Minimum Requirements for Temporary Support with Artificially Frozen Ground

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Use of artificially frozen ground to provide temporary support is increasing in the United States. Temporary frozen ground structures are usually designed by contractors experienced in this type of construction. However, there is an absence of accepted requirements for artificially frozen ground. Sheeting and shoring are typically designed by a professional engineer, stress levels are regulated by code, and computations are often reviewed. Similar standard design procedures do not exist for temporary frozen walls. As a result, the owner and the engineer cannot readily ascertain the adequacy of the design of a project or the risk undertaken by the contractor. This paper describes the ground-freezing process and proposes performance and monitoring requirements for artificial ground freezing. Engineers can then judge whether the design is commensurate with project needs. Case histories are included.

Use of frozen ground to provide excavation support has increased in the United States in recent years. Although it has been used for the construction of deep vertical shafts in soils for more than 100 years, recent applications include much larger circular shafts as well as irregular-shaped structures, emergency measures to stabilize soil, and mining beneath critical structures.

Artificially frozen ground has been used for projects in which it was necessary to limit exterior groundwater drawdown. It has also provided temporary support, which can be completed before excavation, for tunnels extending beneath mainline railroad tracks. It has been used to provide early support for excavations of various shapes, including circular shafts, located near existing structures at shallow depths in a manner similar to that for structural slurry walls. Temporary structures of artificially frozen ground meet safety requirements for some geometric configurations and soil conditions that cannot be met by most other cofferdam methods, even at much higher cost. Increased use of this procedure is enhancing competitiveness, particularly for deeper circular shafts. Because of the significant expense of energy and rental of the refrigeration plant, the economy of this method depends on the duration of construction within the excavation.

Most analysis and design of frozen ground systems are performed by contractors experienced in this type of construction. Some projects have been attempted by contractors with little experience in ground freezing, sometimes with somewhat unsatisfactory results. The purpose of this paper is to summarize the information needed by engineers who consider artificially frozen ground for their projects and to establish minimum performance and monitoring requirements to avoid damage to structures, construction delays, or contractor claims.

There is a lack of accepted minimum requirements for artificially frozen ground. Industry practice for a contractor-designed sheeting and shoring usually requires design by a professional engineer that is submitted for review. Codes limit allowable stresses, and industry standards guide design. Specifications often require that the excavation contractor demonstrate minimum experience. Soil-freezing contractors have widely varying experience, and some are willing to take greater risks than the engineer or owner wants to accept.

Failure of an artificially frozen barrier because of marginal procedures or inadequate knowledge of soil conditions could cause loss of life, subsidence, and damage to adjacent structures or a structure within the excavation. Failure may result in project delay or contractor claims. Catastrophic failures are, in the writers' experience, rare. More common are partial failures due to an unfrozen zone or unplanned delay in forming the frozen structure, sometimes because of equipment breakdown. Minimum performance requirements for a particular project should be tailored to the project's specific needs and be a function of the impact of possible failure.

FREEZING SYSTEM

An installation for ground freezing is composed of a refrigeration plant and a distribution system for controlled circulation of coolant to the ground. Ground-freezing technology was introduced by Poetsch in Germany in 1883. Ground freezing is described in several publications (*1–3*), and only the most common freezing systems are briefly summarized here.

System Components

The most common refrigeration source is a conventional ammonia or freon plant, available in various capacities and typically trailer or skid mounted. It is powered by 100- to 300-hp motors providing freezing capacities ranging between 40 and 120 tons of refrigeration (1 ton of refrigeration =

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3.5 kW). Rated tonnage for ground freezing is highly dependent on brine temperature and is often based on cooling the circulating brine to −20°C. The evaporating temperature of the refrigerant in the chiller will be about −25°C before this brine temperature is obtained. The authors recommend that this relationship be established as a standard in artificial freezing construction. A refrigeration plant will produce more than twice the rated tonnage during startup when the brine is warm and only 70 percent to less than 50 percent of the rated value after the ground is frozen and brine temperatures are approaching practical lower limits. Rated tonnage also depends to a lesser degree on atmospheric conditions and refrigerant temperature.

It is difficult to establish the rated capacity of the refrigeration plant in the field. Refrigeration plants are often modified and may have replacement components differing from the initial assembly. Although the basic components of an appropriate freezing plant are available from many manufacturers dealing with various aspects of refrigeration, these components are usually selected and assembled by a few refrigeration specialists familiar with the particular design and construction requirements for ground freezing. The contractor should always be required to submit data that clearly establish the manufacturer of the refrigeration plant and its rated capacity.

Several plants can be combined if greater capacity is required for a given project. A backup refrigeration unit should be available during all phases of excavation to ensure stabilization of the frozen ground in case of breakdown. A backup unit should also be required during the initial freeze if breakdown delays cannot be tolerated by the project's schedule.

The refrigeration plant comprises a compressor, a condenser, and an evaporator, shown schematically in Figure 1. The compressor liquifies gaseous refrigerant as it is pressurized to several atmospheres. Pressurization raises the temperature of the refrigerant, which is then cooled as it passes through water-cooled coils in the condenser. The refrigerant next passes through an expansion valve and is sprayed onto the coils of the evaporator. Coolant is chilled as it passes through the evaporator coils, which act as a heat exchanger. The ammonia or freon gas then flows into the compressor, where the cycle is repeated. The refrigeration plant is a closed system in which the ammonia or freon refrigerant is continuously circulated.

In the classic ground-freezing system, the coolant is brine. Generally this is a solution of calcium chloride and water that has a specific gravity of 1.24 to 1.28. The brine is pumped into freeze pipes in the ground by means of a supply header. Freeze pipes that are accessible at only one end, such as in a shaft, contain a concentric feed pipe that supplies chilled brine to the end of the pipe. The chilled brine returns back through the annulus formed by the two pipes, extracting heat from the ground as it flows. Brine can be pumped directly from one freeze pipe into another if both ends of the freeze pipes are accessible, as in a tunnel. The warmed brine is collected in a return header and rechilled at the refrigeration plant. The cycle is then repeated. The freeze pipes and headers form a closed system in which the brine is continuously circulated. Calcium chloride brine begins to gel at about −40°C.

