SITE INVESTIGATION FOR SHAFT SINKING PROJECTS REQUIRING ARTIFICIAL GROUND FREEZING (AGF) FOR GROUND STABILITY CONTROL AND OR GROUNDWATER INGRESS PREVENTION

*F. A. Auld
Retired Consultant
Doncaster UK
*(*Corresponding author: faauld@btinternet.com)*

S. Hashemi
Vice-President
Iranian Tunnelling Association (IRTA)
Tehran, Iran

J. A. Sopko
Director-Ground Freezing
Keller-North America
New Jersey, USA

ABSTRACT

This paper sets out the information required from a site investigation necessary for the successful design and construction of shafts which need Artificial Ground Freezing (AGF) for ground stability control and or groundwater ingress prevention during sinking. The information embodied in it will be incorporated into the International Tunnelling and Space Association (ITA) Shaft Design and Construction Working Group No. 23 Site Investigation document to be published.

KEYWORDS

Shafts, Frozen ground strength and deformation, Frozen ground thermal analysis

INTRODUCTION

For the successful design and construction of shafts, either shallow or deep, requiring Artificial Ground Freezing for ground stability control and or groundwater ingress prevention, it is essential to obtain accurate information from a site investigation. The intention of this paper is to set out the requirements for such an investigation. It is not intended to detail all the processes which are involved in producing the data but is a guide to what is needed.

Shallow shafts down to about 100m in depth, which can be constructed using civil engineering methods from the surface, pass generally through soils such as sands, silts and clays which require AGF for both strengthening of the ground and prevention of groundwater ingress during sinking. Deeper shafts, such as mine shafts, generally pass through more competent rock strata and AGF is only needed for water stopping.

The two main areas where the site investigation for ground freezing projects needs to be focussed are firstly, for the determination of frozen ground strength and deformation under load (creep) and secondly, to be able to analyse where the frozen front has reached at any particular time for comparison with measured values during construction (ground thermal analysis).

Site Investigation for Soils (sands, silts, clays) to Determine Frozen Ground Strength and Deformation Under Load (Creep) Properties

Minimum Water Content for Freezing

The minimum water in the ground for freezing needs to be 10% by weight or greater. However, to guarantee the freeze the ground should be fully saturated.

Handling, storage and machining of frozen soil specimens prior to testing

ISGF Working Group 1 (1992) provides details for these aspects. Handling of undisturbed frozen soil samples and remoulded test specimen preparation are covered. The procedure for machining and preparation of the specimens for testing is described.

Freezing of the sample

There are two basic methods of sample freezing:

1. Multiaxial Freezing

The sample is frozen from all its surfaces simultaneously. This produces a frozen sample with a massive soil-ice structure and is most often used for preparing specimens for testing related to artificial ground freezing. The freezing is completed in a triaxial cell that has a displacement transducer on the end of the specimen and a radial caliper transducer around the sample circumference to measure the volumetric expansion during freezing.

2. Uniaxial Freezing

The sample is frozen from one surface and a free water supply is available to allow the sample to expel or take in water during freezing. This produces a sample with a massive soil-ice structure or a layered structure, depending on freezing rate. Such samples are most often used for research purposes.

ISGF Working Group 1 (1992) describes procedures for both methods.

Properties for the determination of frozen ground strength and deformation under load (creep)

ISGF Working Group 2 (1992) sets out the basic principles for design where these properties are required.

1. Unconfined Compressive Strength (UCS)

Uniaxial compression tests are performed generally at -10 or -20°C. The strain rate is 1% per minute related to the initial height of the test specimen (see Figure 1).

