Factor of safety in ground freezing design

J.A. Sopko
Moretrench, A Hayward Baker Company, Rockaway, NJ, USA

ABSTRACT: Design of frozen earth structures for shafts, tunnels, and cross passages has evolved since the publication of guidelines in 2002 by the Working Committee of the International Symposium on Ground Freezing. This evolution is due to the development and implementation of numerical modeling methods, specifically the finite element method (FEM). The design was previously based on determining a thickness of the frozen earth structure with an average frozen temperature. The factor of safety was incorporated by dividing the unconfined compression strength of the frozen soil by two, and thus F.S. = 2. The finite element model allows more specific an analysis of a frozen structure cross section and adjusting strength parameters based on temperature and time-dependent strength reduction. Analyses results permit the evaluation of the internal stress regime of the frozen earth structure. This paper discusses the procedure and reports on key recent projects.

1 INTRODUCTION

1.1 *Factor of safety*

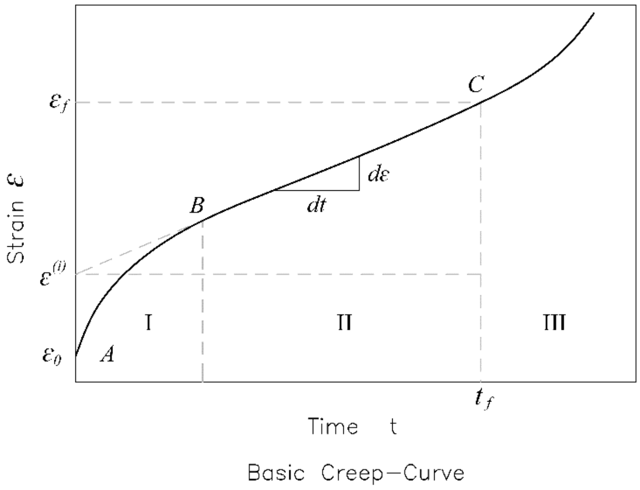
The factor of safety is the allowable stresses of a structural element divided by the actual stresses. In geotechnical engineering, the definition is more complex and somewhat undefined with frozen earth structures. The design engineer is often asked to identify the redundancy or factor of safety. In treating the frozen structure that provides temporary earth support and groundwater control as a structural element, focus is given to the allowable stresses within the structure to withstand the actual stresses imposed by surcharge loads, lateral or vertical earth pressures, and hydrostatic pressures. These allowable stresses are governed by two parameters: the strength of the frozen soil and dimensions of the frozen earth structure.

Published results of safety factors are documented as 1.0 (Harris 1995), 1.1 (Sanger 1968), and 3.0 (Sopko 1990). The author notes from his previous experience on over 50 frozen shafts that 2.0 is most often used in practice.

1.2 *Strength of the frozen soil*

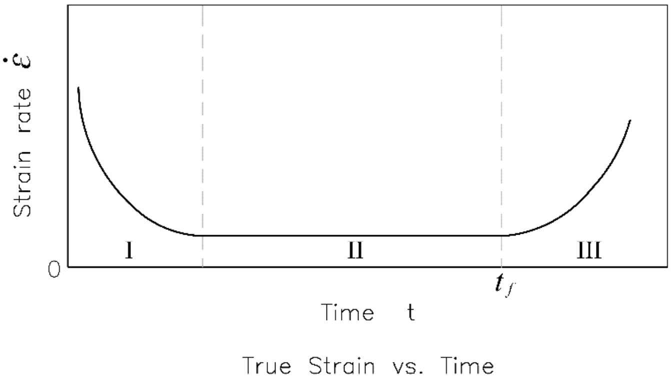
Frozen soil exhibits time and temperature-dependent rheological behavior by deforming (creeping) with time under a constant applied stress, the rate of which is dependent on the temperature. Fine grained soils such as clays and silts are more subject to creep than sands and gravels. Reducing the temperature will reduce the rate of creep and may even prevent it. There are three phases of creep: primary, secondary and tertiary illustrated as I, II, and III in Figure 1.