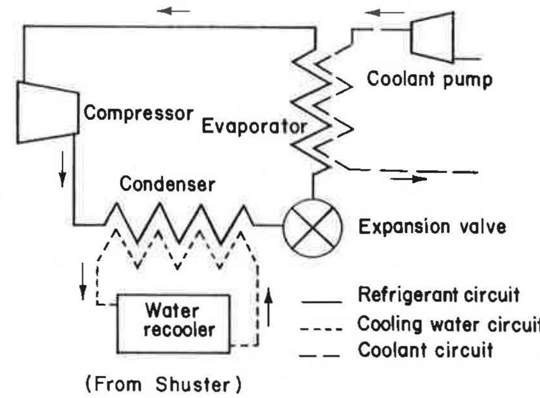

FIGURE 1 Refrigeration plant schematic (*2*).

The closed brine circulation system is simple and is the most commonly used system in ground freezing. Heat transfer is by convection; there is no phase change in the coolant. Time required for freezing is measured in weeks.

Another system that is less frequently used entails the direct injection of a refrigerant, typically liquid nitrogen, into the freeze-pipe assembly, where it evaporates. The resulting gas, still at a very low temperature, is released into the atmosphere. The nitrogen system will freeze soil considerably faster than chilled brine, and the freezing time is typically measured in days rather than weeks. Expandable refrigerants, such as nitrogen, have been used mainly for small projects of short duration or in which emergency stabilization is needed. The principal difficulty with expandable refrigerants is control of the open system. Unconfined venting of the refrigerant often results in a very irregular frozen ground zone. Liquid nitrogen has also been used as a refrigeration plant backup system to cool the brine. This use requires careful control to avoid localized overcooling of the brine.

Other possible freezing systems involve the recovery of evaporated refrigerants and their subsequent reliquification and recirculation. Shuster (*2*) describes these closed refrigeration systems in greater detail. They are not being used at this time.

Freezing Procedures

A wall of frozen ground is created by freeze pipes positioned at a predetermined spacing along the perimeter of the planned excavation. Freeze pipes are generally made of metal and are 80 to 100 mm in diameter. Larger pipe, up to about 250 mm in diameter, is sometimes used, particularly when alignment control is important.

Freeze pipes are installed by either soil removal methods or soil displacement methods. Examples for horizontally installed pipe are (a) rotary wet drilling with following casing, (b) air track drilling with following casing, (c) pipe jacking with interior soil removal, (d) using a pneumatic mole with following casing, (e) using a dry auger with following casing, (f) jacking closed-end pipe, and (g) using a steerable, larger-diameter casing. It is generally easier to control and adjust the alignment of larger-diameter pipe, so it should be considered for horizontal freeze pipes because alignment control is more difficult in horizontal installations than it is in vertical holes. Vertical pipes are usually installed in holes advanced with drilling mud or wet drilled with a following casing.

Following installation, the actual position of the freeze pipes is measured using inclinometers for vertical holes and deflectometers for horizontal holes. A deflectometer measures angle changes between two sections of small pipe sliding inside the freeze-pipe casing. Inclinometer and deflectometer data are used to determine whether the spacing of adjacent freeze pipes exceeds design values at any point along their length. Additional freeze pipes should be installed where spacing exceeds design values. Use of electronic data collection and modem transmission for microcomputer analysis and plotting of the relative location of a series of freeze pipes at various depths has expedited analysis of inclinometer measurements.

Figure 2 is a cross section through a tunnel showing horizontal freeze pipes and the approximate frozen ground envelope at the critical location directly beneath overlying railroad trackage. The measured deviation of the freeze pipe from the intended position is shown with arrows. Such deviations are typical for most freeze-pipe installations.

Freeze pipes should be pressure tested to check for leaks; then freezing is started by circulating brine through the pipes. Flow through individual pipes should be adjusted with valves to provide equal flow along the frozen ground structure. Bleed-off valves should be provided to remove air from the freeze-pipe system.

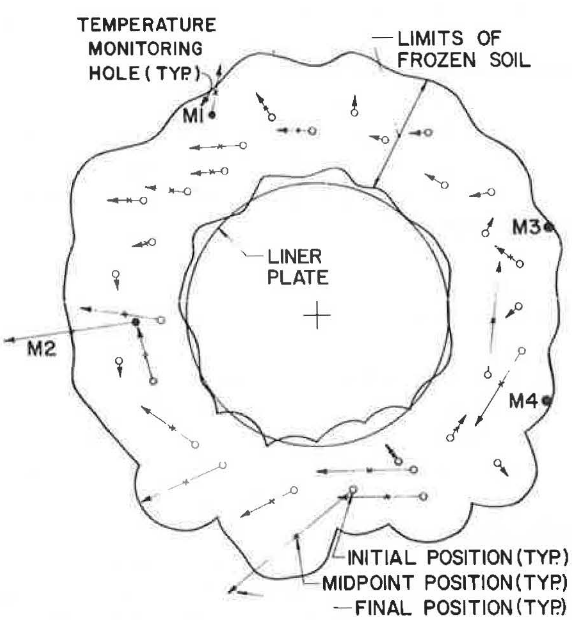

FIGURE 2 Cross-section view through tunnel showing horizontal freeze pipes and approximate frozen ground envelope beneath railroad trackage.

Design of Frozen Soil Walls

Wall thickness is based on limiting stresses in the frozen ground structure. Allowable stress levels are time and temperature dependent. Frozen soil creeps under steady load. Strength is based on plastic failure and Coloumb's law. Deformation is estimated by means of simple equations for creep. Table 1 presents typical frozen-soil properties at −10°C, representing an average soil temperature that varies from about −25°C at the coolant pipe to 0°C at the boundary between frozen and unfrozen soils. These values are not intended to be used for design but to aid in evaluating feasibility of a frozen ground alternative. Formulations for frozen ground strength and deformation are reviewed elsewhere (*4–6*).