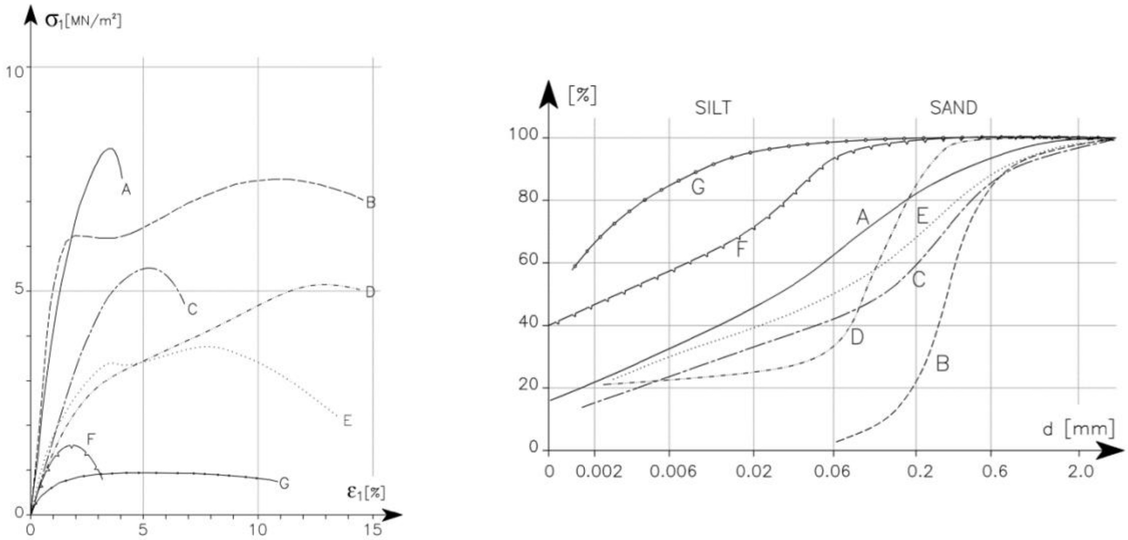


Figure 1. Stress/strain curves for frozen soil samples resulting from UCS tests with a strain rate of 1%/min. at a temperature of -10°C and the corresponding grain size distribution curves – ISGF Working Group 2 (1992)

2. Shear Strength and Cohesion

The shear strength of frozen soil is commonly investigated in triaxial compression tests. Tests are performed at the same test temperatures as for the UCS tests. In this case, the deformation rate is chosen as 0.1% per minute of the initial specimen height. Figure 2 shows the p-q diagram for frozen sand and clay, soil B and soil F respectively at -10°C.The shear parameters ϕ_f and c_f are also indicated.

The angle of internal friction ϕ_f for frozen soil is equal or slightly smaller than for unfrozen soil. Cohesion c_f for frozen soil in general is much larger than for unfrozen soil.

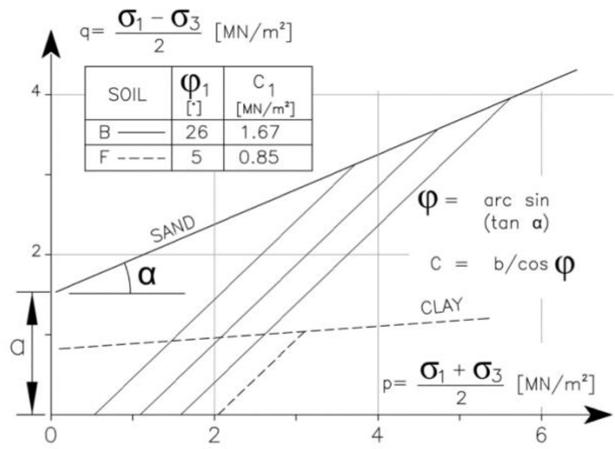


Figure 2. Shear strength diagram for frozen sand and clay at a temperature of -10°C and a strain rate of 0.1%/min. – ISGF Working Group 2 (1992)

While the unconfined compressive strength of frozen soil has been used in most closed form solutions, it has been shown to be conservative. It does not consider the confining stresses that significantly increase the shear strength. Shear stress can be expressed by:

$$s = c + p \tan(\phi) \quad (1)$$

where s = shear strength

c = cohesion

p = compressive stress

ϕ = angle of internal friction

Today's numerical models use the cohesion and angle of internal friction to create Mohr Coulomb material model in the analysis.

