tions for both sharp and dull cutters.

For rolling forces, again the predicted and the measured values agreed very well, but the correlation coefficients were lower than they were for the vertical forces. It was suggested that ignorance of the cutter bearing friction in the development of the rolling force equation and the enormous sensitivity of rolling force to cutter penetration were responsible for the observed deviation of the predicted and measured values.

PREDICTION OF FIELD BORING PERFORMANCE

The field boring data supplied by Jarva, Inc., from one of their machines currently in operation in Chicago provided an excellent opportunity to verify the developed predictor equations for their applicability to predictions of field boring performance. Since the Chicago machine is boring through a relatively homogeneous and competent dolomitic limestone formation with very little jointing, the field data were considered ideal for comparison with the predicted values.

Theoretical advance rates were calculated by using a computer program that incorporated the developed predictor equations and was specifically written for use in predictions of field boring performance. The calculated advance rates and actual rates measured in the field are given in Table 1. The predicted rate of penetrations is clearly very close to those measured. This initial success in predicting field boring performance does not, of course, mean that the equations can be universally applied to cases of tunnel boring. To arrive at such a conclusion, more field data from different machines operating in different rock formations must be collected and compared with predicted data.

SUMMARY AND CONCLUSIONS

The developed predictor equations successfully predicted the forces acting on disc roller cutters in laboratory studies of borability with sharp and artificially dulled disc cutters. Moreover, they closely predicted the field rate of penetration of a Jarva tunnel-boring machine currently operating in dolomitic limestone in Chicago.

It is obvious that more field boring data must be collected and compared with the predicted values before the equations can be considered universal. In addition to parameters of machine design and operation, these data should also include any existing geological features in order to understand their effect on the predicted values.

Future work on this project will concentrate on collecting and procuring more distinct field boring data. The theoretical analysis performed for disc roller cutters will also be extended to include other commonly used rolling cutters such as disc-button and multikerf cutters.

ACKNOWLEDGMENTS

We wish to express sincere appreciation to the National Science Foundation for their financial support and to Jarva, Inc., for supplying the field data.

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Soft-Ground Tunneling by Ground Freezing: A Case History

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A brief introduction to artificial ground freezing for temporary excavation retention during construction is presented. The major aspects that affect the suitability of ground freezing in a particular project are discussed. To illustrate the applicability of artificial ground freezing, a case history in Washington, D.C., is presented. The project consisted of a circular 3.8-m (12.5-ft) diameter sewer tunnel approximately 33.5 m (110 ft) in length that passed 2.7 m (8.9 ft) beneath four sets of railroad tracks. The design process, including the frozen-soil laboratory testing program and the computer modeling, is presented. An instrumentation program was used during construction to monitor the performance of the project. The instrumentation consisted of thermocouples to monitor ground temperatures and elevation monuments to monitor ground movement during construction.

Temporary ground freezing is one of the more promising techniques for soft-ground tunneling. Ground freezing is particularly suitable where more conventional systems such as grouting or compressed air are unfeasible since ground freezing is effective in any soil that contains some pore water. Stratification and variations in permeability have little effect on freezing but can seriously affect the success of grouting. Fine sands and silts can be successfully frozen but are difficult if not impossible to grout because of their low permeabilities. Freezing eliminates the hazards of using compressed air, such as blowouts and dangers associated with working under high pressures.

The purpose of this paper is to provide an insight into the applicability of temporary frozen-earth support systems for tunnels. The paper presents a brief introduction to ground freezing for temporary construction as well as the major factors that affect the suitability of ground freezing. These considerations are then illustrated by a tunneling case history in which ground freezing was used to support loads imposed by trains passing directly over a tunnel.

PROCESS OF GROUND FREEZING

The basic concept in ground freezing is the removal of heat from the ground so that the pore water freezes and acts as a bonding agent (8, 19, 21). The heat removal is accomplished by using coolants that circulate through pipes embedded in the zone of ground to be frozen. Currently, the most commonly used and least expensive freezing method is the Poetsch process, which was developed in Germany approximately 100 years ago. This system consists of a refrigeration plant (ammonia or Freon) used to cool a secondary coolant (usually calcium chloride brine) that is circulated through the freeze pipes embedded in the soil.

Various alternative freezing systems are also available, including a primary Freon plant with an in situ evaporator, a reliquefaction plant with an in situ second stage, and expendable refrigerant systems that use liquefied nitrogen or carbon dioxide (8, 21). These alternative methods generally offer a much lower freezing temperature and faster freezing time but are currently more expensive than the Poetsch process.

The design of any freezing system requires a thorough knowledge of the mechanics of the freezing process and its effects on the soil. Some of the major considerations involved in designing a freezing system include thermal considerations, associated ground movements, strength of the frozen soil, and the cost of the freezing system. Each of these considerations is discussed below.

Thermal Considerations

A detailed discussion of the thermal design of a freezing system is beyond the scope of this paper. Therefore, only a brief overview is presented. Closed-form heat transfer solutions have been developed for very simple geometries (17). These solutions are based on two-dimensional heat conduction theory, which assumes isothermal boundary conditions at the freeze pipes and in the surrounding soil at some large distance from the pipes. The finite element method offers an alternative approach for cases of unusual geometry and complex soil stratigraphy (16).