Once the design thickness and temperature of the frozen ground wall have been achieved, interior excavation can begin.

HEAVE AND SETTLEMENT

Frost heave and thaw settlement of the ground can be small and, unless carefully measured, may not be noticed during and following construction on artificially frozen ground. However, there have been several projects in which movement of the soil during and following ground freezing has been significant.

Soil movement from ground freezing is generally insignificant in clean sands and very stiff to hard clays, even when the groundwater table is close to the surface. Low-plasticity silts and silty fine sands are much more susceptible to frost heave and postconstruction thaw settlement. Soft clays may not experience large heave but may be subject to significant settlement during thaw.

Soil movement from freezing is caused by two distinct but related phenomena. Frost heave, beyond the small volume increase resulting from the phase change of pore water to ice, is caused by the formation of ice lenses along the freezing front that draw water from nearby, more

TABLE 1 TYPICAL FROZEN-SOIL PROPERTIES

	Sand[a]	Clay
Short-term strength		
Tsf	95–160	50–95
MPa	9.1–15.3	4.8–9.1
Stress causing failure at 60 days of load (%)	±70	±70
Allowable strength at 60 days (% of 1)	30–50	—
Elastic modulus		
Frozen soil		
Tsf	6,000	—
MPa	575	—
Unfrozen soil		
Tsf	500	—
MPa	48	—

[a]Saturated soil (partially saturated soils have reduced strength).

permeable soils by negative water pressure (*7*). Thawing of incompressible soil that has experienced frost heave will result in settlement approximately equal in magnitude to the frost heave caused by escape of thawed water from the ice lenses.

The magnitude of heave due to ice lensing can be estimated from laboratory tests. Frost heave tests are performed by subjecting undisturbed samples to a controlled negative heat gradient with vertical confining pressures approximating in situ values. The sample is frozen while access is allowed to free water at its bottom, which permits the formation of ice lenses. The segregation potential method has been used to estimate frost heave (*8, 9*). The segregation potential of soil is determined by measuring the volume of water absorbed by the soil sample during freezing. The estimated magnitude of frost heave is a function of the segregation potential and soil porosity.

Although this test more closely models freezing from the ground surface down rather than radial freezing from a freeze pipe, it is a way of determining relative heave potential for various soils. The magnitude of frost heave can be minimized by rapidly freezing the ground. Because of time constraints in construction, ground is normally frozen rapidly, thus resulting in minimal ice lensing and small amounts of frost heave.

The second phenomenon occurs when compressible soils with natural moisture contents significantly above their plastic limits are frozen (*10*). Ice lenses form during freezing because of segregation of contained pore and film water within the clayey soils. If there is no external source of water, such ice lensing results in small volume increases and heave. However, significant settlement will occur during thaw when water in the ice lenses escapes. The thawed soil finally reaches a lower water content and is denser than it was before freezing. Freezing, in effect, preconsolidates the soil between ice lenses (*10*). If the soil is also susceptible to frost heave from intake of external water, the two phenomena are additive. Thaw consolidation includes both settlement from frost heave and settlement from freeze-thaw preconsolidation.

The term "thaw consolidation" is really a misnomer, because the actual densification of soil occurs during freezing. However, thaw consolidation becomes evident only when the soil thaws. An example of severe thaw consolidation will be described in detail in one of the case histories presented in the next subsection.

Thaw consolidation can be estimated by measuring thaw strain of samples that had been frozen using a controlled negative heat gradient. Thaw consolidation of clay soil can also be estimated from its plastic limit and natural water content (*10*). Special laboratory testing to determine heave and final settlement potential is normally not required for a project. Tests should be performed, however, in special circumstances, such as a frozen tunnel extending beneath critical structures or highly frost-susceptible soil. Testing may aid in quantifying the risk in employing frozen ground construction and the magnitude of soil movement.

The following case histories are not typical of most frozen ground projects, but are presented to illustrate cases of significant soil movement. In some examples, the movement was anticipated and not detrimental. Heave and thaw consolidation often can be accommodated if anticipated. Projects 3 and 4 demonstrate that potential for thaw consolidation must be given consideration.

Project 1

A 3.3-m-diameter tunnel (*11*) was constructed beneath mainline railroad tracks; the soil above the springline was primarily a cinder fill. Below the groundwater table and springline, the soil was a silty fine sand. The crown of the mined tunnel was less than 2.3 m from the surface, and frozen ground extended to less than 0.6 m from the base of the railroad ties. Horizontal freeze pipes were placed 2.3 to 2.6 m from the tunnel center. Some timber and boulder obstructions were encountered in the cinder embankment fill when freeze pipes were installed. Figure 3 shows typical heave of the tracks and postconstruction settlement. The track was periodically reballasted to compensate for settlement following construction. Settlement continued for approximately 7 months following termination of freezing. Thawing was probably prolonged after shutdown of the freeze units by the cold temperatures during the winter months.

Project 2

A second tunnel, about 30 percent larger in diameter, was recently constructed beneath railroad tracks with the crown of the excavated tunnel about 3.3 m below the ground surface and the frozen ground extending to within about 1.5 m of the base of the railroad ties. Subsoils were composed of silty clay fill extending to approximately the tunnel crown and underlain by soft to hard lacustrine silty clay deposits that extended approximately to the tunnel invert. During installation of the freeze pipes, a shallow, buried stone rubble wall was encountered. An estimate of the probable magnitude of frost heave of the tracks based on the range of soil porosities and the segregation potential determined from laboratory tests is shown in Figure 4. The soil characteristics and laboratory test data indicated a larger potential for frost heave here than in Project 1. This was confirmed by actual measurement of track heave. Rapid soil freezing was required on this project, and the actual magnitude of frost heave is superimposed on the estimated magnitude in Figure 4. Figure 5 gives the typical time rate of heave and postconstruction settlement. Tracks were periodically reballasted to compensate for these movements.