The primary (I) phase is where the deformation rate decreases with time, the secondary (II) where deformation remains essential constant with time, and the tertiary (III) where deformation increases with time. The relationship between deformation rates and time is shown in Figure 2.



Basic Creep–Curve

Figure 1. Basic creep deformation curve.



True Strain vs. Time

Figure 2. Deformation rates versus time.

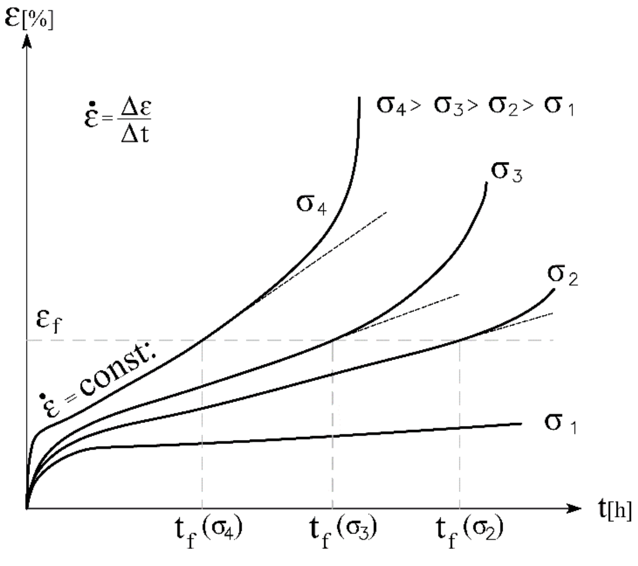


Figure 3. Idealized multiple stress level creep curves.

The deformation of frozen soil with time at a constant temperature and the rate of deformation increases with the applied stress to the soil. Figure 3 shows the relationship between time and creep deformation based on four different stress levels depicting idealized creep curves. It is difficult to get curves of this consistency in the laboratory. At least three constant stress creep tests are typically conducted on samples to evaluate a design strength of the soil and ultimately determine the factor of safety. The constant stress creep test is conducted by applying a load to a frozen sample and measuring the deformation with time. The magnitudes of the theses stresses are typically 0.7, 0.5, and 0.3 of the unconfined compressive strength is measured in a constant strain rate test. The results of the applied constant stress versus time is shown in Figure 3. It is noted that these tests must be conducted until the sample fails. A six percent strain is considered a failure. Sufficient time must be permitted to collect data that can be interpreted. This can take up to 1,000 hours in some cases.

It is necessary to begin with a frozen soil compressive strength in the design of frozen earth structures. As observed in the preceding figures, that strength is based on the length of time the structure will be exposed to combined pressures. This is unique to frozen earth. An excavation or tunnel that will be open for 30 days can be designed with a higher compressive strength than an excavation that will be open for 90 days or longer. The designer is then faced with evaluating a time-dependent compressive strength. There are two methods typically used. The first method, as proposed by the International Symposium on Ground Freezing (ISGF) (Andersland et al. 1991), uses the following equation:

$$q_f(t) = \left(\frac{\epsilon_f}{A * t^C}\right)^{1/B} \tag{1}$$

where Q_f (t) = unconfined compressive strength at a given time; t = time; ϵ_f = strain at the time of failure (Figure 3); and A, B, and C = creep parameters determined from the tests (Andersland et al. 1991).

Another method proposed by Sopko (1990) is shown in Figure 4. By plotting the time to failure versus reciprocal of applied stress, a relationship can be obtained to determine the unconfined compressive strength used in design.

