İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

ARTIFICIAL GROUND FREEZING AND EFFECT OF FREEZING SPEED OVER FROST HEAVE

M.Sc. Thesis by

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Department : **Civil Engineering**

Programme: **Soil Mechanics and Geotechnical Engineering**

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PREFACE

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Mayıs 2006 Mustafa KOÇ

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ABBREVATIONS

AGF	: Artificial Ground Freezing
AFTES	: French Tunneling and Underground Engineering Association
BOKU	: Boden Kultur University of Vienna
LIN	: Liquid Nitrogen
ML	: Clayey Silt
NATM	: New Austrian Tunnelling Method
TU	: Technical University
U2	: U-Bahn 2 (Subway Line 2)
U3	: U-Bahn 3 (Subway Line 3)

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LIST OF SYMBOLS

A	: Area
a	: Thermal diffusivity
C	: Heat Capacity
$\mathbf{c_m}$	: Volumetric heat capacity
$\mathbf{G_s}$	: Specific gravity of solid soil particles
$\mathbf{I_c}$	: Consistency Index
$\mathbf{I_p}$	: Plasticity Index
$\mathbf{i_{ice}}$	: Relative iceness
K	: Thermal conductivity
L	: Latent Heat
n	: Porosity
$\mathbf{S_i}$	: Degree of Saturation
T	: Temperature
$\mathbf{T_f}$	: Initial freezing temperature
V	: Volume
W	: Weight
$\mathbf{W_s}$	: Weight of dry soil
$\mathbf{W_w}$	: Weight of water
$\mathbf{w_l}$	: Liquid Limit
$\mathbf{w_p}$	: Plastic Limit
$\alpha_{(n)}$	: Coefficient
ρ_f	: Bulk density of a frozen soil
ρ_{df}	: Dry density of a frozen soil
ω_t	: Total water content
ω_i	: Ice water content
ω_{uw}	: Unfrozen water content

ARTIFICIAL GROUND FREEZING
AND EFFECT OF FREEZING SPEED OVER FROST HEAVE

SUMMARY

Artificial ground freezing is a technique that has been used extensively for groundwater control and excavation support in the underground construction industry for over a century. The process involves the circulation of a refrigerated coolant through a series of subsurface pipes to convert soil water to ice, creating a strong watertight material. Frozen soil is nearly as strong as concrete and is impermeable, making it ideal for shoring and groundwater cutoff.

There are many ways to freeze the ground, including liquid nitrogen, brine, carbon dioxide. In practice two methods of freezing have been used;

- Slow-rate freezing or closed-loop systems using coolants such as calcium chloride brine, ethylene glycol, carbon dioxide, or ammonia

- Fast freezing or open-loop systems using expendable coolants such as liquid nitrogen

When water is transformed into ice, there would be volume deformation in the soil and it can be seen that an ice lens is formed at 0º C. Ice has 1.09 times volume than the water. It forms a pressure in the soil. If the pressure in the ice exceeds the pressure of overburden material and other pressures, the soil begin to expand. When there is sufficient ice pressure, the new ice lenses will be formed. This upward movement in the site is called frost heave.

In this work the main idea of Artificial Ground Freezing (AGF) is explained, and the method is discussed. In order, soil properties and phase relations in freezing, thermal soil properties and method is given and finally a case study in Vienna Subway System U2 is given as an example.

This thesis reviews and examines the ground freezing as a stabilization method for foundation engineering and tunneling. There were some experiments which were carried out in TU Vienna in order to understand the behavior of frost heave due to relation with water content, and the effect of speed of freezing on heaving. The correlations of the test results showed that when fast freezing put in practice, growing of ice lenses would be less compared to slow rate freezing. This means there will be more heaving in slow rate freezing. Also heaving phenomena is correlated with the water content ratio so that if water content ratio increases, the amount of heaving would increase.

1. INTRODUCTION

At 23% of the world, freezing occurs naturally in specific seasons. In aspect of engineering, freezing is related with the material used in construction and the soil properties. It usually effects the pavements, roads and the natural or man-made materials used in construction. Although the freezing process has been seem to be a big problem, some French engineers changed the natural fact to their own favour in a mine shaft in soft soil conditions at 1850`s. After that engineers began to think about the advantages of freezing.

By means of the thought of acknowledgment of the man made freezing, Civil and Mining Engineers began to use freezing process in their works. And it is used to be named as AGF, Artificial Ground Freezing, in Europe. Despite the fact that it was patented by H. Potesh in Germany, it was first used in South Wales in 1862 in vertical openings. In the development of AGF the main process that Poetch used is taken. Poetsch's main idea involves the circulation of a refrigerated coolant through a series of subsurface pipes to extract heat, thus converting the soil water to ice, creating a strong, watertight material. AGF is mostly used in tunneling and open excavations in mining and civil engineering industries. The main aim is to make a safe and fast stabilization.

In this work the main idea of Artificial Ground Freezing (AGF) is explained, and the method is discussed. In order, soil properties and phase relations in freezing, thermal soil properties and method is given and finally a case study in Vienna Subway System U2 is given as an example.

This thesis reviews and examines ground freezing as a stabilization method for foundation engineering and tunneling. There were some experiments which were carried out in TU Vienna in order to understand the behavior of frost heave due to relation with water content, and the effect of speed of freezing on heaving. The correlations of the test results and the heaving phenomena are tried to be explained and results are given in conclusion.

2. GROUND FREEZING

2.1 Artificial Ground Freezing

2.1.1 What is AGF?

The primary objective in freezing is to stabilize the soil or rock in some bad conditions such as under big values of water contents. By means of freezing, load carrying capacity of the full-partially saturated soils increase and a watertight material is obtained. During ground freezing, ice between soil particles increases soil strength and decreases soil permeability. This is the most important feature of the AGF to be used in Tunnel Engineering. Besides the tunnels and shafts, AGF also is used for stabilisation of slopes, sample weak soils, construct temporary access roads, maintain permafrost overhead pipelines foundations, and below heated buildings. Briefly, AGF is one of the soil improvement method mostly used in some special engineering problems in civil and mining engineering.

As it is mentioned above, the main principle is to overcome the difficulty of ground water during excavation of a hole in ground like a pit shaft or a tunnel. The excavation work becomes difficult when the porosity of the soil and the water content of strata are a bit high. AGF process is very successful and efficient in stabilizing of fine-particled, permeable, saturated soils. Mostly AGF is used to maintain a temporary support converting the soil strata and water into an ice lenses. This is done by cooling and refrigerating of soil by means of some freezing liquids or gases. AGF can be carried out when the velocity of groundwater flow is as much as 2.0 m/day

2.1.2 Why Ground Freezing?

There are many advantages of using AGF. Some are;

1. No chemical additives are applied to the soil, so it has no long term effect on subsurface environment. After thawing process, the environment returns its pre-construction state.

2. It is a combination of sealing and static support.
3. Ground Freezing provides an absolute cut-off wall for the ground water
4. It can be installed in every ground conditions like boulders, gravel, bedrock peat clay, sand etc. (Figure 2.1) **(Harris,1995)**

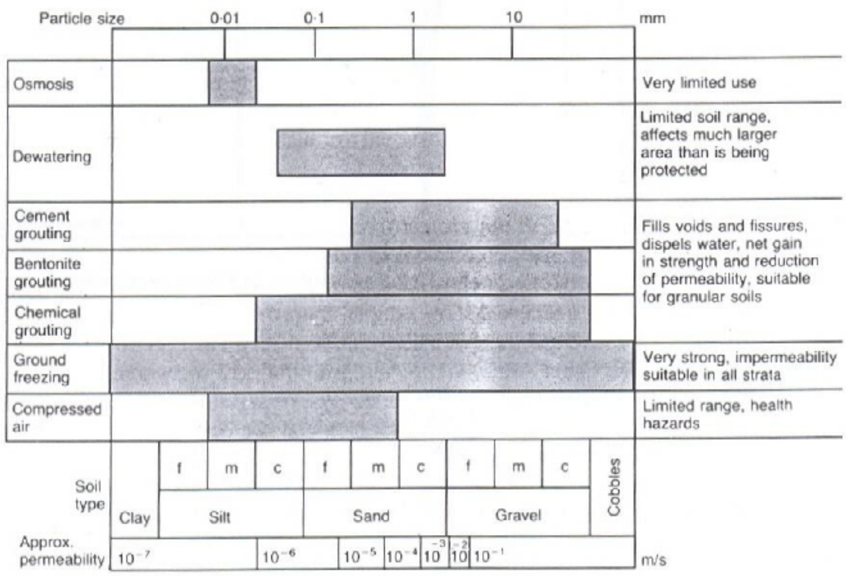


Figure 2.1: Applicability of soil improvement methods according to soil types

5. Most effective support system in hard conditions of ground water, and where groundwater can not be obstructed for the site such as in tunnels.
6. It can be turned on or off during an emergency if cooling coils are incorporated
7. The dimension of the support wall thickness can be controlled by temperature adjustments.
8. Excellent at sensitive sites near buildings or residences.(Low vibration and noiseless)
9. It may be applied during any season of the year.
10. The wall of frozen soil has a relatively great mechanical strength.
11. It can be completely removed after construction

But also it has some disadvantages:

1. Requires large complex freezing installation facilities.

2. In multilayered soil strata, calculations and design of the wall is a bit difficult.
3. Freezing and thawing may destroy the natural structure of the soil especially clayey soils.
4. Is very difficult when the groundwater flow velocity is over 2m/day.
5. It is only economic where other usual methods for stabilisation is not enough and efficient.
6. Takes long time

2.1.3 How Ground Freezing Works?

The fundamental process in ground freezing is the removal of heat from the ground to cause lowering of subsurface temperature below the freezing point of moisture in the pore spaces. The frozen structure acts as a cementing agent binding the soil particles together and supply a structural frame.

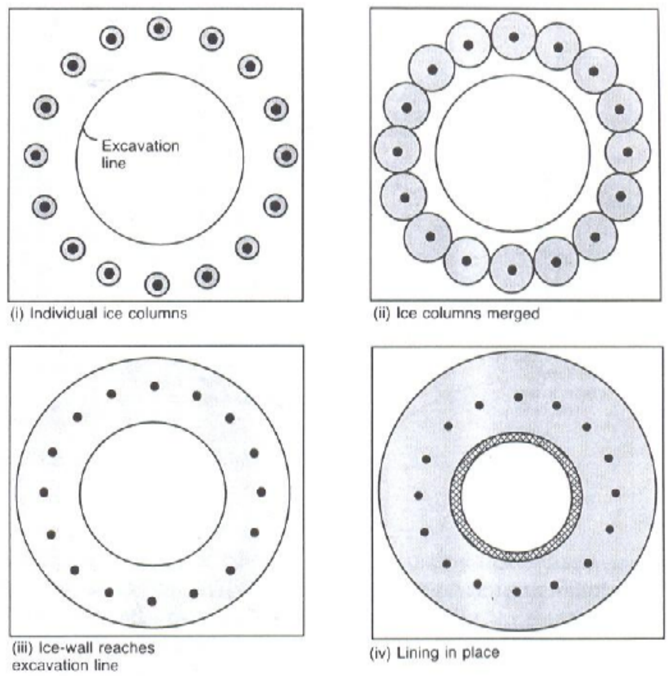


Figure 2.2: Stages of Ground Freezing

There are many ways to freeze the ground, including liquid nitrogen, brine and carbon dioxide. In practice, two methods of freezing have been used:

1) Slow-rate freezing or closed-loop systems using coolants such as calcium chloride brine, ethylene glycol, or ammonia;

A primary refrigerating circuit liquefies the fluid by means of compressors and condensers. By evaporating again, this cools the refrigerant liquid that circulates in the freezing tubes in a closed circuit. The primary liquid may be ammonia or Freon. The refrigerant liquid is generally brine with an operating temperature of between -25˚ C and -30˚ C

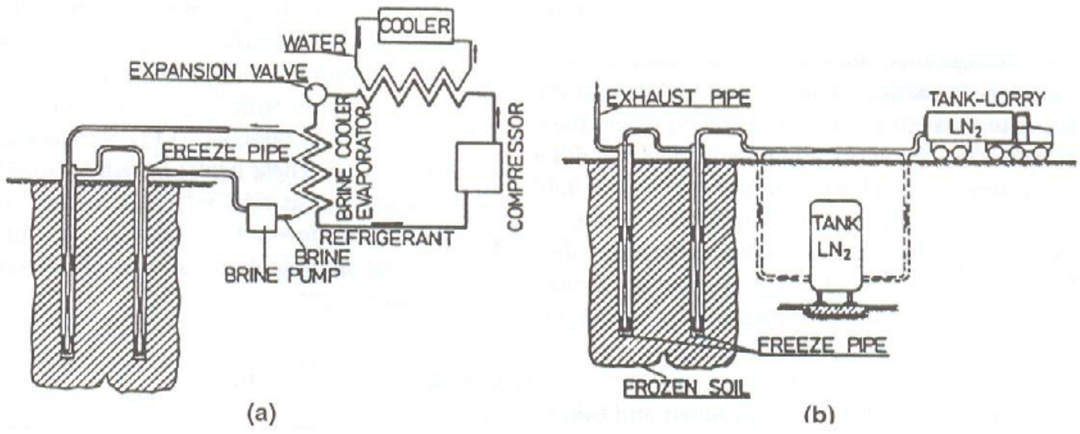


Figure 2.3: Freezing methods a) Closed-loop method b) Open Loop Method (Jessberger, 1985)

2) Fast freezing or open-loop systems using expendable coolants such as liquid nitrogen;

The refrigerant fluid is liquid nitrogen. It is transported to the site by special tanker trucks where it is kept at a temperature of -196˚ C under a pressure about 0.5 MPa. This pressure makes the flow of nitrogen through the probes. After the last probe, the nitrogen is allowed to escape to atmosphere at around -60˚. **(Harris, 1995)**

A third method can be classified as mixed method that combines the open and closed method. This consists of combining the two preceding methods, using the same freezing probes. The rapid cooling of the nitrogen is combined with the economical continued operation of the brine.

The artificial freezing process of the soil is carried out in two stages; active freezing and passive freezing;

In Active freezing required thickness of the wall of the freezing soil is built, and after that active freezing stops and passive freezing begins. The main aim of the passive freezing is to prevent the required thickness of the wall. **(Harris, 1995)**

Most ground freezing systems are quite similar in principle, with some differences in the engineering aspects of the individual sites. The single most important component of a ground freezing system is the subsurface refrigeration system (Figure 2.4), consisting of a series of refrigeration pipes, installed with various drilling techniques. The quantity, spacing, depth and size of the refrigeration pipes are unique to each site, and determined based on the thermal and hydraulic properties of soils, construction period and the cost.

Figure 2.4: Cooler Supplier (left) and connections to the site (right)

2.2 Ground Freezing in Tunnelling and NATM

As it is mentioned above ground freezing is used for sealing and statically support in tunnel engineering. While designing and in progress New Austrian Tunnelling Method, NATM, is considered. Primarily in tunnel constructions, ground freezing is taken as a view of thin sections around the outer tunnel counters in order to improve the roof and side walls.

Figure 2.5: Elements in NATM (Wire mesh, Shotcrete, bolts)

Sometimes shallow tunnels are protected by vertical solutions made from ground surface. This method is less cheap with comparison to horizontal drilling.

2.2.1 Principles of NATM

The New Austrian Tunnelling method (NATM) was developed between 1957 and 1965 in Austria. It was given its name in Salzburg in 1962 to distinguish it from old Austrian tunnelling approach. The main contributors to the development of NATM were Ladislaus von Rabcewicz, Leopold Müller, and Franz Pacher. It was used for rock tunnels for hydropower schemes. First, Rabcewicz applied for a patent in 1948 as defending that a thin temporary support by allowing the deformations is adequate till to last cover and reduce the stress and distribute to the surrounding rocks. So, the final support will be less loaded and much thinner structure. The main idea is to monitor and control the deformations during construction and minimize all risks by applying the right things in sequence. In soft ground conditions it was used in subway constructions in Germany. Now the method is used for all kind of soil conditions from rocks to soft clays.

2.2.2 Concept of NATM

NATM is known as a concept or philosophy rather than a construction method. As defined by the Austrian Society of Engineers and Architects, the NATM constitutes a method where the surrounding rock or soil formations of a tunnel are integrated into an overall ring-like support structure. Thus the supporting formations will themselves be part of this supporting structure. Nevertheless, many engineers already refer to as NATM, when shotcrete is proposed for initial ground support of an open-face tunnel. Especially with reference to soft ground, the term NATM can be misleading. In conventional tunnelling methods tunnel line is the main barrier to support loads whereas the NATM is considered as that surrounding soil or rock mass of a soil is integrated into the overall support structure.

Figure 2.6: NATM tunnelling in U2, Vienna

In NATM tunneling, idea is realized with making the supports as shotcrete, reinforced wire meshes, steel ribs, lattice girders**,** rock bolts by forming a composite structure with soil or rock mass (Figure 6.2). The composite structure will distribute the loads around the tunnel. By means of the composite structure and controlled deformations, lose in strength of the rock is prevented and carrying capacity of the rock arch is preserved. Overall this all ideas are valid with a strong monitoring and geotechnical measurements. Müller listed 22 principles of NATM. There are seven most important features on which are NATM based:

- Mobilisation of the strength of rock mass - The method relies on the inherent strength of the surrounding rock mass being conserved as the main component of tunnel support. Primary support is directed to enable rock support itself.

- Shotcrete protection - Loosening and excessive rock deformation must be minimised. This is achieved by applying thin layer of shotcrete immediately after face advance.

- Measurements - Every deformation of excavation must be measured. NATM requires installation of sophisticated measurement instrumentation. It is embedded in lining, ground, and boreholes.

- Flexible support - The primary lining is thin and reflects recent strata conditions. Used is rather active than passive support and strengthening is not by thicker concrete lining but by a flexible combination of rock bolts, wire mesh and steel ribs.

- Closing of invert - Important is quickly closing of invert and create load-bearing ring. It is crucial in soft grounded tunnels where no section of tunnel should be left open even temporarily

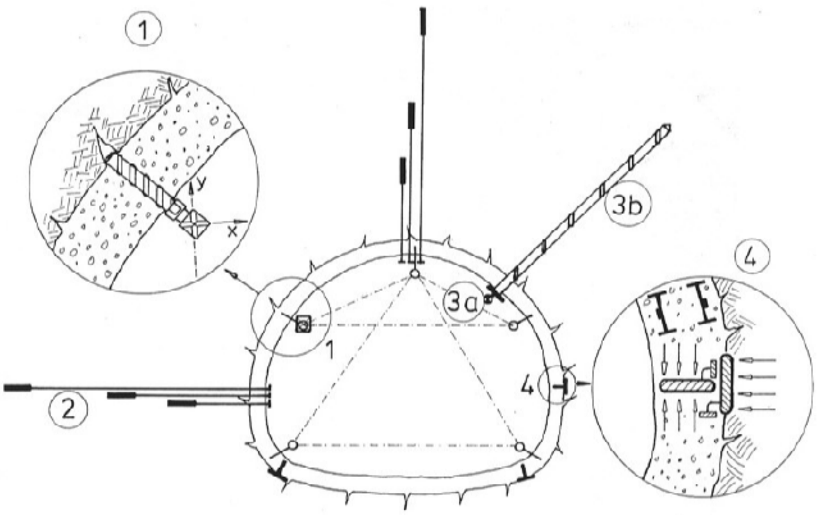
- Contractual arrangements - Since the NATM is based on monitoring measurements, changes in support and construction method are possible. This is possible only if the contractual system those changes enables

- Rock mass classification determines support measures - There are main rock classes for tunnels and corresponding support. These serve as the guidelines for tunnel reinforcement. **(KGM Training Lectures, 1994)**

2.2.3 Observations of Tunnel Behavior

One of the most important factors in the successful application of observational methods like NATM is the observation of tunnel behavior during construction. Monitoring and interpretation of deformations, strains and stresses are important to optimize working procedures and support requirements, which vary from one project to the other. In-situ observation is therefore essential, in order to keep the possible failures under control.

Considerable information related to the use of instruments in monitoring soils and rocks are available from instrument manufacturers. Figure 2.7 shows example instrumentation in a tunnel lined with shotcrete.



Legend	Measuring objective	Instrument
1	Deformation of the excavated tunnel surface	Convergence tape Surveying marks
2	Deformation of the ground surrounding the tunnel	Extensometer
3	Monitoring of ground support element ‘anchor’	Total anchor force
4	Monitoring of ground support element ‘shotcrete shell’	Pressure cells Embedments gauge

Figure 2.7: Examples of NATM tunnel measurement equipment

2.2.4 The Fenner-Pacher curve

The idea of the ground response curve, or otherwise the “characteristic curve” of the ground mass is considered to originate from Fenner (1938) who also proposed a closed form solution for the problem of a circular opening in elastoplastic ground. It was later supported by Pacher (1963) in Austria, as a major design component behind the New Austrian tunneling method. The method was further improved by many researchers such as Gaudin et al. (1981), Guenot et al. (1985), Sulem et al. (1987), Deffayet and Robert (1989), Panet (1993), Panet (1995), Carranza-Torres and Fairhust (1999), Carranza-Torres and Fairhust (2000), Brown and Bray (1982), Brown et al. (1983), Oreste and Peilla (1996), Oreste and Peilla (1997), Oreste (2003), Peila and Oreste(1995) and it has received acknowledgment by the French Tunneling and Underground Engineering Association (AFTES, 1984) for application in rational tunnel designs. The recommendations by AFTES are described by Panet (2001). (http://amadeus.cee.vt.edu)

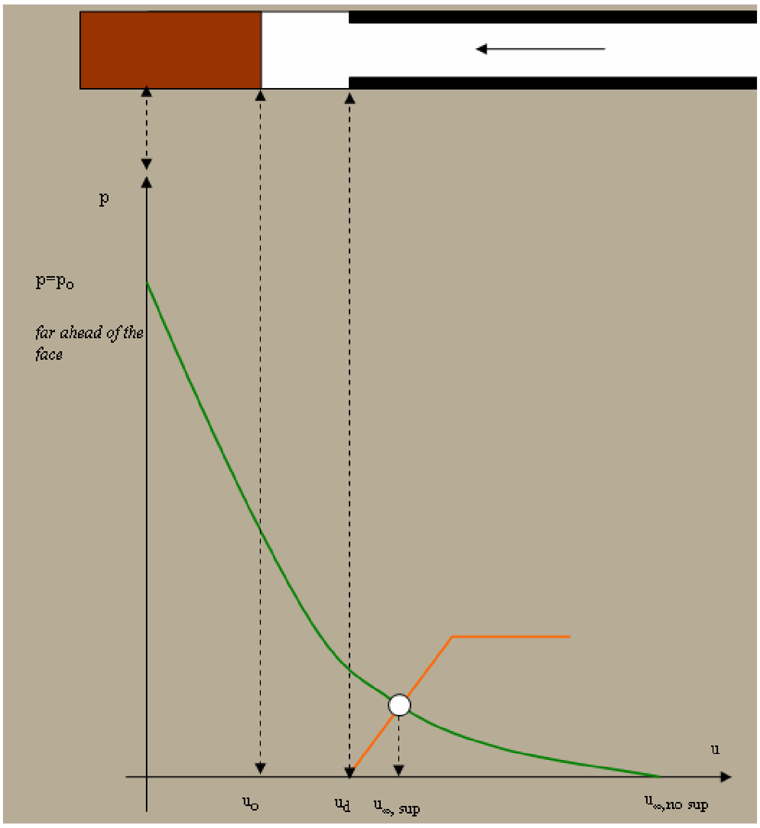


Figure 2.8: Fenner- Pacher Curve (http://amadeus.cee.vt.edu)

The Fenner-Pacher curve shows the relationship between the deformation ΔR/R and required support resistance P_i. Simplistically, the more deformation is allowed, the less resistance is needed. In practice, the support resistance reaches a minimum at a certain radial deformation, and support requirements increase if deformations become excessive.

Fenner-Pacher-type diagrams can be generated to help evaluate the support methods best suited to the conditions. In terms of analysis, it is convenient to carry out quick and simple convergence-confinement calculations with and without support. Resistances of shotcrete, steel reinforcement, and anchors/bolts can be calculated. Functions of support elements, radial stresses, and support resistances of inner and outer arches, deformations, and failures can be analyzed with respect to time.

3. SOIL

3.1 Frozen Soil

3.1.1 Frozen Soil Properties

Ice is defined as the solid state of water, and one of the participants in frozen soil. When the temperature of soil reaches the 0 degrees C or 32 degrees F ice begin to form in soil strata. Ice may be found in soil in different ways and sometimes can not be seen by naked eyes.

3.1.2 Classification System

Tsytovich (1975) classified frozen soil with the description of ice content and physical state categories. According to this classification, three main groups are:

- Hard frozen soils-low temperature (incompressible)
- Plastic frozen soils-warm temperature (with high unfrozen water)
- Friable frozen soils-coarse grained soils (not cemented by ice)

3.2 Phase Relations

3.2.1 Frozen Soil Components

Frozen soil can be thought to be a multiphase complex material as unfrozen soil. But frozen soil has one more phase then unfrozen soil. The phases are:

1. Solid phase, Solid particles
2. Plastic viscous phase or ice
3. Liquid phase or unfrozen water
4. Gaseous Phase or air vapour

The phases may be determined in terms of the physical parameters defined below. Neglecting the vapour phase, the total water content ω_t is given by:

$$\omega_t = \omega_i + \omega_{uw} \qquad \textbf{(3.1)}$$

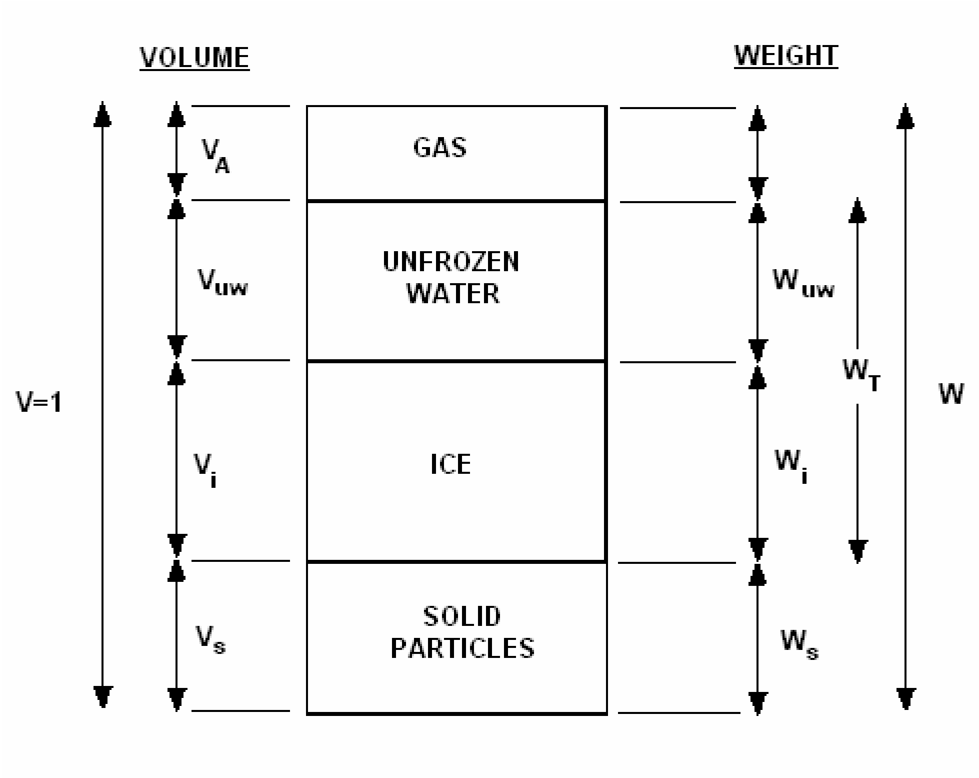


Figure 3.1: Constitution of frozen soil

where

ω_i = Ice water content

ω_{uw} = Unfrozen water content

Water content is defined as

$$\omega_t = \frac{\text{weight of water}}{\text{weight of dry soil}} = \frac{W_w}{W_s}$$

(3.2)

From figure 3.1

$$W_i = W_t - W_{uw} \qquad \textbf{(3.3)}$$

Relative iciness i_{ice} is defined as the ratio between the weight of ice and the weight of total water content in a given unit volume of frozen soil:

$$i_{ice} = \frac{W_i}{W_t} \tag{3.4}$$

or

$$i_{ice} = \frac{\omega_i W_s}{\omega_t W_s} \tag{3.5}$$

$$= \frac{\omega_t - \omega_{uw}}{\omega_t} \tag{3.6}$$

$$= 1 - \frac{\omega_{uw}}{\omega_t} \tag{3.7}$$

The relative proportion of ice and unfrozen water content are dependent on

- Soil characteristics
- Temperature
- Ion concentration of the water
- Externally applied pressure

There are various methods to measure unfrozen water content in frozen soil. These are dilatometry, adiabiatic calorimetry, x-ray diffraction, nuclear magnetic resonance, differential thermal analysis and some other indirect methods. But these methods are not so easy to apply.

The bulk density of a frozen soil is defined as:

$$\rho_f = \frac{\text{weight}}{\text{volume}} = \frac{W}{V} \tag{3.8}$$

The dry density ρ_{df} of a frozen soil is the weight of dry soil divided by the total volume of frozen soil:

$$\rho_{df} = \frac{W_s}{V} \tag{3.9}$$

The relationship between ρ_{df}, ρ_f and ω_t is given by

$$\rho_f = \frac{W}{V} = \frac{W_s + W_{iuw}}{V}$$

$$= \frac{W_s}{V} + \frac{W_{iuw}}{V} \qquad \textbf{(3.10)}$$

$$= \rho_{df} + \omega_t \frac{W_s}{V} = \rho_{df}(1 + \omega_t) \qquad \textbf{(3.11)}$$

Assuming the full saturation (no frozen water but with excess ice in soils);

$$\rho_f = \frac{G_s \rho_w (1 + \omega)}{1 + 1.09 G_s \omega} \qquad \textbf{(3.12)}$$

where G_s is the specific gravity of soil solid particles and ρ_w is the density of water.

The frozen dry unit weight ρ_{df} is given by

$$\rho_{df} = \frac{\rho_f}{1 + \omega} \qquad \textbf{(3.13)}$$

The degree of saturation S_i is given by

$$S_i = \frac{\text{volume of ice}}{\text{volume of pores in frozen soil}}$$

$$= \frac{V_i}{V_v} = \frac{W_i}{\rho_i . e . V_s} = \frac{\omega . G_s . \rho_w}{\rho_i . e} \qquad \textbf{(3.14)}$$

3.2.2 Ice Phase

As shown in Figure 3.2, pore water does not start to freeze until the temperature drops to T_{sc}. After that the formation of ice releases latent heat and cause a rise in temperature to T_f called as initial freezing temperature. In cohesionless soils T_f is close to 0º C whereas in cohesive it is nearly 5 degrees. Afterwards water in will be chilled at temperature T_f. As the water changes to the ice the rate of cooling will go to T_e (about -70º C) where the all free and bonded water will be chilled. **(Andersland & Ladanyi, 2004)**

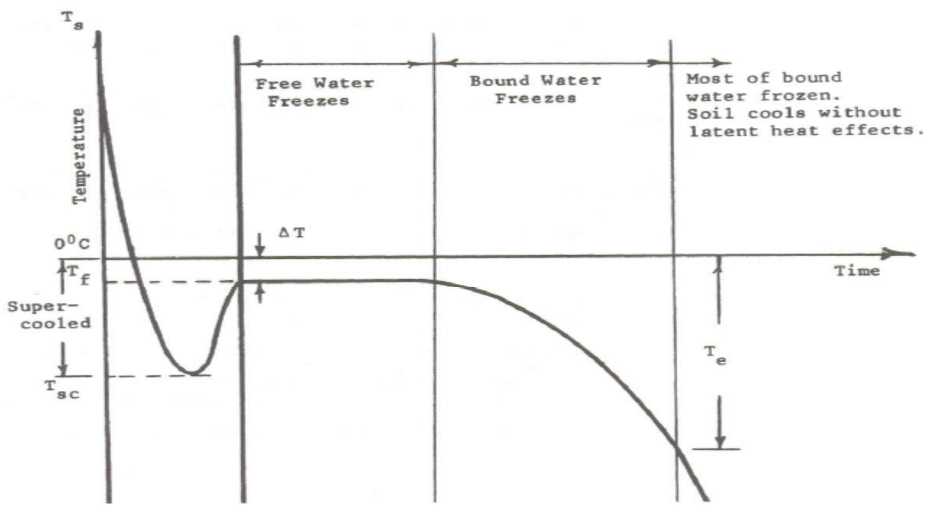


Figure 3.2: Cooling Curve for soil water and ice (Lunardini, 1981)

3.3 Mechanical Properties of Frozen Soils

It is very important to understand the mechanical properties of soils. Because of these both on natural and artificial frozen soil samples, many experiments are carried out. It should be noticed that the structure of frozen soil in site is different from artificial soil samples.

