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ABSTRACT

Artificial ground freezing converts soil pore water to ice. The resulting frozen ground is relatively strong and impervious. It has been used in many geotechnical engineering applications, especially difficult and unusual construction projects. Its applications have been widely used in temporary excavation supports for deep circular shaft construction. Recently, its applications have expanded to provide temporary support for deep open cut excavations and ground stabilization for tunneling. This paper describes artificial ground freezing technology, discusses its engineering properties, illustrates its applications in geotechnical engineering, and concludes with a brief summary of various artificial ground freezing project experiences. Design and performance of the frozen ground will be compared for frozen soil tunnels and a cantilever wall.

INTRODUCTION

Artificial ground freezing was developed in Germany by F.H. Poetsch in 1883. Since then, it has been used in many geotechnical engineering applications such as temporary excavation support, ground stabilization, underpinning, and groundwater cutoff. Artificial ground freezing is the process of converting soil moisture into ice. Freezing fuses the soil particles together, greatly increasing soil strength, and making it impervious. Currently, there are two ground freezing systems being used in geotechnical applications. One is the liquid brine system and the other is the liquid nitrogen system.

Liquid brine (calcium chloride) freezing is a closed system circulating the chilled brine through freeze pipes installed in the ground using a refrigeration system. Figure 1 shows an overall schematic of freeze plant circulating a brine solution through the freeze pipes and Figure 2 shows typical details of a freeze pipe. As shown in Figure 2, chilled brine is pumped down to the bottom of the freeze pipe through an open ended inner pipe and flows up through the annulus. As the circulation continues, the warm energy is extracted from the ground and converts soil pore water to ice. The different frozen thicknesses around the freeze pipe shown in Figure 2 illustrate different soils having different thermal conductivities, which influence the rate of freezing. In most ground freezing projects, the brine circulation temperatures are between -25^{o}C to -30^{o}C. Ground initially freezes around

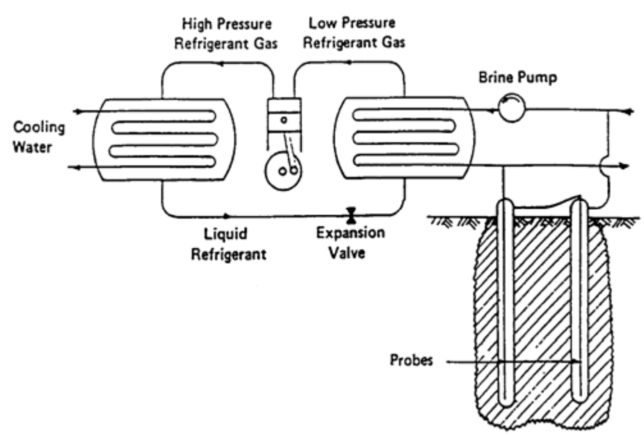

Fig.1 Schematic of freeze plant circulating a brine solution through the freeze pipes

the individual freeze pipes by forming individual frozen ground columns around the freeze pipes. With continuous freezing, these individual frozen columns merge as a frozen ground mass and become the frozen ground structure.

Although the duration of initial freezing to create a frozen ground structure is dependent upon the size and shape of frozen ground and spacing of installed freeze pipes, forming a

1.5 to 2.5 m thick circular ring of frozen ground structure for a 6 to 9 m diameter shaft excavation may take about 4 to 6 weeks of freezing. For large scale freezing projects, such as the one that was used in the Central Artery Big Dig, the freezing took several months to form a frozen ground mass having a frozen volume of 50 m by 23.3 m by 11.6 m. Once the frozen ground is created, it should be maintained throughout the construction. Its stability and performance should also be carefully monitored.

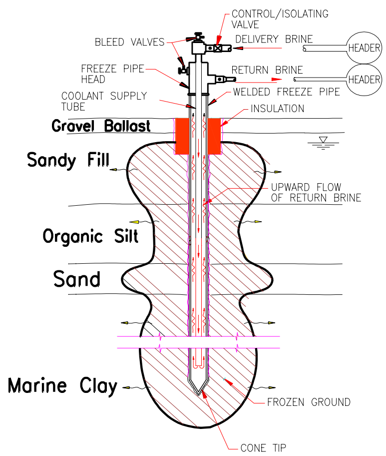

Fig.2 Typical Details of a Freeze Pipe

Liquid nitrogen freezing is an open system because once it is injected into the freeze pipes, it vaporizes as nitrogen gas at extremely cold temperatures, in the range of -82 ^{o}C to -88 ^{o}C, and directly evaporates into the atmosphere as it rapidly freezes ground. Although it is nontoxic to the environment and nonflammable, its exposure to humans can cause serious problems because of the nature of its cold temperatures. It requires a regularly scheduled delivery of liquid nitrogen at the project site to maintain the freezing operation, which often results in a costly freezing operation. Its geotechnical applications, however, have been limited to special situations where it is not practical to use the brine freezing system due to a high groundwater flow or the need to rapidly freeze the ground for a short period time.

Artificial ground freezing technology is beneficial over other ground modification techniques because it is applicable in all types of soil and rock and is easy to monitor the integrity of the frozen ground structure. It can be effectively used in difficult ground conditions, whereas other excavation support and groundwater control methods, such as sheet piling, soldier pile and lagging, slurry wall, or jet grouting, can not be practically installed. Ground freezing has no long term effect on the environment because once the frozen ground melts, the groundwater and soils are returned to their original pre-frozen states. However, it is essential to evaluate and control freezing related issues such as expansion due to freezing, strength reduction of frozen soil when loaded due to long term creep, and the influence of thawing settlement.