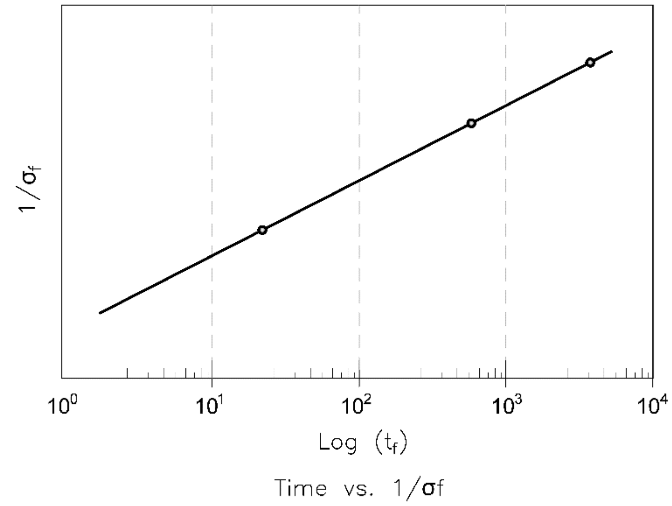


Figure 4. Time versus reciprocal of stress.

It should be noted that the designer is often faced with less than ideal data and forced to use engineering judgement in evaluation of the laboratory test results. The import concept to note is that the compressive strength used in design is totally dependent on the required time the excavation is to remain open or unbraced. After determining this time, the strength can be evaluated by using Equation 1 or by finding the corresponding stress from the line generated line in Figure 4. It is also important to evaluate strength properties at different temperatures to be consistent with the proposed temperature of the frozen earth structure.

2 DESIGN OF FROZEN EARTH STRUCTURES

2.1 *Frozen shafts – conventional design equations*

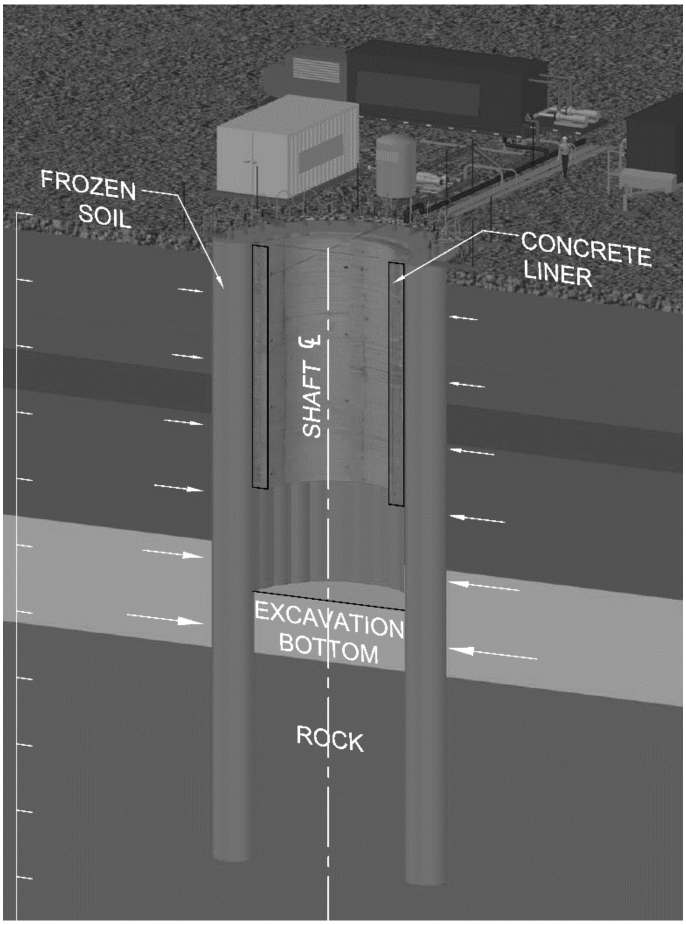


Figure 5. Typical frozen earth shaft design.