Shuster (21) has presented a graph that shows the effect of the size and spacing of the freeze pipes on typical times required to freeze a zone of soil by various types of freezing methods (Figure 1). Additional factors that affect the time required to freeze a zone of soil include the thermal properties of the soil and coolant and the rate of groundwater flow (21). The freezing time is directly proportional to the energy to be removed from the ground and inversely proportional to the required freezing temperature. The thermal energy requirements are also related to the water content of the soil. As a rough rule of thumb, the energy requirement in kilojoules per cubic meter of soil frozen is approximately 2200 to 2800 times the water content in percentage (21).

Available literature (3, 5, 21) indicates that most of the problems and the rare failures associated with ground freezing have been related to high rates of groundwater flow. Soil cannot be frozen if the groundwater flow introduces more energy into the zone to be frozen than is being removed by the freezing system. The maximum rate of groundwater flow that can be tolerated in using the Poetsch process is approximately 0.01 to 0.02 cm/s (0.0039 to 0.0078 in/s) (2, 17, 21).

Ground Movements

Potential ground movements associated with artificial ground freezing come from three sources: frost expansion during the freezing period, stress relief during the excavation, or consolidation during the thawing period.

Ground movements associated with frost expansion are the result of two phenomena (25). The first is the expansion of the pore water during the phase change from water to ice. The volume change during the phase change is about 9 percent; therefore, the maximum expansion is 9 percent of the pore-water volume if all of the pore water freezes and there is no drainage. If the soil can drain at the same rate at which the freezing front progresses, such as in a free-draining sand, no frost heave can occur. The second potential cause of heave is pore-water migration and ice segregation at the freezing front or in the frozen zone. This will not result in ground movement if the confining pressure is greater than the pressure developed by the freezing soil-water system. Williams (25) has developed equations for estimating when the second phenomenon will result in frost heaving. These equations are dependent on the pressures in the ice and in the water and the surface tension and radius of the ice-water interface. Figure 2 shows a graphical representation of Williams' work developed by Shuster (21). Although the figure is not precise, it does illustrate that the more fine-grained the soil is, the higher is the frost expansion pressure that is developed. However, as Figure 2 also shows, the rate of frost expansion also decreases in the types of clayey soils that experience large expansion pressures. The combined pressure curve in Figure 2 represents the maximum pressure that can be developed if a source of water is available. In clayey soils, the pressures are also influenced by temperature—i.e., they are higher for colder temperatures (10).

The second cause of ground movement, stress relaxation during excavation, is common to any excavation.

The third cause of movement is thaw consolidation. Tsytovich (22) has presented equations for calculating the time rate and magnitude of thaw consolidation. Endo (4) has observed that the amount of settlement attributable to thaw consolidation appears to be about 20 percent larger than the amount of heaving during the freezing period. This additional settlement has also been observed by various other investigators (13, 14).

Strength

Frozen soil behaves viscoplastically in that it creeps under stress. The strength and deformation of frozen soils depend on both the internal friction between soil particles and the cohesion. The internal friction component depends on ice content; grain size arrangement, distribution, and shape; and the number of grain-to-grain contacts (20). Sayles (20) has shown that the angle of internal friction of frozen Ottawa sand, after overcoming the initial peak strength, is very nearly that of unfrozen Ottawa sand. The cohesion can be attributed to (a) molecular forces of attraction between particles, (b) physical or chemical cementation of particles, and (c) particle cementation by ice formation in the soil voids (20, 24).

The behavior of frozen soil depends strongly on time, temperature, and stress level. Sayles (19) has presented the following concept of the behavior of frozen soil under stress. When a load is applied to a frozen mass, stress concentrations occur between the soil particles at their points of contact, and this results in melting of the ice. Differential water surface tensions are produced that result in the unfrozen water migrating to regions of lower stress at which the water freezes. As a result of the melting of the ice and movement of the water, a breakdown of the ice and structural bonds occurs with plastic deformation of the pore ice and a readjustment in the soil particle arrangement, which results in the time-dependent deformation phenomenon of creep. As deformation occurs, there is a denser packing of the soil particles that results in a gain in strength attributable to an increase in internal friction between grains. At the same time, there is also a weakening in cohesion and a possible increase in the amount of unfrozen water. If the applied stress is less than the long-term strength of the frozen soil, the weakening process is offset by the strengthening. If the applied stress exceeds the long-term strength, where the strengthening process does not compensate for the weakening process, the rate of deformation increases with time. Structural failure of the frozen mass eventually results. The load-deformation curve for frozen soils is similar to the classical creep curve for metals (19).

The ultimate compressive and tensile strength of frozen soil depends strongly on the freezing temperature. Both compressive and tensile strengths increase with de-

Figure 1. Generalized relation between size and spacing of freeze pipe and required freezing time.