Project 3

A shaft was excavated through 17 m of soft marine clay to the top of glacial till. Nine meters of sand and gravel backfill was placed in the excavation, and a pump station was placed on the top of this backfill. Freeze pipes were

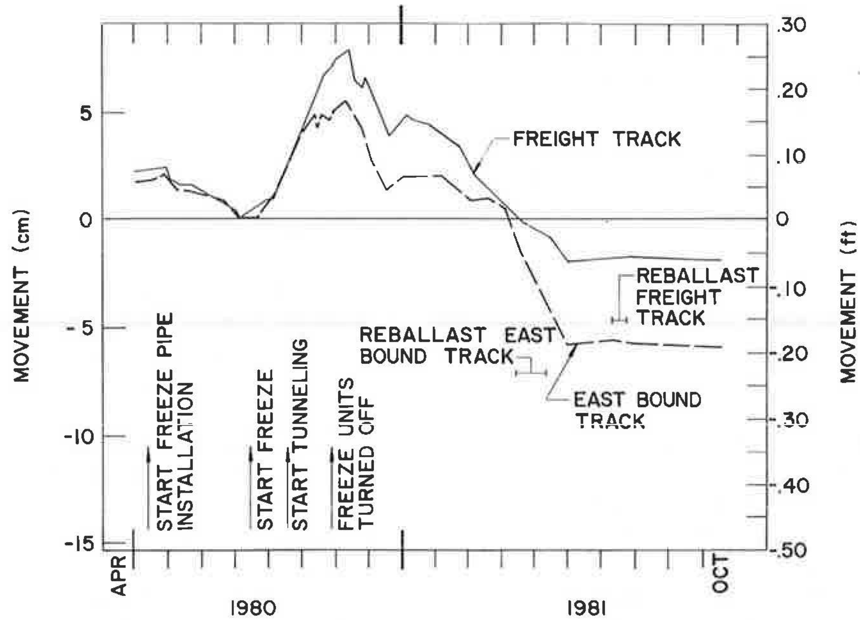

FIGURE 3 Typical heave of tracks and postconstruction settlement.

installed at 0.9-m horizontal spacing in a 16-m-diameter circle to depths of about 21 m to form the artificially frozen ground cofferdam supporting this excavation.

No heaving of the ground surface was observed during soil freezing. Approximately 1 year after construction was completed, settlement of a corner of a one-story building located 3.7 m from the line of refrigeration pipes was noted. Two and a half years later, measured settlement of the ground surface at this location totalled about 0.9 m and was still continuing. One interesting aspect of these measurements is that the most rapid settlement occurred during the last year of measurement. Soil samples obtained more than 3 years after construction revealed that although the soil had completely thawed, some ice lenses were still frozen. It appears that the salty marine clay thawed before the closely spaced ice lenses, which had formed by drawing fresh water from the clay. The fresh-water ice lenses thawed at a higher temperature than the surrounding salty soil. This would explain the relatively slow initial rate of settlement followed by more rapid settlement as the ice lenses thawed.

The soil that had been frozen reached a lower water content and higher strength than adjacent soil that had never been frozen. The decrease in water content in the frozen soils accounted for the total surface settlement. The settlement also caused damage to piping leading into the pump station and as much as 0.4 m of settlement of the pump station because of lateral movement of the supporting sand and gravel fill into the adjacent marine clays that contained thawing ice lenses. Slope inclinometers were used to measure horizontal movement toward the thawing ground from both inside and outside the frozen cylinder.

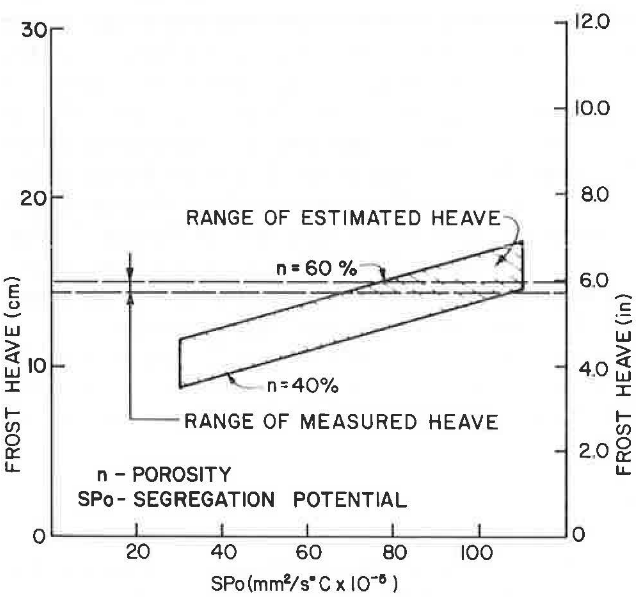

FIGURE 4 Estimated frost heave of tracks based on soil porosities and segregation potential.

Project 4

A second shaft was excavated through medium stiff to stiff clays and silts to a compact sand and gravel layer at 15 m depth. This 29- by 33-m excavation was formed by four parabolic arches that were buttressed at their flat angle corners. The resulting shape approaches a rectangle but has curving walls between corners. There was no noticeable heaving of the ground during freezing; however, no measurements were made. Movement of groundwater from adjacent pumping caused partial wall failure, exposure of freeze pipes, and rupture of brine piping. Automatic shutoff valves minimized brine contamination of the soil. Back-