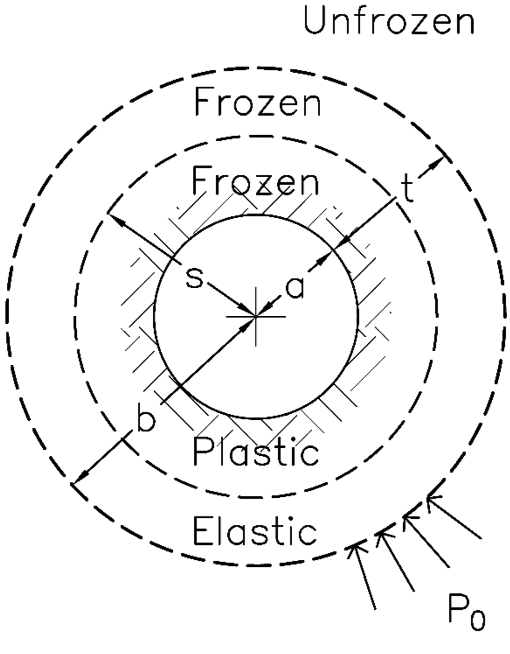


Figure 6. Definition of equation variables.

Ground freezing is most often used to provide temporary earth support and ground water control for the excavation of deep shafts. The refrigeration pipes are typically drilled and installed around the perimeter of an excavation, similar to what is shown in Figure 5.

In the design phase, it is necessary to determine the required thickness of the frozen wall to safely support the excavation and resist lateral earth and hydrostatic pressures. In practice, the thickness in often defined as the -2°C boundary intrados and extrados. Several equations have been developed to determine the required thickness of a thick-walled cylinder subjected to external pressures as shown in section in Figure 6.

One of the more commonly used equations for determining the required frozen wall thickness shown in Equation 2 was proposed by Domke (1915).

$$t = a\,[0.29\left(\frac{P_0}{q}\right) + 2.30\left(\frac{P_0}{q}\right)^2] \qquad (2)$$

This method considers the frozen soil intrados of the refrigeration pipes to be plastic while extrados elastic. It should be noted that this equation does not account for the time that a frozen earth structure will be excavated and subjected to loading. The time-dependent strength is acquired from either Equation 1 or extrapolated from a graph similar to Figure 4.

Equation 3 is based on work from Klein (1981) that was often used in the design of frozen shafts.

$$(3) \qquad t = a\,[(0.29 + 1.42\sin\varnothing)\left(\frac{P_0}{q}\right) + (2.30 - 4.60\sin\varnothing)\left(\frac{P_0}{q}\right)^2]$$

where $s = \sqrt{ab}$.

As with Domke's equation, the dependency is applied to the compressive strength. Klein incorporates the angle of internal friction in his equation. In most cases, the friction angle of frozen soil is assumed to be approximately equal to the angle for the unfrozen soil.

2.2 *Factor of safety for conventional design equations*

Prior to considering the incorporation of a factor of safety, the conservative nature of the assumptions in these equations must be addressed.

The equations are based on a uniformly loaded thick-wall cylinder. This is not the case with a frozen shaft. The load increases substantially with depth. Using a maximum or even average pressure is conservative.

An actual frozen earth shaft is essentially fixed or cantilevered at the base and typically tied into a strong or impermeable stratum. This is not the case with a free-standing cylinder. The cylinder does not transfer any load into the underlying strong stratum.

The equations assume a uniform frozen material. This is not the case as in practice, the core of the frozen earth wall near the refrigeration pipes is substantially colder and stronger than the interior and exterior zone of the wall. This stronger zone is not considered in the equations.

Frozen earth deforms when loaded, resulting in the plastic redistribution of stresses within the structure.

In addition to the inherent conservative nature, designers incorporate a factor of safety into the calculations, but the approaches vary. A simple approach would be increasing the calculated thickness of the frozen wall. For example, if the calculations yield a requirement for a 1 m wall, you could simply assign a factor of safety of 2 and use a 2 m wall. This is not practical in construction. Adding additional thickness to the frozen wall would increase the required freezing time. There are no documented references to using this approach.

As previously noted, the most common approach in design submittals that the author is familiar with is to assign a factor of safety of 2 by using half the tested time-dependent compressive strength. There is no published procedure citing this as a standard practice, but the author knows of no structural failures in using this approach.