The mechanical properties of the frozen soils are as follows:

- Compression strength and tensile strength
- Shear strength
- Creep strength and long term strength
- Compressibility

The strength of frozen soil is highly dependent on temperature and time and is governed by cohesion of soil and internal friction of soil-ice matrix system.

(Harris, 1995)

3.4 Thermal Properties of Soil

The basic thermal properties of soils are thermal conductivity, heat capacity, diffusivity and latent heat. These properties can vary with phase compositions.

Temperature, soil type, water content, degree of saturation, dry density, organic contents are some of the most effective factors on soils thermal properties.

3.4.1 Thermal Conductivity:

Thermal conductivity is defined as the rate at which the amount of heat is transferred upon the existing temperature gradient in a unit time through a unit cross sectional area. Thermal conductivity is given as:

$$K = \frac{q}{A(T_2 - T_1)/L} \qquad (\frac{cal}{s\ cm\ °C}) \qquad \textbf{(3.15)}$$

where the temperature drops from T_2 to T_1 over a length of L of the element.

(Jumikis, 1977)

3.4.2 Thermal Diffusivity:

In unsteady state, thermal behaviour, also heat capacity C of the soil governs the thermal behaviour of the soil. And the ratio of the thermal conductivity over heat capacity is defined as thermal diffusivity

$$a = \frac{K}{C} \qquad (\frac{mm^2}{s}) \qquad \textbf{(3.16)}$$

The thermal diffusivity of ice is nearly 8 times of the water. So the frozen ground has more diffusivity than that of the soil in thawed position. **(Jumikis, 1966)**

3.4.3 Heat Capacity:

One of the important thermal properties of water is defined as the quantity of the heat energy necessary to change the temperature of a unit mass. This is called specific heat capacity. If it is expressed on a unit volume mass, it is known as the volumetric heat capacity.

$$c_m = \frac{Q}{\Delta T} \qquad (\frac{cal}{cm^3 °C}) \qquad \textbf{(3.17)}$$

The total heat capacity is the addition of the heat capacities of constituents like solid, ice, unfrozen water and etc.

3.4.4 Latent Heat:

The volumetric latent heat of fusion L is the amount of heat required to melt the ice or freeze the water in unit volume of soil with no change in temperature. This factor must be calculated for all thermal requirements of freezing program. Because latent heat is the most when compared to all other heat losses. It usually represents the most important factor in the freezing process. It is defined as:

$$L = \rho_d . \omega . (1-\omega_{uw}) . L' \qquad \textbf{(3.18)}$$

$$L = \rho_d . \omega . L' \quad \text{for } \omega_{uw} = 0 \qquad \textbf{(3.19)}$$

3.5 Hydrologic Properties

Hydrologic properties can be related with bulk thermal properties and have important influence on final design. Most important considerations are:

1. Moisture content
2. Subsurface flow rates and direction of flow
3. Permeability of soil
4. Pore water chemistry

4. BASIC PRINCIPLES OF ARTIFICIAL GROUND FREEZING

4.1 Method and Design

4.1.1 Soil Investigation

Soil investigation reports should include all data till below depths of the proposed excavation. Disturbed and undisturbed samples have to be taken and frozen and unfrozen soil tests should be done. Soil type, density, and water contents are needed to estimate the soil thermal properties. In-situ permeability tests and ground temperatures have to be taken regularly. In some cases multilayered soils thermal properties will be so important that all care should be taken for these regions. Also the bottom soil properties are so important like the water existence or permeability. For example free water will go upwards to the frozen earth barrier so that the barrier will grow and much more energy would be needed. Generally frozen earth barrier should be fixed to impervious layer.

4.1.2 Ground Water

If the rate of velocity is more than 2m/day the freezing columns will not merge and a complete wall will not be formed. For liquid nitrogen systems Shuster (1972) reported that flows as high as 50 m/day have been stopped. So the permeability of the soil should be investigated carefully. This can be done by some radioactive solutions or fluorine ion in boreholes.

The second issue is the contamination degree of the soil. The presence of these kind of constituents (organic, inorganic, radionuclide and bacteriological) will cause low freezing temperatures, reduced ice content and lower strength. Also the salinity of the water should be determined. **(Jessberger, 1980)**

4.1.3 Thermal Analysis

The thermal analysis is the most important component of designing the ground freezing system, during the construction of frozen earth barrier walls. Four independent parameters are based on the factors in completing the thermal analysis, assuming hydrostatic conditions are relatively static.

The four parameters are:

- Thermal properties of the soil;
- Required time to freeze;
- Coolant temperatures. and
- Freeze pipe spacing.

4.2 Theories

There are mainly 4 theories based on heat flow in the body. These are:

1. The steady state flow of heat
2. The unsteady state heat conduction
3. F. Neumann`s Theory
4. J. Stefan`s Theory

All these theories are based on

a. Heat flows from regions of higher temperature to regions of lower temperature
b. The amount of heat in a differential element of soil or other material is proportional to its mass and its temperature
c. The rate of heat flow across an area is proportional to its size and to the temperature gradient

4.2.1 The Steady State Flow of Heat

In this theory some assumptions are made as

1. The soil is homogeneous, isotropic material
2. The geotechnical and thermal properties of the soil are constant
3. Freezing in soil begins at 0 °C
4. The thermal conductivity of the soil is independent of the temperature.

The theory was given by Fourier as in the following equation:

$$\frac{dQ}{dt} = K.A.\frac{dT}{dx'} \tag{4.1}$$

where $\frac{dQ}{dt}$ is the rate of heat flow, K is the coefficient of thermal conductivity $\frac{dT}{dx}$ is the rate of change of temperature with respect to thickness, x, of the plate.

When the flow of heat is constant then the temperature remains constant in each point. Hereby:

$$\frac{dQ}{dt} = q \qquad \frac{dT}{dx} = \frac{T_1 - T_2}{x} \tag{4.2}$$

where T_1 and T_2 are temperatures at the top and the bottom of a plate.

Then equation becomes $$q = K.A.\frac{T_1 - T_2}{x} \tag{4.3}$$

4.2.2 The Unsteady State Heat Conduction:

In this phenomenon temperatures can vary with time and position. Also in this state the heat flow is uniformly distributed over the cross sectional area. According to the Biot-Fourier heat equation in this state is derived as:

$$dQ = -(K).(\frac{\delta T}{\delta x}).(dA).(dT) \tag{4.4}$$

where $\frac{\delta T}{\delta x}$ temperature gradient at any point and x is the depth. The minus sign shows the decrease in temperature as the x in the direction of heat flow increases.

4.2.3 Franz Neumann`s Theory

The original theory was given for the formation of pure ice from water. But it can be used in geotechnical engineering by fitting soil constants into the equation.

Two equations for ice and water were given by Neumann. The assumption is the initial temperature of the soil is positive and constant.

For the frozen part of the soil ($0<x<\xi$):

$$\frac{\delta T_1}{\delta t} = \alpha_1 . \frac{\partial^2 T_1}{\partial x^2} \qquad \textbf{(4.5)}$$

and for the unfrozen part ($x>\xi$):

$$\frac{\delta T_2}{\delta t} = \alpha_2 . \frac{\partial^2 T_2}{\partial x^2} \qquad \textbf{(4.6)}$$

Boundary conditions for the equation are:

1. The initial conditions at $x>0$, $t=0$, $T_1=T_0=\text{constant}$
2. The fixed boundary conditions
   a) At $x=0$, $t\geq 0$, $T_2 = T_s$
   b) When $x \to \infty$, $t \geq 0$, $T_2 \to T_0 = \text{constant}$
   c) The condition at the advancing isothermal surface of the frozen layer downwards at $x = \xi$, $t>0$, $T_1 = T_2$ =constant $T_f = 0\,°C$ where T_f is freezing temperature

4.2.4 Stefan`s Solution

Stefan`s solution presents a simple formula for the formation of ice which takes the frost penetration depth as a function of time while computing.

The assumptions of Stefan`s solution are:

- T_0; The temperature of the water at the isothermal boundary surface, between the frozen and unfrozen zone is 32° F, and that this is so below the frost boundary.
- T_s; The surface temperature (air temperature) is constant.
- The temperature in the frozen zone varies linearly.
- The density of ice is the same as that of water.

The main differential equation is similar as Neumann`s Theory, but there is no second equation as in Neumann`s solution.

The main differential equation for the frozen soil is;

$$0 < x < \xi \qquad \frac{\delta T_1}{\delta t} = \alpha_1 \cdot \frac{\delta^2 T_1}{\delta x^2} \tag{4.7}$$

The boundary conditions are;

At $x = 0$, $T_1 = T_s$

At $x = \xi$, $T_1 = T_f = 32^0 F$

The continuity condition for the flow of heat reduces to;

$$\frac{d\delta}{dt} = \frac{1}{Q_L \cdot \rho_s \cdot w} \cdot \left[K_1 \cdot \frac{\delta T}{\delta x} \right]_{x=\xi} \tag{4.8}$$

$\rho_s \cdot w$ can be explained as the amount of moisture in the soil.

(Jumikis, 1977)

5. APPLICATION IN U2/1 VIENNA-AUSTRIA

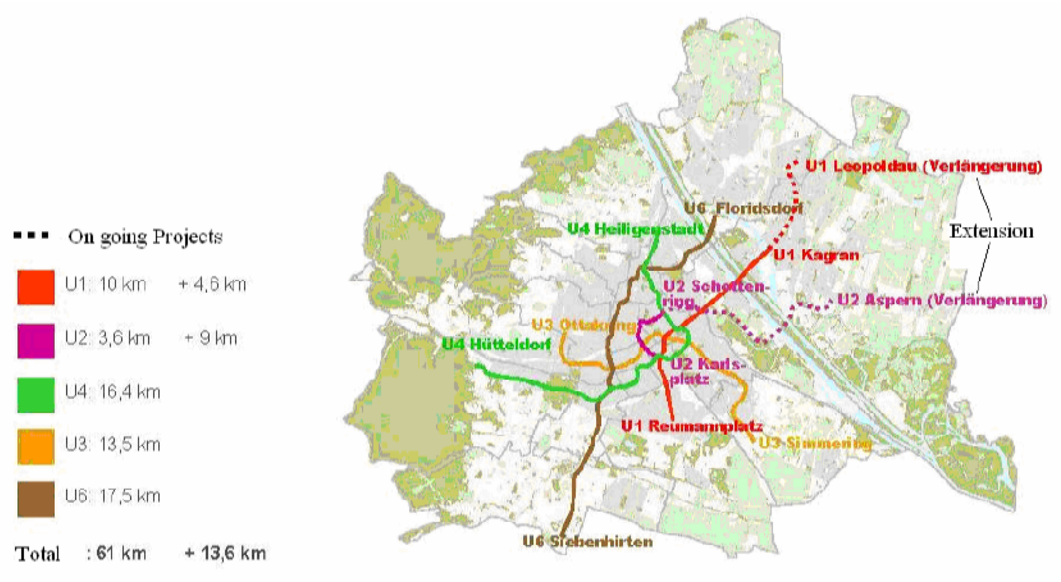


Figure 5.1: Vienna Underground Map

5.1 Definition of the Project

The subject of this project is the extension of the underground line 2 (purple line in Figure 5.1) from the Schottenring to the Aspern Street. Main Underground construction works are planned for the European Football Championship that will be held in 2008 both in Austria and Switzerland.

The route runs along the Maria Theresien Street and then turns to the northern, to the end of the existing station Schottenring. Afterwards the route drops rapidly, in order to make the under crossing of the Danube Canal. (www.magwien.at)

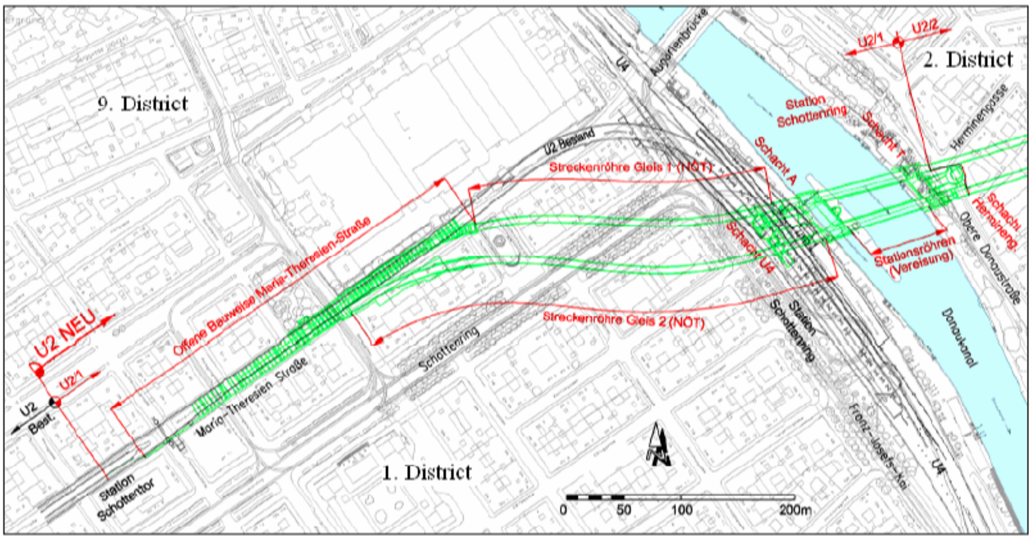


Figure 5.2: Plan of the Danube Crossing

The crossing under the Danube Canal represents the principal item of this underground section. The route takes place from the right Danube Canal bank in the protection of a building and drives under the former and Emperor Bath standing under protection, as well as the Contactor House designed by the architect Otto Wagner (Figure 5.2). The outbreak surface of the two tunnels is enough for approximately in each case 70 m - station pipes amounts to 76 m².

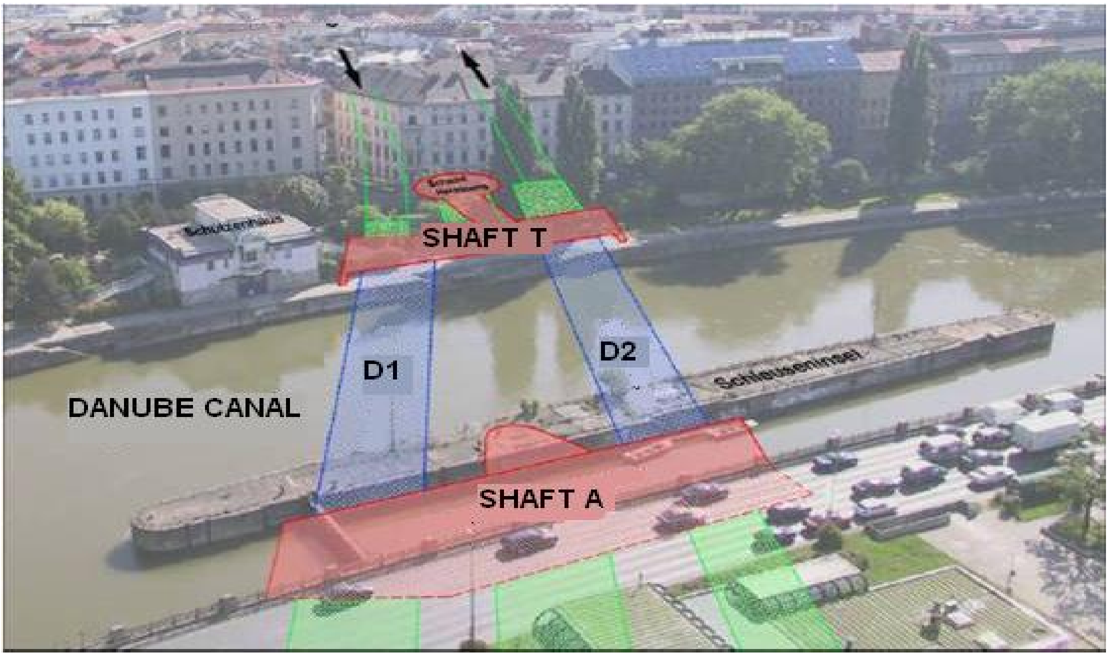


Figure 5.3: View from Ring Tower

Because of shaft construction works people can reach the first and second district from platforms of the new station Schottenring by means of escalator to the existing underground line 4, which will take place under the Danube Canal in the future.

5.1.1 Technical Data:

Freezing Project that lies in 1.District of Vienna, belongs to Viennese lines GmbH & CO and is constructed by ISP Monarth & Tatzber, Bilfinger Berger Bauges.m.b.H, Porr Technobau and Environment AG. The construction period has planned between 05/2003 to 08/2006, with the cost of 43.8 millions Euros. Some technical data about the project are:

First tube is 793.57 m and the second tube is 817.00 m. It was planned that 14,000 m³ frozen soil will be obtained with brine and liquid nitrogen. The lengths of boreholes for freezing pipes are 9700 m. The construction site has been drained by means of 45 wells and 23 levels. **(Joestl etall., 2003)**

5.2 Geology

5.2.1 Geology of Vienna

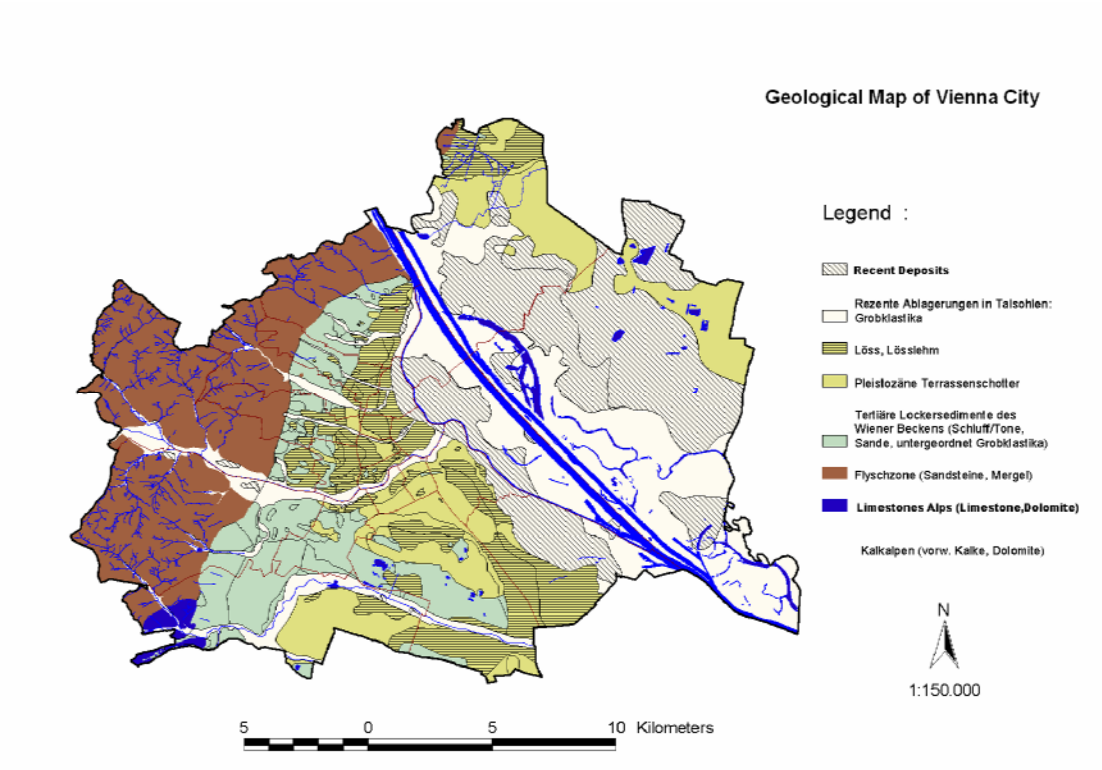


Figure 5.4: Geological Map of Vienna

Vienna lies at the eastern parts of the Alps, because of the west edge of the tertiary times Viennese basin is from the Pleistocene times to today. Viennese area landscape ends at Danube. Due to its geological history, natural soil profile can be seen. General scheme of the city is formed by different geological landscapes:

- Crushed stone and sand of the ice-age and the current unconsolidated Danube sediments of Quaternary
- Loose rocks - tertiary unconsolidated sediments of the Vienna basin
- Rocks of the Flyshzone and the Alps limestone in the western Viennese forest area

Powerful groundwater bodies can be found in the Danube crushed stones.

The Waldviertel belongs to the flattened mountains of the Bohemian Massif, the big crystalline area in the heart of Europe. There one finds crystalline and metamorphic rocks like granite, gneiss, slate etc.

In the south, the Danube embedded itself in the crystalline soil and divided the Dunkelsteiner Forest from the rest of the Bohemian Massif. Following this valley of beautiful scenery, the unique Wachau valley, the Danube enters the flat foothills of the Alpine upland.

On its further course, the Danube cuts through the easternmost foothills of the Alps (the Vienna woods) at Greifenstein-Klosterneuburg and crosses the Vienna Basin, that was formed in the Tertiary and filled with sediments.

The Vienna Basin contains Austria’s biggest oil and gas-fields. At its fracture zones ("thermal line") there are hot springs (Baden, Bad Vöslau). In the west the Vienna Basin borders the Vienna woods and the limestone Alps. In the south the basin borders the crystalline regions Semmering, Bucklige Welt, Rosalien- and Leitha mountains.

During the ice ages Lower Austria was mostly ice-free. Sediments from the detritus of the moraines, which were formed in front of the glaciers, were blown away by the wind. This fine sand covers today wide areas of the Weinviertel and forms the fertile soil for wine growing. (www.magwien.at, Pfleiderer and Hoffmann, 2004)

5.2.2 Geology of Site

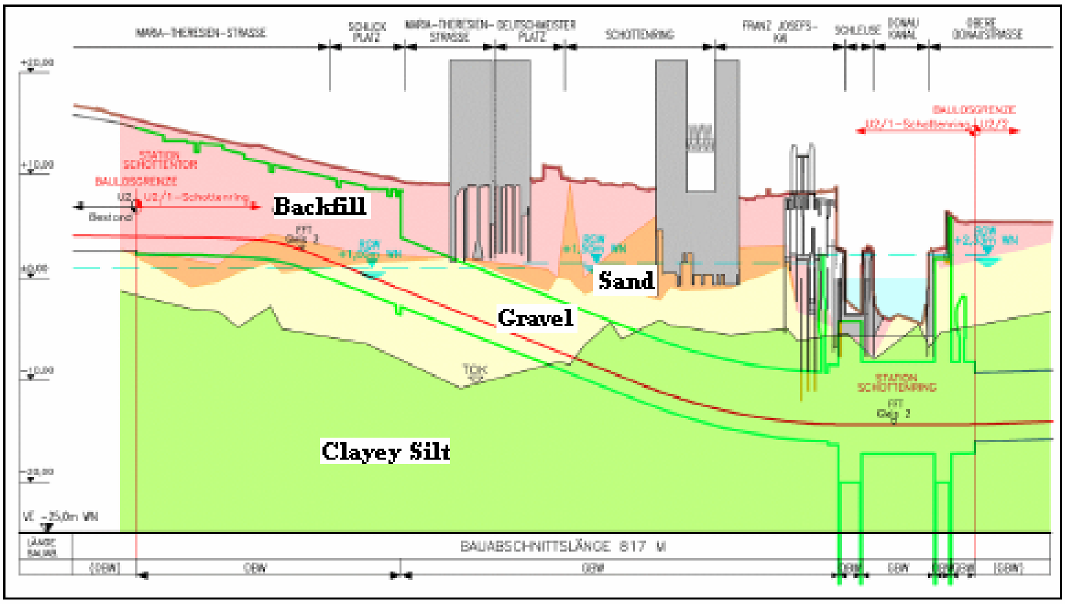


Figure 5.5: Geological cross section of the tunnel

Available favourable geological conditions are consisting of clay in accordance with silt and sandy gravel and the concrete sole of the emperor bath air-lock made the decision possible for a building ground freezing up from soil-mechanical view. The river bed is formed through sandy gravel for differently loose compaction and depths between 0.5 m and 1.5 m. River scours and bomb funnels from 2nd World War that were filled by debris can be found up to 5 m under the Danube Canal.

The freezing up attempts at altogether 8 soil samples from the tertiary, tonus towards and sandy Silt became for the temperatures -5°C, -10°C, -15°C and -20°C accomplished. Freezing and thawing out procedures were simulated. In the unloaded condition the freezing attempts resulted in after 6 frost rope cycles elevations between 12.86 mm and 17.48 mm. In the thawing out procedure, the elevations decreased with all samples to values between 9.65 mm and 14.56 mm.

(Martak and Herzfeld, 2005)

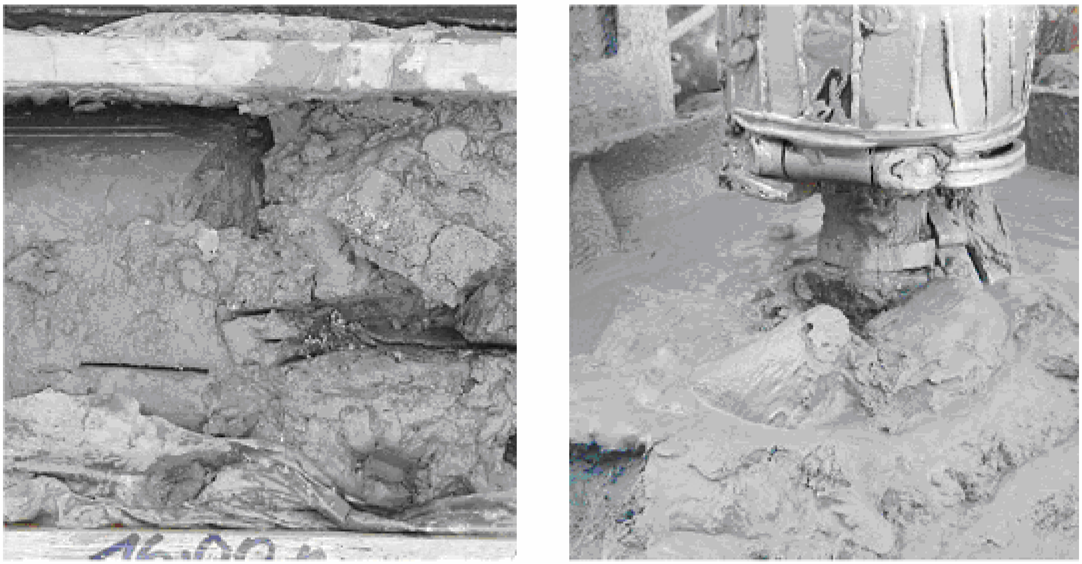


Figure 5.6: Soil Samples taken from boreholes and silty soil in piling bucket.

The creep deformations were appropriate 2.2 % and 9.8 %. The values showed a clear increase as a function of dropping the temperature. As maximum strengths with the fine to medium silts values between 1.99 N/mm² and 3.31 N/mm² are resulted. **(Jöstl etall, 2003)**

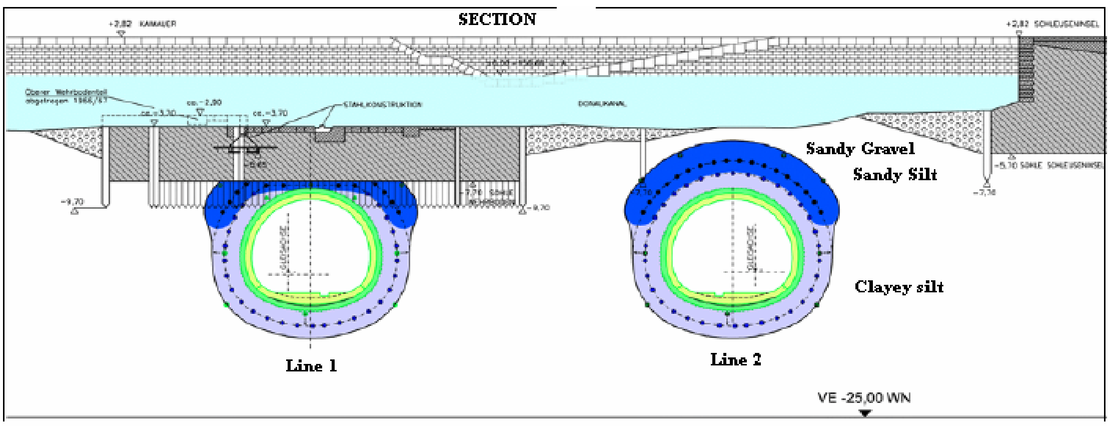


Figure 5.7: Cross section of the tunnel with planned freezing parts

5.3 Before Construction

As it is seen in Figure 5.7, two sections were planned and in progress. But in one tunnel because of the old foundation of the Emperor Bath freezing is bordered with that concrete structure. Both tunnels are driven through Clayey Silt soil and in order to prevent the water from Danube all section is planned to be frozen. But especially for the top parts, the freezing power and distribution of pipes are high with correspondence to bottom parts. Dark Blue Parts are planned to be frozen with LN_2, on the other hand light blue ones are planned to be frozen with brine.

From the first week of December 2001, the temperature variation under the Danube has begun to be measured in 7 boreholes. Seasonally the temperature gradient were measured between +9˚ C to +19˚ C. The seasonal variations of the Danube Canal are measurably, but for the cost of the project and construction period, winter term is selected. The deeper soil strata under Danube gave +12˚ C at average.

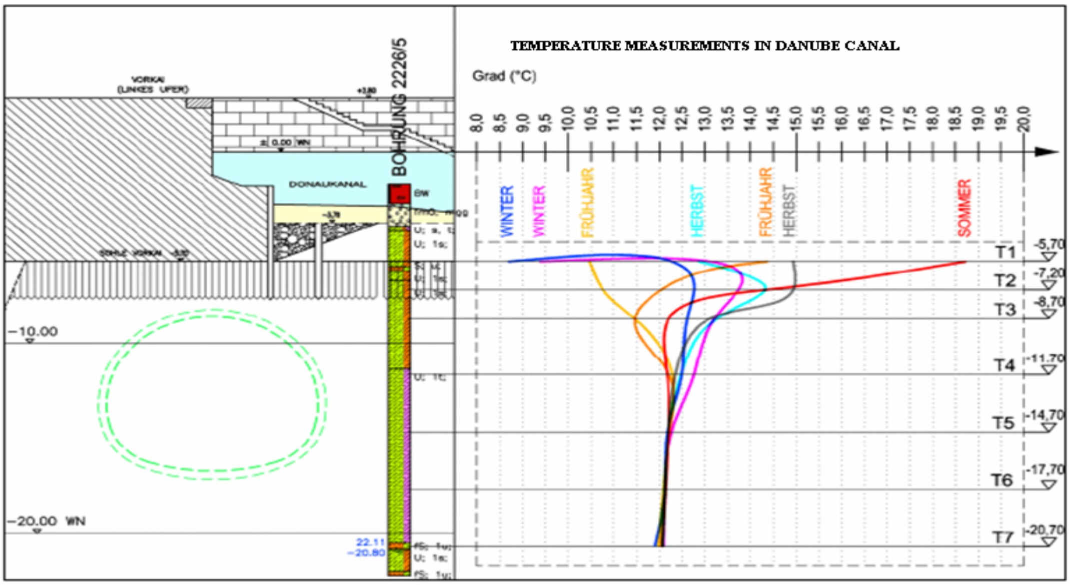


Figure 5.8: Temperature along borehole in Danube Canal (Jöstl etall., 2003)

The current consumption during the freezing-up period of approx. 43 weeks with approx. 3 million kilowatt-hours calculated. Nitrogen consumption (LIN) is proposed as 3 million liters. The soil volume which will be frozen amounts to approx. 14,000 m³, the entire freezing period is approx. 10 months. **(Jöstl etall, 2003)**

5.3.1 Freezing Pipe Installation and Monitoring

The drillings of pipes are parallel to the tunnel axes. Per axes three drillings are to be implemented for temperature measurement due to of defaults from pit geometry diagonally to the tunnel axes. The drilling pattern and the respective drilling lengths are different due to the boundary conditions in tunnel 1 and tunnel 2. The drilling lengths for the production of the circular freezing up body amount to up to 40 m with an overall length of the drillings of approx. 9,700 m, the drilling work was started in June 2004, and the beginning of the freezing up was started at 13th January 2005.