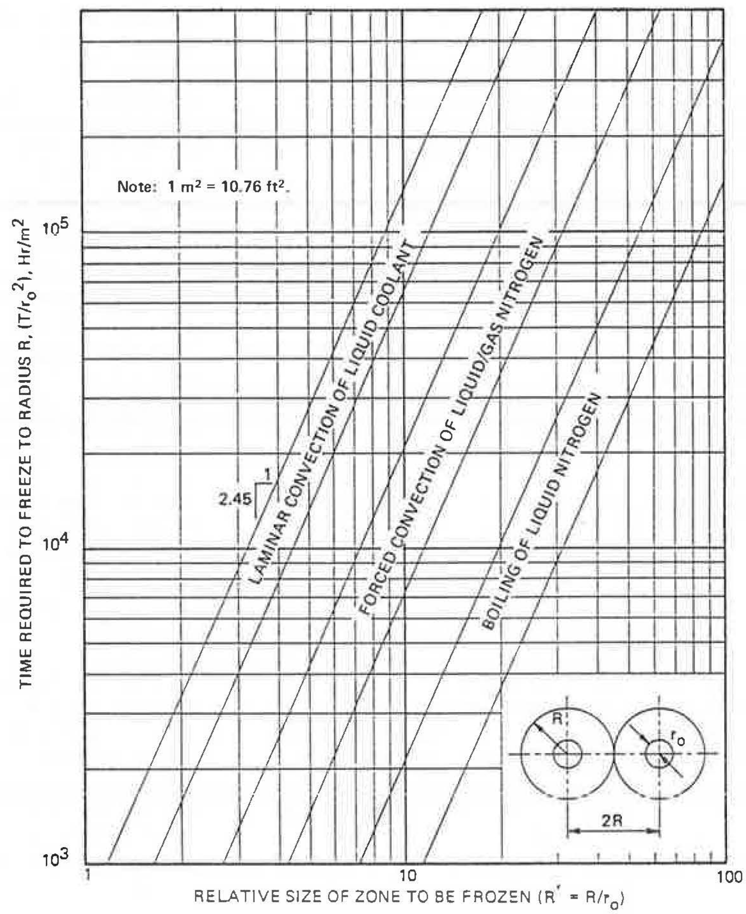
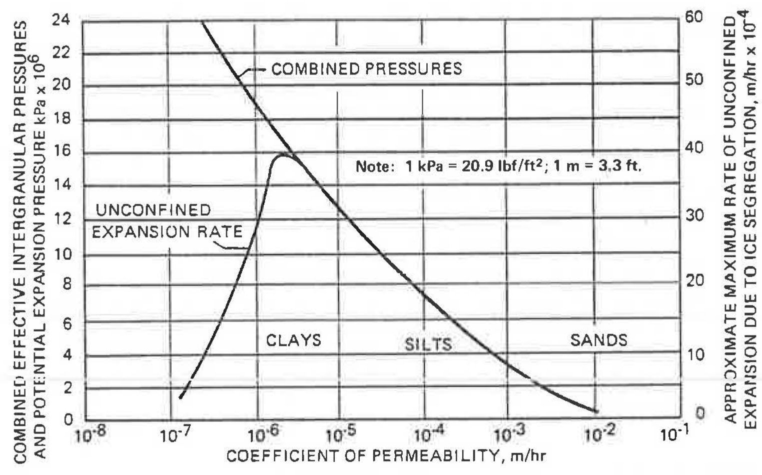


Figure 2. Typical frost expansion pressures and rates as a function of permeability.



creasing temperature. The strength of frozen soil is also a function of the moisture content of the soil. The strength increases with increasing moisture content up to complete saturation.

Vialov (23) has developed equations to describe the creep of frozen soil by using the theory of hereditary creep. Sayles (18, 19) has proposed the use of equations based on strain rate. Rein (16) has proposed the use of two separate equations that define an approximate bilinear stress-strain curve that discounts a continuous stress-strain function over the entire stress range. A detailed discussion of these relations is beyond the scope of this paper.

Cost

Because so many variables influence the cost of a freezing system, it is impossible to assign a cost per cubic meter of material to be frozen. The major items that affect the cost of the Poetsch process include ground conditions, spacing of freeze pipe, the available time for freezing, and the length of time the system has to be maintained. Typical costs associated with freezing are about \$60/m^2 (\$5.60/ft^2) of frozen surface area to install a single row of freeze pipes and around \$2/week/m^2 (\$0.18/week/ft^2) of frozen surface area to maintain the system. Therefore, the length of time the system has to be maintained has a strong influence on the total cost of freezing.

In comparing the cost of a freezing system with that of more conventional systems, one also has to consider the potential cost savings that may be realized by eliminating the necessity for dewatering, compressed air, and the like. Although the freezing process often appears expensive in direct comparison with other methods, our experience indicates that many freezing systems have been less expensive than more conventional systems as a result of time and cost savings in the overall project. This has been particularly true when the contractor was able to complete the freezing portion of the project within a relatively short time period and minimize the energy costs associated with freezing.

STUDY PROJECT

To illustrate the applicability of the freezing method, a case study is presented in which ground freezing was evaluated as temporary support for a tunnel that was to pass beneath multisets of railroad tracks. The railroad company involved in the project specified that the tunnel be designed to support a Cooper E-80 engine loading on two adjacent sets of track above the tunnel. The requirement resulted in a line load of 152 kN/m (10 400 lb/ft) for each track or a distributed stress of 61 kPa (1270 lbf/ft^2).

The project consisted of a circular sewer tunnel 3.8 m (12.5 ft) in diameter in Washington, D.C. The tunnel was approximately 33.5 m (110 ft) in length and passed 2.7 m (8.9 ft) beneath four sets of railroad tracks. Figure 3 shows a plan view of the study project.