Sanger (1968) suggests applying the factor of safety to the structural life or the time factor of strength. If the structure is to stay open for 100 days, assume 100 x 1.1 and use the decreased strength for 110 days from Equation 1 or Figure 4.

Sopko (1990) suggests applying the same approach but with a factor of safety of 3 for the following reasons: material properties are not always satisfactorily determined; refrigeration capacity may not always be available to reach the required frozen temperatures; the excavations may be open for longer than anticipated and sometimes in very warm climates where the ambient air temperatures could raise the temperature of the frozen wall and result in decreased strength; and impurities or dissolved salts may be contained in the groundwater and can weaken the frozen soil. Many improvements in standardized test procedures, refrigeration equipment, and excavation equipment have occurred since, rendering this suggestion obsolete.

Harris (1995) cites an interesting project in China where Equation 4 was used.

$$T = a\,[0.56\left(\frac{P_0}{q}\right)+1.33\left(\frac{P_0}{q}\right)^2] \qquad (4)$$

No factor of safety was applied here. Extreme creep deformation, basal heave, and broken refrigeration pipes occurred, suggesting a factor of safety of less than 1. This equation has not been accepted in the industry.

All of the cited references state the need and advantages using numerical methods, specifically the FEM, to analyze the stress and deformations with a frozen structure. The FEM permits analyses of shapes other than cylindrical and can be readily used in horizontal tunnel projects.

2.3 *FEM design*

Use of the FEM for design with programs such as PLAXIS has significantly changed the approach to the design and analysis of frozen earth structures. Figures 7a and b show typical PLAXIS models for actual projects.

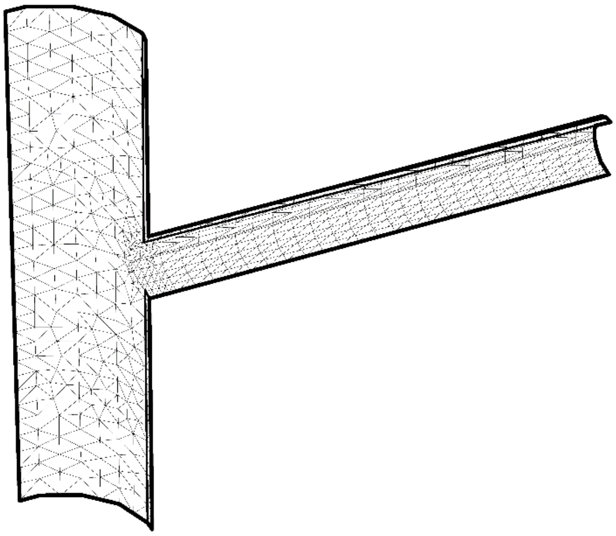


Figure 7a. Typical PLAXIS model for shafts and tunnels.

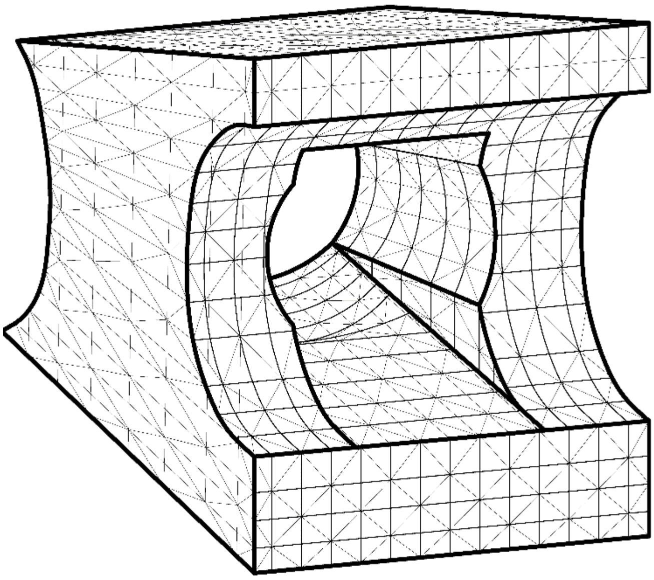


Figure 7b. Typical PLAXIS model for shafts and tunnels.