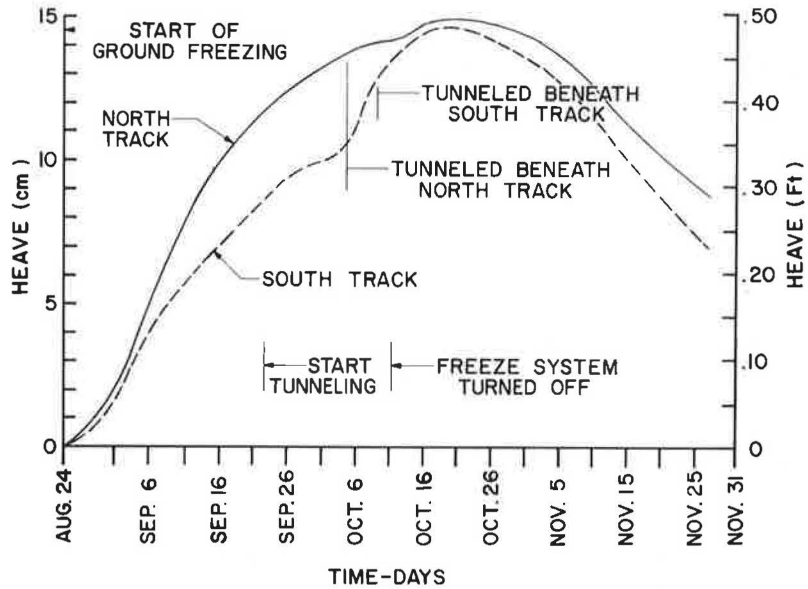

FIGURE 5 Time rate of heave and postconstruction settlement.

filling the hole with loose sand and refreezing the wall to a greater thickness extended the period of freezing substantially. Since completion, there has been nearly 80 mm of settlement of a shallow-supported retaining wall from the new structure across the line of the formerly frozen ground. There is also evidence of pavement cracking and up to 230 mm of pavement settlement, concentrated over the line of the formerly frozen ground. Ground cracks up to 80 mm wide form a circle about 9 m outside the new structure that was constructed in the shaft. Field data indicated that compaction of backfill met normal standards and should not have resulted in such large postconstruction settlement. However, a 1-m-wide band of backfill placed after partial failure was frozen in a loose condition.

REQUIREMENTS FOR ARTIFICIALLY FROZEN GROUND

The following suggested requirements are generalized for average project conditions. Different requirements may be necessary depending on the importance of maintaining a rigid frozen ground structure, the proximity of adjacent structures, and the consequences of failure of the frozen ground structure.

Freeze-Pipe Installation Methods

A variety of freeze-pipe installation methods were described in a previous section. Ground movement during installation of these elements is usually insignificant when pipes are being installed vertically. There is greater potential for ground surface settlement in installing horizontal freeze pipes. This is of particular concern when pipes are passing beneath an existing structure where settlement may be detrimental.

Rotary wet drilling methods should be used, with the casing closely following the drill bit or with drilling mud to stabilize the hole. Soil displacement methods minimize ground settlement, but pipes installed by these methods tend to be more severely misaligned than pipes installed with other methods. Rotary wet drilling must be performed carefully to avoid loss of surrounding soil. Washing a casing into the ground and having the water return outside the casing is generally not permitted for horizontal pipes. Installation of fewer large-diameter horizontal pipes is generally preferable when the pipes are long, when there may be obstructions, and when the soils are variable or dense, because their alignment is easier to control. Smaller-diameter pipes are suitable for lengths less than 30 m in low-strength soils.

Freeze-Pipe Spacing

Pipes for ground freezing are normally spaced 0.9 to 1.2 m apart. A rule of thumb for smaller freeze pipes is that the ratio of spacing to diameter should be ≤ 13. This simple formula appears to apply for pipes that are about 120 mm or less in diameter. The contractor should normally be required to meet this criterion for installed freeze pipes.

Brine Temperatures

Brine temperatures during freezing drop during the first several days of freezing and approach an equilibrium between −20° and −30°C. A required brine temperature of −25°C or less is suggested to ensure that the soil is frozen rapidly, which minimizes frost heave and expedites construction. However, temperature requirements will vary with strength requirements for individual structures, which, in turn, vary with soil type and water content.

Size of Refrigeration Plant

Refrigeration plant size in the United States is normally measured in tons of refrigeration. Some ground-freezing contractors prefer to measure plant size in terms of horsepower because rated tonnage is dependent on several factors, including air temperature, relative humidity, and brine temperature.

Typically, about 4 to 7 tons of refrigeration per 93 m^2 of interior frozen ground surface is required to form the wall of shafts. This corresponds to about 0.013 to 0.025 tons of refrigeration per lineal foot of freeze pipe. Tunnels sometimes have a double row of freeze pipes above the springline, and refrigeration requirements have typically been higher. Refrigeration capacity for a particular project depends on many factors, including desired speed of freezing, design temperatures, and so forth. Formulations to estimate tonnage requirements have been presented by Sanger (*5*). About 50 to 70 percent of the estimated tonnage requirement is typically needed to maintain the frozen ground structure after it is formed. Backup units should be made available in case of refrigeration plant breakdown.

Special Design Considerations at Shallow Depths

When the ground around a vertical shaft is frozen, heat from flowing air inhibits freezing near the ground surface. The cylinder of frozen ground around an individual freeze pipe becomes progressively smaller within 0.9 to 1.2 m of ground surface. This tapering can result in incomplete freezing of the soil between freeze pipes at shallow depths. If this unfrozen soil ravels from the top into the excavation, it will expose underlying frozen soil, which may then gradually melt. To alleviate this problem, contractors often install a horizontal freeze pipe along the center line of the vertical pipe ring 0.3 to 0.6 m below ground.

The strength of frozen soil depends on sufficient moisture between the soil grains to form ice bonds. Saturated soil below groundwater normally obtains high strength when frozen. Clay soils above the groundwater table are nearly saturated and normally obtain high frozen strength. Silty soils near the water table usually have high moisture contents as a result of capillarity, and therefore will also have high frozen strengths.