The subsurface conditions, shown in Figure 4, consisted of clayey sand, sand, and gravel with varying amounts of clay and silt to a depth of 7.6 m (25 ft). Standard penetration resistances in the material varied from 7 to more than 115 blows/m (2 to more than 50 blows/ft) with an average value of 66 (20). The average moisture content for the clayey sand material was about 34 percent. Beneath this material was a thick stratum of silt. The average standard penetration resistance in this stratum was 13 blows/m (4 blows/ft). Typical moisture contents in this lower stratum were in the range of 60 to 80 percent. The gradation characteristics of representative samples of the two materials are given below. D_{90}, D_{60}, D_{30}, and D_{10} are the grain-size diameters at 90, 60, 30, and 10 percent of the sample passing respectively (1 mm = 0.039 in):

	Gradation (mm)			
Type of Material	D_{90}	D_{60}	D_{30}	D_{10}
Clayey sand	0.7	0.2	0.008	-
Silt	0.03	0.003	-	-

DESIGN ASPECTS OF STUDY CASE

Design of a frozen tunnel requires an economic balancing of freezing temperature and time, configuration of freeze pipe, and frozen soil thickness plus verification that freezing will not damage adjacent structures or underground utilities. These factors are all interrelated and depend on such factors as the configuration and depth of the tunnel, subsurface conditions, and loading conditions. For example, as the design temperature is lowered, the strength of the frozen soil increases, thus allowing use of a thinner frozen soil zone. However, the cost of the freezing system or the freezing time increases as the design temperature decreases. As Figure 1 shows, the smaller the spacing of the freeze pipe is, the faster the soil can be frozen; however, the cost also increases as the spacing of the freeze pipe decreases. Therefore, experience and judgment are very important in obtaining an economical and safe freezing system.

Selection of a freezing temperature is an important consideration in the design process for a frozen-soil tunnel. Typical brine temperatures for commercial refrigeration plants are around -25°C to -40°C (-13°F to -40°F). A temperature of -5°C to -15°C (23°F to 5°F) can normally be attained throughout the zone to be frozen at reasonable freezing times and freeze-pipe spacings. Based on previous experience, the thermal properties of the soils, and judgments, a design temperature of -10°C (14°F) was selected for the Washington tunnel.

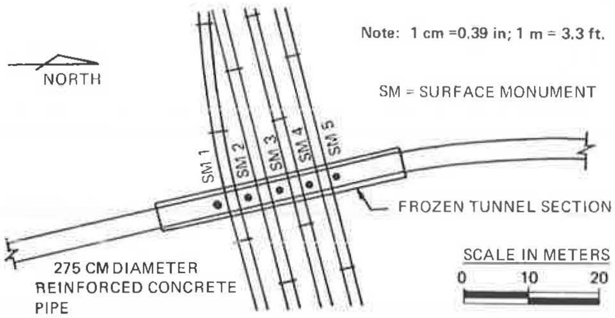
The configuration of the freeze pipe is a very important design consideration. The two major types of freezing configurations are (a) a circular or elliptical frozen zone in which the freeze pipes are placed horizontally around the perimeter of the tunnel and (b) an arch-shaped configuration in which the freeze pipes are placed vertically or inclined from the ground surface. The first configuration is shown in Figure 5. The freezing system must prevent bottom heave; this can be accomplished by using either configuration. Because of the relatively small diameter and length of the Washington tunnel and since access was available from the ends of the tunnel, horizontal placement of the freeze pipes was selected.

A third consideration is the thickness of the frozen soil, which has to be sufficient to provide an adequate safety factor against structural failure or excessive deformation. The selection of the soil thickness is highly dependent on the loading conditions and freezing temperature. The selected thickness must be verified by stress analyses (discussed in a later section). A 1-m (3.3-ft) thick zone of frozen soil was selected for the Washington tunnel.

The potential for frost heave and subsequent damage to adjacent structures or underground utilities also has to be considered in the design of the tunnel system. A method for analyzing the potential for frost heave has already been presented. An evaluation of the potential for frost heave at the study site indicated that approximately 5 to 13 cm (2 to 5 in) of heave could be expected—the larger amount at the south end of the tunnel where the soft silt stratum was at the bottom. Most of the anticipated heave was expected to occur as a result of expansion of the pore water in the soft silt stratum during the phase change from water to ice. It was decided that the expected amount of frost heave could be tolerated without causing damage.

The next step in the design process was to develop a laboratory test program to simulate the field behavior of the in situ frozen soils and to determine the stress and deformation states for the frozen tunnel configuration selected.

Figure 3. Plan view of Washington, D.C., tunnel.



LABORATORY TESTING PROGRAM

The laboratory testing program was designed to determine the strength and deformation characteristics of the frozen soil under the specified design loading and in situ conditions. Design of the testing program consisted of selecting the type of strength tests and test temperatures, considering undisturbed versus remolded samples, and evaluating the necessity of tensile testing.

The specified design criteria required that the frozen tunnels support large-magnitude static loads until the permanent tunnel liners were installed. Therefore, long-term triaxial creep tests were conducted to obtain strength and deformation parameters for the analyses.

A freezing temperature of -10°C (14°F) was used in the testing program to simulate the average freezing temperature in the field. Since high-quality, undisturbed samples of the fill soils were impossible to obtain, remolded samples were used in the laboratory testing program. The use of remolded samples was considered acceptable since the study soils, with the exception of the soft silts near the bottom of the tunnel, did not have a sensitive structure.

Since the laboratory testing program was intended to allow an evaluation of the field behavior of the frozen soil, it was necessary to decide whether tensile stresses would occur in the field and, if so, whether tensile testing would be necessary. Our experience and the avail-

Figure 4. Subsurface conditions at tunnel section.