The following discussions and case histories are solely based on the liquid brine system.

GROUND FREEZING DESIGN AND CONSTRUCTION

In this section, the authors, based on their personal experiences from various ground freezing projects, provide practical background information for designing and constructing frozen ground.

Site Investigation

The site investigation must be performed prior to ground freezing design. Although the scope of the site investigation would be dependent upon the scale and sensitivity of the project, as a minimum it should be performed to obtain the following site specific information:

Subsurface soil and rock conditions – a reasonable number of borings / test pits should be performed to interpret the site conditions, including different soil and rock layers and their thicknesses. It is also important to identify various obstructions and/or boulders that may influence the installation of freeze pipes and freezing operations.

Disturbed and undisturbed soil samples for laboratory testing – it is important to obtain disturbed and undisturbed soil samples to be used for frozen and unfrozen laboratory testing. Soils with plasticity are more commonly subjected to laboratory testing as their frozen soil properties are more variable than those of granular soils.

Groundwater condition – the groundwater level, its flow direction, and its flow velocity should be determined. Groundwater movement will have an impact on the ground freezing operation.

Salinity – soil pore water samples should be obtained from observation wells to perform salinity content tests. Soil with high salinity content will significantly increase the duration of freezing as it depresses the freezing point and accelerates creep of the frozen ground, which reduces both to short and long term frozen ground strengths.

Unfrozen and Frozen Soil Laboratory Testing

Unfrozen and frozen soil tests should be performed on samples obtained from the site investigation to determine thermal and strength characteristics of the soils. The following laboratory tests should be performed:

Tests for predicting thermal characteristics of frozen ground:

- Frozen and unfrozen Water content
- Particle size gradation
- Atterberg Limits
- Salinity
- Thermal conductivity and mineralogy

Tests for predicting frozen ground strength:

- Unfrozen compression tests
- Frozen short term compression tests
- Frozen long term creep tests

Ground Freezing Design Parameters

Accurate interpretation of the laboratory test results is one of the critical steps for designing an appropriate ground freezing system. It is important to verify the test results based on previous ground freezing experience and information provided in related technical papers on ground freezing in similar soil conditions. The following design parameters should be established prior to the frozen ground design analysis:

- Strength of frozen ground at various frozen temperatures
- Reduced strength of frozen ground due to creep at various temperature conditions
- Frozen and unfrozen water contents
- Salinity Content
- Thermal conductivity of the unfrozen and frozen soils
- Segregation potential (frost-heave susceptibility)

Design of Ground Freezing System

The first step for designing a ground freezing system is to layout freeze pipes based on the intended frozen ground structure, site information, and ground freezing experience. For a massive block freezing, it is reasonable to install freeze pipes in a triangular pattern, except along the boundaries where single lines of closer spaced freeze pipes are installed. The reason for this single line of freeze pipes is to form groundwater cutoff boundaries quickly such that the freezing in the central area where freeze pipes are more widely spaced is not delayed by moving groundwater. A practical rule of thumb is that if the groundwater velocity is greater than 2 m/day, freezing may be difficult. For a circular shaft, freeze pipes are typically installed around the shaft to form a frozen ground ring structure. The number of rows of freeze pipes is dependent upon the diameter of the shaft, types of soils to be frozen and the required structural design capacity.

The initial ground freezing design should then be performed by thermal and structural analyses. The thermal analysis will provide shape and temperature profiles of the frozen ground. The structural analysis will use the resulting frozen ground structure to evaluate whether the frozen ground structure is stable during the construction (excavation).

Thermal Analysis

A closed form thermal analysis solution (i.e. Sanger's solution) has been used for frozen circular shaft construction. However, in many ground freezing projects, the size and geometry is complex and can not be solved by the closed form solution. It is necessary to perform thermal analysis using computer modeling, which simulates a more comprehensive heat transform mechanism.

Geoslope's TEMP/W modeling program, based on a finite element modeling technique, has been successfully used in practical applications to predict a shape of frozen ground mass and its temperature profiles (contours) as a function of freezing time. The thermal properties obtained from the laboratory tests, such as frozen and unfrozen thermal conductivity, unfrozen and frozen water contents, and other freezing related information such as brine circulation temperature and ambient temperature are required as input parameters for the analysis. The thermal conductivity controls the rate of freezing and is largely dependent upon mineralogical constituents and unfrozen water contents of the soil. In general, sand has a higher thermal conductivity than clay.

Structural Analysis

The structural analysis evaluates the frozen ground structure established from the thermal analysis. For simple circular frozen ground ring structures, closed form solutions based on elastic and plastic theories (i.e. pure compression) can be used to estimate safety factors against structural instability. However, if the shape of the frozen ground is not a simple geometry and the sequence of construction is complex, it is necessary to use computer modeling to evaluate the structural capacity of frozen ground. Finite element method (FEM) modeling programs, such as PLAXIS, can be used to simulate frozen ground mass supporting the excavation and to predict stresses and deformations induced in the frozen ground mass.