The time dependent cohesion for frozen soil is related to the time dependent UCS q_f and is expressed as:

$$c = q_f (\sin \phi)/2(\cos \phi) \ldots\ldots\ldots\ldots(2)$$

3. Creep Behaviour of Frozen Soil

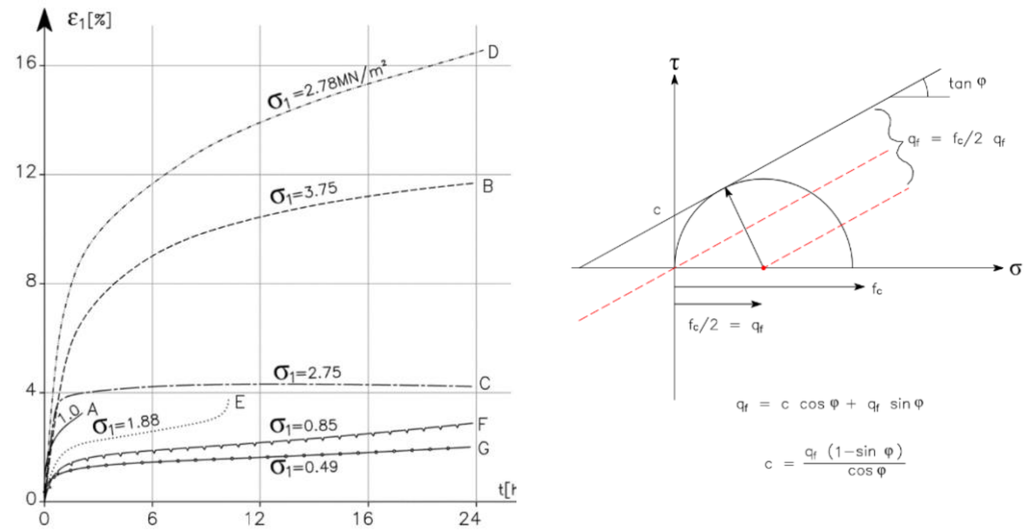


Figure 3. Creep curves of soils in Figure 1 – ISGF Working Group 2 (1992)

At constant stress the stress-strain behaviour of frozen soil is strongly dependent on time. This effect can be shown in UCS creep curves with different stress levels σ_1. Figure 3 gives the creep curves for the soils A to G in Figure 1 at temperature -10°C. The vertical pressure σ_1 is equal to 50% of the instantaneous UCS value at the same temperature. The influence of the soil type is obvious.

The creep behaviour can be described by Equation (3):

$$\epsilon_1 = A.\sigma_1^{B}.t^{C} \quad \text{……………………………………………………………}(3)$$

with ϵ_1 = creep induced deformation

σ_1 = constant vertical pressure

t = time

A, B, C = creep parameters of the soil

The creep parameters A, B and C of the soils in Figures 1 and 3 are given in Table 1.

Table 1. Creep parameters A, B and C for the soils in Figures 1 and 3 – ISGF Working Group 2 (1992)

Soil	Creep Parameters A	B	C
	$\left(\frac{m^2}{MN}\right)^B \bullet h^{-C}$	(−)	(−)
A	$7{,}3\times10^{-4}$	1,48	0,13
B	$3{,}4\times10^{-3}$	2,10	0,25
C	$4{,}2\times10^{-3}$	2,20	0,072
D	$8{,}2\times10^{-3}$	2,25	0,24
E	$5{,}0\times10^{-3}$	2,15	0,095
F	$2{,}0\times10^{-2}$	2,14	0,20
G	$5{,}8\times10^{-2}$	3,40	0,48

The creep parameters A, B and C are evaluated by the following method. It starts by determining the UCS of the soil. However, it is not easy to obtain a good representative value for the soil UCS due to difficulties in obtaining a sufficient amount of soil for testing and sometimes having to reconstitute the soil sample. A minimum of four UCS values is recommended for each type of soil for averaging out purposes. After the UCS value is obtained, samples are tested under creep conditions at different constant stress levels below the UCS value to produce the Creep Strain versus Time curves as shown in Figure 4.