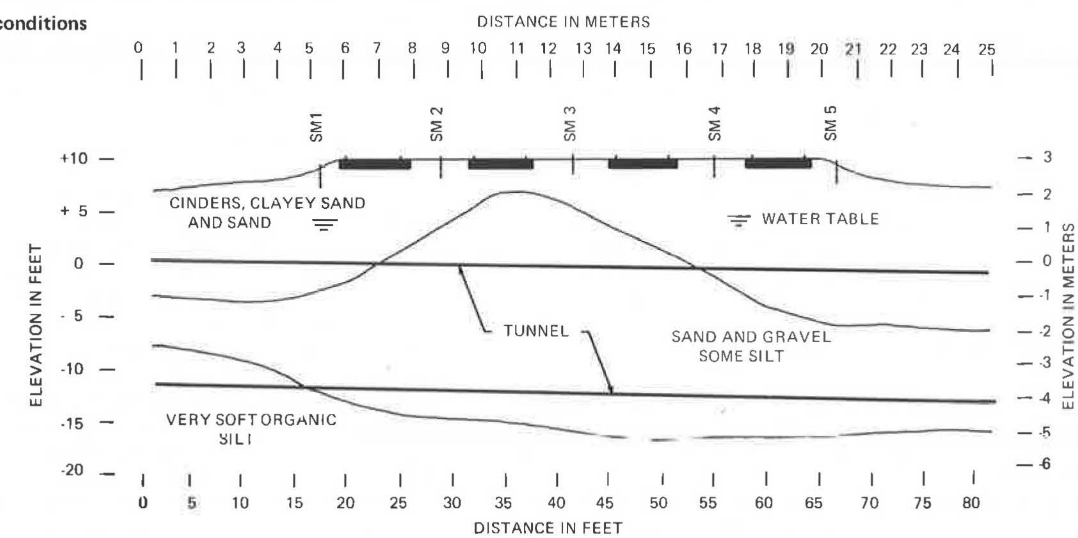
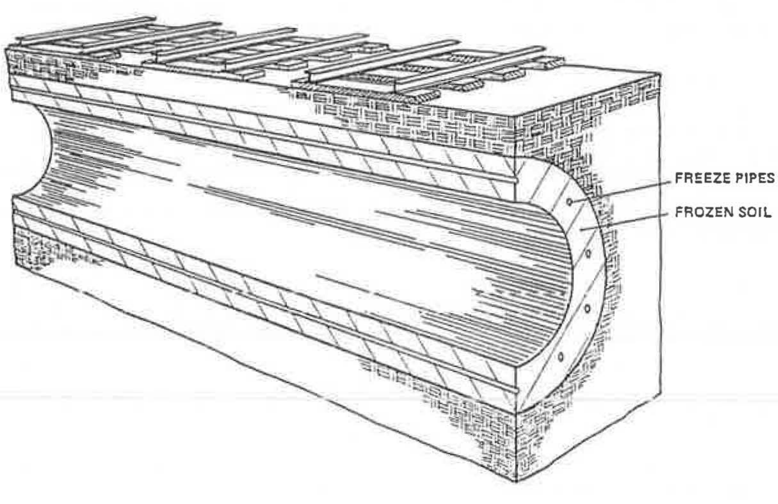


Figure 5. Frozen-soil tunnel using horizontal freeze pipes.



able literature both indicated that only nominal tensile stresses could be expected in the smaller tunnels. The available literature (9, 19, 22) indicated that the frozen soil would have tensile strengths in excess of the anticipated stresses. It was therefore decided not to conduct tensile tests.

Since only the very bottom of the tunnel passed through the organic silt, the strength of the frozen fill material would provide the major support for the tunnel. It was therefore not considered necessary to test the strength of the frozen silt.

Preparation of Samples

The samples were uniaxially frozen to a temperature of -10°C (14°F). Uniaxial freezing was used to simulate actual field freezing behavior since the in situ soil freezes in a radial direction emanating from the freeze pipes. A temperature of approximately 1°C (33.8°F) was maintained around and at the top of the uniaxial freezing chamber while the bottom of the chamber was maintained at the freezing temperature. The freezing chamber was sufficiently rigid to prevent radial expansion. Water was supplied to the samples since they were obtained below the water table. After a minimum freezing period of 24 h, each sample was ejected from the freezing chamber and inspected for formation of ice lenses. No ice lenses were observed during the testing program, however. Each sample was then placed in the triaxial cell after a rubber membrane was placed around the sample. The process of transferring the samples from the uniaxial freezing chamber to the triaxial cell was performed in a cold room at a temperature of approximately -10°C (14°F).

The remolded samples were formed in a split lucite mold at a predetermined water content. The split mold was designed to fit directly into the controlled freezing chamber to allow uniaxial freezing. A moisture content of 34 percent was used for the clayey sand fill in the Washington tunnel. The samples were uniformly compacted to average field densities in five equal layers. The compaction was performed with a 2.5-cm (1-in) diameter tamper. The surface of each layer was scarified before placement of an additional layer. The samples were then frozen in the uniaxial freezing chamber to the design temperature for a minimum period of 24 h. After that time, the sample was ejected from the mold, the rubber membrane was placed around the sample, and the sample was placed in the triaxial cell.

Equipment

A schematic of one of the triaxial cells is shown in Figure 6. The freezing unit used to cool the sample consists of a primary Freon refrigeration plant that cools a 190-L (50-gal) bath of agitated ethylene glycol. The ethylene glycol is pumped through the copper coils surrounding the sample as shown in Figure 6. The ethylene glycol in the coils cools the antifreeze that is used as the triaxial chamber fluid. To maintain a uniform temperature throughout the sample, the antifreeze is agitated by a motorized propeller. In addition, the samples are isolated from the base pedestal and top cap by lucite discs. The temperature throughout the system is monitored by copper-constantan thermocouples. The temperature was regulated to ±0.4°C (±32.7°F).