The basic concept behind the structural evaluation of the frozen ground is that the induced stresses in the frozen ground due to the excavation should not be greater than the capacity of frozen ground strength, meaning that the ratio between the available frozen ground strength and the induced maximum stress in the frozen ground mass should be greater than 1.0. For safer construction, this ratio should be higher than 2.0. The most critical part of the analysis is assigning appropriate frozen ground strength values for the analysis. Because frozen ground creeps under a constant stress, the analysis should take this into account for a long term creep condition, which will result in lower long term frozen ground strength in comparison to short term strength. In addition, the deformations of frozen ground should also be evaluated to ensure that the deformations of the frozen ground mass are within the allowable limits.

If the results of structural analysis indicate that they would not meet the minimum stability and deformation requirements for the frozen ground structure, the thermal and structural design should be repeated by adjusting the design of the ground freezing system (i.e. revising the layout of the freeze pipes or freeze longer to create colder and stronger frozen ground). This process should be continued until the frozen ground structure meets the design requirements.

Design Issues regarding Ground Expansion due to Freezing

It is important in some ground freezing applications to control expansion caused by ground freezing. When ground freezing converts soil pore water into ice, it results in about 9% expansions by pore volume of water (approximately equal to 4% of the total soil volume). This effect is most pronounced in high plasticity soils where water contents are high and permeability is low. This expansion will exert additional stresses on the surrounding soils. If ground freezing is implemented near existing buildings or other structures, such as underground utilities, these structures can be damaged from the expansion forces. It is important to evaluate the potential impact on the structures prior to the ground freezing operation and develop appropriate methods to remediate the problems. There are several methods that have been successfully used in the authors' ground freezing projects. These include a method of installing warming pipes near the structures such that the freezing can not be advanced near the structures, a method of drilling a line of holes near the structures such that the incoming expansion forces can be relieved, and/or a method of cycling ground freezing circulation (i.e. on and off the freezing operation in proper intervals) such that the freezing expansion can be moderated. Practical appreciations of these methods are discussed in the case histories.

It is also important to note that in clean granular soils, which exhibit high permeability, the excess pore water induced by the ground freezing drains out faster than the advancement of the freezing front. Therefore, there is little or no volume expansion. However, in cohesive soils and silts, which exhibit low permeability, the excess pore water induced by the ground freezing will not drain out faster than the advancement of the freezing front. Therefore, in this case, volume expansion will result from freezing.

Implementation of Ground Freezing System

After the thermal and structural analysis demonstrates that the proposed ground freezing design provides the intended frozen ground structure that can be established within a reasonable construction schedule, the designed ground freezing system can be implemented by installing freeze pipes, connection of the freeze pipes to the coolant distribution manifold, and circulating cold brine using the freezing unit. Installed freeze pipes should be surveyed for their verticality and their final positions with respect to the adjacent freeze pipes. If there are significant gaps at depth between the pipes, additional freeze pipes should be installed to reduce the gaps. If the freezing starts without identifying the gaps, there could be weak and/or unfrozen zones in the frozen ground mass, which may cause structural instability during the excavation.

Instrumentation for ground freezing monitoring should also be installed. These include temperature pipes to measure the ground temperatures, in some cases inclinometers to measure lateral ground movements near the existing structures, and observation wells to measure water levels.

Monitoring and Performance Evaluations of Frozen Ground Structure

As the freezing progresses, continuous monitoring of temperatures should be required to evaluate the progress of the freezing. It is often necessary in the early stage of freezing to check the results obtained from the thermal analysis against the field temperature measurements. This process will refine the thermal analysis to better predict the freezing progress. If inclinometers are installed near the structures, they should be monitored regularly to evaluate when to operate the pre-installed remediation system(s) to relieve the expansion forces advancing toward the structures. In frozen ground shaft construction, water levels in observation wells located in the center of the shaft can be used to indicate whether the frozen ground has been formed to completely cutoff the groundwater. The groundwater level will rise up inside the observation well where the frozen wall has closed to cutoff exterior ground water.

After the integrity of the frozen ground structure is established and the temperature monitoring indicates that the frozen ground has sufficient frozen ground temperatures and has gained enough frozen ground strength and thickness, the excavation can be implemented. It is common at this point that the freezing operation is switched to a maintenance mode (typically from -25^{o}C/-30^{o}C to about -15^{o}C) to save energy cost. In addition to the previously discussed instrumentation, the frozen ground structure movement should be monitored during the excavation. An optical survey of pre-set points can often be used to monitor the frozen ground movements. At any time during the excavation, if the movement of frozen ground is accelerating with time or creeping under the constant stress level, excavation should be stopped and the colder brine circulation should be restored to regain the frozen ground strength and to reduce the rate of movements. It is also important that the frozen ground exposed to the air should be insulated as soon as possible to protect melting of the frozen ground surface. Melting can result in raveling of the exposed face of the frozen ground, which could be dangerous and hazardous to the working crew. Polyurethane foam is widely used as an insulation material, but the use of polyurethane should also be evaluated to prevent a possible fire of the insulation and its potential harm on the working crew.

Retrieve Ground Freezing System and Thawing Effects

It is important to remember that the ground freezing operation is not complete after the freezing circulation is shut down because the frozen ground will begin to thaw out after shutting down the freezing operation. The duration of the thawing process will depend on the volume of the frozen ground, the ambient temperature, the depth of the frozen ground, and the type of frozen soil. Generally, the duration of the thawing process is longer than the duration of the initial ground freezing.

It is known that the thawing process, especially thawing of clay soils, may induce settlements and potentially damage the surrounding utilities and structures. Since the thawing process is slow and the anticipated thaw settlements will occur gradually, it is often feasible to plan and implement remedial actions. In some instances, thawing can be accelerated by circulating warm or heated water through the freeze pipes. The freezing designer and contractor should thoroughly evaluate the thawing process and its potential impacts on the existing structures prior to the start of the thawing process.