These models permit the evaluation of the compressive hoop stresses defined by Figure 8. Evaluation of these stresses has permitted a new, straight forward approach to the determination and implementation of a factor of safety.

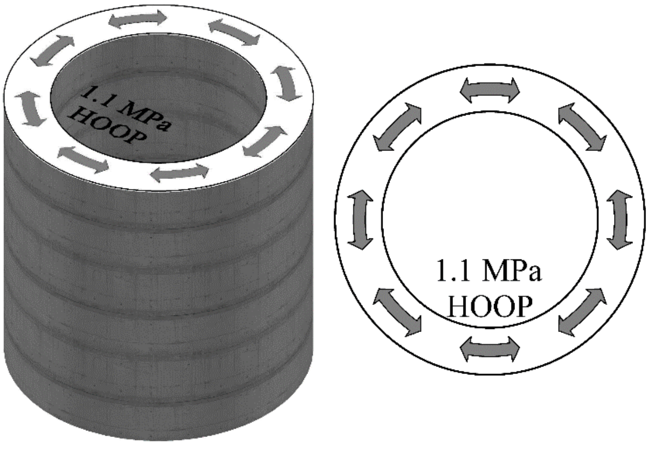


Figure 8. Definition of hoop stresses for factor of safety computation.

2.4 *Suggested factor of safety method*

The proposed standard method of applying a factor of safety in frozen earth shafts is based on the evaluation of the internal hoop stresses and comparing them to the time-dependent unconfined compression strength as shown in Equation 5.

$$\text{F.S.} = \text{Maximum Hoop Stress} / q_{(t)} \tag{5}$$

The factor of safety can be increased by increasing the thickness of the frozen earth wall, re-evaluating the stresses, and then repeating equation 4.

This approach has been successfully used on several projects in North and South America. A factor of safety ranging from 1.2 to 2.0 has been used, resulting in safe structures and no measurable deformation. This variation is based on the confidence level of the laboratory testing and the number of tests run to confirm the strengths.

Established laboratory testing procedures must be adhered to. American Society for Testing and Materials (ASTM) D5520 (2011) and ASTM D7300 (2011) for the frozen compression tests are recommended. The samples must be representative of the field conditions and are of a high quality. The designer must be experienced with the FEM for frozen ground and have observed actual field performance. Appropriate shaft insulation and lining methods are required, as well as the implementation of a well-established quality control/assurance plan. A modification to this method has been used on two projects subsequently discussed.

Certain soils are highly susceptible to creep deformation. Situations have arisen where no deformation is permitted, making it difficult to assign a factor of safety. These cases require additional laboratory testing.

The constant stress creep compression tests were run at increasingly lower stress levels until a level was reached where there was simply no time-dependent deformation. The thickness of the frozen earth wall was increased to match those low levels and creep deformation was mitigated. This is not always practical but the only option in some cases.

The elastic modulus directly impacts the results of the FEM analysis. A very high value will result in lower deformations but much higher stresses, while lower values will result in lower stresses (thus affecting this approach to factor of safety determination) but increased deformation. Values for the elastic modulus can be obtained from the constant strain rate test. A time-dependent modulus was defined by Klein (1981) and is sometimes used.

3 CASE HISTORIES

3.1 *Verglas, Rouyn-Noranda Quebec*

The Quemont Mine located approximately 626 km northeast of Toronto required a large shaft excavation. The approach included a 61-m-diameter, 30-m-deep excavation support by ground freezing. One of the soil strata was a soft, water-bearing clay. The clay had a very high water content and would be susceptible to time-dependent creep deformation when frozen.

A series of constant strain rate and constant stress creep tests were conducted. Using the method previously described, it was determined that time-dependent deformation did not occur when stress levels in the laboratory were 1,000 kN/m2 (Sopko et al. 2012) at -10°C.

Several iterations of the FEM stress analysis were conducted to determine the required frozen earth dimensions.