Procedure and Results

Static creep tests were performed on remolded samples of the clayey sand material, which would be expected to have the lowest strength and deformation properties of the fill material. Figure 7 shows a plot of the reciprocal of the applied axial stress versus the recorded time to failure. Figure 8 shows the stress-strain curves obtained for various loading times.

SUMMARY OF FROZEN SOIL PROPERTIES

The frozen soil parameters and the unfrozen soil properties selected for use in the analyses are given below (1 kPa = 20.9 lbf/ft^2 and 1 kg/m^3 = 0.062 lb/ft^3):

	Clayey Sand and Sand		Silt	
Parameter	Frozen	Unfrozen	Frozen	Unfrozen
Modulus of elasticity, kPa	95 760	28 728	-	4788
Unit weight γ, kg/m^3	1 857	1 857	-	1440
Poisson's ratio μ	0.3	0.3	-	0.45
At-rest earth pressure coefficient K_o	0.54	0.54	0.35	0.38
Angle of internal friction ϕ, °	0	20	-	0
Cohesion C, kPa	814	0	-	24

The slope of the stress-strain curve for the frozen soil for various loading times indicated the material could be modeled by hyperbolic functions. However, our previous experience and the available literature indicated the stresses in a 1-m (3.3-ft) thick zone of frozen soil would be in the linear portion of the stress-strain curve and would be nominal in the tensile range. The moduli were therefore selected for stress levels of 50 percent of the ultimate stress.

STRESS ANALYSES

The tunnel investigated in this study is transverse to the direction of train travel. To obtain an accurate determination of the stress and deformation states in the frozen soil, a three-dimensional analysis was considered. Because of economic constraints, however, it was decided to use conservative two-dimensional models.

Stress analyses were conducted in which all the tracks were assumed to be fully loaded with Copper E-80 engines. Since the ratio of the tunnel diameter to the available loading distance along the longitudinal axis of the tunnel was small, the train load could be approximated by a large area load. Therefore, a large area load that was infinite along the longitudinal axis of the tunnel was applied at the ground surface to simulate the train loading.

As discussed previously, linear elastic stress-strain moduli were used in the finite element analyses to model the behavior of the frozen soil. A plane strain finite element model was used to calculate tunnel stresses and deflections. The maximum calculated shearing stress in the frozen zone was 337 kPa (7040 lbf/ft^2); the corresponding normal stress in compression was 370 kPa (7730 lbf/ft^2). The maximum shearing stress occurred at the springline. The calculated levels of stress in the Washington tunnel were considered to be well within acceptable ranges. The maximum tensile principal stress, which occurred at the crown, was 262 kPa (5470 lbf/ft^2). The maximum shear stress at the crown was 154 kPa (3220 lbf/ft^2).

A factor of safety of two was maintained on the shearing stresses. The maximum calculated tensile stress of 154 kPa (3220 lbf/ft^2) was considered to be within an acceptable range. Haynes (9) has reported uniaxial tensile strengths greater than 2500 kPa (52 250 lbf/ft^2) for frozen Fairbanks silt at -9.5°C (14.9°F) tested at very slow strain rates.

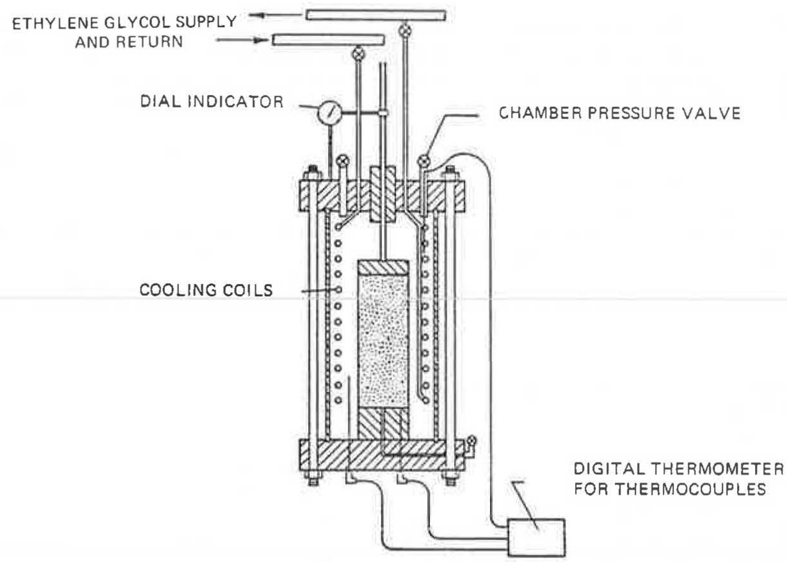


Figure 6. Frozen soil triaxial cell.