CASE HISTORIES

There are numerous examples of deep shafts being constructed using the frozen ground method to provide initial structural support and a groundwater cutoff, including 10 fairly recent shafts for the New York City Water Tunnel No. 3 and a similar number in Milwaukee. Two rather unusual artificial ground freezing projects, which the authors have experienced in the US, are briefly summarized in this section. These case histories provide general project information, the purpose of using ground freezing, specific design issues, implementation of ground freezing, and performance of the frozen ground compared to the design predictions.

Central Artery Frozen Ground Tunnel Jacking

The Massachusetts Turnpike Authority's (MTA) Central Artery/Tunnel (CA/T) project in Boston required constructing three tunnels below active railroad tracks leading to South Station using the tunnel jacking method, which would maintain full activity and use of the tracks during construction. Figure 3 shows the locations of the three tunnels. Since poor ground conditions at the jacked tunnel construction sites would not provide a stable tunnel face during mining of the jacked tunnel, the Owner's engineer (Bechtel / Parsons Brinckerhoff) proposed ground stabilization using a combination of jet grouting columns, chemical grouting, soil nailing, and dewatering to provide a stable tunnel mining and jacking operations. Full tunnel face breast board support was also required allowing only small sections of the face to be mined at any one time, a procedure similar to that used by Brunel in 1825 to 1843. The Contractor, a joint venture of Slattery, Interbeton, J.F. White, and Perini (SIWP), after a value engineering evaluation, replaced the Owner's ground stabilization approach with artificial ground freezing stabilization. The Contractor determined that frozen ground would provide less field installation problems, positive groundwater cutoff, a more stable mining face, and more reliable temporary overall structure for tunnel jacking. In addition, the full frozen soil face could be left exposed permitting more efficient and rapid mining of the tunnel face. The MTA approved the Contractor's ground freezing approach. SIWP hired freezeWALL to design, install, and operate the ground freezing system and hired Mueser Rutledge Consulting Engineers (MRCE) as a ground freezing consultant.

Fig.3 Locations of the Three Jacked Tunnels

Subsurface conditions consisted of fill over compressible organic clay, which was underlain by marine clay, locally known as Boston Blue Clay. The fill contained numerous obstructions. The proposed jacked tunnels, located from about 1m to 8 m below the existing grade, had a tunnel cross section, a 23.8 m wide and 11.6 m high. The length of the tunnels at each location varied from 50 to 109 m long. A typical cross section showing the intended frozen ground mass and jacked tunnel box is provided in Figure 4.

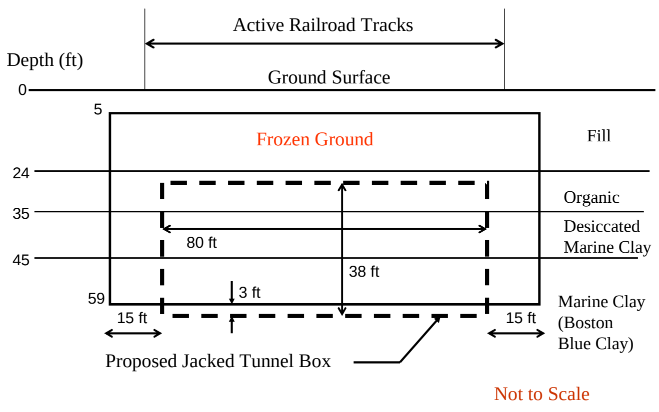

Fig.4 Typical Cross Section Showing the Frozen Ground Mass and Jacked Tunnel Box (1 ft = 0.305 m)

Laboratory testing was performed on unfrozen and frozen soils to determine soil parameters for site specific ground freezing design. A total of 28 undisturbed tube samples were obtained at the three jacked tunnel areas for the laboratory testing. Unfrozen soil tests included undrained triaxial compression, consolidated undrained triaxial compression, Atterberg limits, salinity contents, water contents and particle size gradation. Frozen soil laboratory tests included unfrozen water contents, thermal conductivity, and mineralogy, creep strength, and a heave and thaw test. Testing details and results were presented in Deming, Lacy, and Chang [2000]. The test results were used to design the ground freezing system and to analyze the stability of the frozen ground mass during the tunnel excavation.

114 mm O.D. vertical steel closed-ended pipes were installed. The freeze pipes along the perimeter of each side of the tunnel were installed with an average center-to-center spacing of 1.4m, while the interior freeze pipes were installed in a triangular pattern with an average spacing of 2.1 m to 2.4 m. The length of the pipes, ranged from 14 m to 18 m. The closely spaced perimeter freeze pipes were intended to freeze faster to establish groundwater cutoff boundaries. In addition, warming pipes, which circulate hot water through the pipes to arrest the advancement of cold energy, were installed around the frozen ground boundaries to control the external growth of the frozen ground. A freezing period of about 3 months for each jacked box tunnel was required to create the intended shape of frozen ground mass having an average frozen ground temperature of -10^{o}C. The thermal analysis performed prior to freezing correlated well with the shape and temperature profile of the frozen ground mass.