Evaporation at shallow depths tends to dry the soil, often resulting in low frozen strengths. Sands above the water table are normally too dry to form strong ice bonds when frozen, and may require the addition of moisture to obtain adequate frozen strength. This is accomplished by wetting the soil surface from a ditch or by installing slotted polyvinylchloride (PVC) pipe along the circle of freeze pipes to act as a soaker. Excessive application of water will delay formation of the frozen ground wall. Horizontal slotted pipes have been installed above tunnel alignments to add moisture where the groundwater table is low. In one instance (*11*), a bentonite slurry was used to increase moisture in a highly permeable cinder fill when it was found that moisture content was too low and that water drained away too rapidly to obtain the intended frozen strength.

Strength Testing for Design

The strength of the frozen ground is more critical at certain locations. An example would be the need for consistent frozen ground strength across the crown of a tunnel when it is immediately beneath a heavy structure. Other examples include heavy train loads above a tunnel, a highly variable rock level in a large-diameter shaft causing unbalanced loadings, a shaft that has rather flat curvature or straight walls along one or more of its sides, structures that require penetration through frozen ground walls, and curved frozen shaft segments buttressed against separate frozen shafts.

A general knowledge of the soils and their degree of saturation is usually adequate for most projects to estimate frozen soil strength with sufficient accuracy using published information. However, it is sometimes necessary to determine the frozen strength of a particular soil for design of a critical frozen ground structure. It is then necessary to obtain undisturbed samples of the in situ soils for laboratory frozen strength testing.

Strength of frozen ground is generally a function of its temperature below freezing. In other words, frozen soil at −10°C is significantly stronger than frozen soil at −5°C. Frozen soil creeps under load. As a result, strength of frozen ground decreases with time of loading (*11*). The rate of creep deformation is a function of the stress level. Creep parameters can be determined from laboratory testing. Repetitive train loading above a frozen tunnel has been simulated in the laboratory (*11*). High repetitive loads cause creep deformation and a reduction from the short-term strength in a manner similar to a smaller load applied over a long period of time.

Another method of establishing frozen strength and modulus is by artificially freezing a small test section of ground to perform in situ testing (*11*). A soil auger is used to make a hole through the frozen ground, and a pressuremeter appropriate for rock testing is inserted into the hole and expanded against its sides. Results of such tests correlate reasonably well with tests performed on soil samples frozen in the laboratory.

It is sometimes necessary to determine the elastic modulus of both the frozen ground and the surrounding unfrozen soil to analyze the distribution of unbalanced load such as a train or other moving vehicle above a frozen ground tunnel. When the frozen ground structure is close to these moving loads and the track or roadway has heaved during freezing, the dynamic loads will be attracted to the stiff frozen soil structure. The ratio of frozen to unfrozen soil modulus, as determined by laboratory test, is commonly about 12 but could be 40 or more (*11*).

Analyses of stresses within a frozen ground structure can vary from relatively simple empirical methods to elaborate finite-element techniques, which produce detailed contours of stress levels throughout the structure. An example of the results of the latter type of analysis for the first tunnel project described earlier is shown in Figure 6.

Protection of Frozen Ground Structures

Sunlight and high air temperatures at some sites can thaw frozen ground exposed in an excavation. It is then necessary to protect the frozen ground from deterioration. White plastic or canvas tarpaulins can be draped over exposed areas or foam insulation sprayed on 50 to 75 mm thick and lightly reinforced with wire mesh anchored to the exposed frozen wall.

An important requirement for a frozen ground shaft or tunnel is to prevent surface water from contacting the frozen ground structure. Normal-temperature water flowing over the top of a shaft and into the excavation can rapidly cause the frozen ground to deteriorate. It is necessary to divert drainage around the shaft and prevent inflow if there is a possibility of flooding during wet weather.

Groundwater Control

Artificial ground freezing is often used where the groundwater table cannot be lowered, typically where existing structures are underlain by compressible deposits or where watertight cofferdams cannot readily be installed. Design of frozen ground structures must consider existing groundwater conditions. Frozen ground shafts normally penetrate

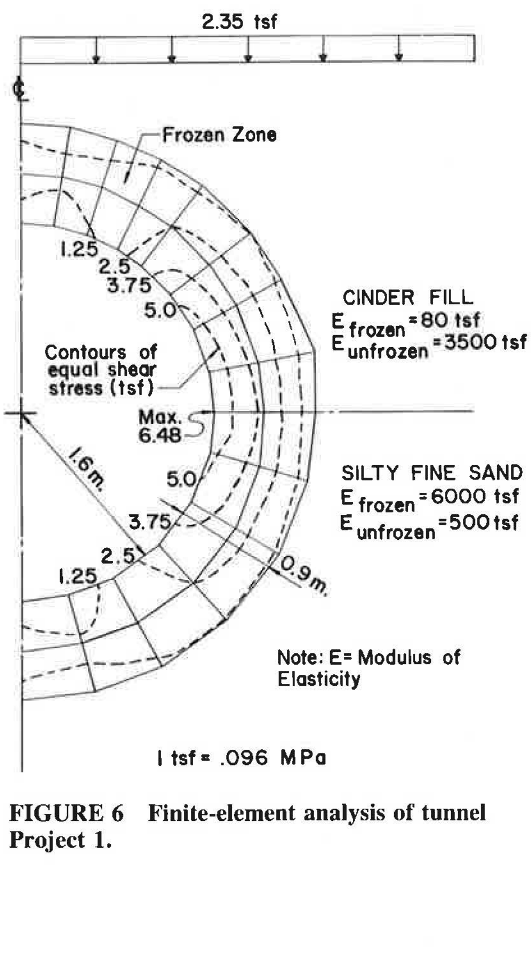

FIGURE 6 Finite-element analysis of tunnel Project 1.

an aquiclude below the subgrade, if practical. If the shaft extends to or below the bedrock surface, freeze pipes are toed into the rock to ensure a good seal. Bedrock within the shaft can be grouted to minimize water inflow, depending on rock jointing and its permeability.

The stability of soil subgrades in shafts must be analyzed in the same way as that for normal shaft construction. If necessary, underlying aquifers should be depressurized by pumping before excavation. If no aquicludes exist, the shaft interior must be dewatered with wells. Groundwater inflow from below a frozen ground shaft because of a dewatering system malfunction can cause rapid deterioration of frozen ground.