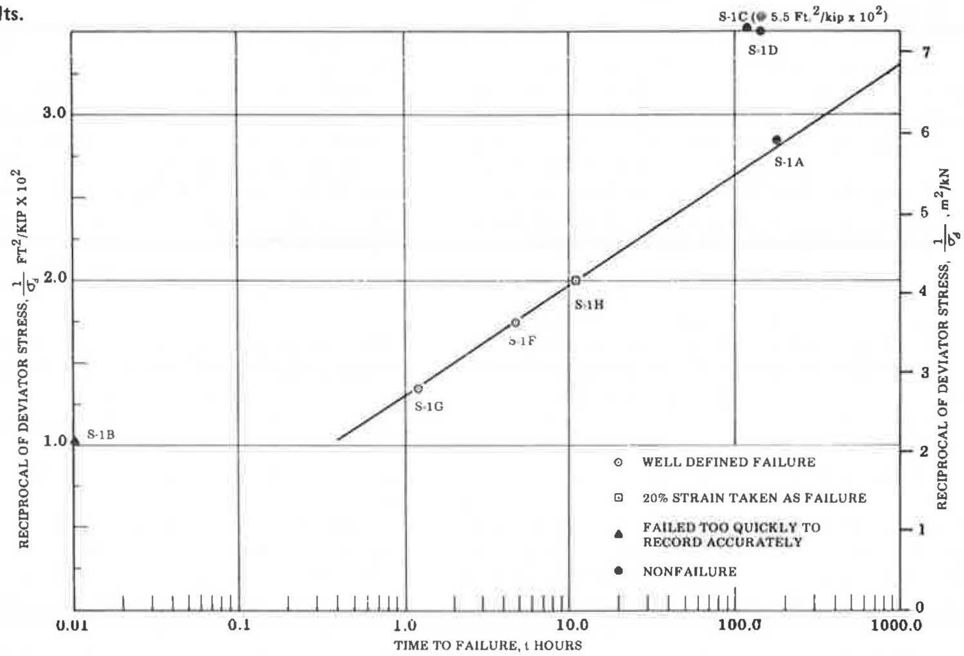


Figure 7. Triaxial test results.

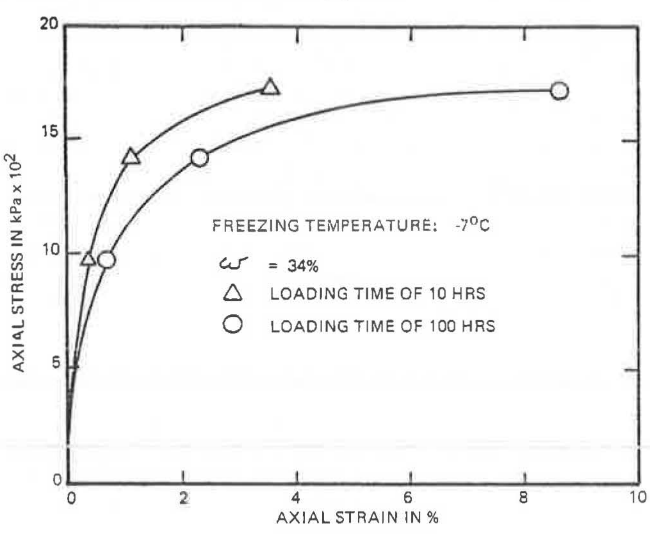


Figure 8. Stress-strain curves for clayey sand.

Although the calculated stresses indicated that a frozen zone of 0.9 m (3 ft) would be sufficient for structural support, the governing agencies required the thickness of the frozen zone to be 1.5 m (5 ft) to add an additional safety factor.

FIELD PERFORMANCE AND EVALUATION

Steel pipes 9 cm (3.5 in) in diameter and spaced approximately 0.9 m (3 ft) on center were used as the refrigeration pipes. To ensure closure of the freeze wall during a reasonable time period, it was necessary to have fairly accurately placed pipes or, if not, to know the deviation of the pipes. The pipes were driven from two cofferdams placed at each end of the tunnel. Placement of the pipes was first attempted by use of an air-actuated "down-hole" device that pulled the freeze pipes behind it. It was extremely difficult to maintain accurate alignment with this self-drilling device; therefore, it was decided to abandon this system in favor of horizontally driving the pipes. To improve the alignment of each

Figure 9. Readings of ground temperature over time.