Prior to the tunnel excavation, a tunnel face stability analysis was performed to determine whether the free standing face would be safe during mining of the tunnel and jacking of the pre-cast rectangular box lining. The tunnel jacking process required complete removal of frozen ground ahead of the tunnel so that the prefabricated box would be jacked into a void at the tunnel face. The planned total advancement of tunnel jacking was about 1 to 1.5 m per day. The stability analysis was performed assuming that potential instability was developed by soil mass contained within an assumed failure surface rotating into the unsupported tunnel excavation face including the live load of three trains passing above (see Figure 5). Sliding on the failure surface was resisted by the shear forces developed along the frozen ground failure surface, which would prevent rotational movement. The safety factor was computed as the ratio of the resisting and driving moments about the center of rotation. One of the main concerns for the stability analysis was to assume that shear stresses would be well below values that limit creep deformation over the period that it would take for the tunnel to pass through the area.

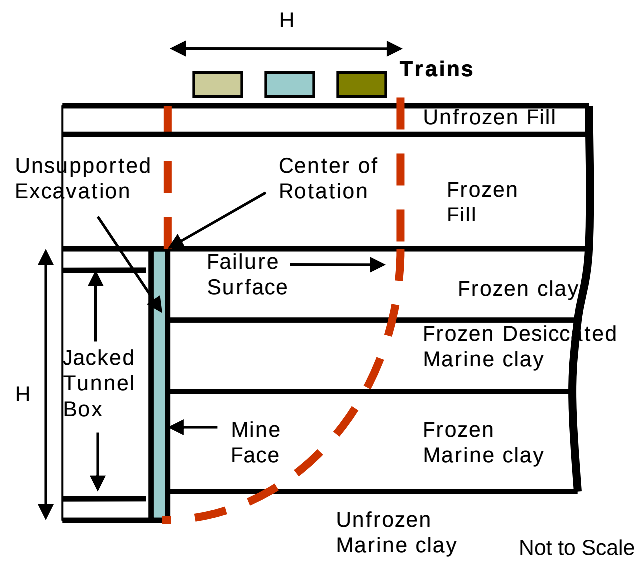

Fig.5 Typical Cross Section Showing 2-D Plane Strain Failure Surface and Center of Rotation

Frozen ground provided groundwater control and locked obstructions in place during the tunnel excavation and jacking. The mining and jacking process was advanced without significant interruptions. Although the maximum track heave predicted was about 114 mm, the actual track heave that resulted from the freezing in some localized areas for the RAMP D tunnel was higher than the maximum 178 mm allowed by AMTRAK. However, the heave that occurred slowly provided sufficient time to adjust track levels without impacting train movements. In the subsequent tunnels (I-90 EB and I-90 WB tunnel), the actual maximum heave was close to a predicted estimate of 114 mm. Experienced gain from the

RAMP D freezing was applied to the remaining two tunnels. The freezing subcontractor used an alternating freezing operation (on and off cycles) to control heave. The impact of lateral movement at the tunnel jacking pits was addressed by drilling holes in back of the sheeting to release excess load and by providing jacks on the bracing system that controlled brace loads by allowing the sheeting to deflect into the jacking pit. The sheeting moved in about 150 mm during ground freezing.

Soil modification by the ground freezing performed well providing a stable ground mass for the jacked tunnel operation. The frozen ground strength was sufficient to provide a stable unsupported vertical face almost 12 m high prior for each stage of jacking. The frozen ground held obstructions in place for mining with road-header equipment as the Contractor had projected. The thermal and structural analyses performed prior to the freezing predicted the actual frozen ground behavior reasonably well.

<u>Central Artery Cantilever Frozen Ground Support for Cut and Cover Tunnel Excavation</u>

As discussed above, part of the Central Artery /Tunnel (CA/T) project required three tunnels (Ramp D, I-90 West Bound, and I-90 East Bound) below active railroad tracks constructed by tunnel jacking through artificially frozen ground that provided a stable material and groundwater control. These jacked tunnels were connected to immersed tube tunnels, installed across the Fort Point Channel, by cut and cover tunnel segments. However, developing an excavation support system for I-90 EB cut and cover tunnels was a challenging task since there were numerous obstructions consisting of abandoned timber pile supported masonry piers, just east of the I-90 EB jacked tunnel. This made it impossible to install the originally planned soil-cement ground stabilization and T-shaped slurry wall excavation support system. After evaluating several options, it was determined that forming a massive cantilever frozen ground block (approximately 9 m wide, 26 m long and 43 m deep) would be the most reliable option to support the cut and cover tunnel excavation. This approach, however, required providing a stable and durable 18 m deep vertical frozen ground face, which would be exposed for at least 6 months including the hot summer months to support the excavation. In addition, it was required that potential heave and the subsequent thaw settlement resulting from the ground freezing, especially at the adjacent active railroad tracks, should be properly evaluated and addressed.

Subsurface conditions at the site (in descending order from the ground surface) consisted of approximately 10 m of granular fill, 4 m of compressible organic clay/silt, 26 m of marine clay (locally known as Boston Blue clay), 8 m of glacial deposits (till), and argillite bedrock. Groundwater levels were observed about 4.5 to 6 m below the ground surface and influenced by the tidal fluctuations. It was known that the fill contained numerous obstructions, including granite and concrete bridge pier foundations, timber piles, bricks, rubble, and buried abandoned railroad track structures.

