate set is essential owing to the gravity of mechanical failure. Some design data follow:

**Coolant:** Ethylene glycol solution (50%), or propylene glycol solution, is customary. (These are poisonous liquids.)

**Piping:** Solid-drawn hard copper tube is usual owing to a mild corrosive action of the glycols. Mains are from 1 to 2 in. in diameter and grids \(\frac{5}{8}\) to 1\(\frac{1}{8}\) in. in diameter. A typical modern installation uses \(\frac{5}{8}\) in. for grids and 1 in. for supply and return. Length of pipe per grid is from 80 to 100 ft.

**Spacing:** Cooling grid pipes are from 1 to 2 ft apart.

**Temperatures:** A soil temperature of 29°F or 30°F is maintained by the cooling grid system and the coolant is kept about 10°F below that, with a rise in temperature of about 5°F between inlet and return.

**Pressures in system:** A differential should be not more than 5 lb/in.² in design.
Fluid velocity: Low speeds at turbulent flow should be about 3 ft/sec maximum.

Heat transfer: Limited tests show that the contact between soil and pipe is very good and that there is little heat loss at the pipe surface. For soil a value of 2 (large diameter pipe) to 5 (small diameter pipe) Btu/ft² hr °F is reasonable and for still air 2½ Btu/ft² hr °F. For air moving over the pipe, consult standard references on heat transfer. About 4 or 5 Btu/ft² hr °F is possible. The heat transfer per unit of surface area is much more for small diameter pipes than for large diameter pipes, but the smaller diameter means higher speeds, larger pressure drops and more circulating pump horsepower so that a balance is necessary. ¼-in. tubing for cooling grids is almost standard practice.
APPENDIX C: RECOMMENDATIONS FOR TESTING PILES UNDER STATIC LOADS (USSR Guide, 1964)*

For information only: not used in North America

1. The necessity of carrying out static tests on experimental piles, their number and design, is established by the planning organization in relation to the available experience, purpose of construction and working conditions.

The tests are carried out by the planning organization with the participation of the permafrost station or research organization where necessary.

The cost of static tests is included by the planning organization in the construction estimates.

2. The equipment for testing the piles is specified in the plan. The recommended installation for testing piles in axial compressive loading is given in Figure C1. Figure C2 shows the reference installation.

3. The ultimate strength of foundation soils of the pile is determined when the ground temperature reaches its maximum at the time of greatest thaw of the active layer in the autumn.

4. The tests are carried out on the construction site after the engineering preparations have been completed.

5. To eliminate the effect of the active layer, the pile may be separated from the ground during tests within the design thickness of the active layer and the resultant space filled with material of low thermal conductivity.

6. The test pile must be provided with a tube for possible temperature measurements along the pile and at its lower end. The location of additional temperature holes at the testing installation is indicated on the plan.

7. The pile tests are allowed to begin when it is established that the thermal regime in the foundation soils is close to that specified.

8. Loading of the experimental pile is carried out in stages. The first two stages of loading are assumed equal to 0.5 \( P \), where \( P \) is the design strength of foundation soils determined according to section 23.

The subsequent stages are assumed equal to 0.25 \( P \).* Each stage of loading is continued until settlement ceases but for not less than 3 days.

The settlement of the pile of not more than 0.5 mm during the last day of the tests is taken as attenuating settlement, which makes it possible to proceed to the next loading stage.

*The value of 0.5 \( P \) in the translation is an error.
9. The pile tests are stopped if continuous settlement greater than 0.5 mm per day is observed in one stage of loading over a 10-day period.

10. The unloading of the pile is carried out in stages equal to the loading stages, with each stage lasting at least one day.

11. The tests establish the following:
   (a) the ultimate strength of the pile, equal to the load under which the pile continues to settle for a period of 10 days at a rate equal to or somewhat exceeding 0.5 mm per day
   (b) design settlement - the settlement under the design load
   (c) rate of settlement of the pile during each stage of loading
   (d) elastic and residual deformation of the pile.

12. During the pile tests, records are kept in accordance with GOST 5685-51. Test records must contain the following:
   (a) observation logs of ground temperatures and pile settlements
   (b) "settlement vs loading" curve constructed on a scale: 1 cm = 1 mm of settlement, 1 cm = 5 tons of load
   (c) settlement vs time curve on a scale: 1 mm = 1 hr, 1 cm = 1 mm of settlement
   (d) rate of pile movement vs time curve on a scale: 1 mm = 0.2 mm/hr, 1 mm = 1 hr
   (e) a soil profile showing the soil types, main properties and thickness of soil layers
   (f) temperature profiles and graphs of isopleths in the soil at the pile
   (g) temperature graphs of foundation soils at the moment when the piles reach their ultimate condition.
This monograph describes the various kinds of foundations used for structures on permafrost with a brief discussion of foundations in areas of seasonal frost. Special attention is given to piled foundations in permafrost and the design of ventilation systems for controlling thaw under heated buildings. Appendixes outline techniques for computing the depth of freezing or of thawing, the design of refrigeration systems for artificial freezing, and the recommended procedure in the USSR for static pile tests. Included in the main text are 51 figures and 62 selected references.