Flowing groundwater will impede formation of the ice wall. If the flow velocity through the soil pores exceeds about 1 to 2 m per day, formation of a continuous frozen ground wall may be inhibited (*12, 13*). This seepage velocity refers to the resultant actual seepage velocity in the soil pores, and not to the superficial velocity as defined by Darcy for flow through soils. Seepage velocity is simply the superficial Darcy velocity divided by the soil porosity.

It is essential to determine the groundwater gradient at a given site and to estimate its magnitude. This can be done by installing several observation wells across the site and measuring areawide water levels. The seepage velocity in an aquifer with known gradient can be estimated if the permeability and porosity of the aquifer are also known.

In a recently completed frozen ground shaft, a window in the frozen ground wall was discovered as the shaft was excavated. The window was attributed to groundwater flow, which prevented closure on the upstream side of the shaft. The window was eventually closed by adding extra freeze pipes and grouting the soil on the upstream side to block water movement through the frozen wall.

MONITORING

Every frozen ground project should be carefully monitored to evaluate the performance of the system and to track growth and temperature of the ice wall.

For a chilled-brine system, both the brine flow and the brine temperature are normally monitored, as well as ground temperatures within and adjacent to the frozen ground wall. For shafts, the groundwater level inside the frozen ground ring should be monitored for a characteristic rise in interior water level, which indicates closure of the frozen cylinder. Heave and settlement of adjacent and new structures should be monitored by establishing survey points on the structures. Suggested monitoring procedures and requirements for typical projects are outlined in the following paragraphs. Monitoring for any given project will vary depending on site-specific conditions.

Brine Flow

Brine flow into and out of freeze pipes or groups of freeze pipes is measured to identify blockage or air pockets in a pipe. Air is normally bled from high points on a daily basis to ensure efficient circulation. Brine flow data are used to balance the freezing system; that is, flow into individual freeze pipes or groups of freeze pipes is regulated with valves to ensure approximately uniform rates of growth along the ice wall.

Brine flow rates should, as a minimum, be monitored shortly after startup of the system to check for blockage and to tune the system. Thereafter, the flow rate could be checked periodically. It is also prudent to recheck brine flow shortly before excavation to ensure that there is no blockage and that brine is being distributed evenly to all parts of the frozen ground wall. Brine flow in segments of the system is commonly estimated by measuring brine temperature differences as described in the following subsection.

Brine Temperature

Brine delivery temperatures from the refrigeration plant and brine return temperatures to the plant should be monitored daily. Brine delivery temperatures will drop gradually during the freeze and thereafter stabilize at a temperature largely dependent on the volume of soil to be frozen and the plant capacity. When two or more refrigeration plants are operating together in series, the brine delivery temperature will rise dramatically if one of the plants is shut down temporarily. The difference between brine delivery and return temperatures, indicating the amount of heat transfer, will narrow as the frozen ground wall develops.

It is also instructive to monitor delivery and return temperatures at each freeze pipe or group of freeze pipes. These data can be used to pinpoint inefficient or overcooled pipes. Groundwater flow that is preventing freezing may be identified by large temperature differentials in individual pipes or groups of pipes, indicating a heat source. Brine temperatures in individual freeze pipes or groups of freeze pipes should be measured periodically, and as a minimum, just before excavation.

A temperature profile can also be obtained in a freeze pipe by temporarily disconnecting it from the system and then profiling the temperature of the brine with a thermocouple or other temperature sensor. Usually it is informative to leave individual pipes disconnected for several hours to monitor changes in the temperature profile with time. Warm spots or windows of unfrozen ground can readily be detected by this method. The freeze pipe must be disconnected from the system for a sufficient time to allow the brine temperature in the pipe and the temperature of the surrounding frozen ground to equilibrate. Such temperature profiling is not done routinely but is useful if there is a problem or a suspected problem with a frozen ground wall.

Soil Temperature

Ground temperature measurement by thermocouples in probe holes is the primary control system for ground freezing. Temperature probe holes are not installed until all freeze pipes have been completed and inclinometer surveys have been made in each freeze-pipe hole to measure deviation from the intended position. The probe holes are then installed at locations where freeze-pipe spacing is maximum. Sets of probe holes should be installed at a minimum of two locations; each set should be made up of at least two probe holes. Probe holes are normally positioned midway between freeze pipes and near the center and exterior of the zone to be frozen.

Temperatures should be monitored in each probe hole at given intervals, say every 1.5 to 7 m, with at least one thermocouple located in each soil stratum. Extra ther-

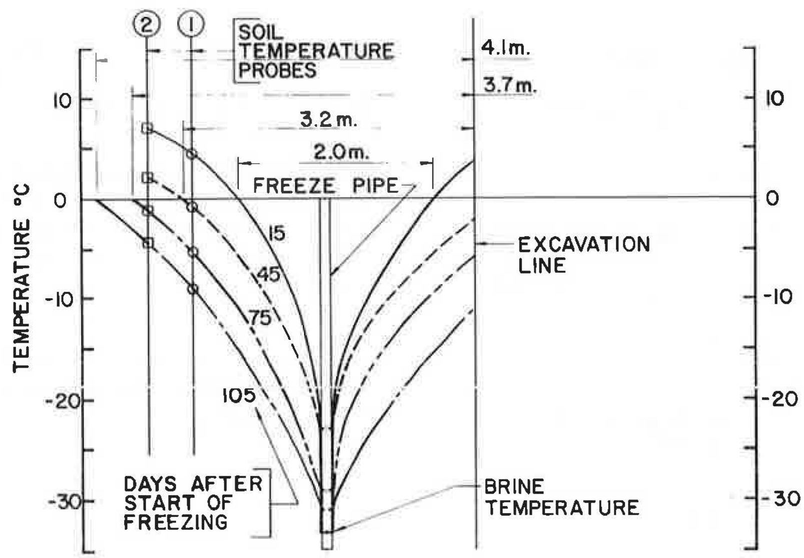

FIGURE 7 Temperature versus radial distance from freeze pipe.

mocouples can be positioned at critical locations, such as where closure is expected to take the longest.