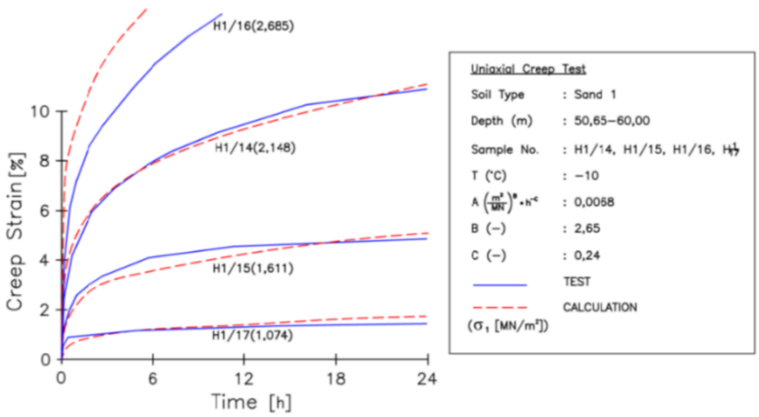
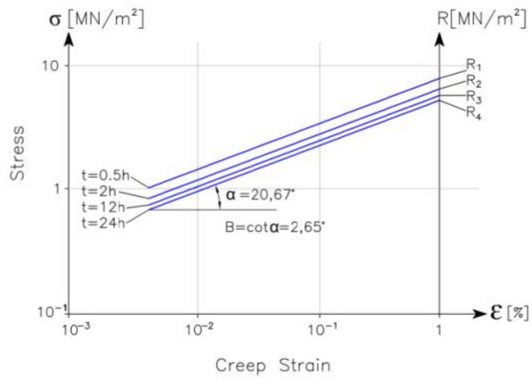


Figure 4. Creep Strain versus Time curves at different constant stress levels below the UCS value



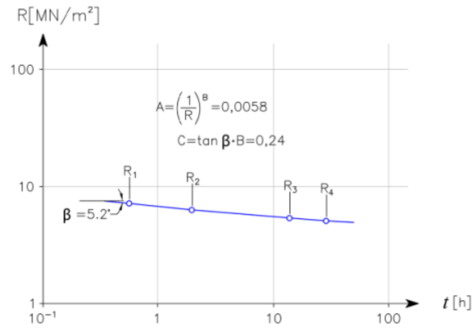


Figure 5 (a)
Stress versus Creep Strain graphs plotted logarithmically

Figure 5 (b)
R function versus Time plotted logarithmically

From the constant stress Creep Strain versus Time Curves, plots of Stress versus Creep Strain plotted on logarithmic scales on both axes at various time intervals of t = 0.5 hours, t = 2 hours, t = 12 hours and t = 24 hours produce the straight-line graphs shown in Figure 5 (a). The value of B = cot α where α is the slope angle of the straight lines. From Figure 5(a), the R functions obtained plotted against Time in Figure 5(b), again on logarithmic axes, produce the A and C values. $A = (1/R)^B$ and C = tan β.B where β is the slope angle of the straight line. Figure 6 is a photograph of the creep testing apparatus.

Figure 6. Creep testing apparatus

The creep parameters A, B and C are used to get the time dependent UCS $q_f(t)$. It is assumed that the elapsed time until failure depends on the applied load but the deformation at failure $Є_f$ itself does not change much with time (see Figure 7).

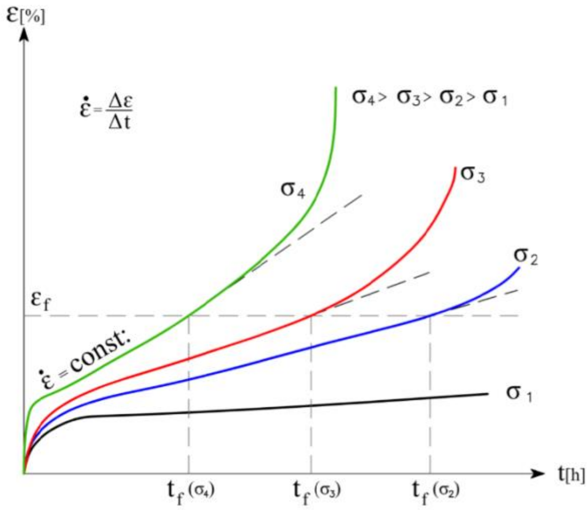


Figure 7. Idealized creep curves - ISGF Working Group 2 (1992)

Based on this assumption and with the free-standing time of the project the time dependent UCS can be estimated with Equation (4).

$$q_f(t) = (Є_f/A/t^C)^{1/B} \quad \ldots\ldots (4)$$

with $q_f(t)$ = time dependent UCS

$Є_f$ = deformation at failure (this is normally assumed to be about 6%)

t = time

A, B, C = creep parameters of the soil

In a similar way the time dependent Young's Modulus of Elasticity is calculated with Equation 5.

$$E(t) = (Є^{(1-B)}/A/t^C)^{1-B} \quad \ldots\ldots (5)$$

As the Young's Modulus of Elasticity expressed by Equation (3) takes into account creeping of the soil, the values which are obtained by using Equation (3) are much smaller than those directly taken from a σ-Є diagram of a short-term test.