The ground freezing design for the excavation support wall was subdivided into two parts. One was the shallow ground freezing to a depth of about 18 m and the other part was the lower freezing to a depth of 43 m. Figure 6 shows the ground freezing area designations (A, B, & C) and the locations of the freeze pipe installation. In the shallow freezing areas, the pipes were typically placed on a 2 m by 2.3 m spacing. But, at the perimeter of the excavation they were placed on 1.5m spacing and extended 1.5 m deeper to provide an early frozen groundwater barrier. The perimeter piles also maintained the exposed excavated frozen ground face at a colder temperature. The surface of the excavation was sloped 1H to 12V and insulated with about 100 mm of polyurethane foam reinforced with light weight (chicken) wires and secured by long nails drilled or driven into the frozen ground. The insulated face was painted white to reflect the sunlight. In deep freezing areas, the piles were installed 43 m deep, penetrating through the marine clay (Boston Blue Clay) and bearing into the till. They were installed in rows perpendicular to the excavation face to provide "barrette" shapes. The deep frozen barrettes were intended to support the shallow frozen ground mass and provide lateral stability of the frozen ground mass. The frozen soil barrettes prevented global instability of the excavation through the deep marine clay (Fig. 8). The deep pipes were spaced 2.2m within a barrette, and each barrette was spaced 4.5m along the excavation support. A total of 18 deep barrette freeze pipes were added to resist ground movement into the excavation. The basic focus of laying out the freeze pipes was to provide a continuity of frozen ground block such that there were no obvious weak zones in the frozen ground mass.

A thermal analysis was performed to evaluate the ground freezing system described above. The 2-D TEMP/W FEM computer program from Geoslope, Inc. was used to determine the rate of freezing, the extent of the frozen ground, and the frozen ground temperature profiles. Both the shallow and deep freezing were modeled independently using the freeze pipes as constant temperature sources. An average brine (freezing)

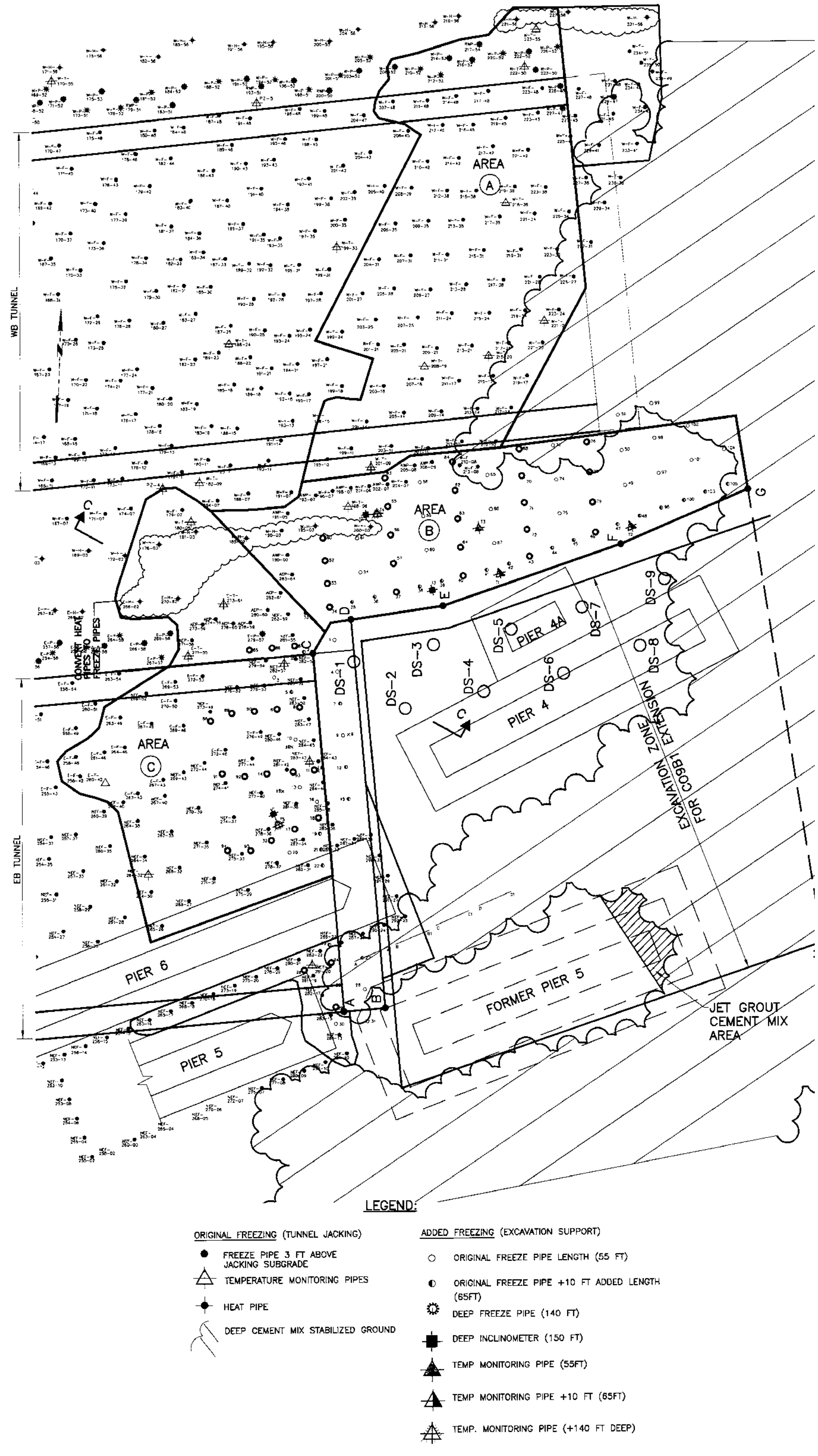

Fig.6 Ground Freezing Area Designations and Locations of Freeze Pipe Installation