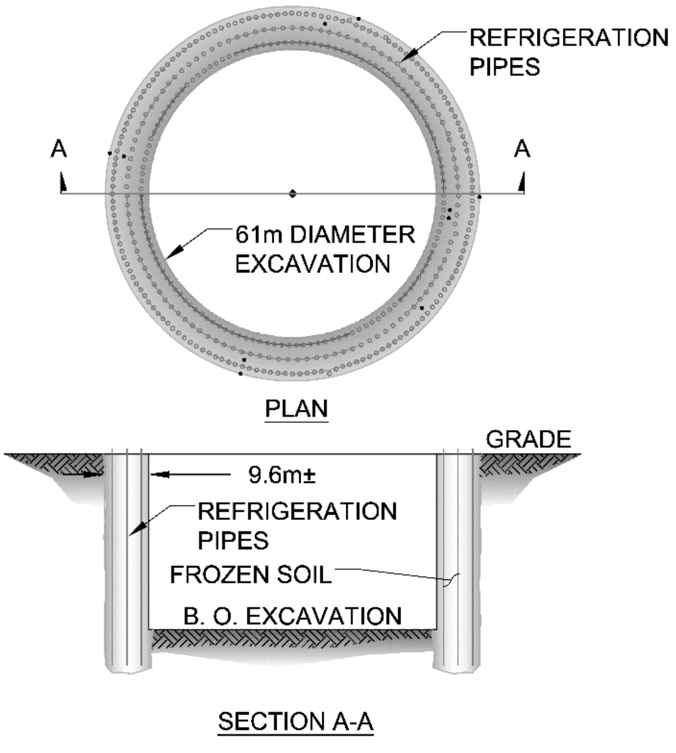


Figure 9. Ground freezing for Verglas.

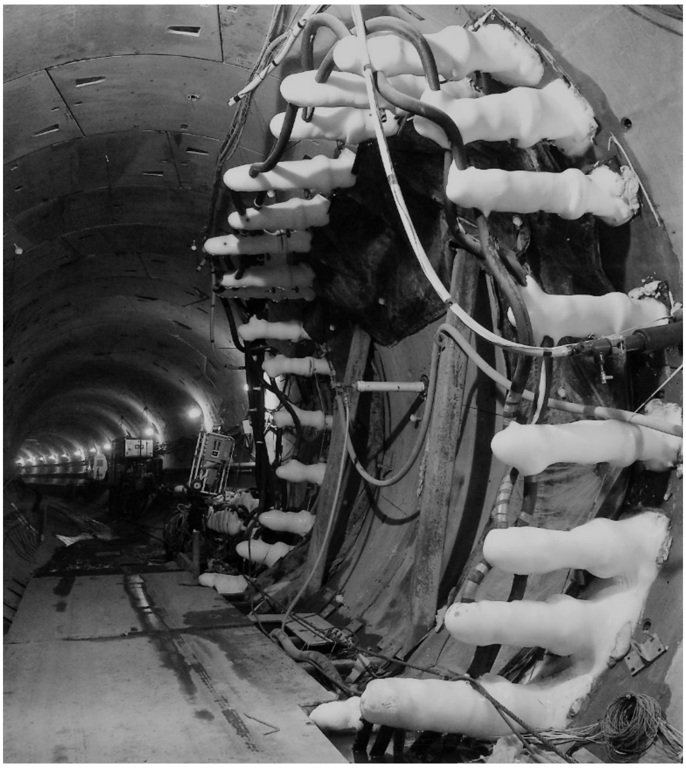


Figure 10. Ground freezing system CP-23.

The analyses concluded that with a frozen wall thickness of 9.1 m, maximum internal stress would be on the order of 621 kN/m^2 as shown in Figure 9.

One of the problems with a project of this size was the need to insulate the exposed fact of the frozen wall when excavated. The insulation costs were prohibitive, requiring the contractor to complete the excavation from November 1 until March 31 when higher ambient temperatures would induce melting. This permitted five months of loading on the frozen earth structure.