4. Water Content

The strengthening component in frozen soil is water which is converted into ice during the freezing process. In the case of cohesionless soil above ground, the water content is relatively low with the consequence that measures may have to be taken to increase the water content (see Figure 8).

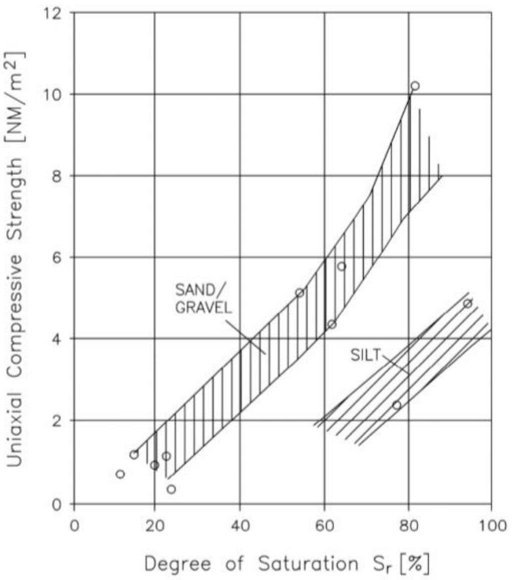


Figure 8. UCS as a function of the degree of saturation at T = -10°C - ISGF Working Group 2 (1992)

5. Salinity

Salt content in the soil water depresses the freezing temperature point which for sea water is approximately 1.8°C. The in situ salinity controls the reduction in strength of the frozen soil and the soil water salinity should be checked for each project.

6. Ground Water Movement

Groundwater velocities of up to 1m/day at brine temperatures are not normally a problem but are dependent on freeze pipe spacing and groundwater temperature It is possible to freeze higher velocities with liquid nitrogen, but it is often more practical to reduce the permeability of the soil with permeation grouting. Higher velocities cause difficulties for soil freezing.

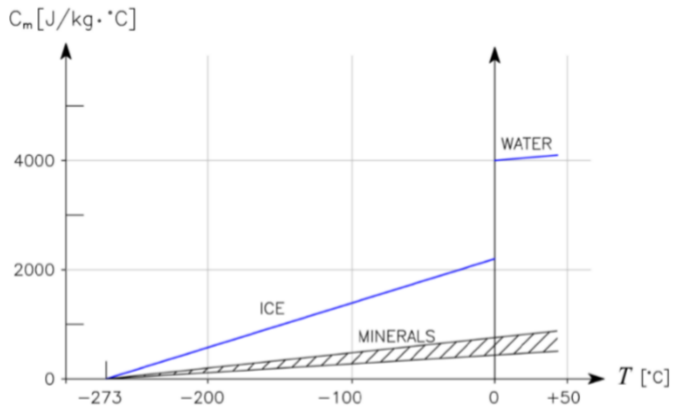


Figure 9. Mass heat capacity versus temperature – ISGF Working Group 2 (1992)

Site Investigation for Soils (sands, silts and clays) to Carry Out Ground Thermal Analysis

For the purpose of carrying out a thermal analysis of the frozen ground, the two main properties needed are one, the heat capacity and two, the thermal conductivity.

<u>Determination of the Heat Capacity</u>

The heat capacity C per unit volume of soil is the heat energy required to raise the temperature of this unit volume by 1°C. Figure 9 shows the heat capacity of ice/water and of minerals versus temperature.