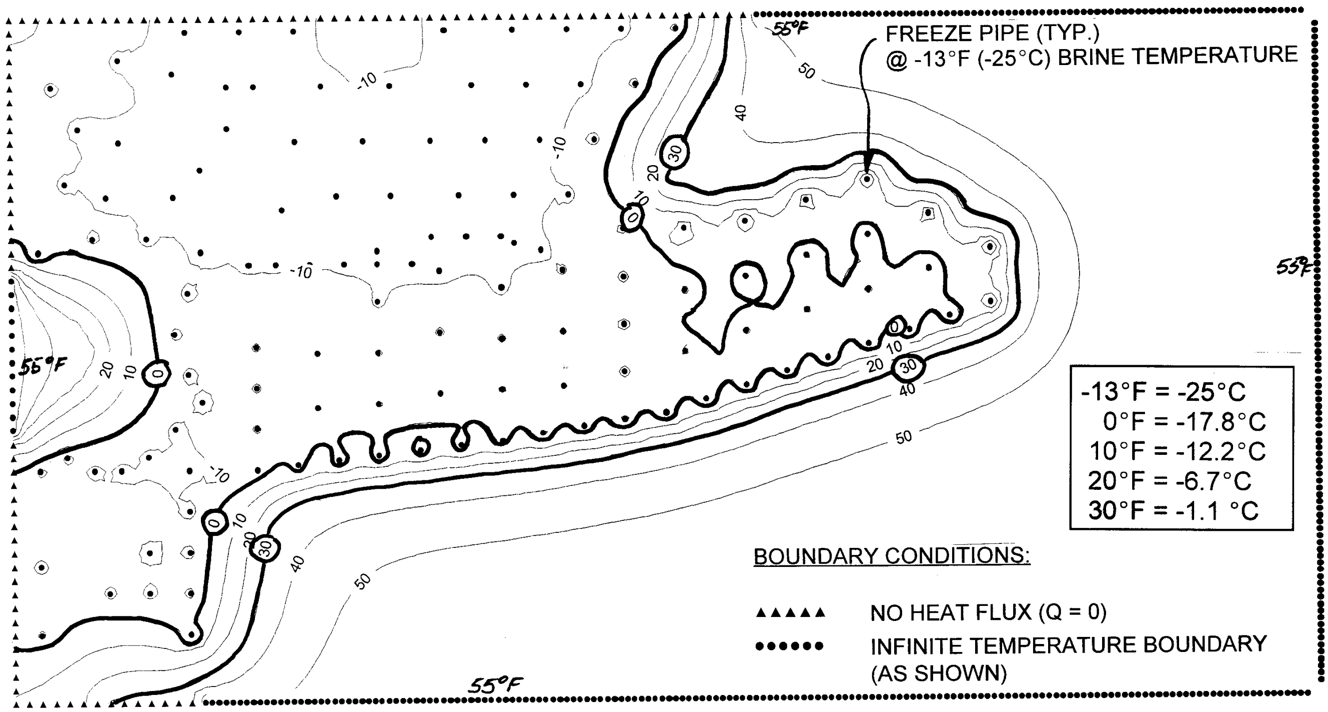

Fig.7 Typical Output from the TEMP/W Finite Element analysis (Shallow Frozen Ground Temperature Profile after 90 Days of Freezing)

circulation temperature of –25 ^{o}C was used. The thermal properties obtained from the laboratory tests were used for the analysis. The results of the thermal analysis indicated that the shallow freezing should provide sufficient shallow frozen ground mass after 90 days of continuous freezing. Additional freezing would make the frozen ground colder and stronger. The results of the deep freezing analysis indicated that at 90 days of continuous freezing, the ground would be frozen between the deep freeze pipes in each barrette and the area of frozen ground would cover about 45 percent of the area below the shallow frozen ground. At 120 days, the barrettes would merge to become a deep frozen mass and the frozen ground area would cover about 65 percent of the area below the shallow frozen ground. Figure 7 shows a typical output (shallow frozen ground temperature profile after 90 days of freezing) from the TEMP/W FEM analysis.

Based on the results of the thermal analysis, the stability analyses were performed. The results of the stability analyses indicated that after 90 days of continuous freezing, the frozen ground would provide sufficient strengths to support the first 10.5 m of excavation. At 120 days of freezing, the excavation could safely advance an additional 7.5 m of excavation to the final subgrade level. Figure 8 shows a typical cross section with the proposed excavation stages and various stability failure modes that were analyzed. In order to evaluate the most critical stability condition, the stability analyses assumed that the excavation would remain open for 150 days for the cut and cover tunnel construction and the brine circulation temperature would be switched to maintenance temperature after the initial 120 days of freezing to hold an average frozen ground temperature of –7.5 ^{o}C. This analysis considered that the frozen ground would creep and the frozen ground strength would be decreased. The long term laboratory creep test results were used to assign reduced creep strengths for the analysis. The results of the stability analyses indicated that the safety factors under the most extreme circumstances were about 2.0.