Figure 5.9: Drilling of boreholes of freezing pipes

Temperature measurements of frozen body over the cross section of tunnel-1 are done with 12 temperature measuring tubes, and in cross section of tunnel-2 are done by 9 distributed temperature measuring tubes. In the temperature measuring tubes as rule stationary temperature primary detectors at a distance of 2.0 m are arranged, thus an accurate determination of the temporal development of the frost body thickness and temperature distribution can be accomplished. At the beginning and at final range of the temperature measuring tubes, the distance of the temperature primary detectors is reduced to 0.5 m at connection to the Emperor Bath. The temperature measuring tubes were filled with a $CaCl_2$ solution, in order to guarantee a good heat transfer between soil and measuring level.

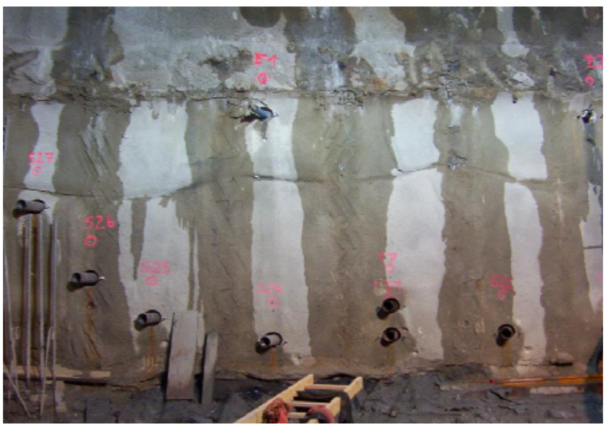


Figure 5.10: Freezing pipes and temperature pipes

The freezing procedures were already used several times in the past 25 years successfully in the Viennese underground constructions. For example as sealing and improvement of the soil in construction of the line U3 under the department store Herzmansky in the Mariahilfer road and Telecommunication Center in Meidling freezing process was done in success . **(Martak and Herzfeld, 2005)**

The same process was decided to be used in U2/1 Project passing through Danube Canal with common view of Municipality and TU Vienna and BOKU Vienna. The heart of the section U2/1 - Schottenring represents the station Schottenring within the range of the monument-protected emperor bath air-lock, which can be established again under the Danube Canal.

When planning of the under crossing of the Danube Canal a solution with half lateral excavations in the river was examined. It was however denied, because it would have stopped the trip traffic of the navigation at the Danube Canal over two seasons. This would have meant a substantial decrease of incomes for the trip navigation, whose subsequent costs could be calculated for the tourism only inaccurately.

The Danube Canal has its main arm in the northeast of the city in the last centuries. Vienna Municipality shifted Danube to the east districts several times. Today's situation goes on adjustments at the beginning 19th Century.

Figure 5.11: Freezing process has begun

5.3.2 Considerations in Freezing

In this freezing project it is decided to use the system as mentioned above combined method as third solution. For the choice of the combined freezing process the following considerations were decisive:

- High cooling capacity would minimize the effect of thermal supply from the Danube Canal flowing which can not be excluded, fast up to 2 m/s
- The route of the tunnel drives under the air-lock island and the military field diagonally with the station pipes, whereby the monument-protected hydraulic engineering calculations are to be done;
- The clayey silt soil around the tunnel driving is inclined approximately with the freezing procedure over the duration of several months to intensive elevations,
- If soil ranges with the tunnel driving, which are unexpectedly seem to be soft during the work, a fast possibility high-energy of the post freezers, in particular toward the river bed of the Danube Canal is of planning;
- It is necessary, around the entire edge of tunnel outbreak a freezing up ring of approx. 2 m thickness, so that the hydraulic lift security of the station tunnel outer shells is given

These will be realized with liquid nitrogen (LN_2) for the shock freezing and full rings forward around and afterwards the brine ($CaCl_2$) will push freezing process.

With this procedure the rapid formation of a high-energy cooling potential (LN_2) can be fulfilled for the minimization of the ice lens formation. Nitrogen has a temperature of -196 as liquefied gas at 1 bar °C. **(Martak and Herzfeld, 2005)**

With the brine procedure an economical and reliable freezing process is available over several months, which can hold the circular frost body after some weeks and can enterprise the desired temperature range between -10 °C and for -20 °C. The brine used in the brine freezing process has a maximum freezing carrier temperature of approx. -40° C. The freezing procedure in the soil lasts however clearly longer than with the LN_2 (according to experience 5 to 7 weeks), which will particularly lead in clay towards silt to ice lens formation and elevations, which can be kept relatively small however by the load soil already frozen with LN_2 and the present buildings over it. Geotechnical control of the current temperatures in the freezing bodies is guaranteed by temperature measuring probes over the entire length of the frost bodies and distributed over the extent.

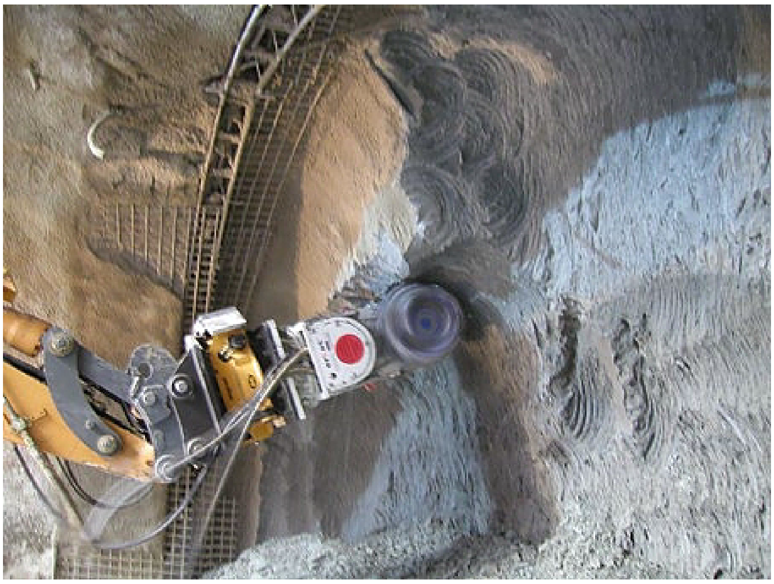


Figure 5.12: Excavation in the Tunnel

It was necessary to manufacture purposeful freezing drillings for the filling with brine/nitrogen from both sides of the Danube Canal which is 40 m long. From both sides pipes will be parallel to the tunnel axes and will be intersected in the center approximately 4 m. Their distance between each other are 1.1 m to 1.4 m. Drilled pipes diameters were 139 mm and exhibit in individual cases vertical deviations to 0.70 m and horizontal deviations to 0.12 m. According to plan altogether 4 x 55 drillings (30 drillings for the brine freezing ring, 13 drillings for the liquid nitrogen partial drillings for each tunnel driving) only few auxiliary drillings had to be implemented.

The regular thickness of the freezing up body amounts to within the range of the brine freezing 2.0 m, within the range of the combined freezing process (in the roofridges) up to 3,5 m with station pipe D1. With the station pipe D2 the freezing up body is enough to the military soil near and has thereby a smaller thickness. As edge of frost body the extent is defined, that exhibits a temperature of -2° C. For the formation of the frozen body with liquid nitrogen is estimated one period of 15 days, for the employment of the brine 50 to 55 days. Current control of the air conditioning probes supplies the information about the development of the frost body thickness.

After the frost body achieved the intended thickness, further increasing is no longer desired. The phase of the frost attitude begins. The frost border is to be kept constant by intermittent brine.

Figure 5.13: General view of excavated part from Shaft1

5.4 Costs

The offered total sum amounts to approx. 48 millions Euro. 50 % of the costs to the open building method and 50% are allotted to the closed building method. The distribution of the costs of the closed building method shows that approx. 50% of the costs on the building remedial measures (ground freezing, ground-water lowering, jet procedure, etc..) not applicable. The offered costs of the building ground freezing up, based on the mass determination of the advertisement, amount to approx. 4.6 millions Euros. During a theoretical building ground freezing up of approx. 14,000 m^3 and means 329 Euro/m^3. Related to the section length of approx. 817 m means amount to the net costs 55,080 Euro/m. **(Joestl etall, 2003)**

6. LABORATORY WORK

6.1 Freezing and Thawing Experiments

6.1.2 Laboratory Soil Parameters:

The aim of these experiments was to define the theoretical action of the frozen ground. The site behaviour was tried to be related with the results taken from experiments. Furthermore phenomena as Frost heave, thawing process of the site were tried to be understood.

Due to the experiments carried out on two type of silt in both TU Vienna and BOKU Vienna, soil parameters written below were found.

Table 6.1: Soil parameters (TU Vienna)

Parameters		Silt 2	Silt 3
Bulk Density	ρ_s [g/cm^3]	2.73	2.74
Density	ρ [g/cm^3]	1.96	2.02
Dry Density	ρ_d [g/cm^3]	1.61	1.63
Water Content	w [%]	22	24
Porosity	$n = 1- \frac{\rho_d}{\rho_s}$	0.41	0.41
Liquid Limit	w_L [%]	50	46
Plastic Limit	w_P [%]	28	27
Plasticity Index	$I_p = w_L - w_P$ [%]	22	19

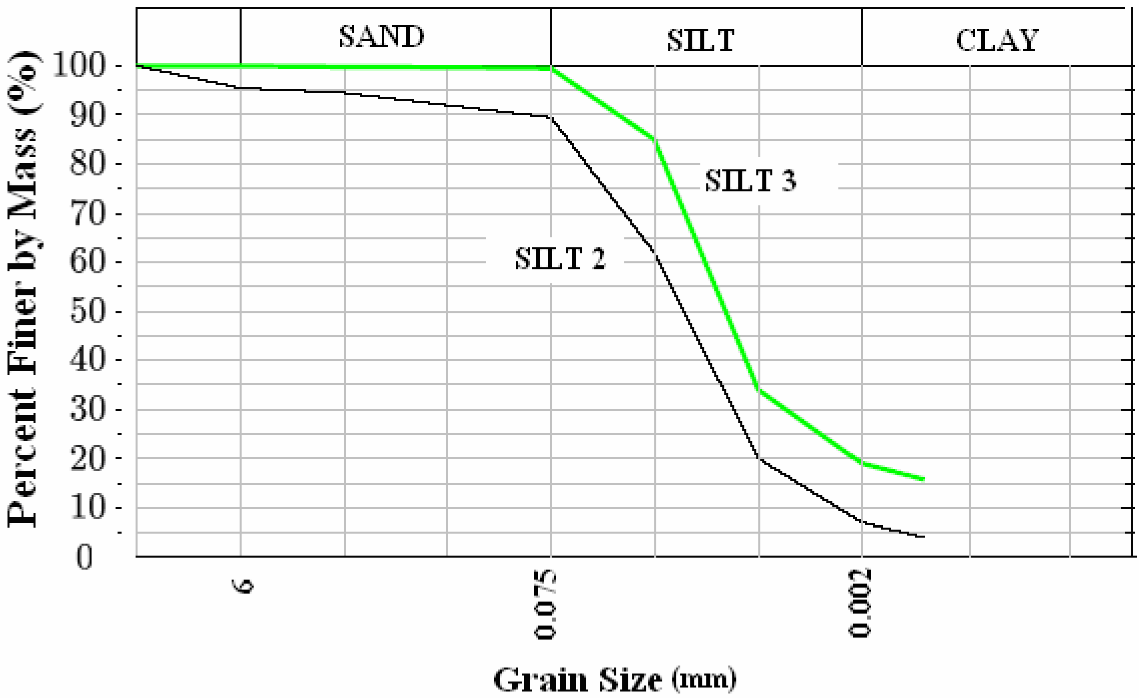


Figure 6.1: Grain size distribution of the soil

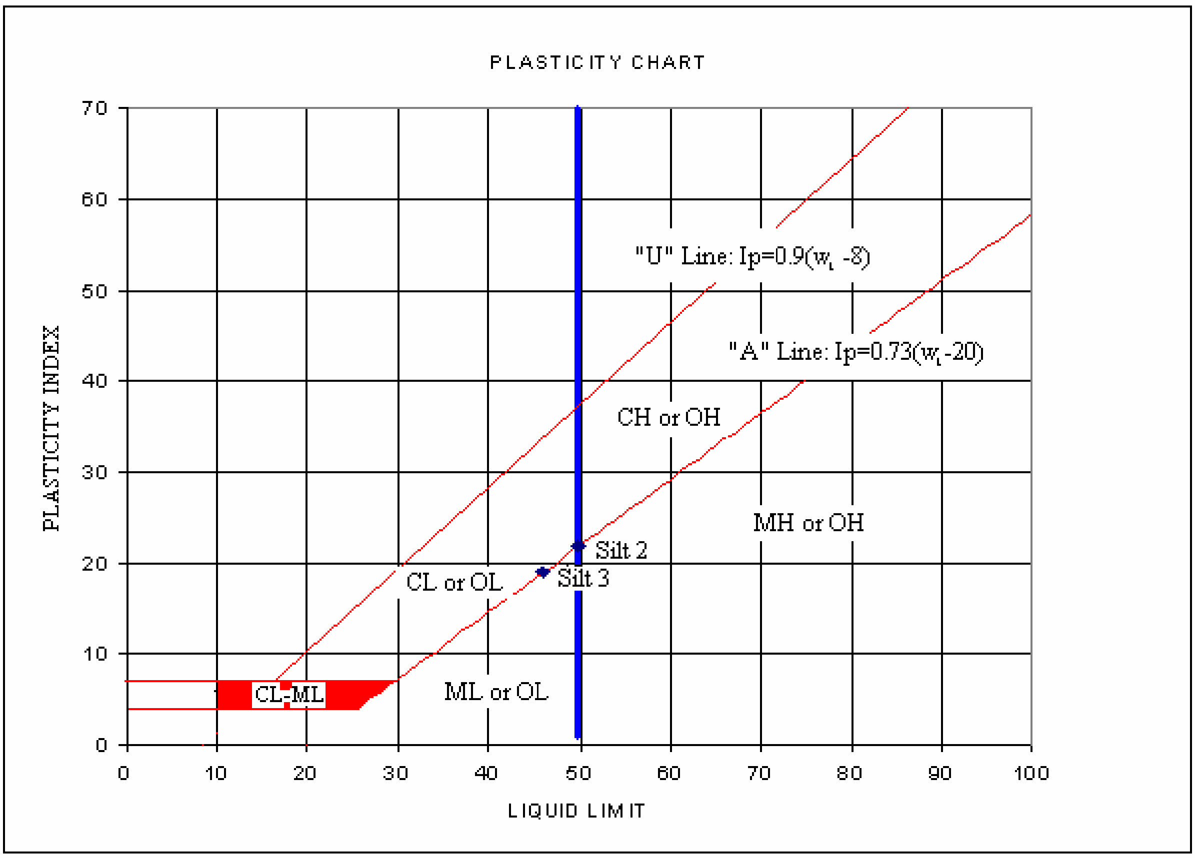


Figure 6.2: Plasticity Card: Samples can be seen as ML

According to the plasticity chart Figure 6.2, it can be classified as Clayey silt (ML). It is seen that it is poor graded silt with 27% of clay from grain size distribution Figure 6.1. Also the mineralogy of the clay was examined. As can be seen from table 2, 16 % of the all sample is with smectite group of clay minerals.

Table 6.2: Mineralogy of the clay in samples (Potz, 2005)

Silt	Percentage in Clay				Total Clay	Percentage in Total			
	Smectit	Illite	Caolinite	Chlorite	%	Smectit	Illite	Caolinite	Chlorite
2	58%	22%	10%	10%	26.8%	16%	6%	3%	3%
3	62%	20%	9%	9%	26.7%	17%	5%	2%	2%

6.2 Experiments

All the experiments were carried out by Institute of Material Science in Vienna University of Technology. 29 experiments were done on three different specimens taken from site, and these are classified as Silt 1, Silt 2, and Silt 3. There were 5 specimens from Silt 1, 10 specimens from Silt 2 and 14 specimens from Silt 3.

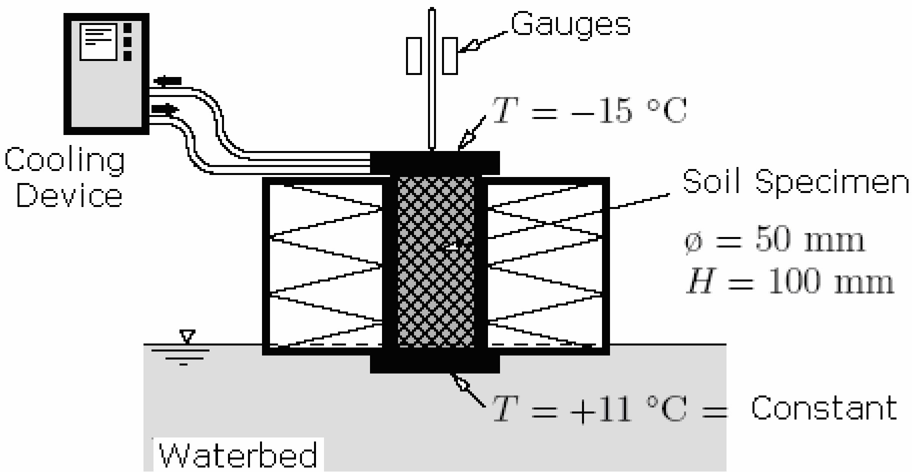


Figure 6.3: Schematic description of freezing

As shown in the figure 6.3, a cooling device is connected to the soil specimen with dimensions of the radius 50mm and height 100 mm. In order to reflect the site parameters bottom side of the specimen is filled with water at +11˚ C and freezing and thawing behaviour of the soil was tried to be understand. From the top of the soil is chilled to -15 degrees with a cooling device. In order to insulate the samples, all samples are prevented in insulation foam to get better results. (Figure 6.4)

Figure 6.4: View from experiment, insulation foams are used to prevent heat loses.

As can be shown in the figure 6.5, sample is monitored by 4 temperature gauges directly connected to the computer. All data from the soil was gained at every 2 minutes. All gauges have same gap between each other with 2 cm.

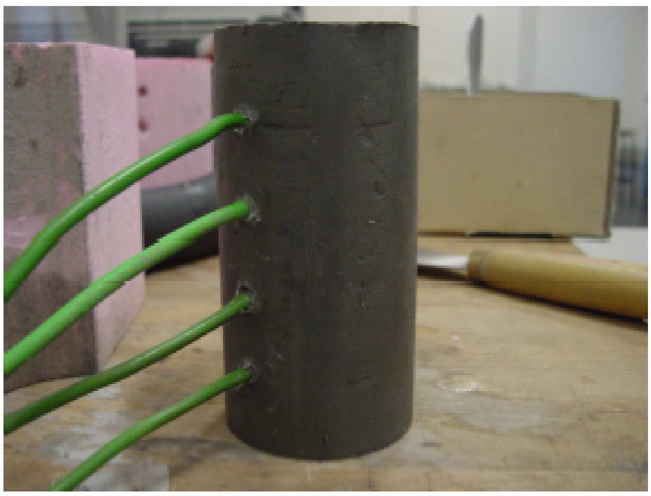


Figure 6.5: Specimen with temperature gauges

Experiments were done in 4 ways. Nine samples were examined with cooling rate -50˚C/h, three samples were examined with cooling rate -0.4˚C/h, seven experiments were done with thawing cycle after freezing, and four samples were examined with two step freezing.

6.3 Results and Discussion

6.3.1 Frost Action

After the experiments data the real ice can be seen at freezing point near 0 ˚C. This kind of behaviour is same as figure idealized frost heave soil column of Nixon 1992. Nixon has explained this phenomenon as the rate of heat extraction exceeds the rate of heat supply (water flow) to freezing point. The frozen fringe is defined as the region of the impeded flow caused by partial filling of soil pores by ice (Nixon, 1991). If the pressure in the ice exceeds the pressure of overburden material and other pressures, the soil begin to expand. When there is sufficient ice pressure, the new ice lenses will be formed. This upward movement in the site is called Frost Heave. In slow freezing process the temperature gradient will be formed near freezing fringe and unfrozen zones. In the fast freezing process the permeability of the frozen fringe will decrease and this causes the less ice lenses in the body. As a result the relation of the water and heat flow would cause thinner ice lenses upwards and thicker ice lenses in downwards away from the supply of cooler. **(Andersland, 2004)**

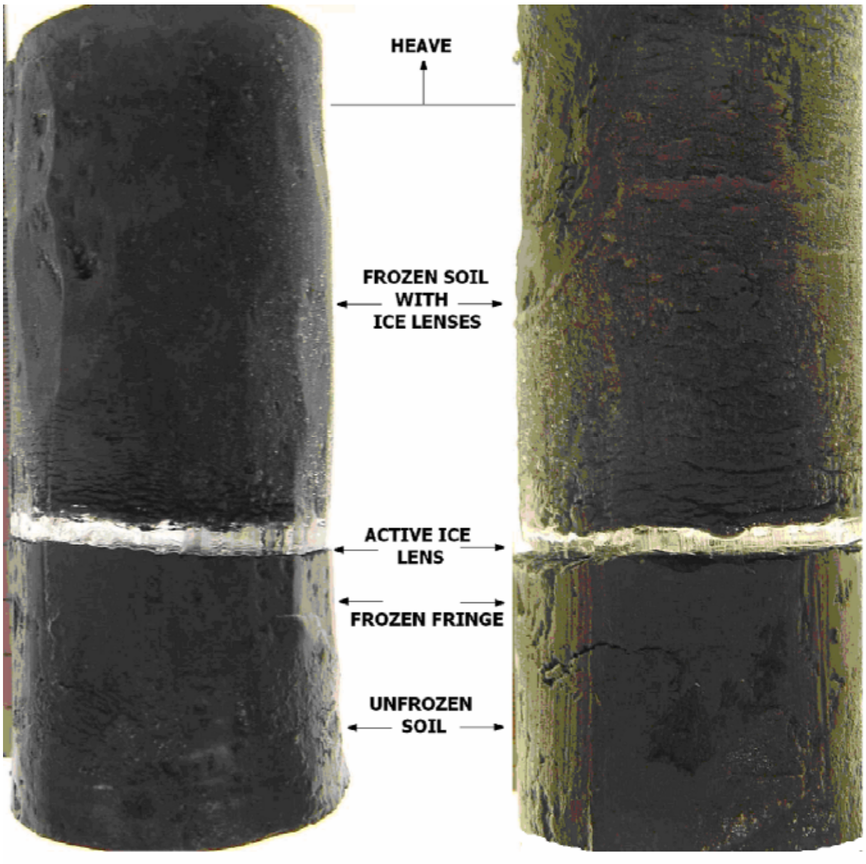


Figure 6.6: Frozen Soil Samples, Ice can be seen by naked eyes

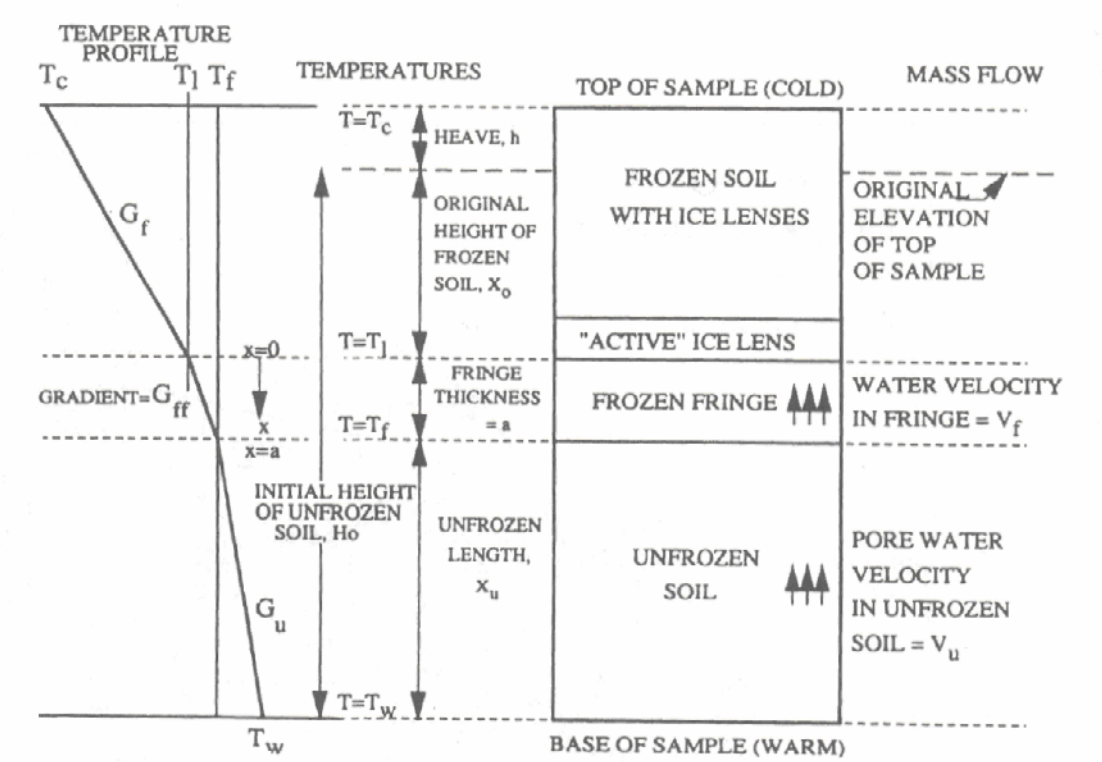


Figure 6.7: Frost heave in idealized one dimensional soil sample (Nixon, 1992)

6.3.2 Correlations and Statistical Evaluation of Results

Table 6.3: Correlation of heaving and water contents

Sample	Heaving (mm)	Δω (%)	x^2	y^2	xy
1 (F)	12.5	6	156.25	36	75
2 (F)	10	4	100	16	40
3 (F)	20	12	400	144	240
4 (F)	10	7	100	49	70
5 (F)	19	14	361	196	266
6 (F)	14	9	196	81	126
7 (S)	17	10	289	100	170
8 (S)	18	14	324	196	252
9 (S)	20	17	400	289	340
10 (S)	23	20	529	400	460
11 (S)	22	16	484	256	352
Total	$\Sigma x =$ 185.5	$\Sigma y =$ 129	$\Sigma x^2 =$ 3339.25	$\Sigma y^2 =$ 1763	$\Sigma xy =$ 2391
Mean	16.86	11.73			

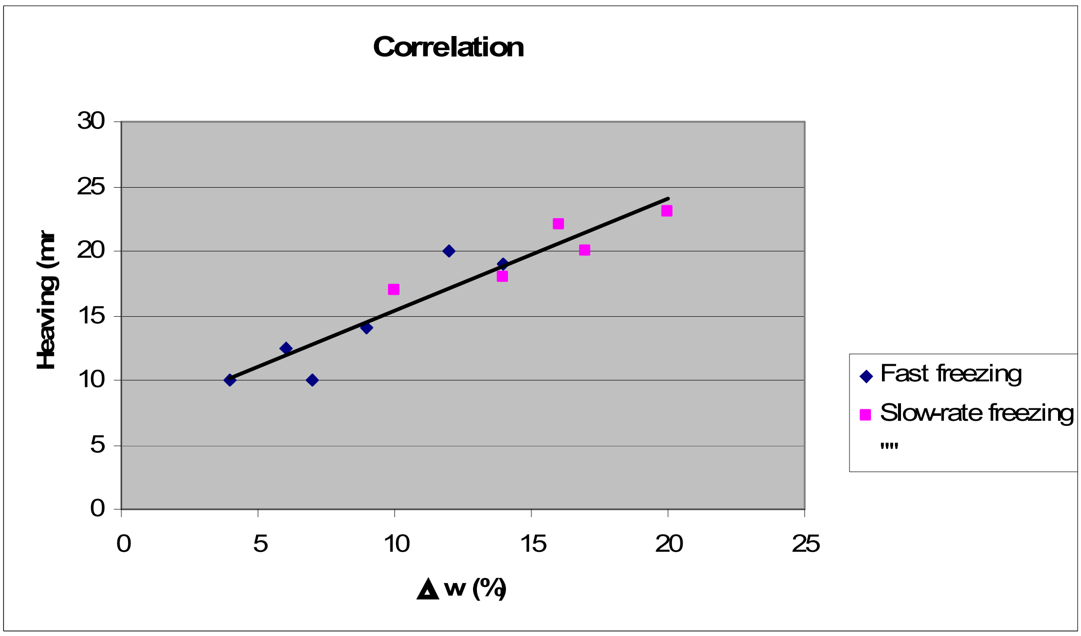


Figure 6.8: Correlation of Heaving with Δw (water contents)

$\Sigma d\, x^2 =$	211.05
$\Sigma d\, y^2 =$	250.18
$\Sigma d\, x d\, y =$	215.59

r_c= 0.938
r= 0.602 From "r" Table; p=0.05
r_c> r

The calculated value (0.938) does exceed the tabulated value (0.602) at p=0.05 value. It also exceeds the tabulated value for p = 0.01 and for p = 0.001. So it can be said that 99.9% confident that heaving and increase in water content are positively correlated in experiments.

Table 6.4: Mann Whitney U Test data

Heave (mm)	10	10	13	14	17	18	19	20	20	22	23
Process	F	F	F	F	S	S	F	F	S	S	S
Row no	1.5	1.5	3	4	5	6	7	8.5	8.5	10	11

According to nonparametric alternative test (The Mann-Whitney U Test), null hypothesis of equal means is rejected at 95th percentile level.

R_x = 5+6+8.5+10+11= 40.5
R_y = 5 x (6+5+1)-40.5 = 19.5
R = 20> R_y (rejected; from U Test table, n=5 m=6 R can be found at α=0.05)

This means the heaving ratio at slow rate freezing is more than that of fast freezing.

Table 6.5: Change of Consistency Index in Freezing Thawing cycle in specimen [before freezing/After experiment upper part/ After experiment bottom]

Sample Nr	Water Content (%)			Liquid Limit	Plastic Limit	Plasticity	Consistency Index (I_c)
	Before	After Upper	After bottom	(%)	(%)	Index (%)	
13(2)	28.0 /	32.7 /	34.1	49.5	28.0	21.5	1.00 / 0.78 / 0.72
14(2)	26.3 /	30.3 /	31.0	49.5	28.0	21.5	1.08 / 0.90 / 0.86
15(2)	27 /	30.6 /	33.3	49.5	28.0	21.5	1.04 / 0.88 / 0.76
16(2)	27.4 /	32.4 /	32.9	49.5	28.0	21.5	1.03 / 0.79 / 0.77
17(2)	27.7 /	33.9 /	32.9	49.5	28.0	21.5	1.01 / 0.72 / 0.77
22(3)	24.7 /	30.3 /	36.6	46.0	27.0	19.0	1.12 / 0.83 / 0.50
31(3)	26.8 /	36.2 /	31.2	46.0	27.0	19.0	1.01 / 0.52 / 0.78

As it is seen from the Table 6.5 the consistency indexes of the specimen are above 1 before freezing. After freezing, the amount became smaller than “1”. This show the properties of the soil is physically changed and the soil become more plastic.

Table 6.6: Water contents in specimen [Before freezing/After experiment upper part/ After experiment bottom/ Overall]

Specimen no	Silt		Cooling Rate [°C/h]	Heaving (mm)	After Thawing (mm)	Water Contents (%)			
						Before	After Top	After Bottom	Overall
8	1	Freezing	-50	21		26	43	32	39
9	1	Freezing	-50	14		27	40	36	38
10	2	Freezing	-50	12.5		26	36	27	32
11	2	Freezing	-50	10		27	34	26	31
12	2	Two Step Freezing	-50	11		27	34	28	32
13	2	Freezing/Thawing	-50	11	6	28	33	34	33
14	2	Freezing/Thawing	-50	14	2	26	30	31	31
15	2	Freezing/Thawing	-50	13	2	27	31	33	32
16	2	Freezing/Thawing	-50	14	3	27	32	31	32
17	2	Freezing/Thawing	-50	15	3	28	34	33	34
18	2	Freezing	(0.4)	17		28	42	31	38
19	2	Two Step Freezing	-50	9		27	33	27	30
20	3	Freezing	(0.4)	18		26	46	34	40
21	3	Two Step Freezing	-50	26		28	58	30	43
22	3	Freezing/Thawing	-50	9	-0.5	25	30	37	33
23	3	Freezing	(0.4)	14		25	52	37	44
24	3	Freezing	-50	20		26	45	30	38
25	3	Freezing	-50	10		24	31	30	31
26	3	Freezing	-50	19		23	48	29	37
27	3	Freezing	-50	14		25	37	28	34
28	3	Freezing	(1.0)	20		26	47	36	42
29	3	Freezing	(1.0)	23		27	60	37	46
30	3	Freezing	-25	17		26	41	29	35
31	3	Freezing/Thawing	-50	20	3	27	36	31	34
32	3	Two Step Freezing	-50	30		25	56	28	43
33	3	Freezing	(1.0)	22		27	47	35	42

7. CONCLUSION

Artificial ground freezing is one of the most reliable methods in stabilizing the soft ground tunnels where the other methods cannot be effective or more expensive and where the ground water is concern. In this thesis the main idea for artificial ground freezing techniques and the design considerations with NATM tunnelling were discussed. All cases were considered from a geotechnical view and tried to be explained by principles of soil mechanics. Additionally one case study in Vienna Underground system has been given in details. Finally, the experiments carried out by Institute of Material Science at TU Vienna were given and some comments about the freezing process were tried to be explained. Cylindrical soil samples were tested in one-dimensional case and the relationship of heaving in slow-rate freezing and fast freezing and behaviour of soil during freezing were examined. The results can be summarized as below:

1- Heaving amounts for both slow rate and fast frozen samples are significantly different if p is set at 0.05 level (95% percentile level). This means the heaving ratio in slow rate freezing is more than that of fast freezing. There will be more heaving if the process is slow. This is because; growing of ice lenses in slow-rate freezing is more than that of fast one.