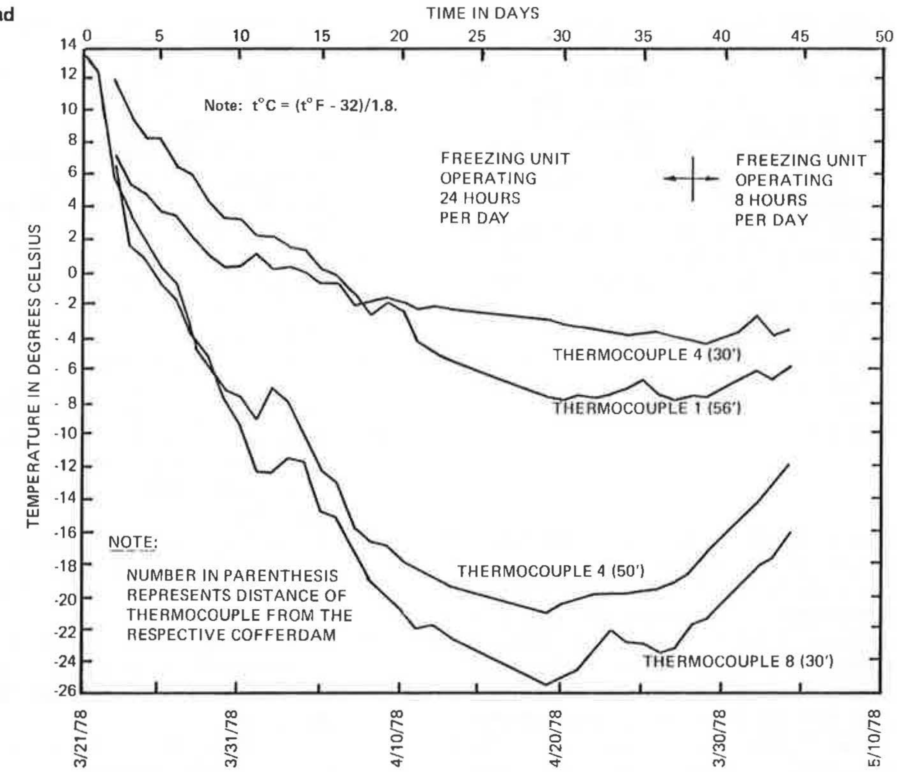
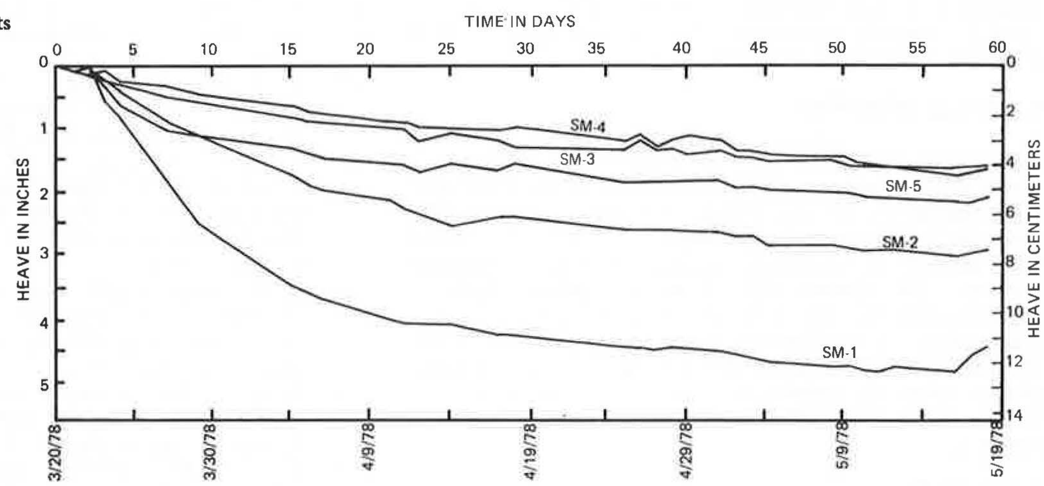


Figure 10. Heave measurements over time.



pipe, the freeze pipes were not driven from one cofferdam completely through to the other cofferdam. Instead, the pipes were driven only halfway through the tunnel distance with a 1.5-m (5-ft) overlap midway along the tunnel. The location of each freeze pipe was determined by use of a borehole deflectometer. In several instances the deflectometer revealed significant deviations of the pipe. In the areas where large deviations occurred, a second freeze pipe was installed to ensure that the required freeze zone would freeze within a reasonable time period.

Freezing was begun on March 21, 1978. Twelve thermocouples were used to record the temperature of the soil to be frozen. Figure 9 shows a time history of ground temperatures at selected locations throughout the period of freezing. Thermocouple 1 was located in a horizontal monitoring pipe installed from the north access shaft and was positioned at the eight o'clock position (southwest quadrant) in the zone to be frozen. Thermocouple 4 was in a horizontal monitoring pipe installed from the north access shaft and was positioned at the six o'clock sector. Thermocouple 1 was in a horizontal monitoring pipe installed from the south access shaft and was positioned in the three o'clock sector of the tunnel. The readings indicated that an average temperature of less than -10°C (14°F) (the design temperature) was attained in the planned frozen zone.

Mining operations were initiated on April 21, 1978, 1 month after the freezing plant was turned on. However, based on the temperature measurements, mining operations could have begun earlier than after the 1-month freezing period. Because of the extended waiting period, the inside section of the tunnel—and not just the 1.5-m (5-ft) thick annular ring as planned—was frozen. Before April 28, 1978, the freezing unit operated continuously. After that date, the freezing plant was operated on a maintenance mode, which required operation only 8 h/d. Mining operations were completed on May 22, 1978; however, the freezing unit was turned off on May 16, 1978.

Heave measurements were obtained throughout the course of the freezing operations. In addition to measuring the elevations of the railroad tracks, five permanent settlement monuments were embedded in the ground surface. The locations of the monuments are shown in Figure 3. Figure 10 shows a record of heave measurements. As expected, the major portion of heave occurred during the phase change of the pore water to ice and mainly in the soft silt material. Also as expected, the greatest amount of heave occurred at the south end of the tunnel because a greater thickness of soft silt was frozen at that end.

Unfortunately, there is currently no acceptable method for directly measuring stress in frozen soil. Therefore, there was no verification that the calculated stresses in the frozen zone were correct.

CONCLUSIONS

The design of a frozen soil lining for temporary support of a tunnel beneath mainline railroad tracks in Washington, D.C., illustrates the basic decisions that must be made during the design process for an artificially frozen ground-support system. Instrumentation data in the form of temperature readings and measurements of ground movement were obtained and presented for the study project. Although it was not possible to obtain direct data on stresses in the frozen soil, the instrumentation data provided verification of several of the design decisions. Based on this case history study and the instrumentation data obtained during the actual construction of the tunnel, ground freezing offers a viable system for use in soft-ground tunneling.

ACKNOWLEDGMENTS

The freezing project described in this paper was performed by Terrafreeze Corporation of Lorton, Virginia. Law Engineering Testing Company served as design consultant to Terrafreeze Corporation. The tunnel is owned by the District of Columbia Department of Environmental Services. The authors wish to express special appreciation to John Shuster of Terrafreeze Corporation and William Kelly of the District of Columbia Department of Environmental Services for their invaluable cooperation and assistance throughout the course of the project. Permission to publish the results of this project is appreciated.

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