Measured temperatures are plotted versus radial distance from the freeze pipe as shown in Figure 7. These data can be used to estimate the approximate thickness of the freeze wall, as shown. Progressive expansion of frozen ground around a freeze pipe in a shaft is shown in Figure 8. Because the wall thickness will be minimal midway between freeze pipes, this is where the probe holes should ideally be positioned. Electronic data collection of large volumes of soil and brine temperatures is now being used to speed microcomputer evaluation of the variation in temperatures with time and to permit more detailed analysis of the growth of frozen ground with time.

Other methods of determining the thickness of artificially frozen ground include frost indicators and test pits. Both are best suited for shallow frozen ground structures. Simple frost indicators consist of small-diameter (about 25 mm) plastic pipe installed in the ground. A smaller-diameter clear plastic tube, which is filled with a mixture of water and methylene blue, is then inserted into the pipe. As the mixture freezes, the blue fluid turns white. The level of adjacent frozen ground can be determined by periodically removing the insert tubes and measuring the location of the color change. This low-cost measuring device can provide a detailed profile of the freeze boundary.

Test pits can be used to directly determine the limits of frozen ground where it is shallow. This technique is particularly suited to examining the crown of shallow frozen ground tunnels. Test pits allow direct visual examination of the extent and quality of frozen soil.

Locations of temperature probe holes may be dictated by special conditions of the project. For a tunnel excavation, for example, the thickness and temperature along the crown would be of primary importance, because this area may carry heavy loads. In a tunnel project recently constructed beneath railroad tracks, four horizontal temperature probe holes were specified to be distributed across the tunnel crown to the springline. Vertical temperature probes and frost indicators were also used to measure the top of the frozen tunnel at regular intervals.

Another example is where known groundwater flow conditions exist. Under these circumstances, it is most important to place probe holes on the up-gradient side of the frozen barrier. In a recent frozen-shaft project, for example, temperature probes were placed at the widest freeze-pipe spacing, which happened to be on the downstream side of the groundwater flow. A window in the ice wall, apparently caused by the flowing groundwater, was discovered only after excavation had begun. This window might have been discovered earlier and a blow-in prevented had temperature probes been placed on the upstream side of the shaft.

Settlement

Ice lensing and resulting heave and thaw consolidation and settlement are most severe for silt-sized soil. Because of potential heave and settlement, survey monitoring points should be established before construction on all nearby structures and major utilities, including new structures after their completion. These survey points should be monitored periodically during ground freezing and subsequent thaw.

MINIMUM DATA NEEDED BY CONTRACTOR

For proper design of a frozen ground structure, the exploration should meet normal requirements for the particular project, including number and type of borings, undisturbed samples, groundwater measurements, and so on. Obstructions should be identified, because they may slow installation of freeze pipes and affect the cost of construction and the schedule.

Of great importance in ground freezing is the water content of the soils to be frozen, particularly cohesive soils. A large amount of heat energy must be removed to change pore water to ice, and freezing typically develops slowest in high-water-content cohesive soils. Of secondary importance is the density of the soils to be frozen. This can readily be determined by measuring and weighing undisturbed samples of cohesive soils. As a minimum, the water content of each soil type encountered in the exploration should be determined. Densities of cohesive soils should also be measured directly if they cannot be reliably es-

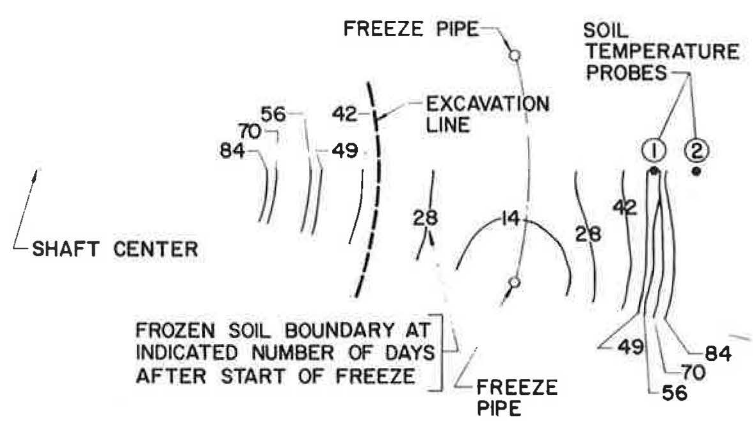

FIGURE 8 Progression of frozen ground around freeze pipe in shaft.

timated. Densities of granular deposits can generally be estimated with sufficient accuracy. The degree of saturation of granular soils above the water table should be determined.

A detailed investigation should be made of groundwater conditions to determine the gradient across the site as well as the grain size and permeability of each aquifer. These data are used to estimate seepage velocity through the soil pores. Temperature of the ground and groundwater should also be determined. If ground freezing is contemplated during design, consideration should be given to obtaining undisturbed samples of critical strata for laboratory testing of both frozen and unfrozen strength and deformation. Frost heave and thaw consolidation tests should be performed if heave or settlement will adversely affect existing or new structures.

CONCLUSIONS

Use of artificially frozen ground to provide temporary support of excavations has increased in the United States in recent years. It is commonly used for projects on which it is required that support be completed before excavation and on which the water table must not be depressed during construction. Ground freezing is occasionally specified for a project, but more commonly it is proposed and designed by the contractor when project requirements make this method of temporary support cost-effective.

This paper establishes general performance and monitoring requirements for artificial ground freezing. These are intended to enable the engineer to review a proposed frozen ground design for a particular project.

Artificial ground freezing has been successfully used for numerous projects. Nevertheless, unexpected problems have occurred even on successful projects. Examples of possible problems are presented. These can generally be anticipated by thorough subsurface investigation at the project site.

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