2- The means of water content values at slow rate freezing and fast freezing are giving reasonable differences. It can be said that the difference in two experiments is significant at the 0.05 level (95% percentile level) statistically. This means the water content ratio in slow rate freezing is more than that of fast freezing.

3- If the correlation of water contents and heaving values are studied statistically, it can be said 99.9% confident that heaving and increase in water content are positively correlated in experiments. So when the water content of the soil increases, the heaving ratio would increase. (Figure 7.1)

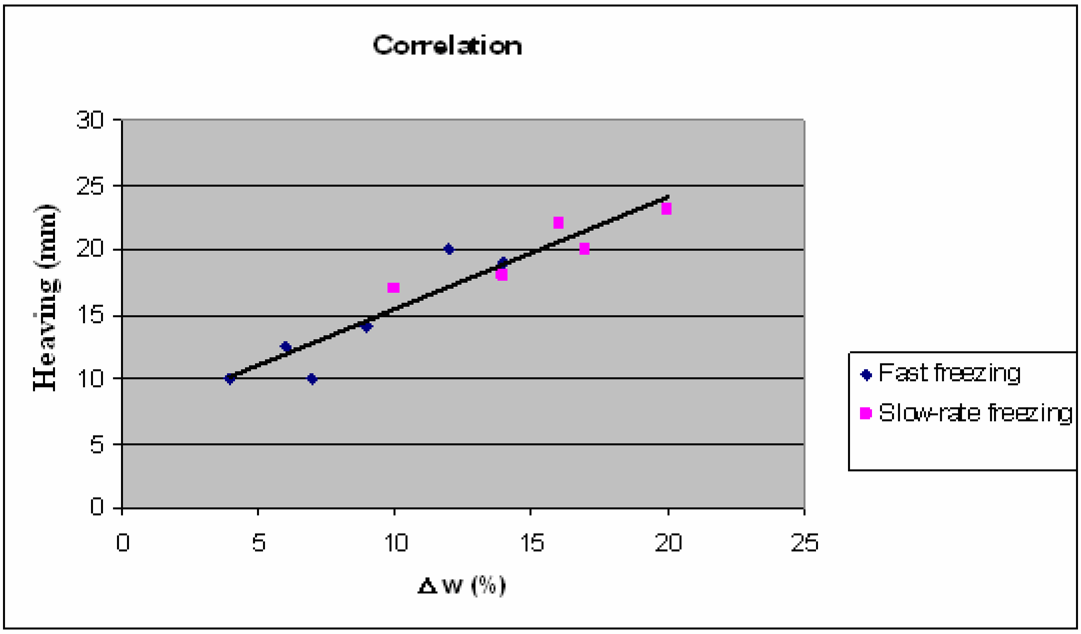


Figure 7.1: Correlation of water content versus heaving ratio.

4- The behaviour of heaving phenomena is related with the increase of water content in the soil. The water content increases with the decrease of temperature under freezing point. With the increase of water content, the heaving ratio increases. There is a physical relationship. When the water goes into ice phase, there would be 1.09 times volume change. So the difference between the last and the first water content ratio in frozen part is the change in volume.

5- The main process is the heat and water flow in the body. The heat is in water in soil. So the heat by means of water is transported to the cold and at 0º C the water is transformed into the ice. With the continuity of the heat transfer, the ice lens is formed and grown. So the heaving phenomenon occurs.

6- After thawing of the soil, due to changes in water content ratio the physical properties of the soil changes. If consistency indexes are studied, it can be seen that in silty soil, the soil becomes more plastic compared with the situation before freezing. This would be because of high percentage of clay minerals such as smectite.

7- Growing of crystals in slow reactions is regular and bigger than that of fast ones. So the more bigger crystals means the more volume deformation, if the freezing phenomenon is realised. This leads to the more heaving ratio in slow rate freezing.

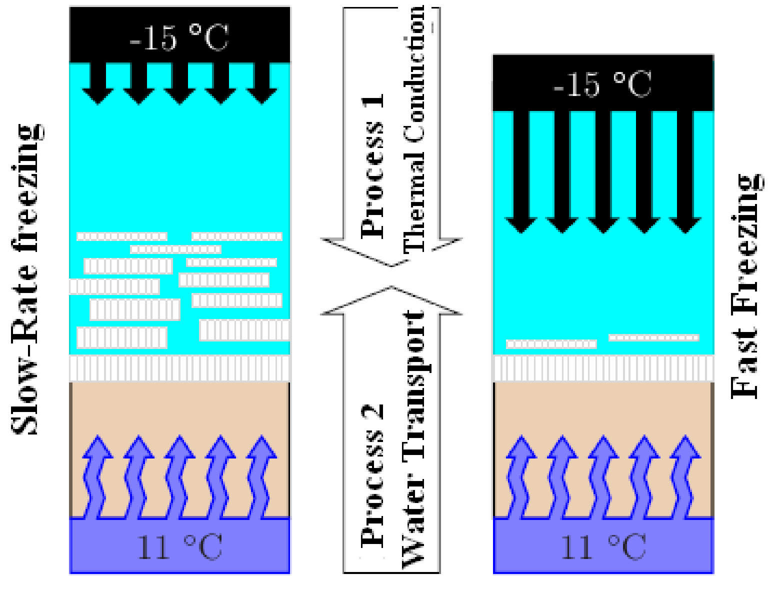


Figure 7.2: Behavior of one dimensional soil column during freezing (Martak, 2005)

8- The aim of this study to be used in practise was that; during lengthening of U2, the under crossing of the Danube Canal, artificial ground freezing method was chosen due to the circumstances in the site and its benefits given in section 2.

Moreover while chilling the ground, according to the results in slow-rate and fast freezing, first it was decided to freeze fast in order not to have much heaving amounts (Figure 7.2). After having the desired strength and thickness of the frozen wall, just to keep the amount of thickness, slow rate freezing would be applied. Because, there are some monumental buildings adjacent to the proposed project site and no more heaving is desired.

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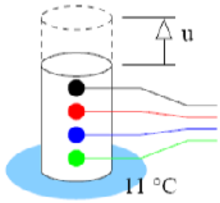
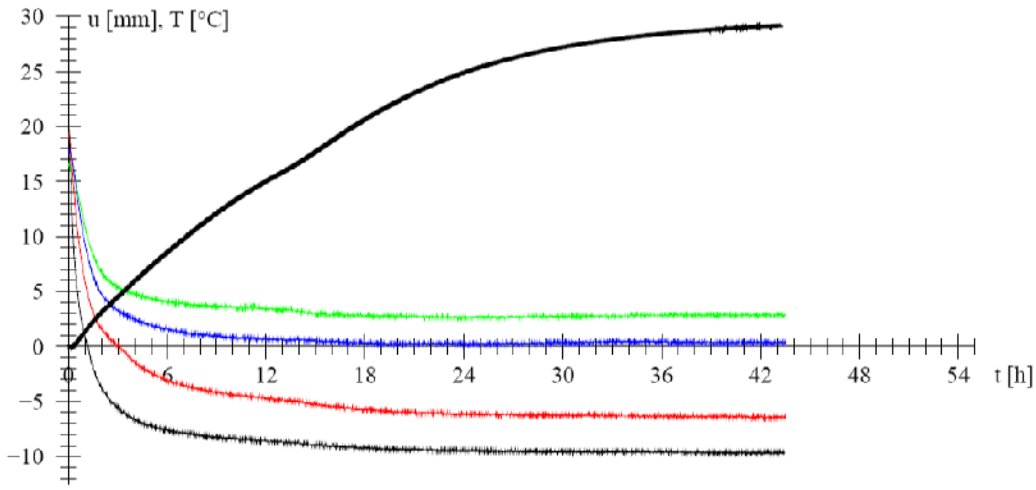
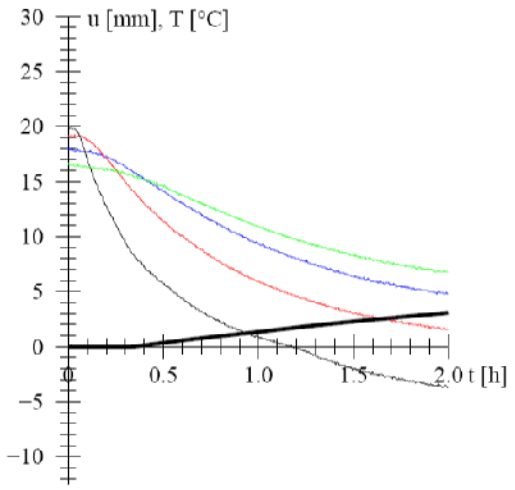
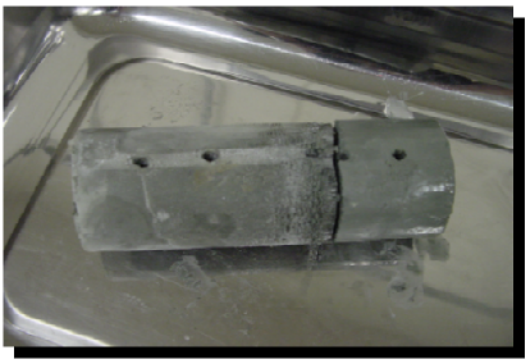
APPENDIXES

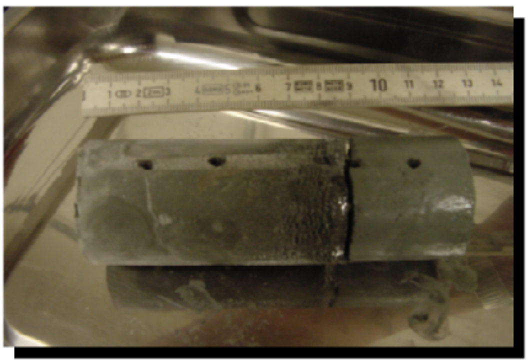
sample: 005 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	27.7 % (before)
remark: −−	42.8 % (after − overall) −− (after − frozen) −− (after − unfrozen)

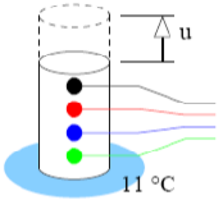


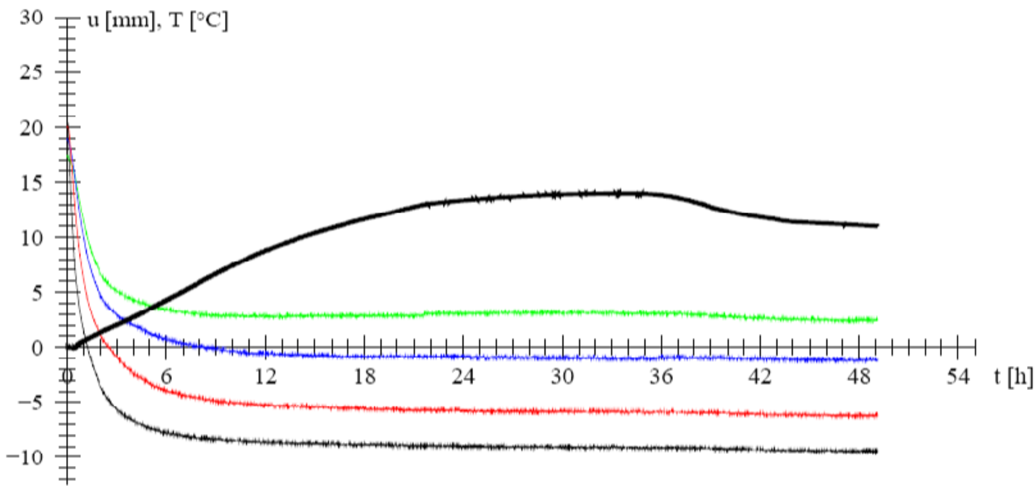


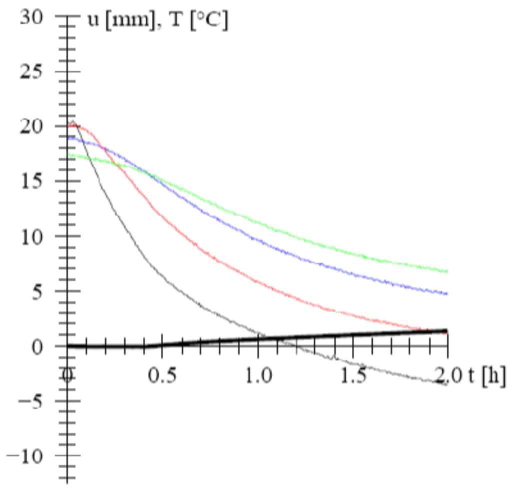


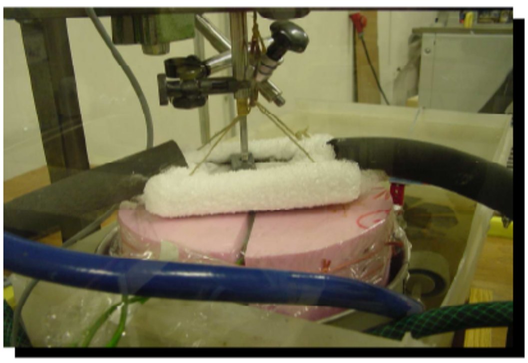


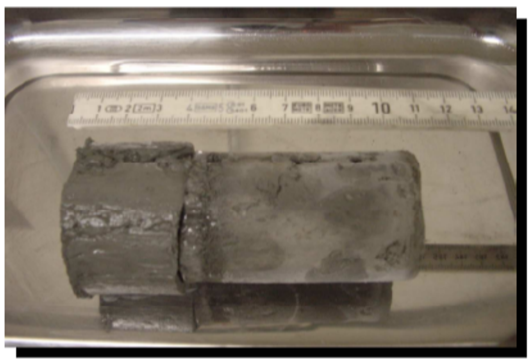


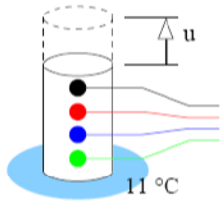
sample: 006 (silt 1) dimensions (H/D): 100/50 mm	water content:	
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.7 % (before)	
remark: −−	37.3 % (after − overall) −− (after − frozen) −− (after − unfrozen)	

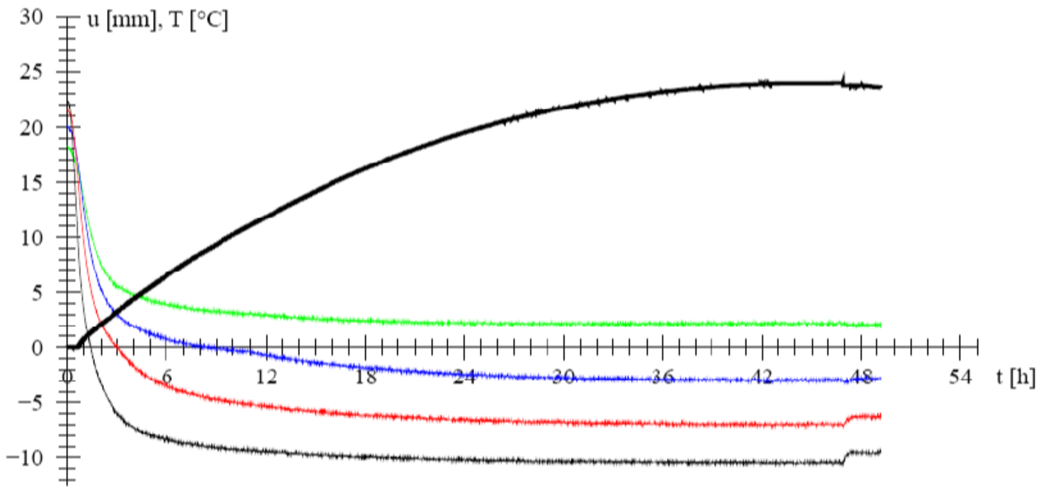


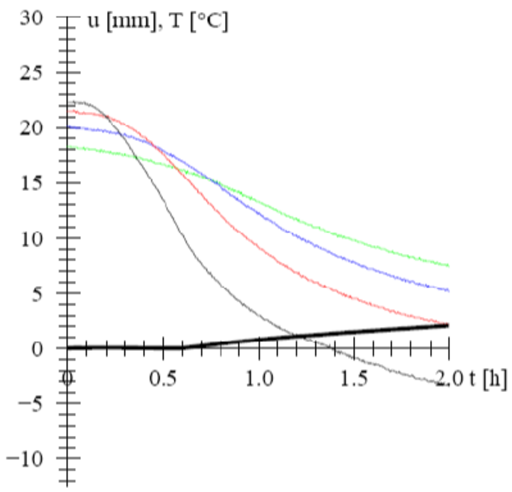


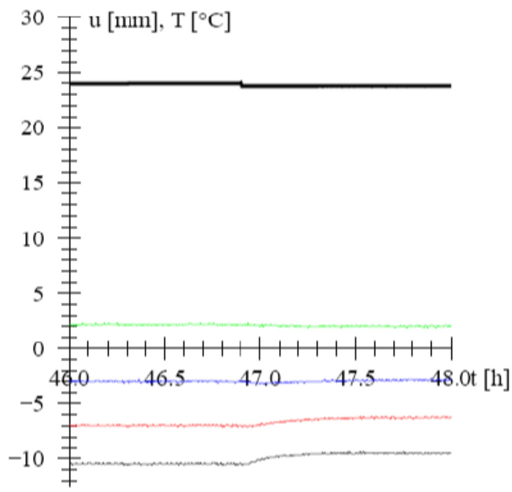




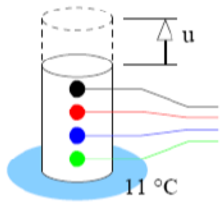
sample: 007 (silt 1) dimensions (H/D): 100/50 mm	water content:	
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.2 % (before)	
remark: −−	40.4 % (after − overall) −− (after − frozen) −− (after − unfrozen)	

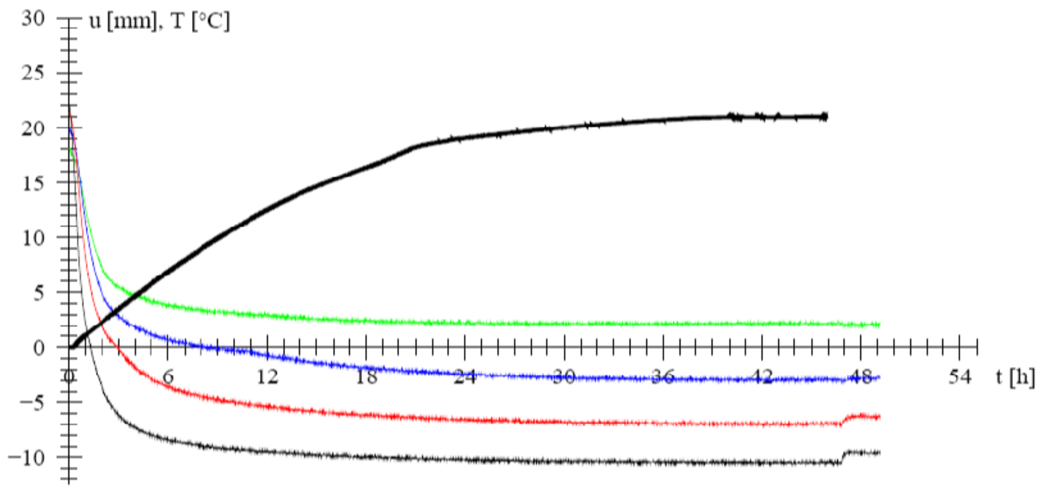


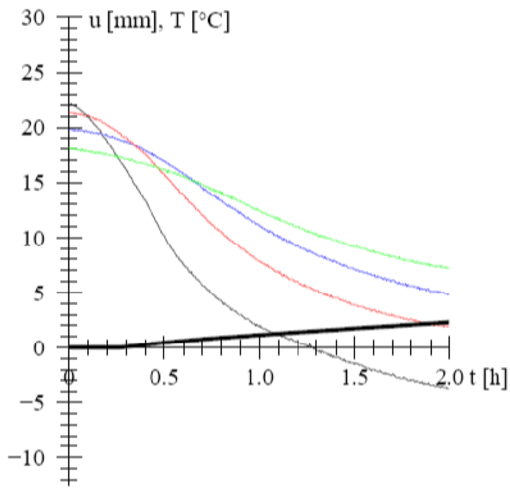


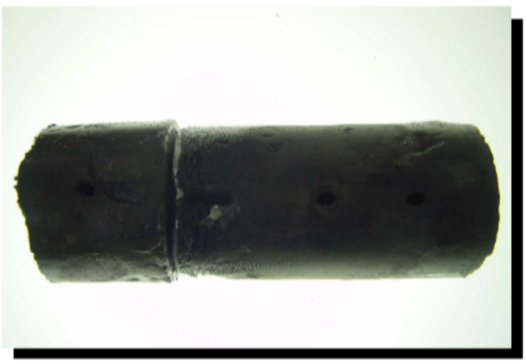


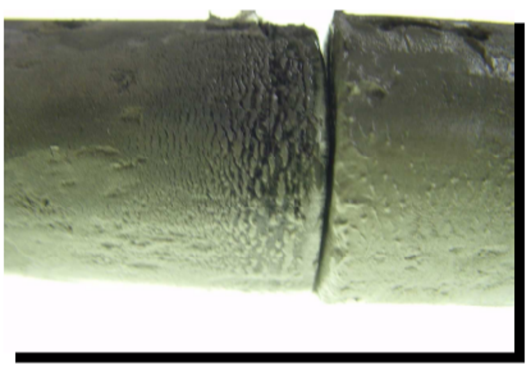
sample: 008 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.4 % (before)
remark:	39.2 % (after − overall) 43.4 % (after − frozen) 31.9 % (after − unfrozen)



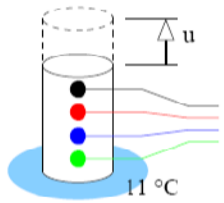


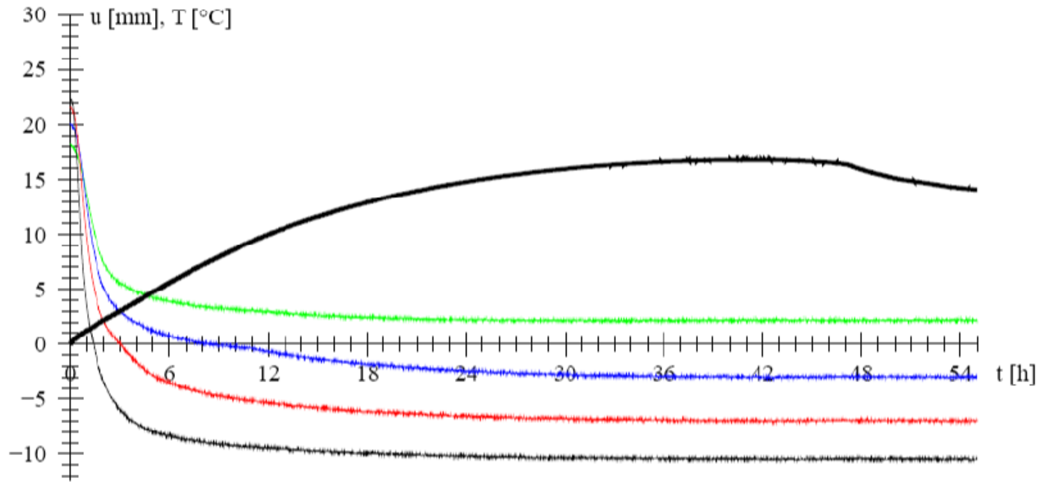


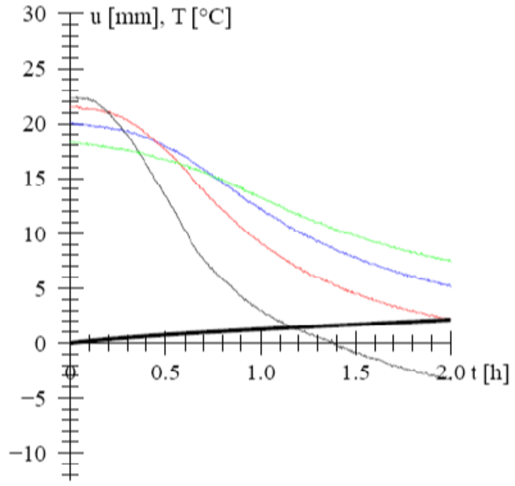


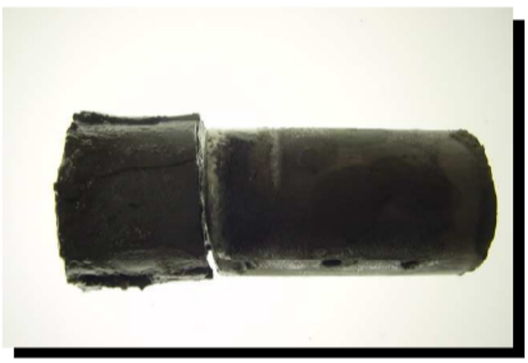


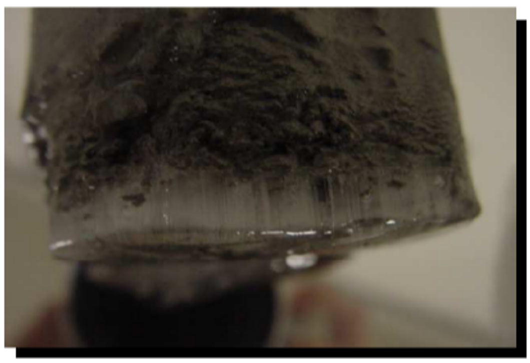
sample: 009 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	27.1 % (before)
remark: −−	38.2 % (after − overall)
	40.0 % (after − frozen)
	35.5 % (after − unfrozen)



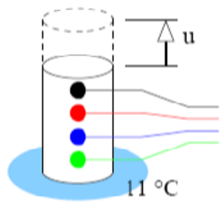


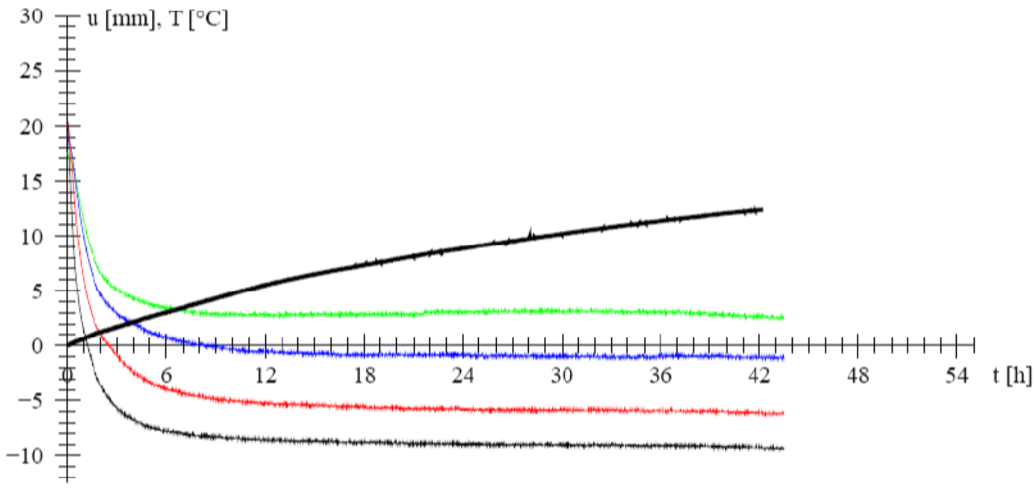


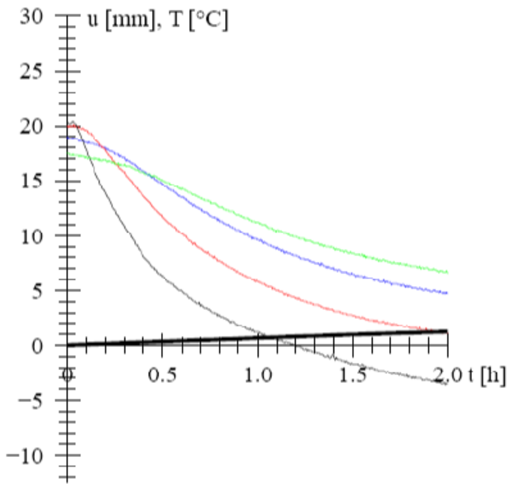


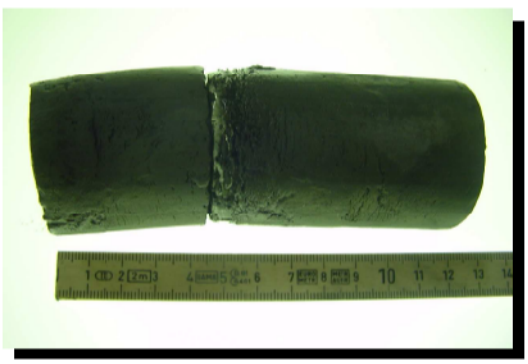


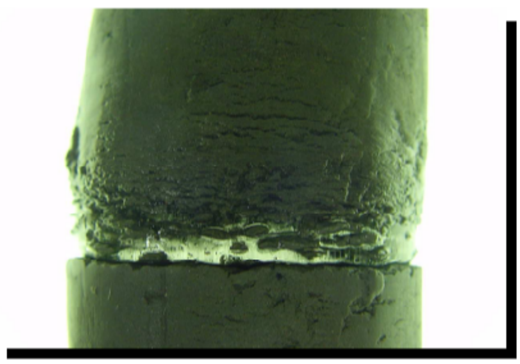
sample: 010 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.2 % (before)
remark: −−	32.2 % (after − overall) 35.7 % (after − frozen) 27.1 % (after − unfrozen)











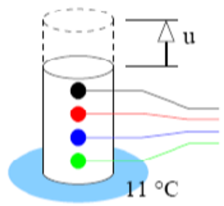
sample: 011 (silt 2) dimensions (H/D): 100/50 mm

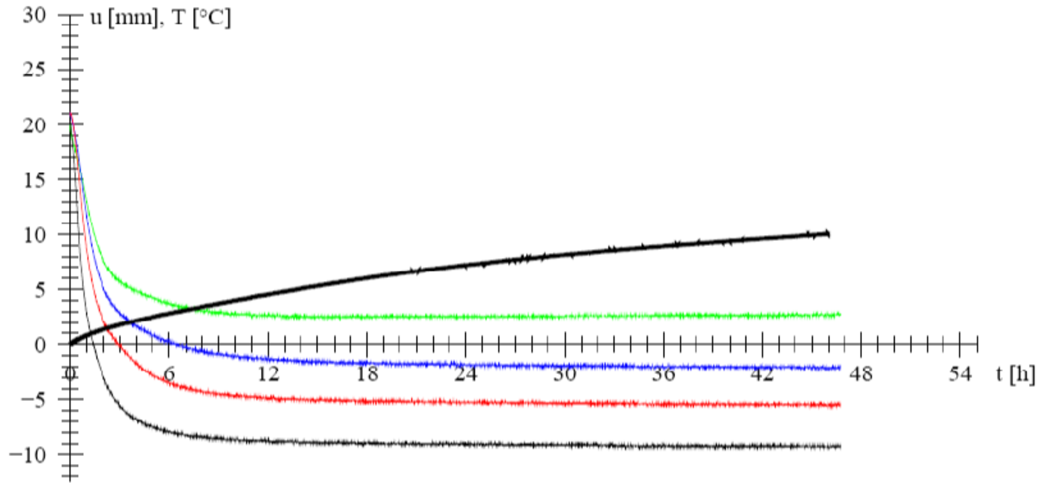
experiment: one−step freezing (cooling rate: 50.0 °C/h)