Heat capacity is a product of the mass specific heat C_m (kJ/kg°C) and the density γ_d (kg/m^3). The average volumetric heat capacities for unfrozen (C_u) and frozen (C_f) soil are given in Equations (6) and (7):

$$C_u = \gamma_d.(C_{ms}+C_{mw}.w) \qquad (6)$$

$$C_f = \gamma_d.(C_{ma}+C_{mw}.w_u+C_{mi}.(w-w_u)) \qquad (7)$$

With C = vol. heat capacity (kJ/m^3°C)

C_{ms} = heat capacity of soil particle 0.7 to 0.84J/g°C

C_{mw} = heat capacity of water 4.2J/g°C C_{mi} = heat capacity of ice 2.1J/g°C

w = total water content (weight ratio) w_u = unfrozen water content

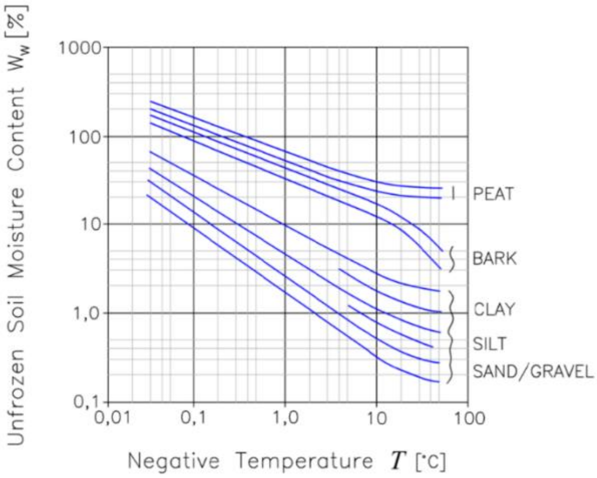


Figure 10. Unfrozen water content – ISGF Working Group 2 (1992)

The unfrozen water content for different soils depending on temperature are given in Figure 10.

<u>Thermal Conductivity</u>

Thermal conductivity is a measure of the quantity of heat that will flow through unit area of the soil of unit thickness in unit time under a unit temperature gradient. Since the thermal conductivity of ice is much higher than that of water, thermal conductivity values for frozen soil are usually larger than those for unfrozen soil.

Andersland and Ladanyi (1994) provide graphs for the determination of the thermal conductivities for sands and gravels (Figure 11) and silts and clays (Figure 12). These are based on the dry density of the soil, γ_d, the water content, w(%) and the percentage saturation of the soil.

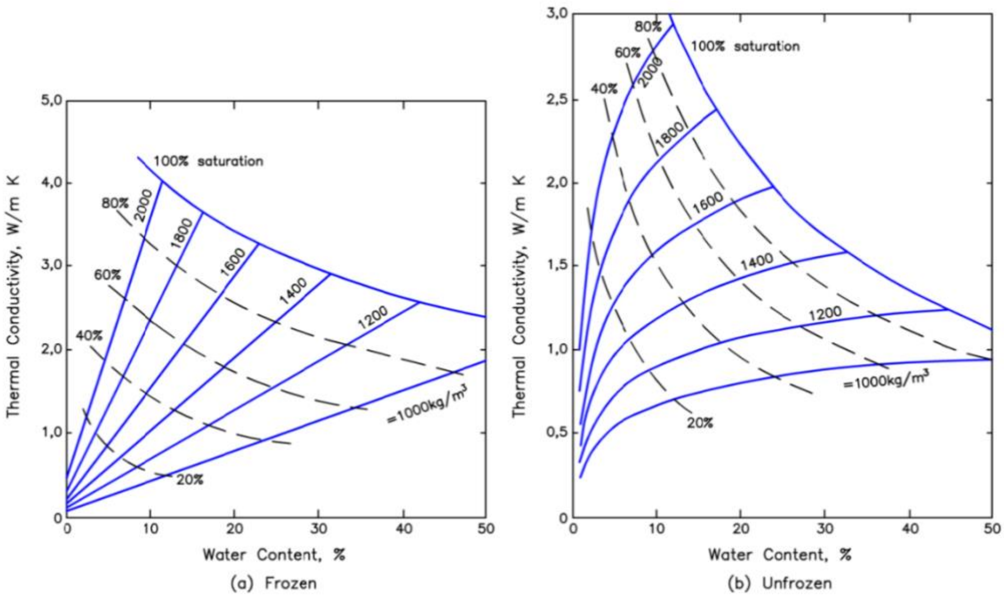


Figure 11. Average thermal conductivities for sands and gravels – Andersland and Ladanyi (1994)