Deformation of the frozen earth wall was monitored using three inclinometers drilled and installed within the frozen wall, as well as several monitoring points on the face of the wall. During the entire five months of excavation, there was no measurable movement, indicating the design approach was a success.

3.2 *Frozen cross passages, Northgate Link Tunnel, Seattle U.S.A.*

Seattle's Northgate Link project had 5.5 km of twin tunnels. There were 23 cross passages through water-bearing, unconsolidated soils. Ten of these cross passages were completed using ground freezing for temporary earth support and ground water control. Cross Passage 23 (CP-23) had been planned to use a conventional system of dewatering and bracing for elevation when unexpectedly unstable and running silt was encountered. Ground freezing was selected as the method for support as shown in Figure 10.

Unlike the previous project, a temporary shotcrete liner was installed at approximately 1 m intervals as the excavation was completed. This lining mitigated any potential time-dependent deformation. The design was based on the constant strain rate test conducted at -10°C, with a strain rate of 0.1 percent per minute). Results yielded a peak axial stress of 5.2 MPa and a tangent modulus of 1.5 MPa.

A three-dimensional finite element analysis was completed as shown in Figure 7. This analysis yielded a maximum stress of 0.148 MPa using the following equation for determining the factor of safety:

$$F.S. = 5.2/.148 = 35 \quad (6)$$

The frozen earth wall had a thickness of 2 m governed by spacing of the individual refrigeration pipes at 1 m. A smaller thickness and more reasonable factor of safety would not be practical. It should be noted that if a time-dependent analysis was used, so would a much lower compression strength.

4 CONCLUSIONS

This paper has reviewed previously used methods of analysis and current state of the practice methods that have been highly successful. The convention methods incorporated a high level of conservatism in the assumptions and equations. The industry never established a standard for incorporating the factor of safety. Previous methods used high factor of safety values to compensate for unreliable equipment and non-standardized testing methods. Current equipment and test methods are very reliable and do not require consideration while determining the factor of safety. The method of relating the time-dependent compressive strength of a frozen soil to the internal hoop stresses of a frozen earth structure has proven to be safe and reliable.

REFERENCES

Andersland, O.B., Berggren, A.L., Fish, A., Fremond, M., Gallavresi, F., Gonze, P., Harris, J., Jessberger, H.L., Jordon, P., Klein, J., Maishman, D., Ohrai, T., Rebhan, D., Ryokai, K., Sego, D.C., Shuster, J.A., Thimus, J.F., Changsheng, W. & Williams, P. 1991. Frozen ground structures – basic principles of design. *Ground freezing 91*; *Proc. intern. symp., Beijing, 10-12 September 1991.*

ASTM (American Society for Testing and Materials). 2011. *D-5520 Laboratory determination of creep properties of frozen soil samples at constant rate of strain.* West Conshohocken: ASTM Press.

ASTM (American Society for Testing and Materials). 2011. *D-7300 Laboratory determination of strength properties of frozen soil samples by uniaxial compression.* West Conshohocken: ASTM Press.

Domke, O. 1915. On the stresses in a frozen cylinder of ground used for shaft sinking (in German). *Glückauf:* 51-47.

Harris, J.S. 1995. *Ground freezing in practice.* London: Thomas Telford Services Ltd.

Klein, J. 1981. Dimensioning of ice-walls around deep freeze shafts sunk through sand formations under consideration of time (in German). *Glückauf-Forschung* 43: 112-120.

Sanger, J.S. 1968. Ground freezing in construction. *Journal of the Soil Mechanics and Foundations Division* 94 (1): 131-158.

Sopko, J.S. 1990. New design method for frozen earth structures with reinforcement. *Unpublished Ph.D. Thesis.* East Lansing: Michigan State University.

Sopko, J.S. Braun, B. & Chamberland, R., 2012. *Largest frozen ground excavation in North America for excavation of crown pillar; Proc. third intern. conference on shaft design and construction, London, 24-26 April 2012.*