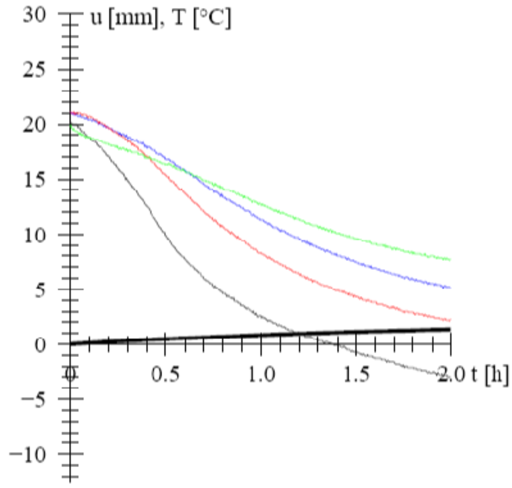
remark: −−

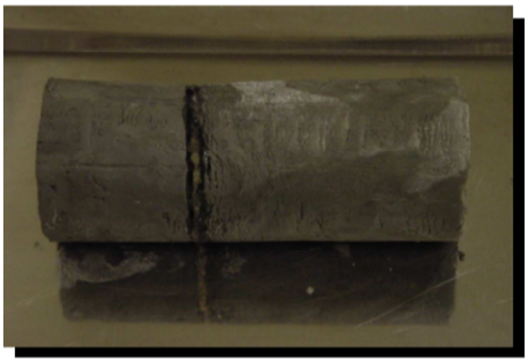
water content:

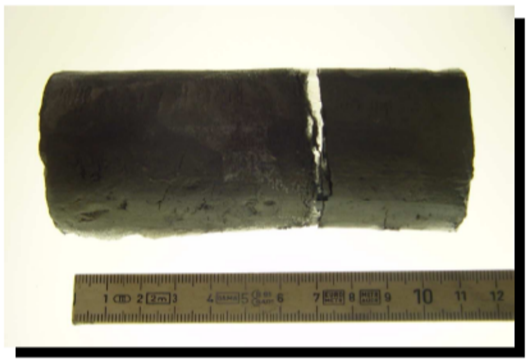
27.1 % (before)
30.8 % (after − overall)
33.6 % (after − frozen)
26.1 % (after − unfrozen)



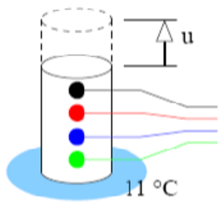


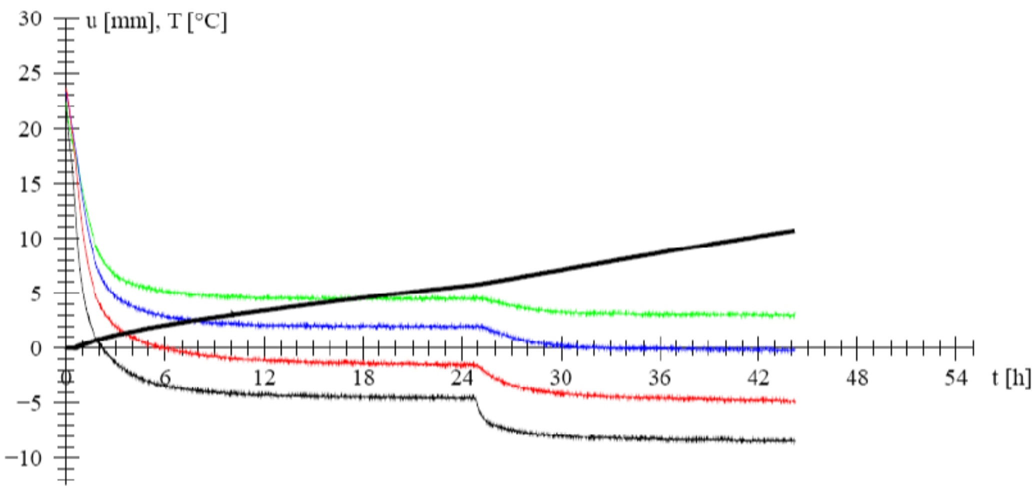


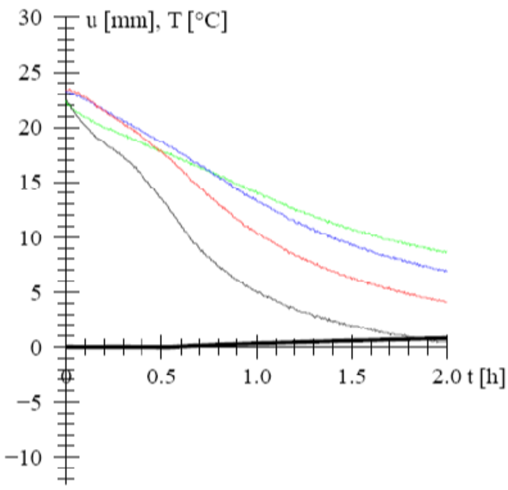


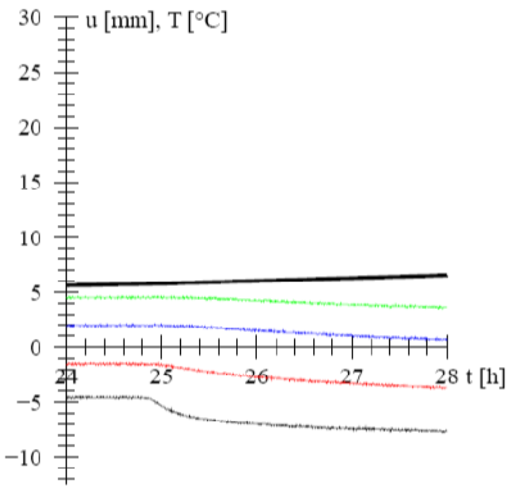


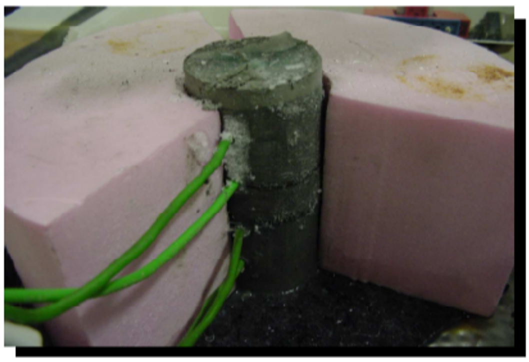
sample: 012 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.3 % (before) 31.5 % (after − overall) 34.4 % (after − frozen) 27.5 % (after − unfrozen)
remark: −−	



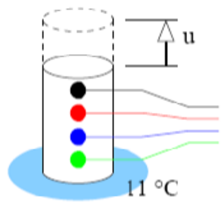


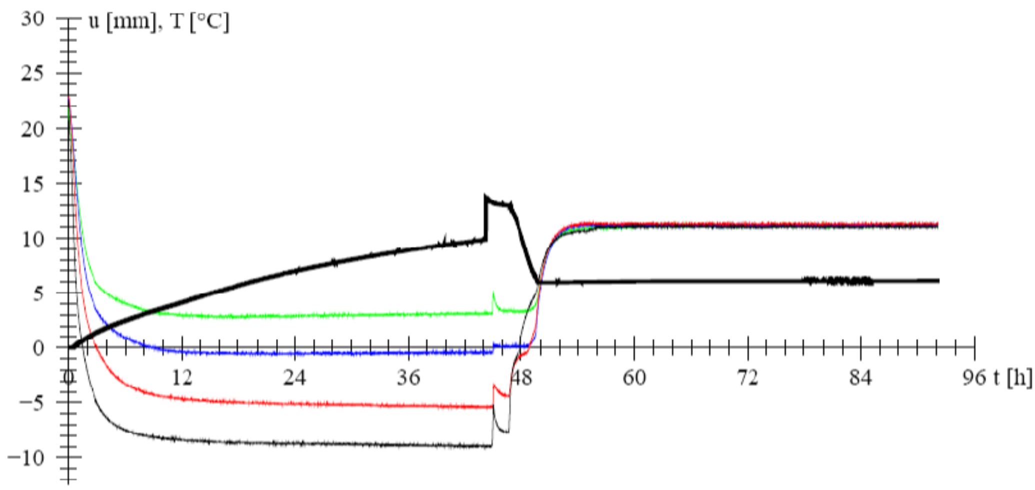


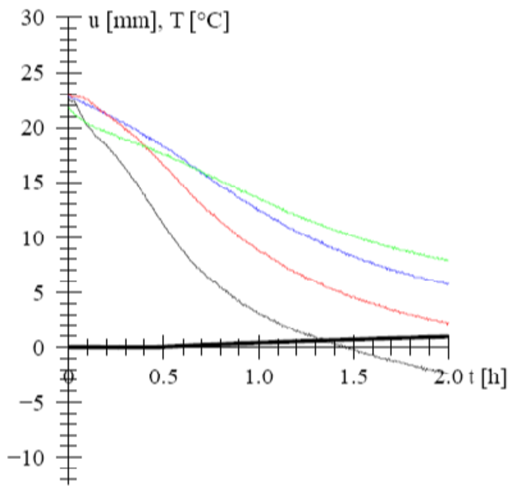


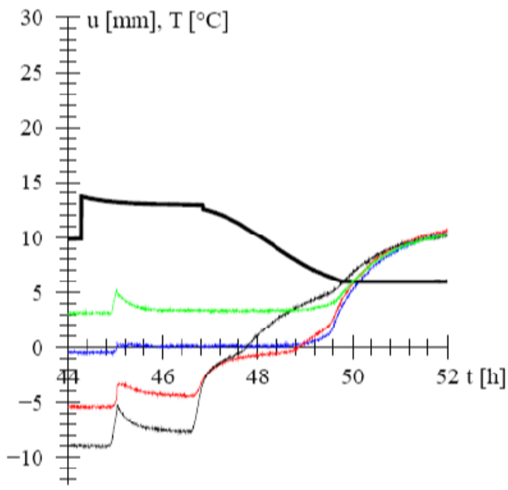


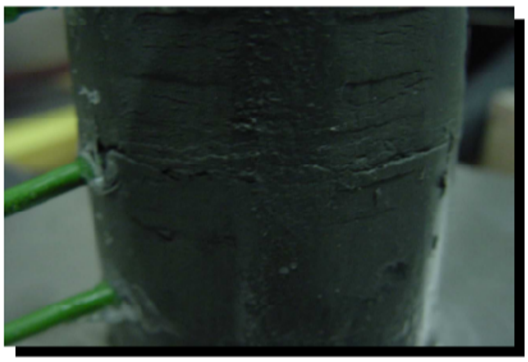
sample: 013 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	28.0 % (before)
remark: sample was uninstalled after freezing and reintegrated for thawing to get the weight after the freezing process	33.2 % (after − overall) 32.7 % (after − frozen) 34.0 % (after − unfrozen)



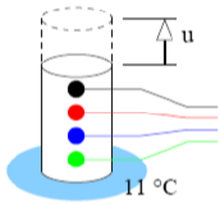


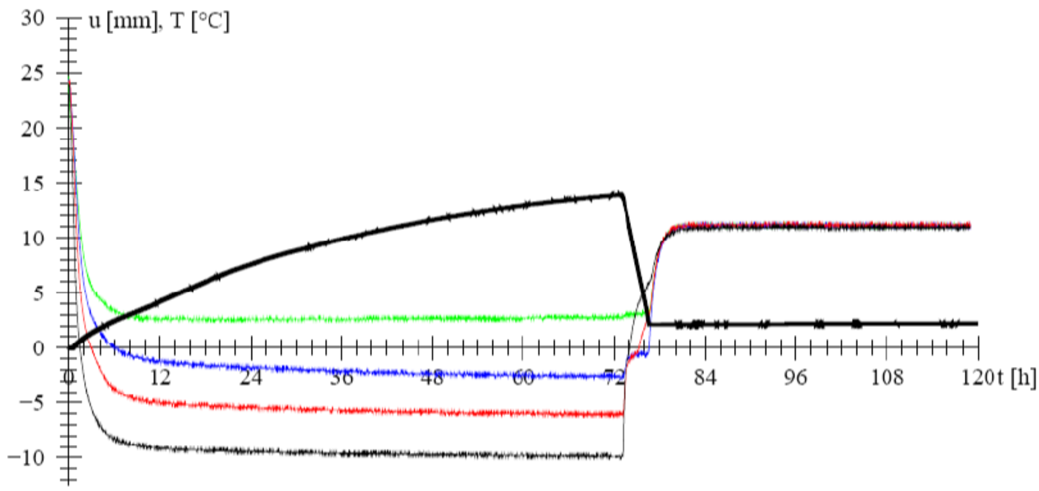


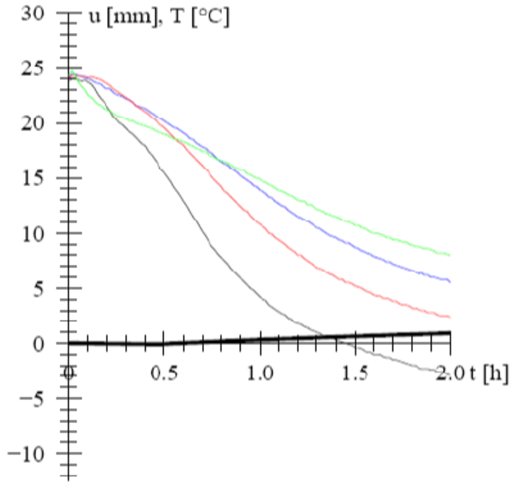


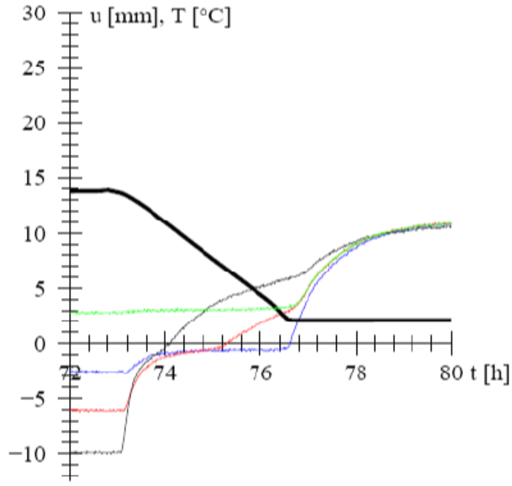


sample: 014 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	26.3 % (before)
remark: electrical power outage for one hour and ten minutes during the thawing process, measurement continued after this	30.6 % (after – overall) 30.2 % (after – frozen) 31.0 % (after – unfrozen)

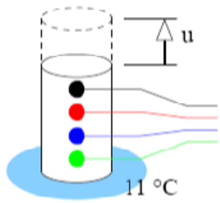


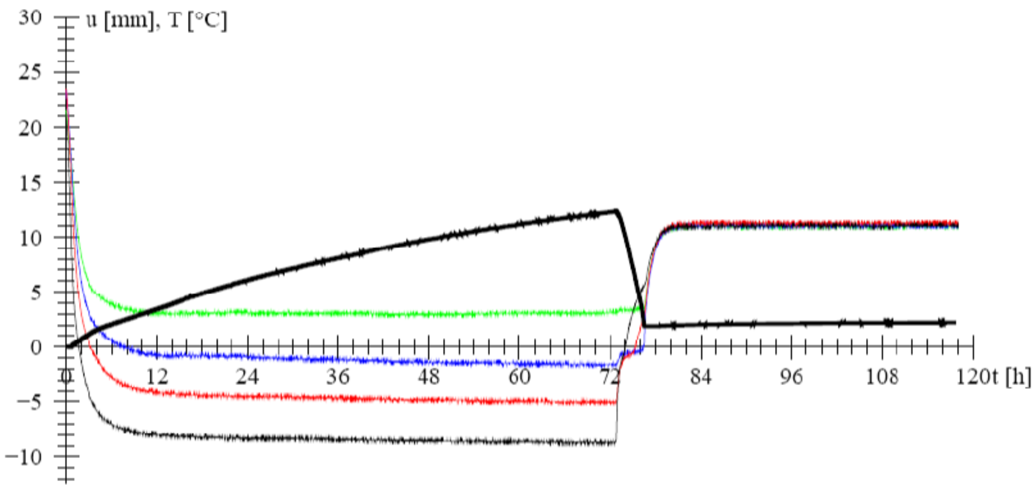


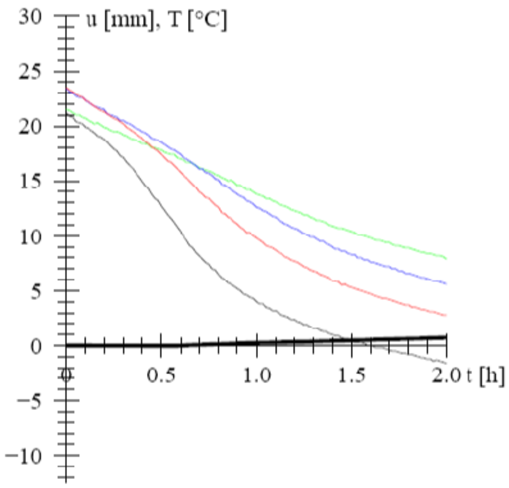


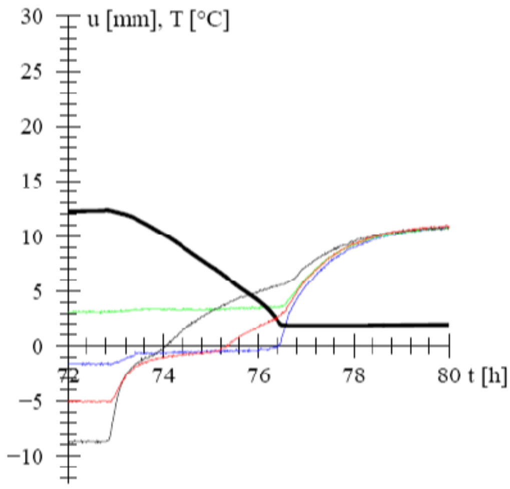


sample: 015 (silt 2) dimensions (H/D): 100/49.5 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.2 % (before)
remark: —	31.7 % (after – overall)
	30.6 % (after – frozen)
	33.3 % (after – unfrozen)

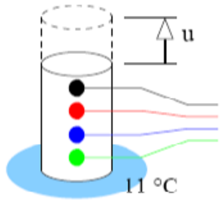


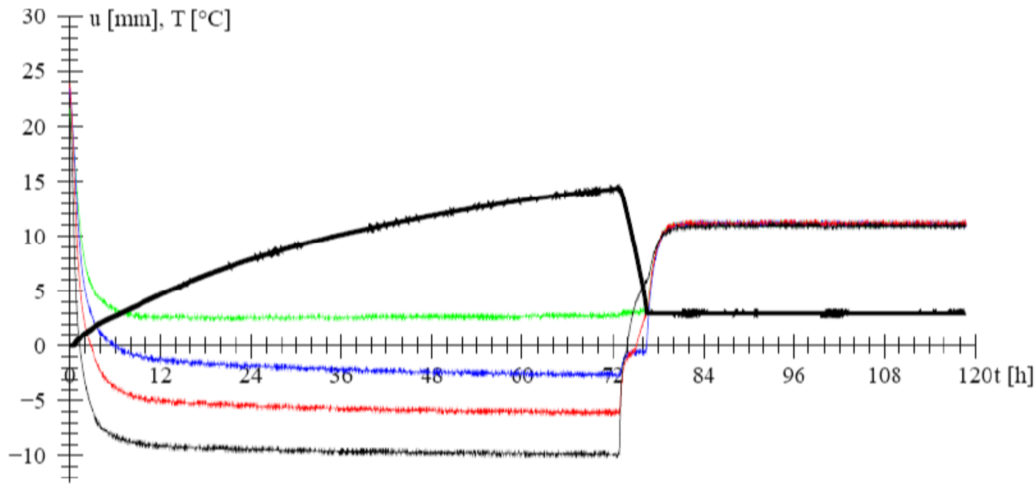


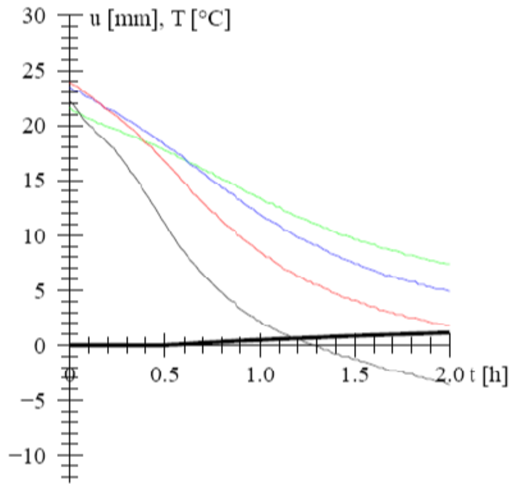


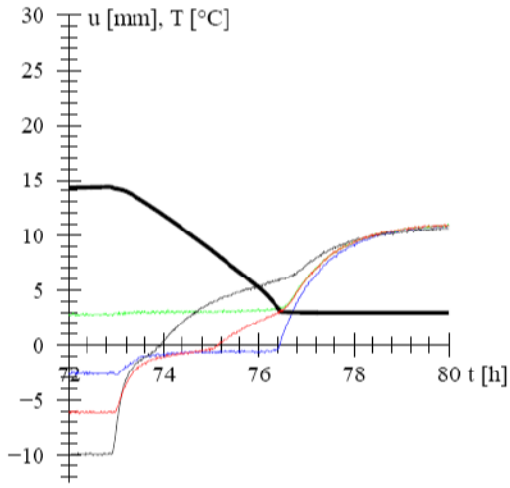


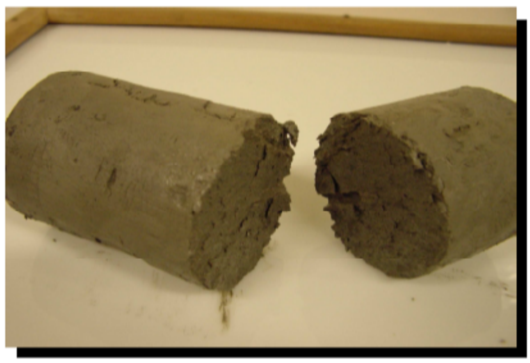
sample: 016 (silt 2) dimensions (H/D): 100/49.9 mm	water content:	
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.4 % (before)	
remark: ––	32.1 % (after – overall) 32.4 % (after – frozen) 31.2 % (after – unfrozen)	



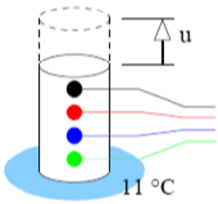


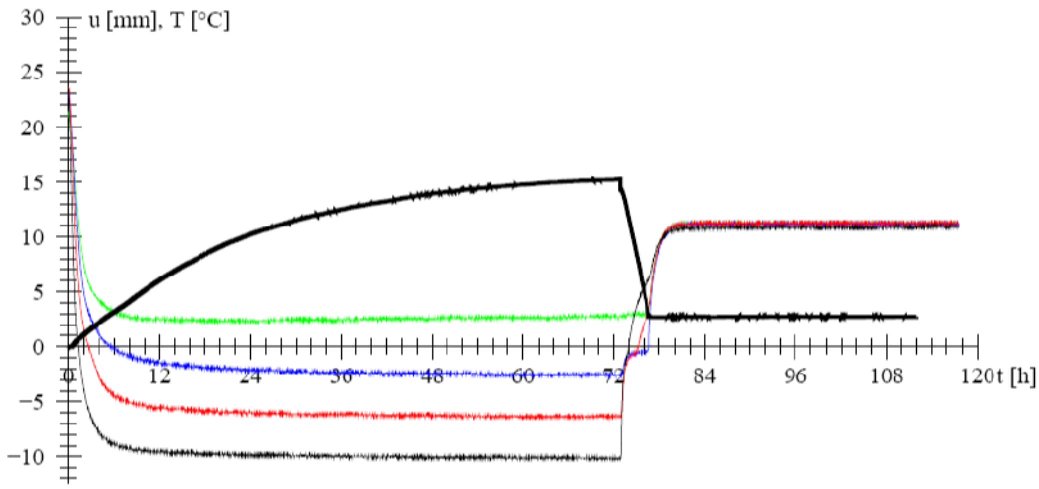


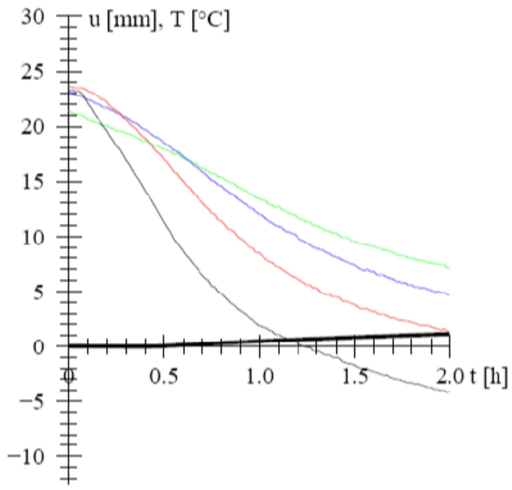


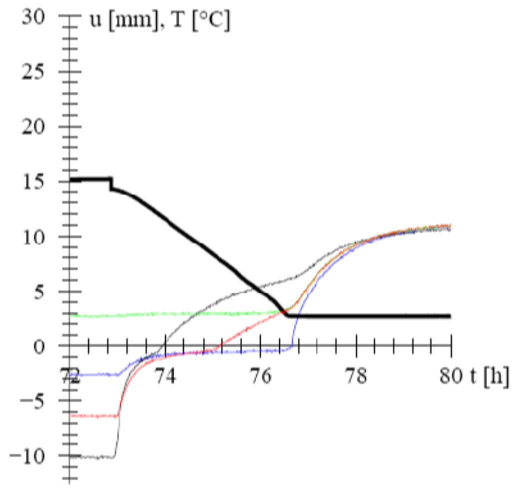


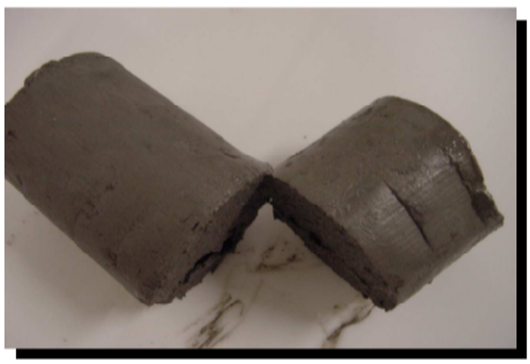
sample: 017 (silt 2) dimensions (H/D): 100/49.9 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.9 % (before)
remark: —	33.6 % (after – overall) 33.9 % (after – frozen) 32.9 % (after – unfrozen)



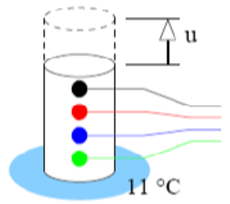


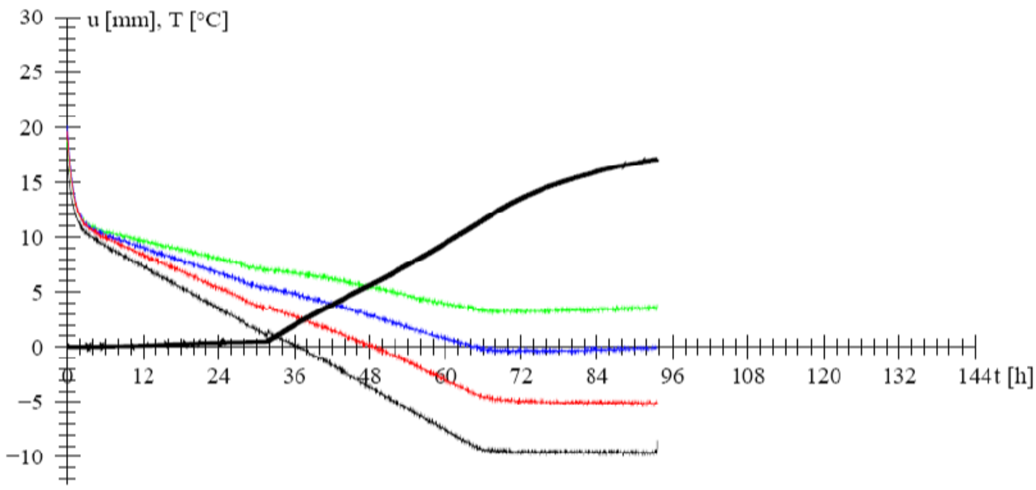


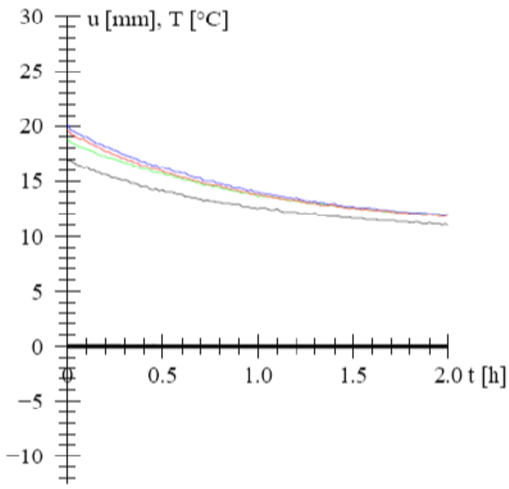


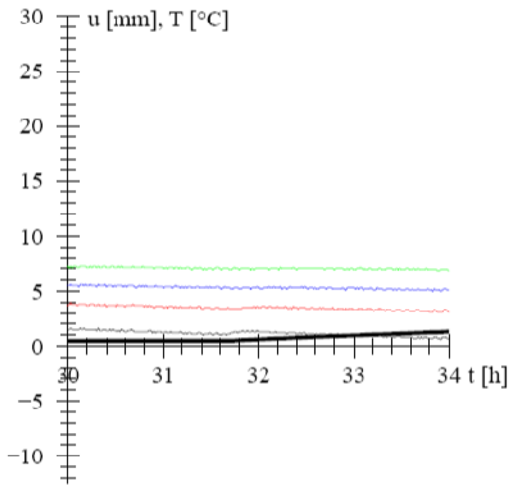


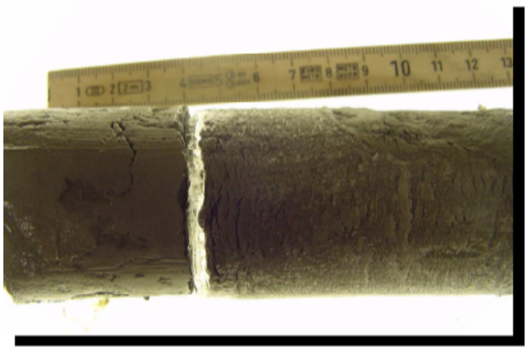
sample: 018 (silt 2) dimensions (H/D): 100/49.6 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	27.8 % (before)
remark: −−	37.6 % (after − overall)
	42.4 % (after − frozen)
	30.5 % (after − unfrozen)