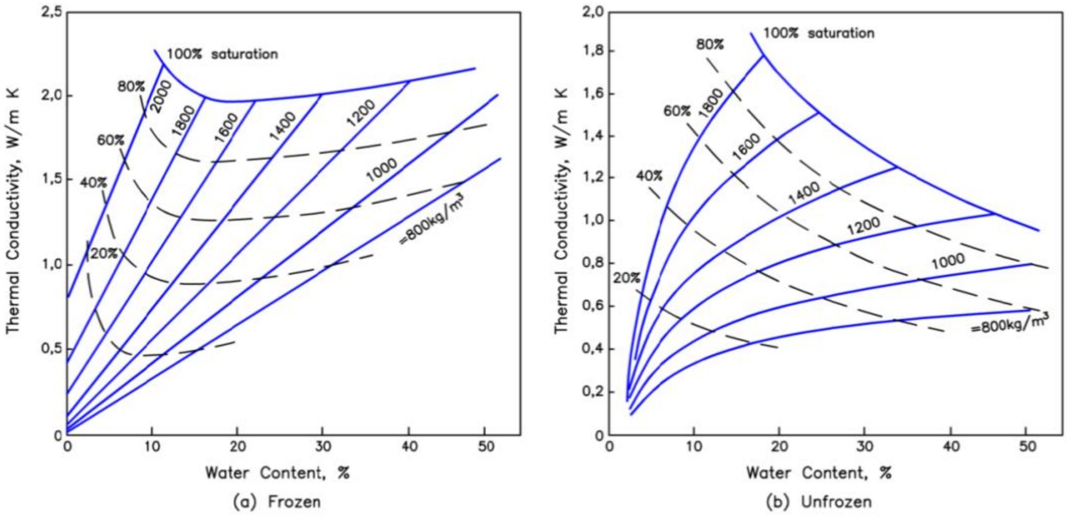


Figure 12. Average thermal conductivities for silts and clays – Andersland and Ladanyi (1994)

ISGF Working Group 2 (1992) also quote values for soil thermal conductivities based on the dry density of the soil and percentage saturation (see Table 2).

Table 2. Thermal conductivities for sand clay and peat – ISGF Working Group 2 (1992)

		Unfrozen state: k_e Degree of saturation			Frozen state: k_f Degree of saturation		
	0_σ \ S_t	20%	60%	100%	20%	60%	100%
Sand	1400 1600 1800	1,05 1,28 1,52	1,40 1,74 1,59	1,63 2,03 2,44	0,70 0,81 1,05	1,80 2,09 2,38	2,38 3,37 3,66
Clay	1400 1600 1800	0,58 0,64 0,70	1,05 1,22 1,40	1,28 1,51 1,80	0,47 0,52 0,58	1,28 1,28 1,40	1,98 1,98 1,98
Peat	300 to 400	0,12	0,29	0,47	0,12	0,47	1,05

<u>In situ Measurements of Heat Capacity and Thermal Conductivity</u>

In situ measurements of Heat Capacity and Thermal Conductivity can be made during the investigative boring program (providing the undisturbed sample is large enough) or during shaft sinking to confirm the values assumed in the design before construction. A Tempos Meter is used for this purpose (see Figure 13).

Figure 13. Tempos Meter

Heave and Thaw Settlement

In sandy soils, where the permeability is such that groundwater can be expelled through the soil in an unrestricted manner in front of the freeze wall as it develops during the expansion of the retained water in the freeze wall, ground heave generally does not occur. However, with clay and silty soils, the movement of the groundwater through the soil is restricted, due to the low permeability, and the retained water creates the heave expansion when it freezes. Estimates of heave can be made based on the amount of retained water in the soil and an expansion of 9%. Thaw settlement generally does not recover fully after the frozen soil thaws.

SUMMARY COMMENTS

This paper sets out the information required from a site investigation necessary for the successful design and construction of shafts which need Artificial Ground Freezing (AGF) for ground stability control and or groundwater ingress prevention during sinking. The information embodied in it will be incorporated into the International Tunnelling and Space Association (ITA) Shaft Design and Construction Working Group No. 23 Site Investigation document to be published.

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