Heave prediction was an important design issue because the nearby railroad operation would have been influenced by the heave from the ground freezing. It was assumed that the fill and till strata would not produce volume expansion, but the organic clay and marine clay were expected to produce heave. It was estimated that the ground freezing would produce up to

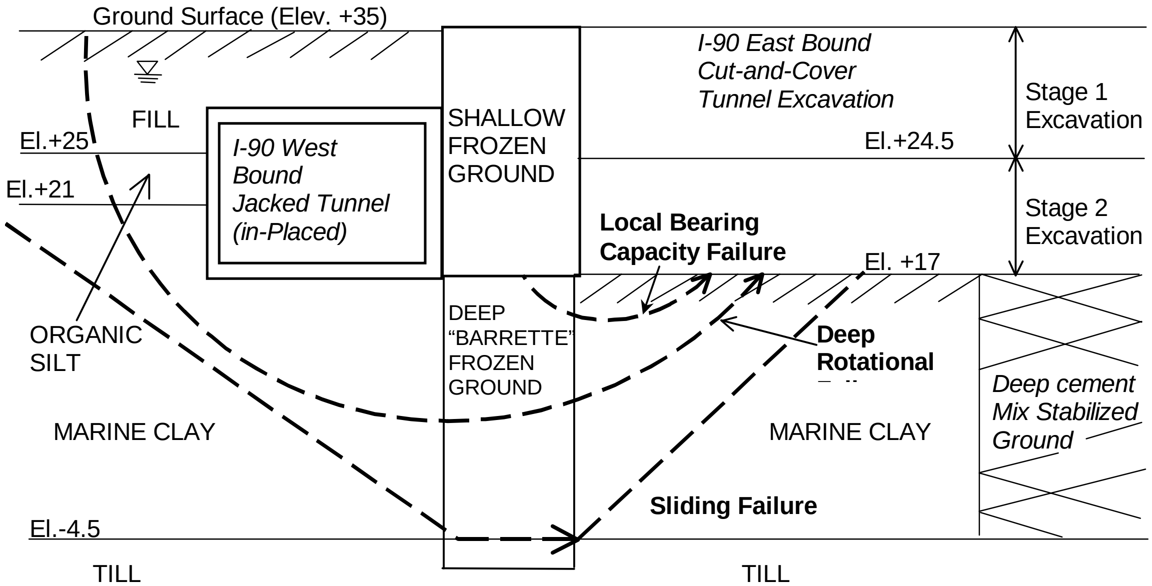

Fig.8 Typical Cross Section with the Proposed Excavation Stages and Various Stability Failure Modes

75 mm of heave in the railroad area. But, it was determined by the railroad that the slow rate of heave would not influence the railroad operation because the railroad would have sufficient time to make track adjustments.

Deformation of the exposed frozen ground walls was minor and the walls were stable for more than 18 months, which was much longer than the anticipated duration of 6 months. Optical survey data indicated that a maximum lateral frozen ground wall movement of 15 mm toward the excavation occurred during 267 days of freezing while a maximum vertical settlement of 12 mm was monitored. The only significant wall movement occurred in short duration in a small area where the insulation was accidentally removed (due to a small fire). Movement stopped as soon as the insulation was replaced.

Whenever the deformation rate of the wall increased on an increasing trend, the freezing system was quickly switched back to the colder brine temperature to re-gain frozen ground strength. This flexible strength control system was one of the advantages of using ground freezing. There were no stability problems for the frozen ground mass. Conservative design assumptions used for the stability analyses demonstrated the effectiveness of the frozen ground mass.

Figure 9 shows an overall view of the frozen ground support walls. Figure 10 shows the completed base slab of the cut and cover tunnels.

Fig.9 Overall view of the frozen ground support walls and Movement Monitoring Points on the Frozen ground Wall (also showing optical survey points on the exposed face of the frozen ground wall)

Fig.10 Overall view of the frozen ground support walls and the completed base slab of the cut and cover tunnels.

Although it was predicted that thawing of the frozen ground would be very slow and would produce ground surface settlement of up to 400 mm at the center of the frozen ground, the magnitude of thaw settlements would diminish with distance. A total of about 75mm of thaw settlements were expected at the train tracks. The railroad determined that the tracks could be adjusted to remediate settlement. The occurrence of long-term thaw settlements took longer than we anticipated (it continued for more than 2 years). However, the actual thaw settlements were significantly less than the estimated settlements and there was no reported interruption in railroad operation.

CONCLUSIONS

1. Artificial ground freezing used for support of excavation and groundwater cutoff is being employed with increasing frequency. It is suitable for deep shafts, temporary cutoff of contaminated groundwater and where obstructions prevent the use of other types of retaining structures.

2. Ground freezing had also been employed as ground stabilization for constructing tunnels and to facilitate the breakout/breakin of tunnel mining to or from shafts.

3. Ground freezing often requires careful analysis and design based on laboratory testing of soil samples frozen to simulate in-situ conditions.

4. Artificial ground freezing has proven to be a reliable and useful method for certain types of construction. The most common problem that has delayed construction had been flowing groundwater in permeable soils that slowed closure of frozen soil walls. This can be prevented by careful investigation and planning.

Ground freezing in geotechnical engineering is often perceived by the practitioner as an expensive construction option and its application is limited to unusual and difficult construction. The authors' past and present experience suggests that the artificial ground freezing is steadily gaining its popularity in geotechnical engineering applications. Better understanding of the physical and mechanical properties of the frozen ground and its engineering characteristics has made ground freezing technology a more attractive and viable construction option. Continuous efforts to improve ground freezing technology and to develop methodology of controlling freezing related issues, as discussed in this paper, will enhance the advancement of ground freezing technology and will promote more ground freezing applications in the geotechnical engineering field.

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