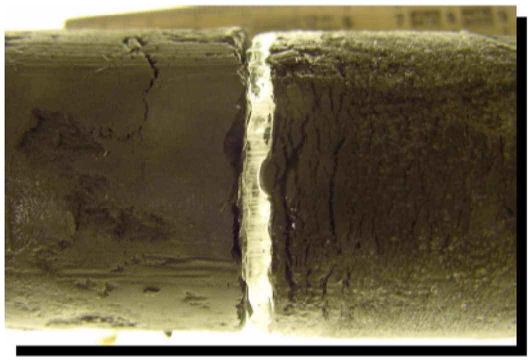




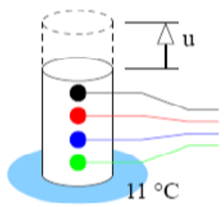


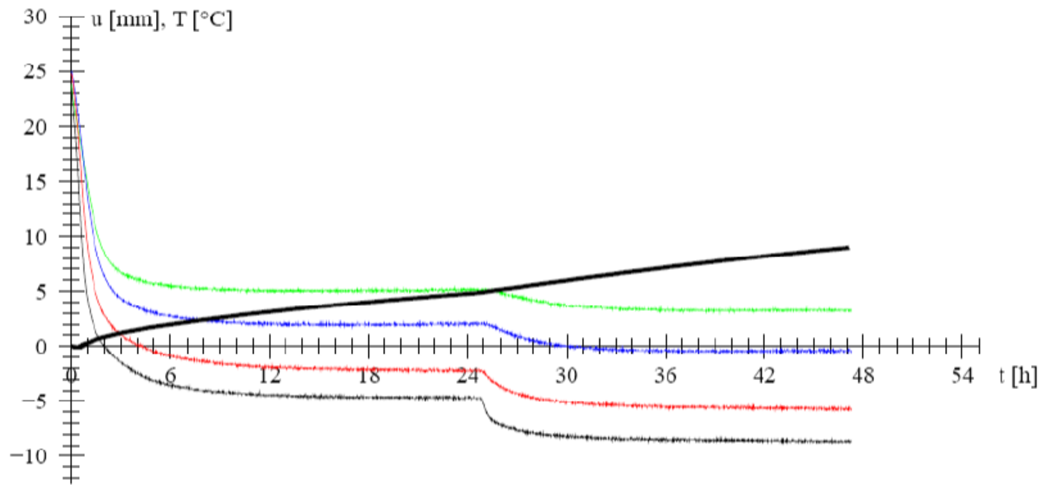


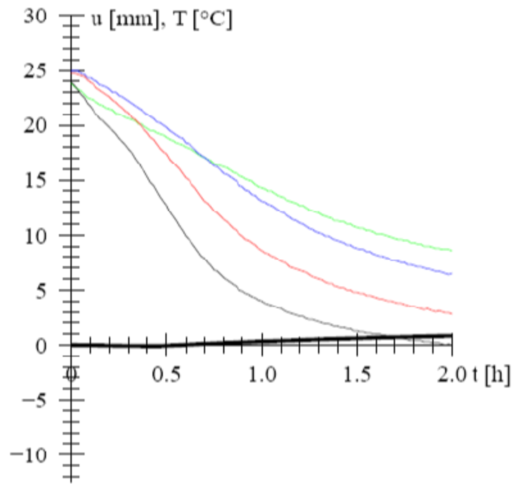


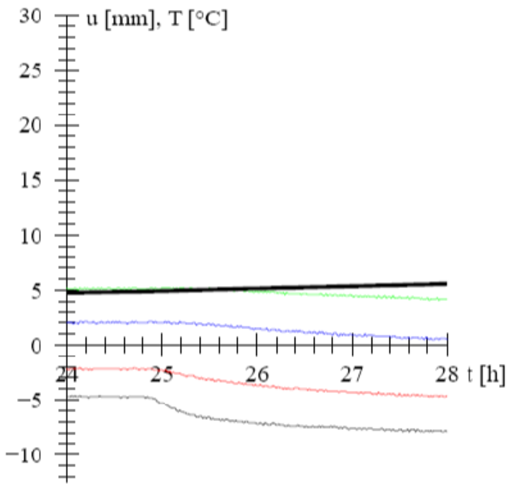


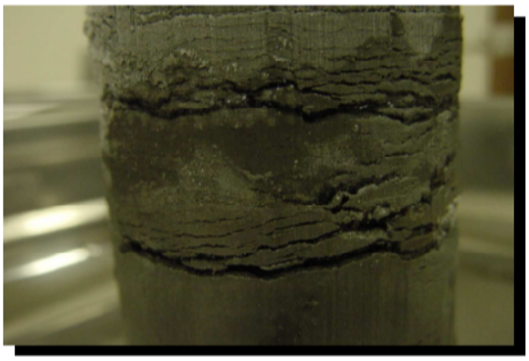
sample: 019 (silt 2) dimensions (H/D): 100/49.7 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.1 % (before) 30.3 % (after − overall) 33.1 % (after − frozen) 26.9 % (after − unfrozen)
remark: ––	

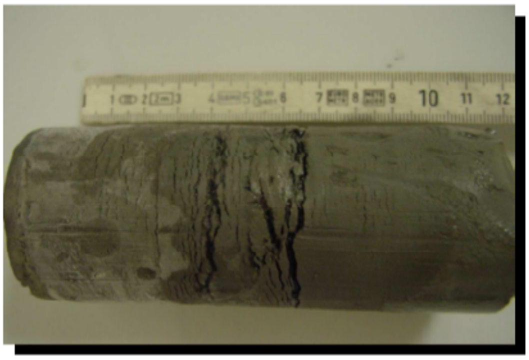




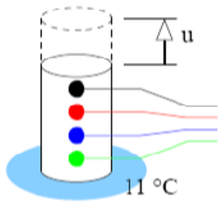


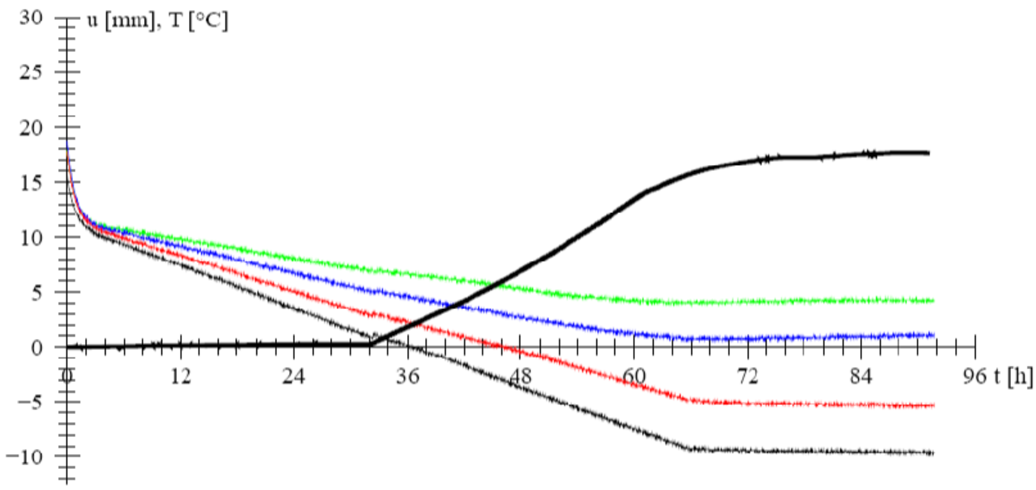


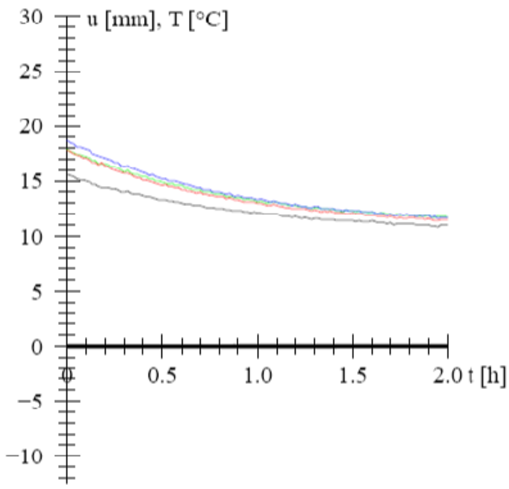


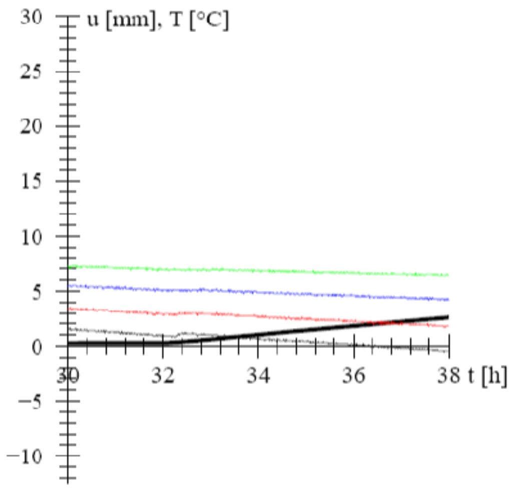


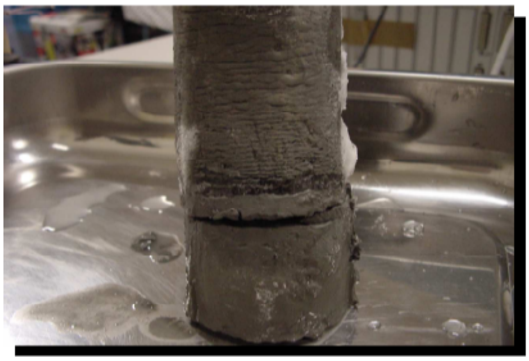
sample: 020 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	26.1 % (before)
remark: ––	40.3 % (after − overall) 45.9 % (after − frozen) 33.7 % (after − unfrozen)



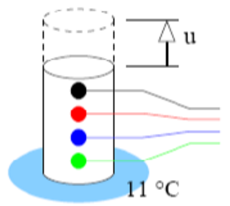


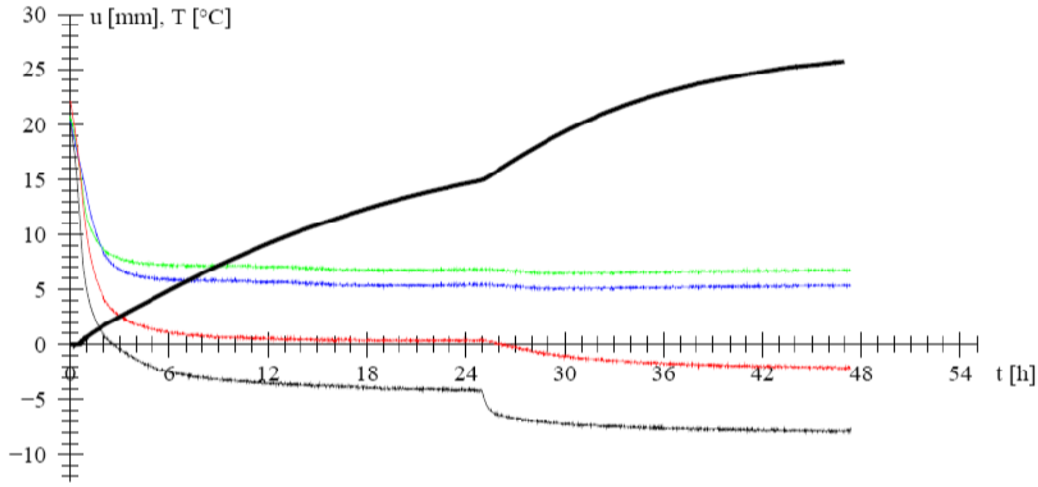


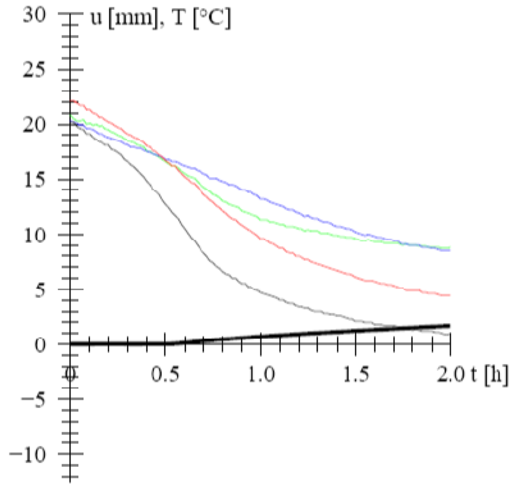


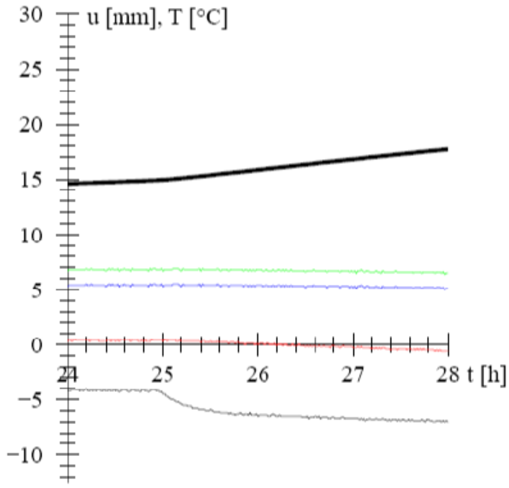


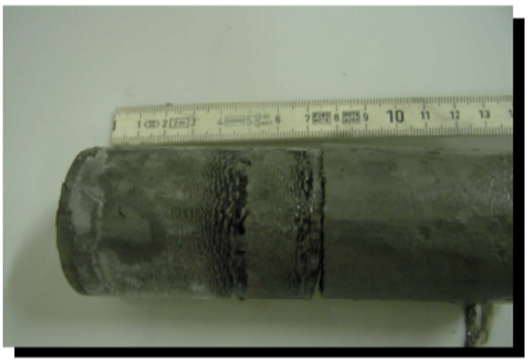
sample: 021 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.5 % (before)
remark: ––	42.8 % (after − overall)
	58.0 % (after − frozen)
	30.2 % (after − unfrozen)

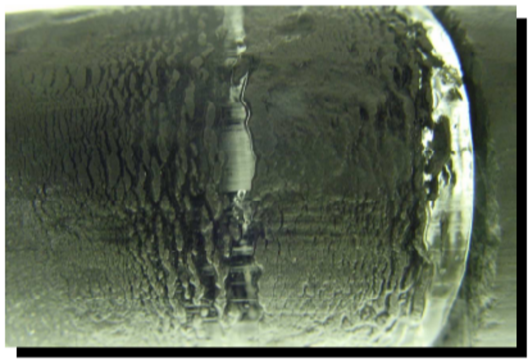




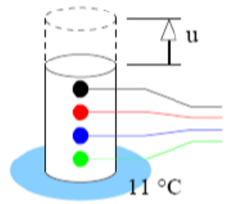


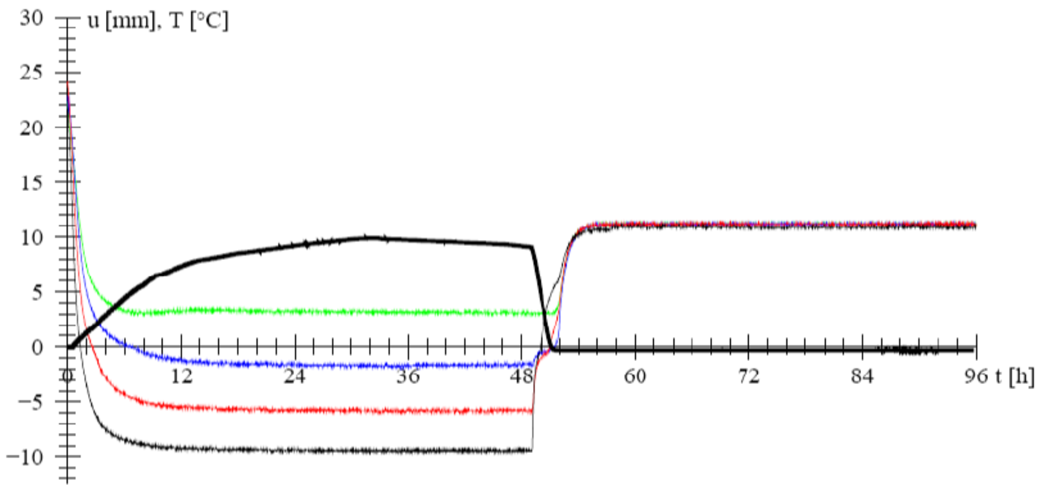


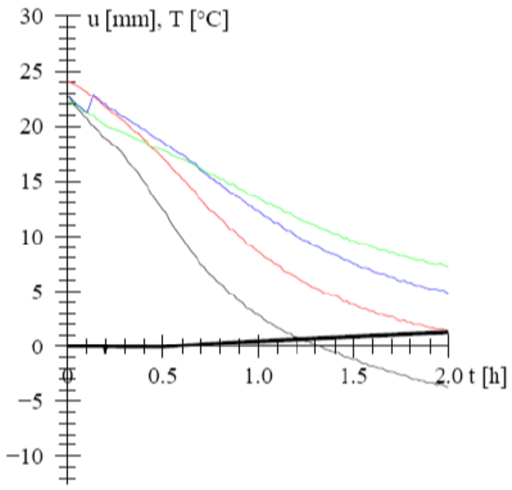


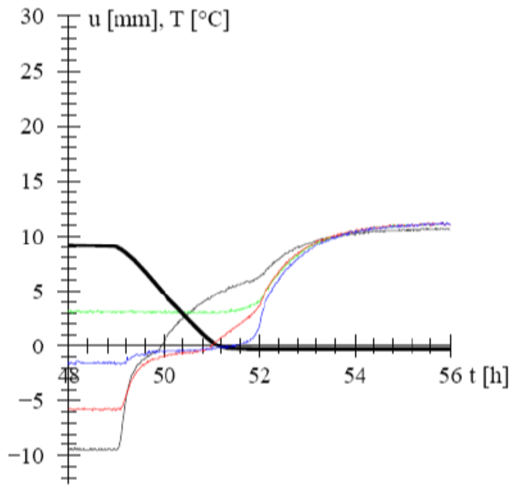


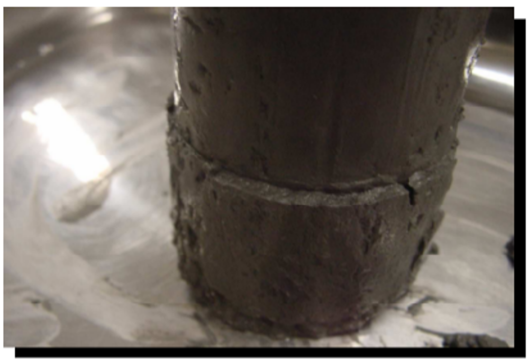
sample: 022 (silt 3) dimensions (H/D): 100/50 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	24.7 % (before)
remark: sample stressed with extra load of 1300g on the top	32.8 % (after – overall) 30.3 % (after – frozen) 36.6 % (after – unfrozen)



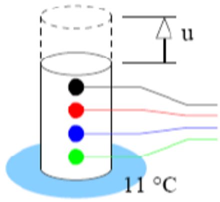


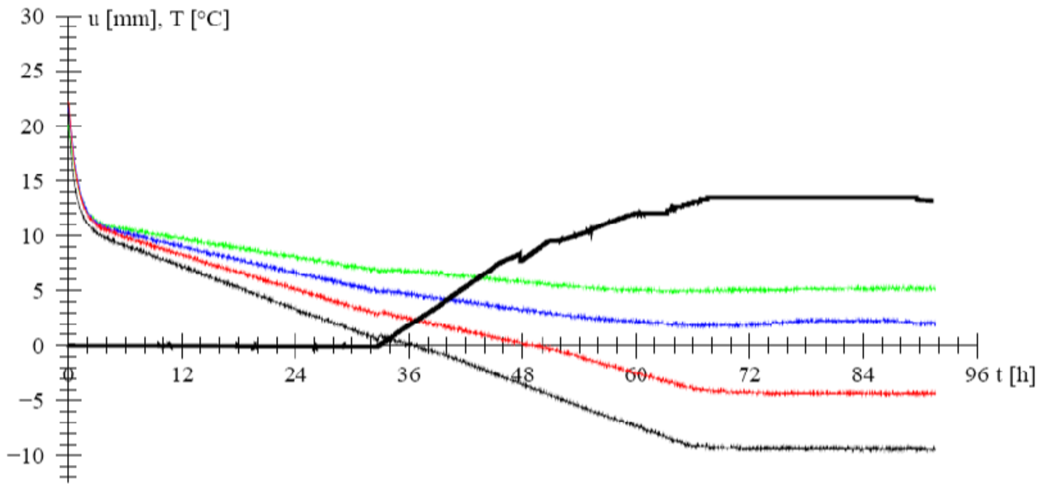


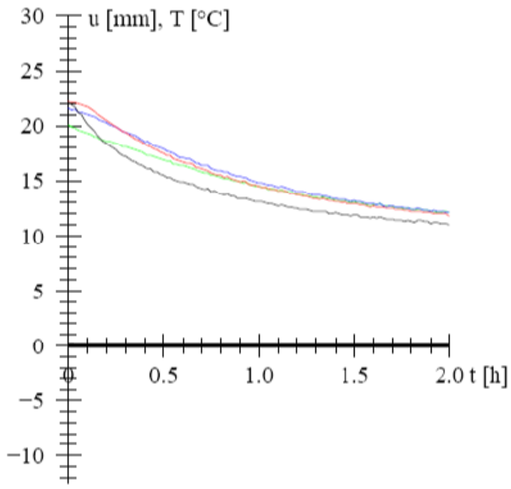


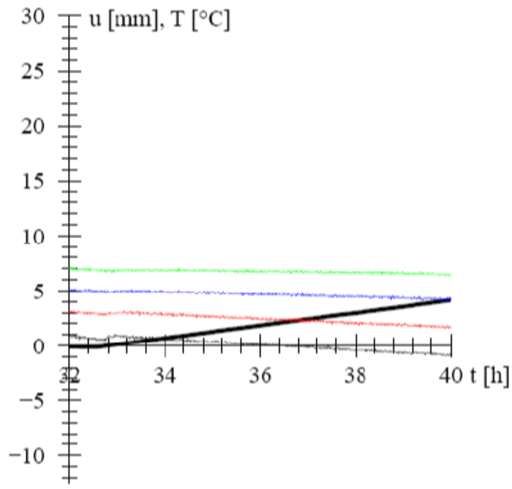


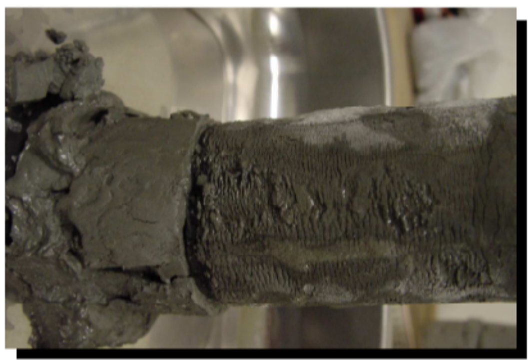
sample: 023 (silt 3) dimensions (H/D): 100/49.6 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	24.6 % (before)
remark: sample stressed with extra load of 1300 g on the top	44.1 % (after − overall) 51.6 % (after − frozen) 36.9 % (after − unfrozen)



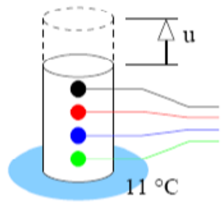


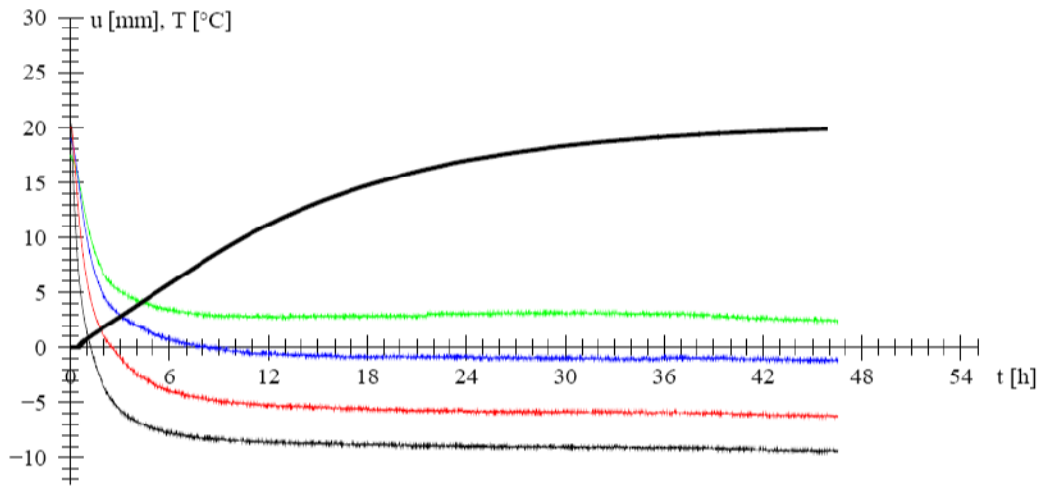


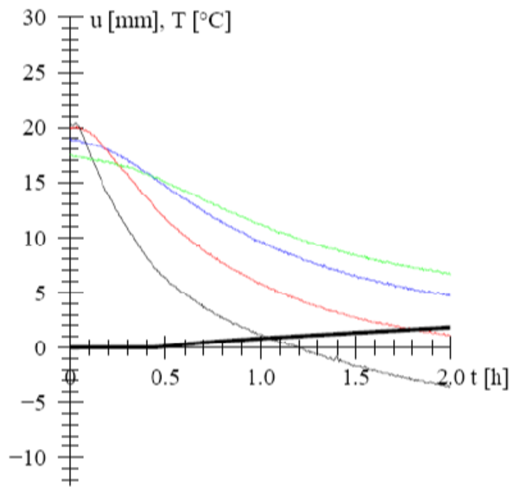


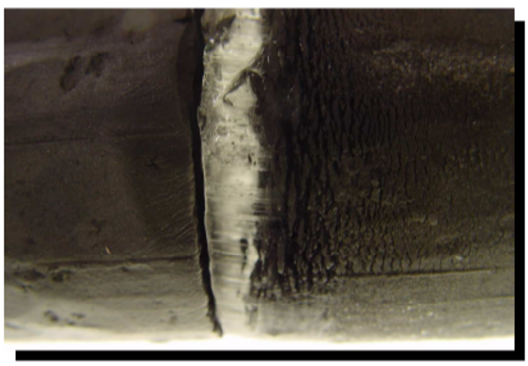


sample: 024 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	25.9 % (before)
remark: −	38.2 % (after − overall) 44.9 % (after − frozen) 29.8 % (after − unfrozen)

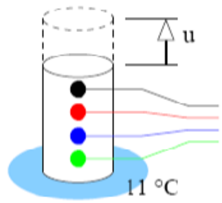


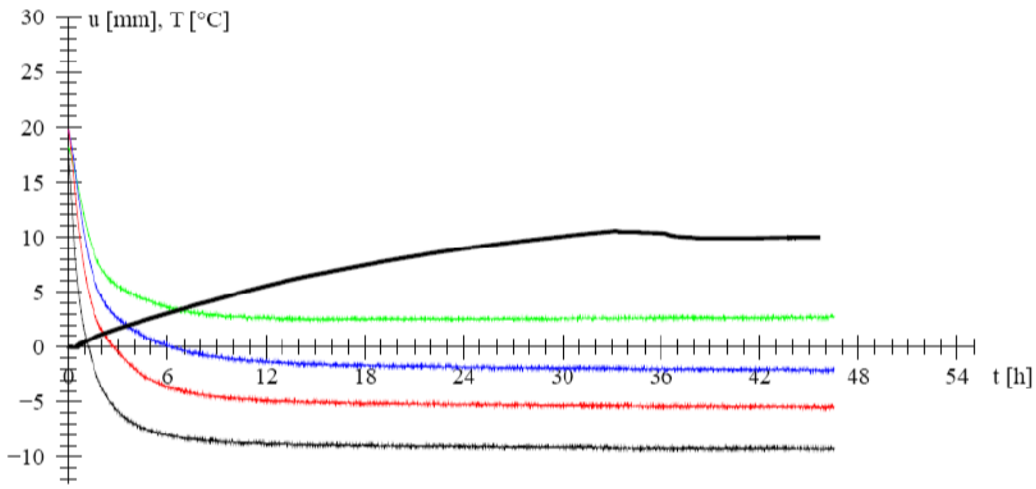


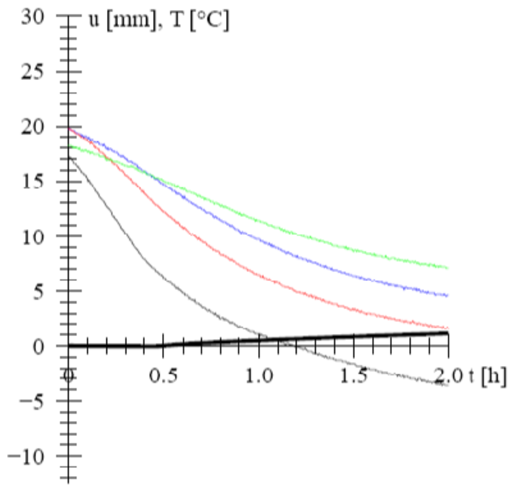


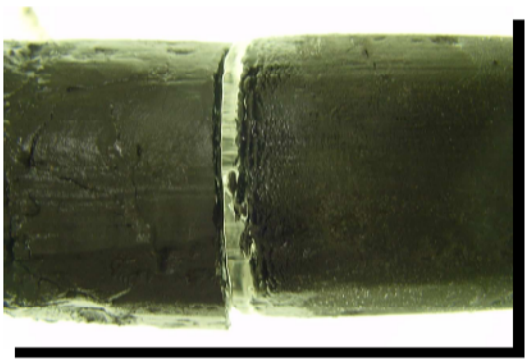


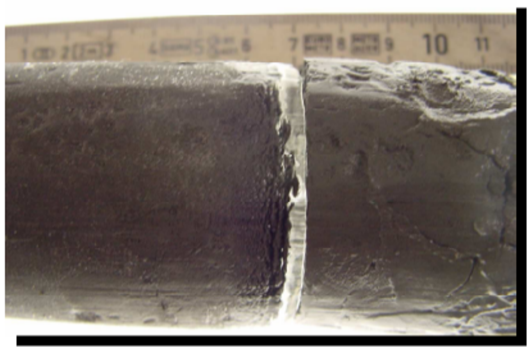
sample: 025 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	23.9 % (before)
remark: ––	30.6 % (after − overall) 30.9 % (after − frozen) 30.0 % (after − unfrozen)



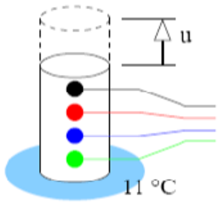


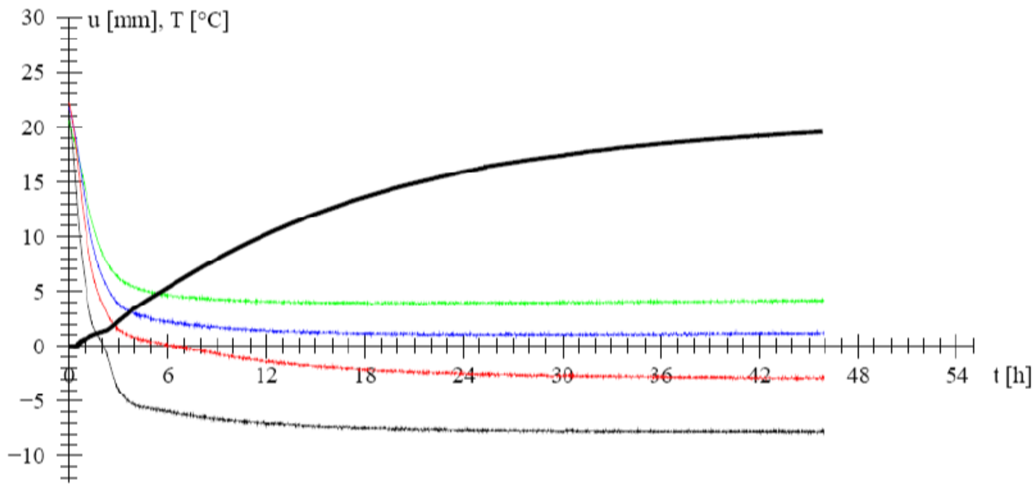


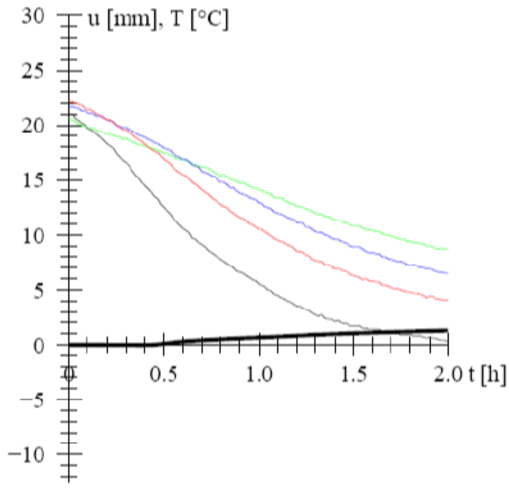


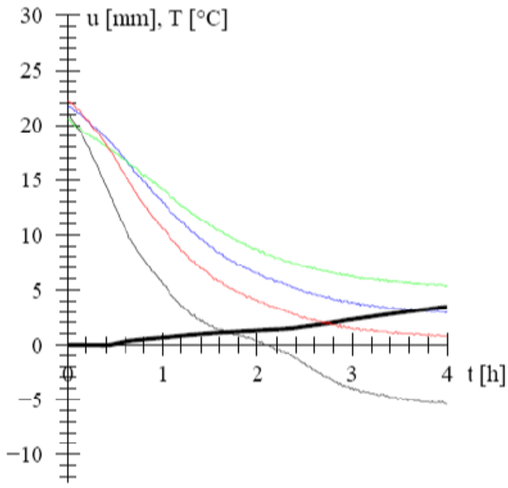


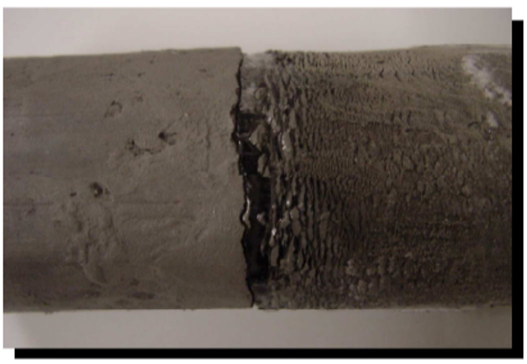
sample: 026 (silt 3) dimensions (H/D): 100/50.2 mm	water content:	
experiment: one−step freezing (cooling rate: 50.0 °C/h)	22.8 % (before)	
remark: −−	37.2 % (after − overall) 48.4 % (after − frozen) 28.6 % (after − unfrozen)	

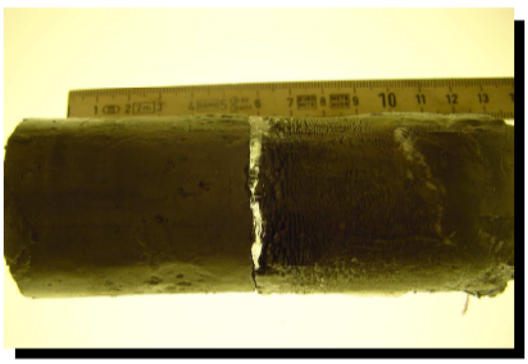




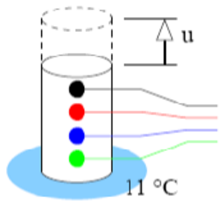


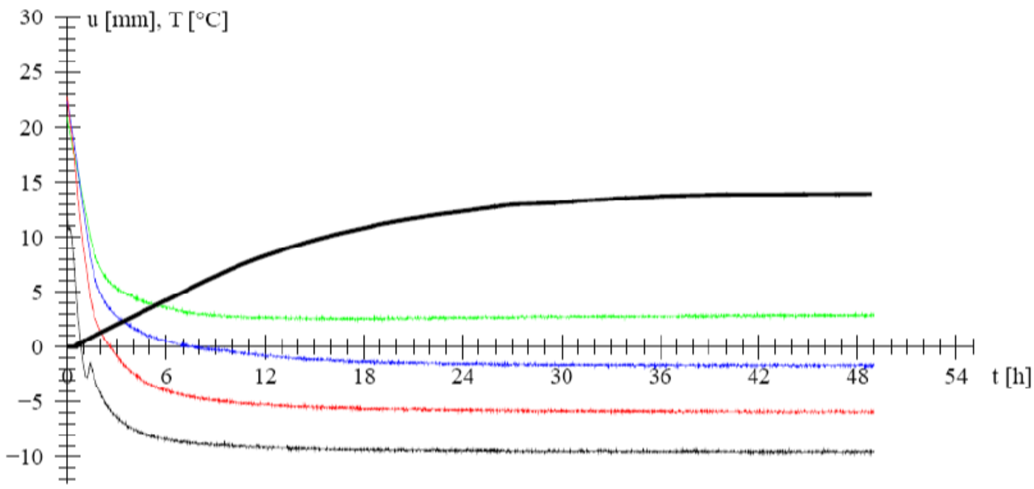


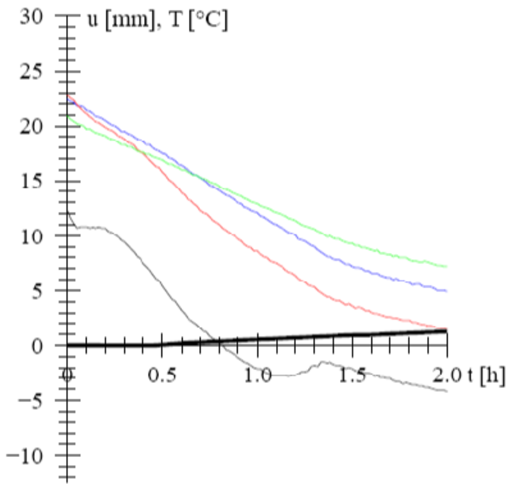


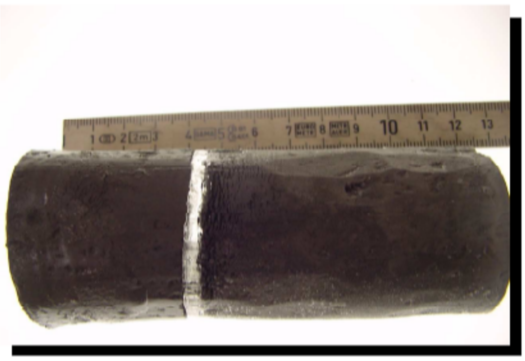


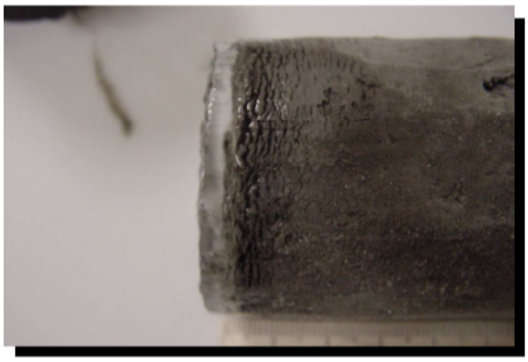
sample: 027 (silt 3) dimensions (H/D): 100/49.3 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	24.6 % (before)
remark: −−	33.6 % (after − overall)
	37.4 % (after − frozen)
	28.1 % (after − unfrozen)



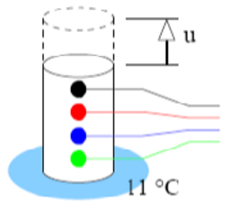


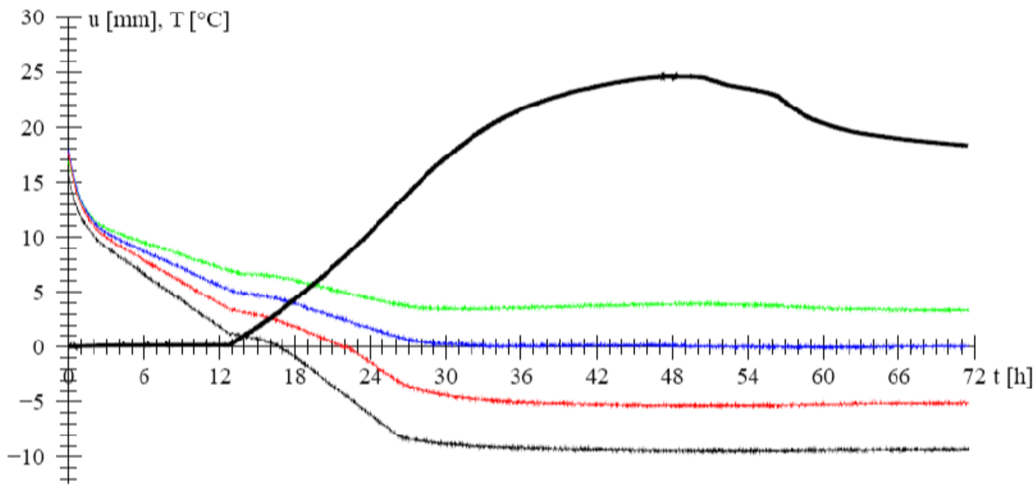


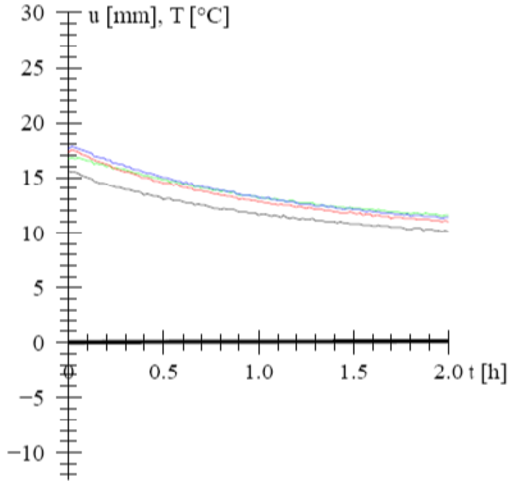


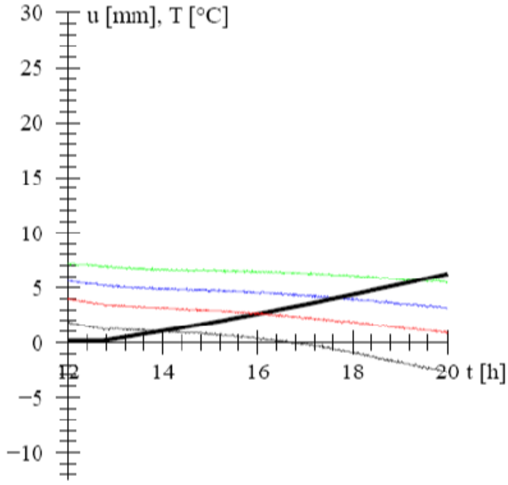


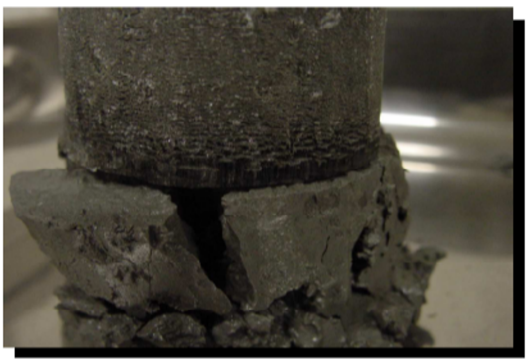
sample: 028 (silt 3) dimensions (H/D): 100/49.9 mm	water content:
experiment: one−step freezing (cooling rate: 1.0 °C/h)	25.5 % (before)
remark: −−	42.0 % (after − overall)
	47.1 % (after − frozen)
	35.7 % (after − unfrozen)











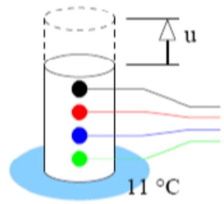
sample: 029 (silt 3) dimensions (H/D): 100/48.8 mm

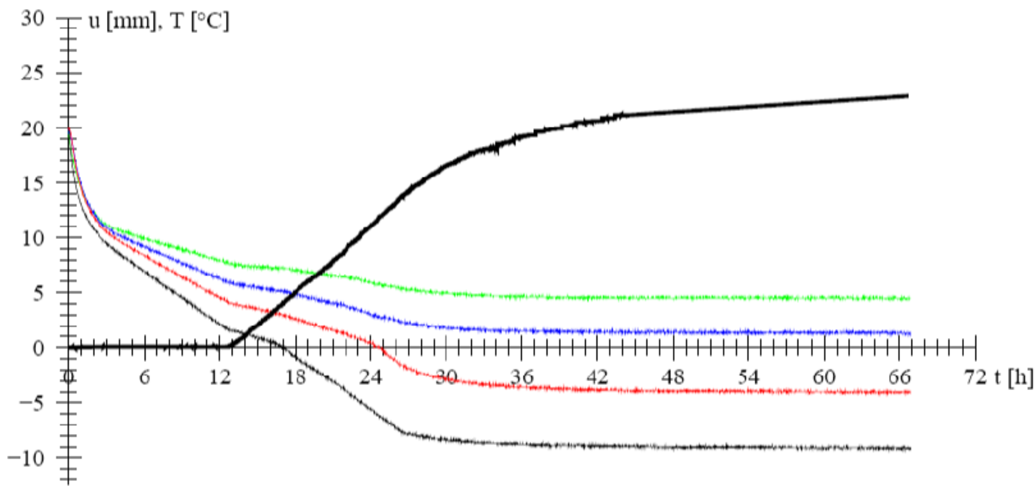
experiment: one−step freezing (cooling rate: 1.0 °C/h)

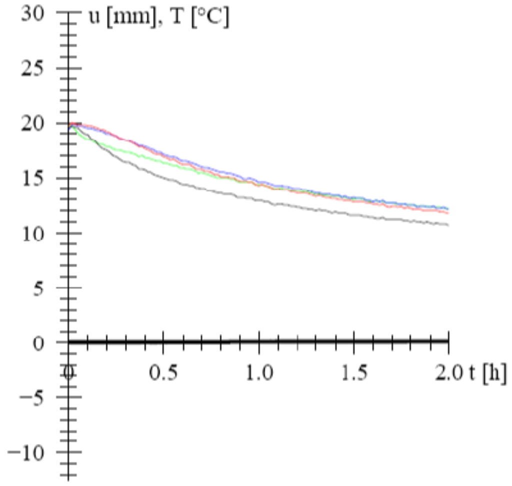
remark: —

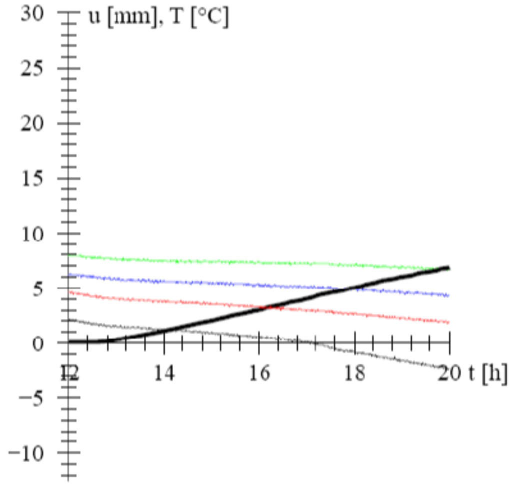
water content:

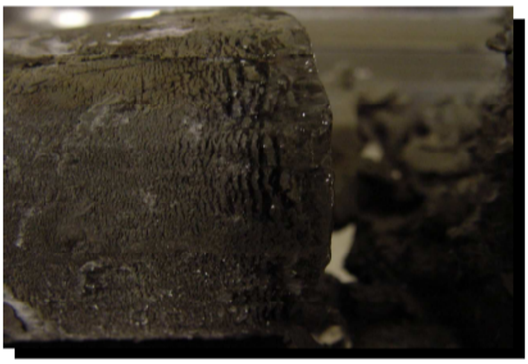
26.7 % (before)
46.4 % (after – overall)
59.9 % (after – frozen)
36.6 % (after – unfrozen)

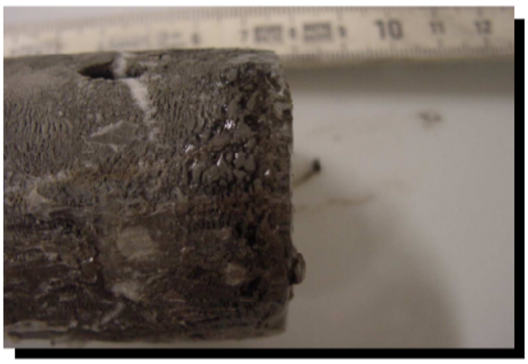












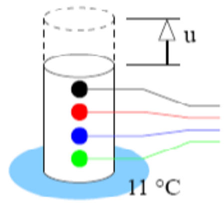
sample: 030 (silt 3) dimensions (H/D): 100/50 mm

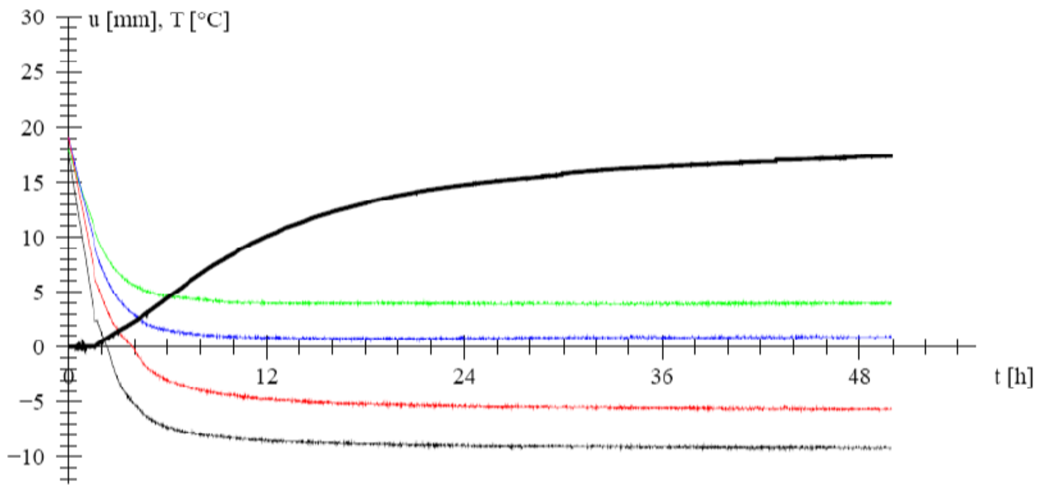
experiment: one−step freezing (cooling rate: 25.0 °C/h)

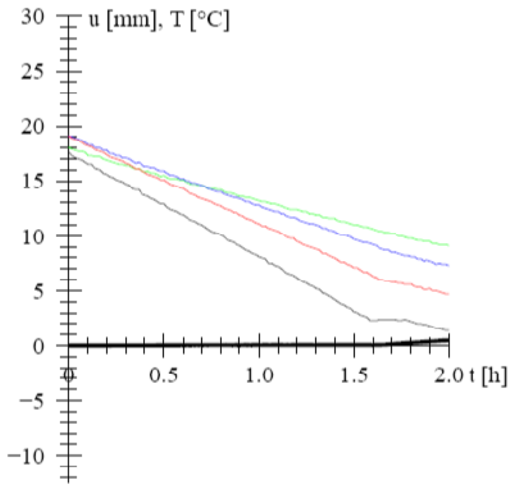
remark: −−

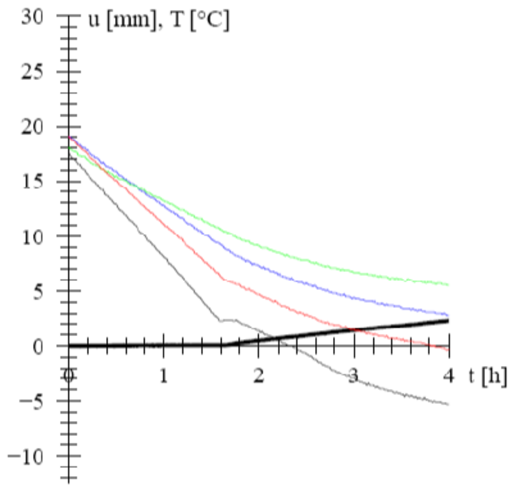
water content:

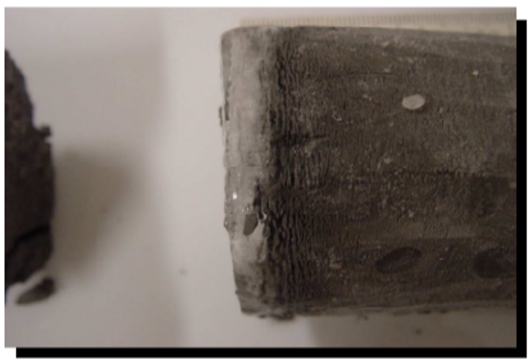
25.8 % (before)
35.1 % (after − overall)
40.7 % (after − frozen)
28.8 % (after − unfrozen)



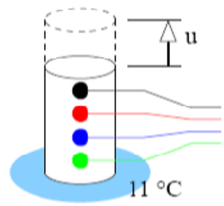


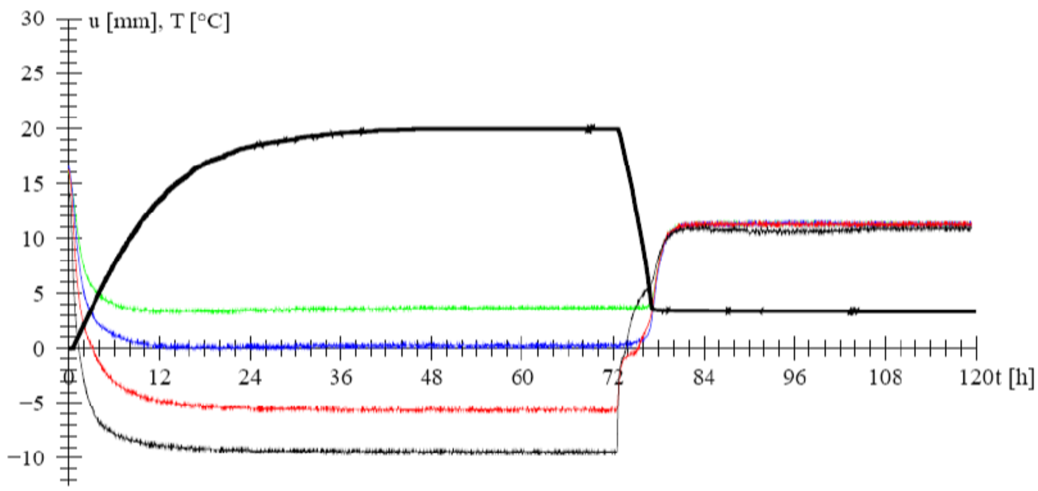


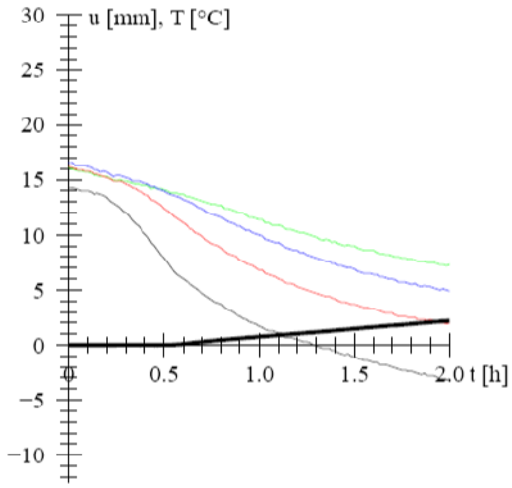


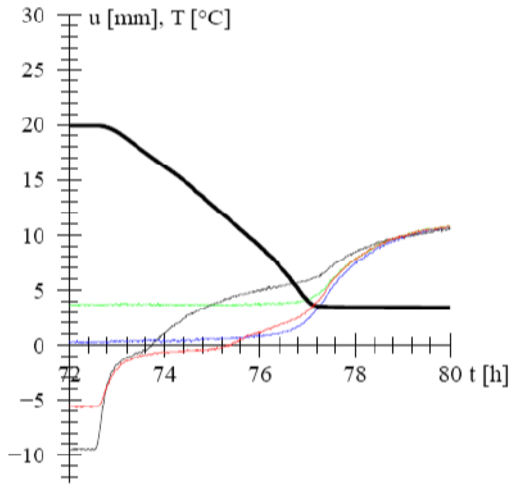


sample: 031 (silt 3) dimensions (H/D): 100/50.2 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	26.8 % (before)
remark: ––	34.1 % (after – overall)
	36.2 % (after – frozen)
	31.2 % (after – unfrozen)

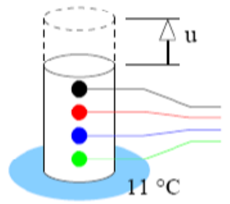


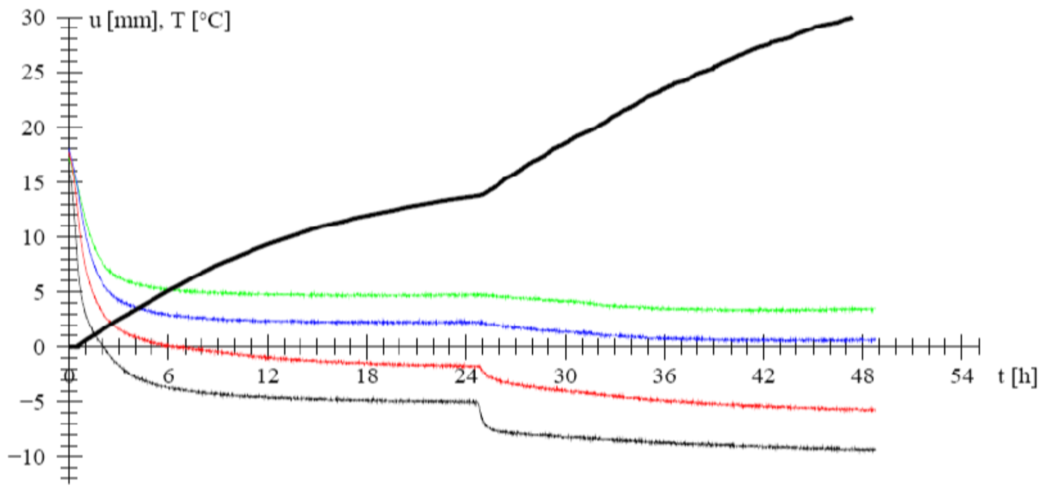


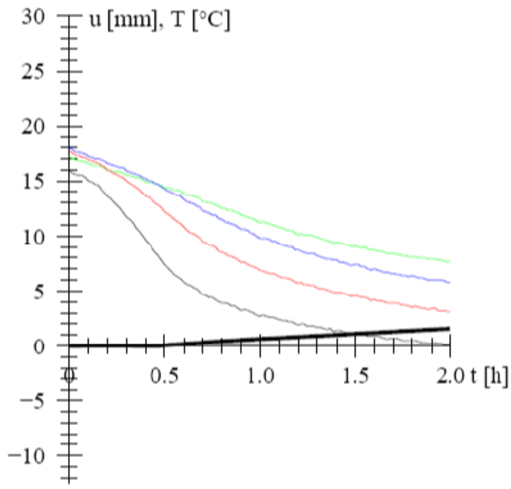


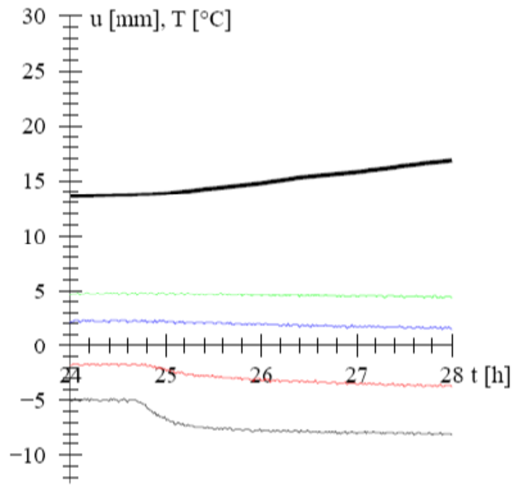


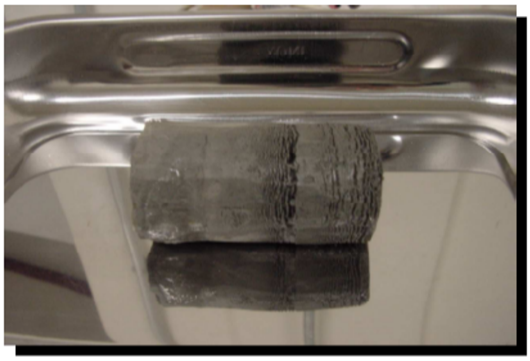
sample: 032 (silt 3) dimensions (H/D): 100/48.5 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	25.1 % (before)
remark: −−	42.6 % (after − overall) 56.0 % (after − frozen) 28.2 % (after − unfrozen)



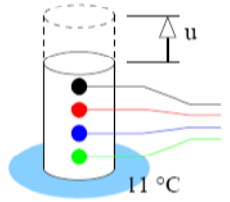


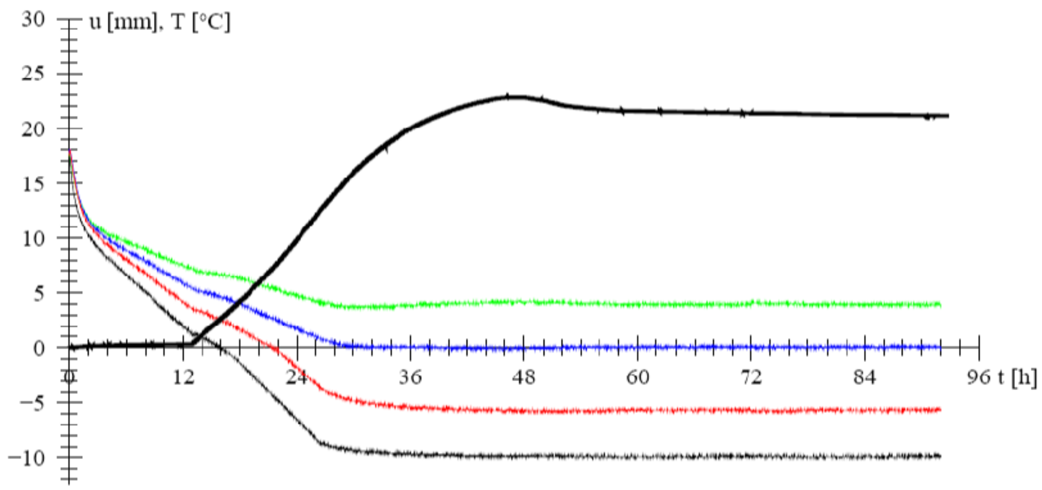


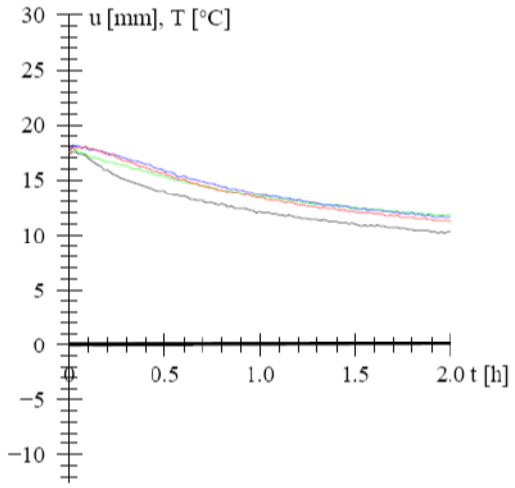


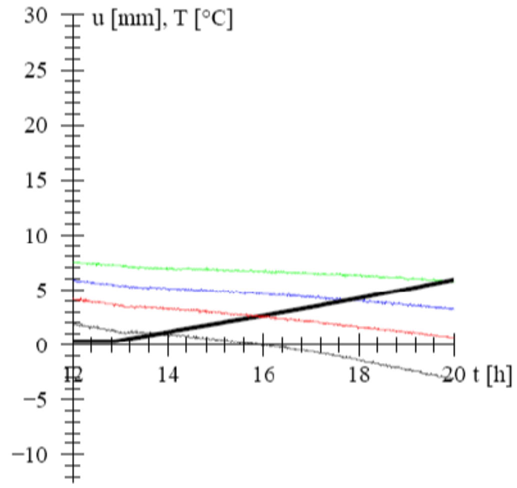


sample: 033 (silt 3) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 1.0 °C/h)	26.6 % (before)
remark: −−	42.3 % (after − overall) 47.3 % (after − frozen) 35.4 % (after − unfrozen)









ÖZGEÇMİŞ

Mustafa KOÇ 1979 yılında İzmir'de doğdu. İlk öğrenimini Altinordu İlköğretim Okulu'nda, liseyi Sekine Evren Anadolu Lisesi'nde tamamladı. 1998 yılında girdiği İstanbul Teknik Üniversitesi İnşaat Mühendisliği Bölümü'nden 2003 te mezun oldu. 2001 yılında başladığı Çift Anadal Programı dahilinde Jeoloji Muhendisliğinden 2004 yılında mezun oldu. 2003 yılında İstanbul Teknik Üniversitesi Fen Bilimleri Enstitüsü'ne bağlı Zemin Mekaniği ve Geoteknik Mühendisliği programında Yüksek Lisans öğrenimine başladı.

Mustafa KOÇ, 2004-2005 yılı Eylul –Temmuz ayları arasında Erasmus Socrates burslu öğrencisi olarak, TUWIEN (Technical University of Vienna), Avusturya bünyesinde çeşitli araştırma çalışmalarında bulunmuştur ve Yüksek Lisans tez çalışmasını adı geçen üniversitede yürütmüstür. 2005 yılı Ağustos ayında başladığı Geomar adlı şirket ile Türkiye ve Dubai'de Geoteknik Mühendisliği alanında çalışmıştır.

İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

ARTIFICIAL GROUND FREEZING AND EFFECT OF FREEZING SPEED OVER FROST HEAVE

M.Sc. Thesis by

Mustafa KOÇ, B.Sc.

Department : **Civil Engineering**

Programme: **Soil Mechanics and Geotechnical Engineering**

JUNE 2006

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(501031312)

Date of submission : **12 May 2006**

Date of defence examination: **15 June 2006**

Supervisor (Chairman): **Assoc. Prof. Dr. Oğuz Tan (İTÜ)**

Members of the Examining Committee **Prof.Dr. Ahmet Sağlamer (İTÜ)**

Prof.Dr. Gökhan Baykal (BÜ.)

JUNE 2006

PREFACE

I want to begin with thanking to ITU European Union members and also administration team that gave me the chance to prepare my study abroad.

First of all, I would like to thank my supervisor Hon. Prof. Dr. Lothar Martak, Prof. Dr. Heinz Brandl and Miss Aigner from Vienna University of Technology, and especially in ITU my supervisor Assoc. Prof. Dr. Oğuz Tan helping me to finish this study, Ass. Prof. Berrak Teymür, Prof. Dr. Ahmet Sağlamer and Prof. Dr. Mahir Vardar, Ass. Prof. Aykut Şenol due to their support in this interval.

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Finally I want to dedicate this study to my family.

May 2006 Mustafa KOÇ

ÖNSÖZ

Bu çalışmayı yurtdışında yapmamı sağladıkları için, İTÜ Avrupa Birliği üyelerine ve de yönetime, teşekkür ederek başlamak istiyorum.

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Bu çalışmayı beni her zaman yürekten destekleyen aileme hediye etmek istiyorum

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ABBREVATIONS

AGF	: Artificial Ground Freezing
AFTES	: French Tunneling and Underground Engineering Association
BOKU	: Boden Kultur University of Vienna
LIN	: Liquid Nitrogen
ML	: Clayey Silt
NATM	: New Austrian Tunnelling Method
TU	: Technical University
U2	: U-Bahn 2 (Subway Line 2)
U3	: U-Bahn 3 (Subway Line 3)

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LIST OF SYMBOLS

A	: Area
a	: Thermal diffusivity
C	: Heat Capacity
$\mathbf{c_m}$	: Volumetric heat capacity
$\mathbf{G_s}$	: Specific gravity of solid soil particles
$\mathbf{I_c}$	: Consistency Index
$\mathbf{I_p}$	: Plasticity Index
$\mathbf{i_{ice}}$	: Relative iceness
K	: Thermal conductivity
L	: Latent Heat
n	: Porosity
$\mathbf{S_i}$	: Degree of Saturation
T	: Temperature
$\mathbf{T_f}$	: Initial freezing temperature
V	: Volume
W	: Weight
$\mathbf{W_s}$	: Weight of dry soil
$\mathbf{W_w}$	: Weight of water
$\mathbf{w_l}$	: Liquid Limit
$\mathbf{w_p}$	: Plastic Limit
$\alpha_{(n)}$	: Coefficient
ρ_f	: Bulk density of a frozen soil
ρ_{df}	: Dry density of a frozen soil
ω_t	: Total water content
ω_i	: Ice water content
ω_{uw}	: Unfrozen water content

ARTIFICIAL GROUND FREEZING
AND EFFECT OF FREEZING SPEED OVER FROST HEAVE

SUMMARY

Artificial ground freezing is a technique that has been used extensively for groundwater control and excavation support in the underground construction industry for over a century. The process involves the circulation of a refrigerated coolant through a series of subsurface pipes to convert soil water to ice, creating a strong watertight material. Frozen soil is nearly as strong as concrete and is impermeable, making it ideal for shoring and groundwater cutoff.

There are many ways to freeze the ground, including liquid nitrogen, brine, carbon dioxide. In practice two methods of freezing have been used;

- Slow-rate freezing or closed-loop systems using coolants such as calcium chloride brine, ethylene glycol, carbon dioxide, or ammonia

- Fast freezing or open-loop systems using expendable coolants such as liquid nitrogen

When water is transformed into ice, there would be volume deformation in the soil and it can be seen that an ice lens is formed at 0º C. Ice has 1.09 times volume than the water. It forms a pressure in the soil. If the pressure in the ice exceeds the pressure of overburden material and other pressures, the soil begin to expand. When there is sufficient ice pressure, the new ice lenses will be formed. This upward movement in the site is called frost heave.

In this work the main idea of Artificial Ground Freezing (AGF) is explained, and the method is discussed. In order, soil properties and phase relations in freezing, thermal soil properties and method is given and finally a case study in Vienna Subway System U2 is given as an example.

This thesis reviews and examines the ground freezing as a stabilization method for foundation engineering and tunneling. There were some experiments which were carried out in TU Vienna in order to understand the behavior of frost heave due to relation with water content, and the effect of speed of freezing on heaving. The correlations of the test results showed that when fast freezing put in practice, growing of ice lenses would be less compared to slow rate freezing. This means there will be more heaving in slow rate freezing. Also heaving phenomena is correlated with the water content ratio so that if water content ratio increases, the amount of heaving would increase.

1. INTRODUCTION

At 23% of the world, freezing occurs naturally in specific seasons. In aspect of engineering, freezing is related with the material used in construction and the soil properties. It usually effects the pavements, roads and the natural or man-made materials used in construction. Although the freezing process has been seem to be a big problem, some French engineers changed the natural fact to their own favour in a mine shaft in soft soil conditions at 1850`s. After that engineers began to think about the advantages of freezing.

By means of the thought of acknowledgment of the man made freezing, Civil and Mining Engineers began to use freezing process in their works. And it is used to be named as AGF, Artificial Ground Freezing, in Europe. Despite the fact that it was patented by H. Potesh in Germany, it was first used in South Wales in 1862 in vertical openings. In the development of AGF the main process that Poetch used is taken. Poetsch's main idea involves the circulation of a refrigerated coolant through a series of subsurface pipes to extract heat, thus converting the soil water to ice, creating a strong, watertight material. AGF is mostly used in tunneling and open excavations in mining and civil engineering industries. The main aim is to make a safe and fast stabilization.

In this work the main idea of Artificial Ground Freezing (AGF) is explained, and the method is discussed. In order, soil properties and phase relations in freezing, thermal soil properties and method is given and finally a case study in Vienna Subway System U2 is given as an example.

This thesis reviews and examines ground freezing as a stabilization method for foundation engineering and tunneling. There were some experiments which were carried out in TU Vienna in order to understand the behavior of frost heave due to relation with water content, and the effect of speed of freezing on heaving. The correlations of the test results and the heaving phenomena are tried to be explained and results are given in conclusion.

2. GROUND FREEZING

2.1 Artificial Ground Freezing

2.1.1 What is AGF?

The primary objective in freezing is to stabilize the soil or rock in some bad conditions such as under big values of water contents. By means of freezing, load carrying capacity of the full-partially saturated soils increase and a watertight material is obtained. During ground freezing, ice between soil particles increases soil strength and decreases soil permeability. This is the most important feature of the AGF to be used in Tunnel Engineering. Besides the tunnels and shafts, AGF also is used for stabilisation of slopes, sample weak soils, construct temporary access roads, maintain permafrost overhead pipelines foundations, and below heated buildings. Briefly, AGF is one of the soil improvement method mostly used in some special engineering problems in civil and mining engineering.

As it is mentioned above, the main principle is to overcome the difficulty of ground water during excavation of a hole in ground like a pit shaft or a tunnel. The excavation work becomes difficult when the porosity of the soil and the water content of strata are a bit high. AGF process is very successful and efficient in stabilizing of fine-particled, permeable, saturated soils. Mostly AGF is used to maintain a temporary support converting the soil strata and water into an ice lenses. This is done by cooling and refrigerating of soil by means of some freezing liquids or gases. AGF can be carried out when the velocity of groundwater flow is as much as 2.0 m/day

2.1.2 Why Ground Freezing?

There are many advantages of using AGF. Some are;

1. No chemical additives are applied to the soil, so it has no long term effect on subsurface environment. After thawing process, the environment returns its pre-construction state.

2. It is a combination of sealing and static support.
3. Ground Freezing provides an absolute cut-off wall for the ground water
4. It can be installed in every ground conditions like boulders, gravel, bedrock peat clay, sand etc. (Figure 2.1) **(Harris,1995)**

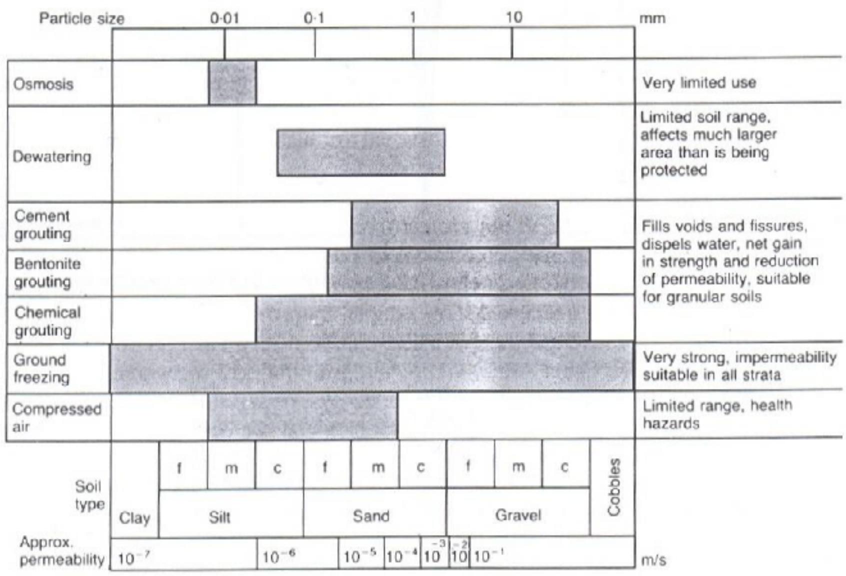


Figure 2.1: Applicability of soil improvement methods according to soil types

5. Most effective support system in hard conditions of ground water, and where groundwater can not be obstructed for the site such as in tunnels.
6. It can be turned on or off during an emergency if cooling coils are incorporated
7. The dimension of the support wall thickness can be controlled by temperature adjustments.
8. Excellent at sensitive sites near buildings or residences.(Low vibration and noiseless)
9. It may be applied during any season of the year.
10. The wall of frozen soil has a relatively great mechanical strength.
11. It can be completely removed after construction

But also it has some disadvantages:

1. Requires large complex freezing installation facilities.

2. In multilayered soil strata, calculations and design of the wall is a bit difficult.
3. Freezing and thawing may destroy the natural structure of the soil especially clayey soils.
4. Is very difficult when the groundwater flow velocity is over 2m/day.
5. It is only economic where other usual methods for stabilisation is not enough and efficient.
6. Takes long time

2.1.3 How Ground Freezing Works?

The fundamental process in ground freezing is the removal of heat from the ground to cause lowering of subsurface temperature below the freezing point of moisture in the pore spaces. The frozen structure acts as a cementing agent binding the soil particles together and supply a structural frame.

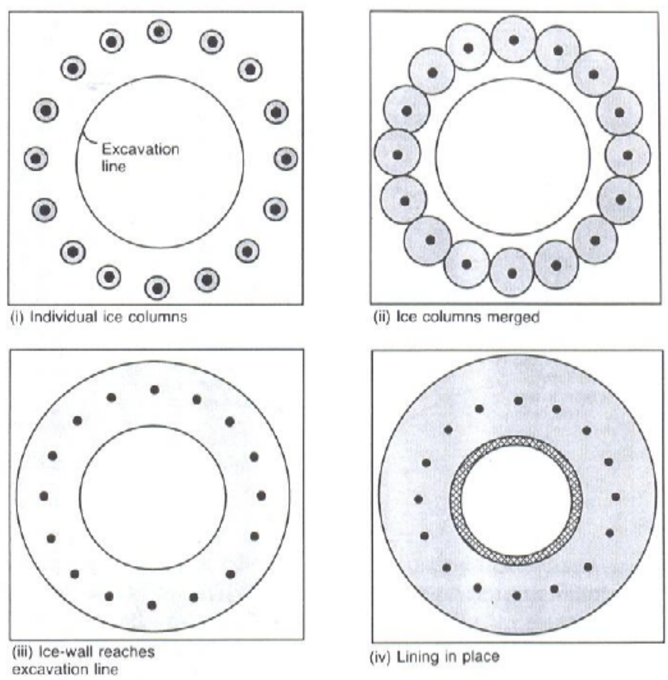


Figure 2.2: Stages of Ground Freezing

There are many ways to freeze the ground, including liquid nitrogen, brine and carbon dioxide. In practice, two methods of freezing have been used:

1) Slow-rate freezing or closed-loop systems using coolants such as calcium chloride brine, ethylene glycol, or ammonia;

A primary refrigerating circuit liquefies the fluid by means of compressors and condensers. By evaporating again, this cools the refrigerant liquid that circulates in the freezing tubes in a closed circuit. The primary liquid may be ammonia or Freon. The refrigerant liquid is generally brine with an operating temperature of between -25˚ C and -30˚ C

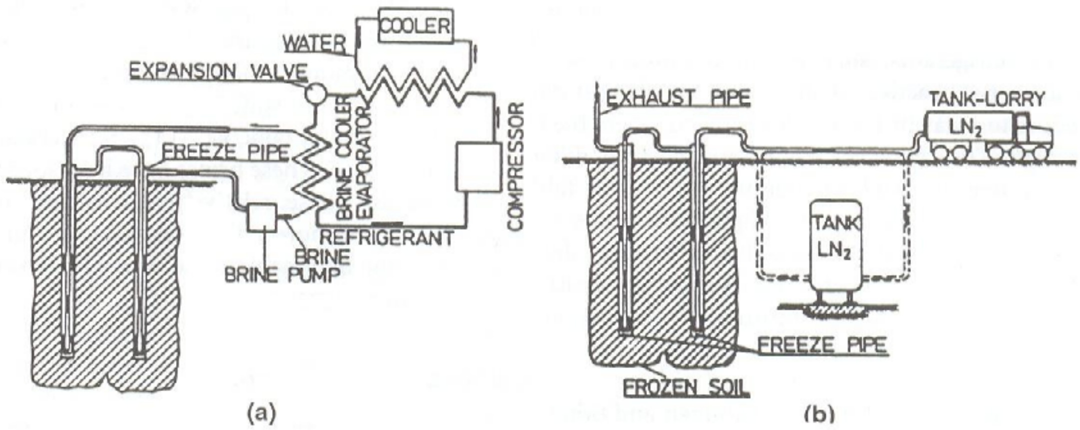


Figure 2.3: Freezing methods a) Closed-loop method b) Open Loop Method (Jessberger, 1985)

2) Fast freezing or open-loop systems using expendable coolants such as liquid nitrogen;

The refrigerant fluid is liquid nitrogen. It is transported to the site by special tanker trucks where it is kept at a temperature of -196˚ C under a pressure about 0.5 MPa. This pressure makes the flow of nitrogen through the probes. After the last probe, the nitrogen is allowed to escape to atmosphere at around -60˚. **(Harris, 1995)**

A third method can be classified as mixed method that combines the open and closed method. This consists of combining the two preceding methods, using the same freezing probes. The rapid cooling of the nitrogen is combined with the economical continued operation of the brine.

The artificial freezing process of the soil is carried out in two stages; active freezing and passive freezing;

In Active freezing required thickness of the wall of the freezing soil is built, and after that active freezing stops and passive freezing begins. The main aim of the passive freezing is to prevent the required thickness of the wall. **(Harris, 1995)**

Most ground freezing systems are quite similar in principle, with some differences in the engineering aspects of the individual sites. The single most important component of a ground freezing system is the subsurface refrigeration system (Figure 2.4), consisting of a series of refrigeration pipes, installed with various drilling techniques. The quantity, spacing, depth and size of the refrigeration pipes are unique to each site, and determined based on the thermal and hydraulic properties of soils, construction period and the cost.

Figure 2.4: Cooler Supplier (left) and connections to the site (right)

2.2 Ground Freezing in Tunnelling and NATM

As it is mentioned above ground freezing is used for sealing and statically support in tunnel engineering. While designing and in progress New Austrian Tunnelling Method, NATM, is considered. Primarily in tunnel constructions, ground freezing is taken as a view of thin sections around the outer tunnel counters in order to improve the roof and side walls.

Figure 2.5: Elements in NATM (Wire mesh, Shotcrete, bolts)

Sometimes shallow tunnels are protected by vertical solutions made from ground surface. This method is less cheap with comparison to horizontal drilling.

2.2.1 Principles of NATM

The New Austrian Tunnelling method (NATM) was developed between 1957 and 1965 in Austria. It was given its name in Salzburg in 1962 to distinguish it from old Austrian tunnelling approach. The main contributors to the development of NATM were Ladislaus von Rabcewicz, Leopold Müller, and Franz Pacher. It was used for rock tunnels for hydropower schemes. First, Rabcewicz applied for a patent in 1948 as defending that a thin temporary support by allowing the deformations is adequate till to last cover and reduce the stress and distribute to the surrounding rocks. So, the final support will be less loaded and much thinner structure. The main idea is to monitor and control the deformations during construction and minimize all risks by applying the right things in sequence. In soft ground conditions it was used in subway constructions in Germany. Now the method is used for all kind of soil conditions from rocks to soft clays.

2.2.2 Concept of NATM

NATM is known as a concept or philosophy rather than a construction method. As defined by the Austrian Society of Engineers and Architects, the NATM constitutes a method where the surrounding rock or soil formations of a tunnel are integrated into an overall ring-like support structure. Thus the supporting formations will themselves be part of this supporting structure. Nevertheless, many engineers already refer to as NATM, when shotcrete is proposed for initial ground support of an open-face tunnel. Especially with reference to soft ground, the term NATM can be misleading. In conventional tunnelling methods tunnel line is the main barrier to support loads whereas the NATM is considered as that surrounding soil or rock mass of a soil is integrated into the overall support structure.

Figure 2.6: NATM tunnelling in U2, Vienna

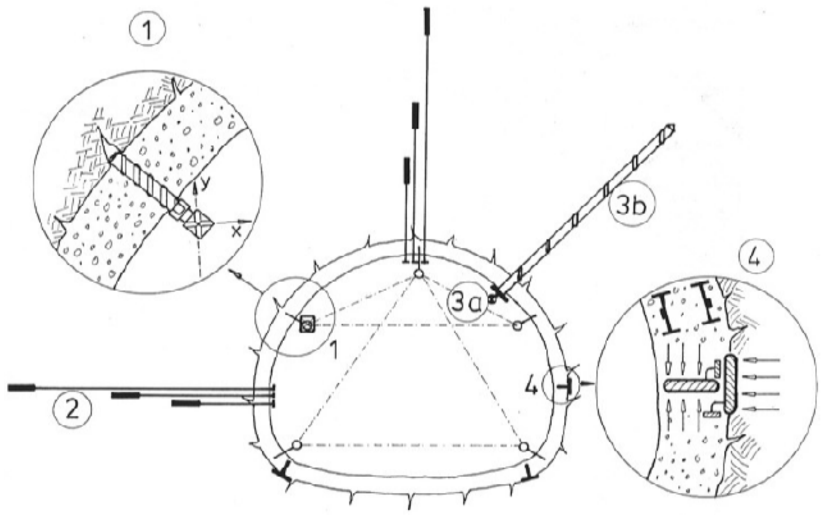
In NATM tunneling, idea is realized with making the supports as shotcrete, reinforced wire meshes, steel ribs, lattice girders**,** rock bolts by forming a composite structure with soil or rock mass (Figure 6.2). The composite structure will distribute the loads around the tunnel. By means of the composite structure and controlled deformations, lose in strength of the rock is prevented and carrying capacity of the rock arch is preserved. Overall this all ideas are valid with a strong monitoring and geotechnical measurements. Müller listed 22 principles of NATM. There are seven most important features on which are NATM based:

- Mobilisation of the strength of rock mass - The method relies on the inherent strength of the surrounding rock mass being conserved as the main component of tunnel support. Primary support is directed to enable rock support itself.
- Shotcrete protection - Loosening and excessive rock deformation must be minimised. This is achieved by applying thin layer of shotcrete immediately after face advance.
- Measurements - Every deformation of excavation must be measured. NATM requires installation of sophisticated measurement instrumentation. It is embedded in lining, ground, and boreholes.
- Flexible support - The primary lining is thin and reflects recent strata conditions. Used is rather active than passive support and strengthening is not by thicker concrete lining but by a flexible combination of rock bolts, wire mesh and steel ribs.
- Closing of invert - Important is quickly closing of invert and create load-bearing ring. It is crucial in soft grounded tunnels where no section of tunnel should be left open even temporarily
- Contractual arrangements - Since the NATM is based on monitoring measurements, changes in support and construction method are possible. This is possible only if the contractual system those changes enables
- Rock mass classification determines support measures - There are main rock classes for tunnels and corresponding support. These serve as the guidelines for tunnel reinforcement. **(KGM Training Lectures, 1994)**

2.2.3 Observations of Tunnel Behavior

One of the most important factors in the successful application of observational methods like NATM is the observation of tunnel behavior during construction. Monitoring and interpretation of deformations, strains and stresses are important to optimize working procedures and support requirements, which vary from one project to the other. In-situ observation is therefore essential, in order to keep the possible failures under control.

Considerable information related to the use of instruments in monitoring soils and rocks are available from instrument manufacturers. Figure 2.7 shows example instrumentation in a tunnel lined with shotcrete.



Legend	Measuring objective	Instrument
1	Deformation of the excavated tunnel surface	Convergence tape Surveying marks
2	Deformation of the ground surrounding the tunnel	Extensometer
3	Monitoring of ground support element ‘anchor’	Total anchor force
4	Monitoring of ground support element ‘shotcrete shell’	Pressure cells Embedments gauge

Figure 2.7: Examples of NATM tunnel measurement equipment

2.2.4 The Fenner-Pacher curve

The idea of the ground response curve, or otherwise the “characteristic curve” of the ground mass is considered to originate from Fenner (1938) who also proposed a closed form solution for the problem of a circular opening in elastoplastic ground. It was later supported by Pacher (1963) in Austria, as a major design component behind the New Austrian tunneling method. The method was further improved by many researchers such as Gaudin et al. (1981), Guenot et al. (1985), Sulem et al. (1987), Deffayet and Robert (1989), Panet (1993), Panet (1995), Carranza-Torres and Fairhust (1999), Carranza-Torres and Fairhust (2000), Brown and Bray (1982), Brown et al. (1983), Oreste and Peilla (1996), Oreste and Peilla (1997), Oreste (2003), Peila and Oreste(1995) and it has received acknowledgment by the French Tunneling and Underground Engineering Association (AFTES, 1984) for application in rational tunnel designs. The recommendations by AFTES are described by Panet (2001). (http://amadeus.cee.vt.edu)

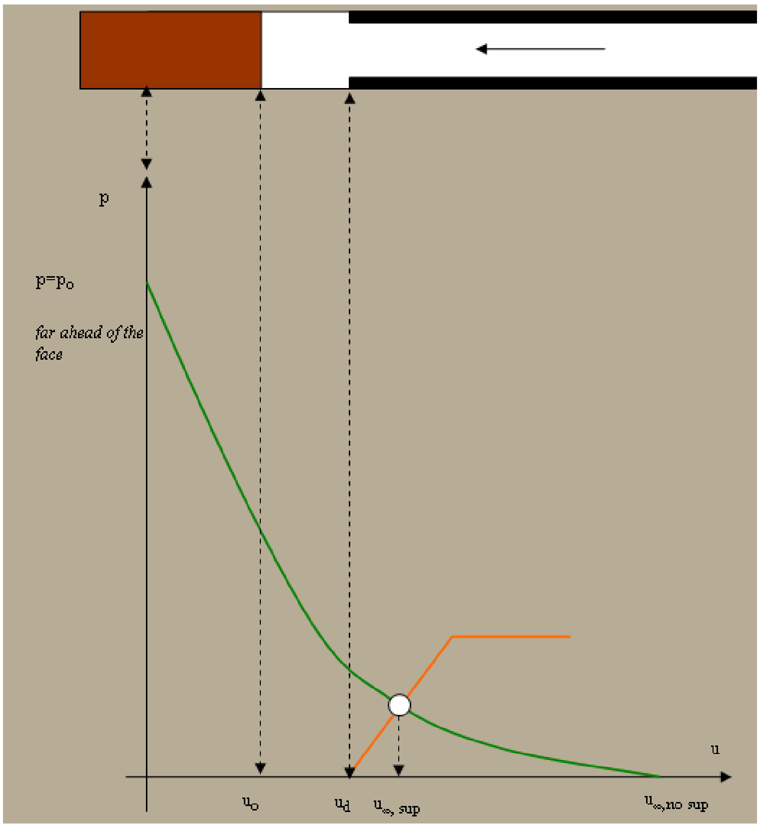


Figure 2.8: Fenner- Pacher Curve (http://amadeus.cee.vt.edu)

The Fenner-Pacher curve shows the relationship between the deformation ΔR/R and required support resistance P_i. Simplistically, the more deformation is allowed, the less resistance is needed. In practice, the support resistance reaches a minimum at a certain radial deformation, and support requirements increase if deformations become excessive.

Fenner-Pacher-type diagrams can be generated to help evaluate the support methods best suited to the conditions. In terms of analysis, it is convenient to carry out quick and simple convergence-confinement calculations with and without support. Resistances of shotcrete, steel reinforcement, and anchors/bolts can be calculated. Functions of support elements, radial stresses, and support resistances of inner and outer arches, deformations, and failures can be analyzed with respect to time.

3. SOIL

3.1 Frozen Soil

3.1.1 Frozen Soil Properties

Ice is defined as the solid state of water, and one of the participants in frozen soil. When the temperature of soil reaches the 0 degrees C or 32 degrees F ice begin to form in soil strata. Ice may be found in soil in different ways and sometimes can not be seen by naked eyes.

3.1.2 Classification System

Tsytovich (1975) classified frozen soil with the description of ice content and physical state categories. According to this classification, three main groups are:

- Hard frozen soils-low temperature (incompressible)
- Plastic frozen soils-warm temperature (with high unfrozen water)
- Friable frozen soils-coarse grained soils (not cemented by ice)

3.2 Phase Relations

3.2.1 Frozen Soil Components

Frozen soil can be thought to be a multiphase complex material as unfrozen soil. But frozen soil has one more phase then unfrozen soil. The phases are:

1. Solid phase, Solid particles
2. Plastic viscous phase or ice
3. Liquid phase or unfrozen water
4. Gaseous Phase or air vapour

The phases may be determined in terms of the physical parameters defined below. Neglecting the vapour phase, the total water content ω_t is given by:

$$\omega_t = \omega_i + \omega_{uw} \tag{3.1}$$

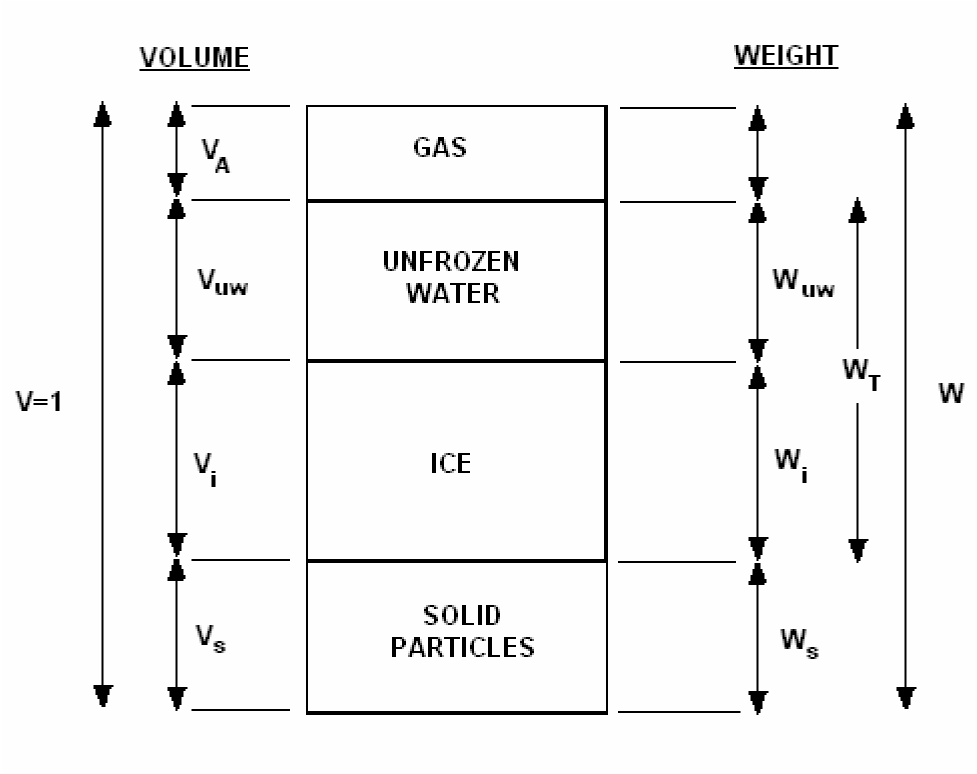


Figure 3.1: Constitution of frozen soil

where

ω_i = Ice water content

ω_{uw} = Unfrozen water content

Water content is defined as

$$\omega_t = \frac{\text{weight of water}}{\text{weight of dry soil}} = \frac{W_w}{W_s}$$

(3.2)

From figure 3.1

$$W_i = W_t - W_{uw} \tag{3.3}$$

Relative iciness i_{ice} is defined as the ratio between the weight of ice and the weight of total water content in a given unit volume of frozen soil:

$$i_{ice} = \frac{W_i}{W_t} \tag{3.4}$$

or

$$i_{ice} = \frac{\omega_i W_s}{\omega_t W_s} \tag{3.5}$$

$$= \frac{\omega_t - \omega_{uw}}{\omega_t} \tag{3.6}$$

$$= 1 - \frac{\omega_{uw}}{\omega_t} \tag{3.7}$$

The relative proportion of ice and unfrozen water content are dependent on

- Soil characteristics
- Temperature
- Ion concentration of the water
- Externally applied pressure

There are various methods to measure unfrozen water content in frozen soil. These are dilatometry, adiabiatic calorimetry, x-ray diffraction, nuclear magnetic resonance, differential thermal analysis and some other indirect methods. But these methods are not so easy to apply.

The bulk density of a frozen soil is defined as:

$$\rho_f = \frac{\text{weight}}{\text{volume}} = \frac{W}{V} \tag{3.8}$$

The dry density ρ_{df} of a frozen soil is the weight of dry soil divided by the total volume of frozen soil:

$$\rho_{df} = \frac{W_s}{V} \tag{3.9}$$

The relationship between ρ_{df}, ρ_f and ω_t is given by

$$\rho_f = \frac{W}{V} = \frac{W_s + W_{iuw}}{V}$$

$$= \frac{W_s}{V} + \frac{W_{iuw}}{V} \qquad \textbf{(3.10)}$$

$$= \rho_{df} + \omega_t \frac{W_s}{V} = \rho_{df}(1 + \omega_t) \qquad \textbf{(3.11)}$$

Assuming the full saturation (no frozen water but with excess ice in soils);

$$\rho_f = \frac{G_s \rho_w (1+\omega)}{1 + 1.09 G_s \omega} \qquad \textbf{(3.12)}$$

where G_s is the specific gravity of soil solid particles and ρ_w is the density of water.

The frozen dry unit weight ρ_{df} is given by

$$\rho_{df} = \frac{\rho_f}{1+\omega} \qquad \textbf{(3.13)}$$

The degree of saturation S_i is given by

$$S_i = \frac{\text{volume of ice}}{\text{volume of pores in frozen soil}}$$

$$= \frac{V_i}{V_v} = \frac{W_i}{\rho_i . e . V_s} = \frac{\omega . G_s . \rho_w}{\rho_i . e} \qquad \textbf{(3.14)}$$

3.2.2 Ice Phase

As shown in Figure 3.2, pore water does not start to freeze until the temperature drops to T_{sc}. After that the formation of ice releases latent heat and cause a rise in temperature to T_f called as initial freezing temperature. In cohesionless soils T_f is close to 0º C whereas in cohesive it is nearly 5 degrees. Afterwards water in will be chilled at temperature T_f. As the water changes to the ice the rate of cooling will go to T_e (about -70º C) where the all free and bonded water will be chilled. **(Andersland & Ladanyi, 2004)**

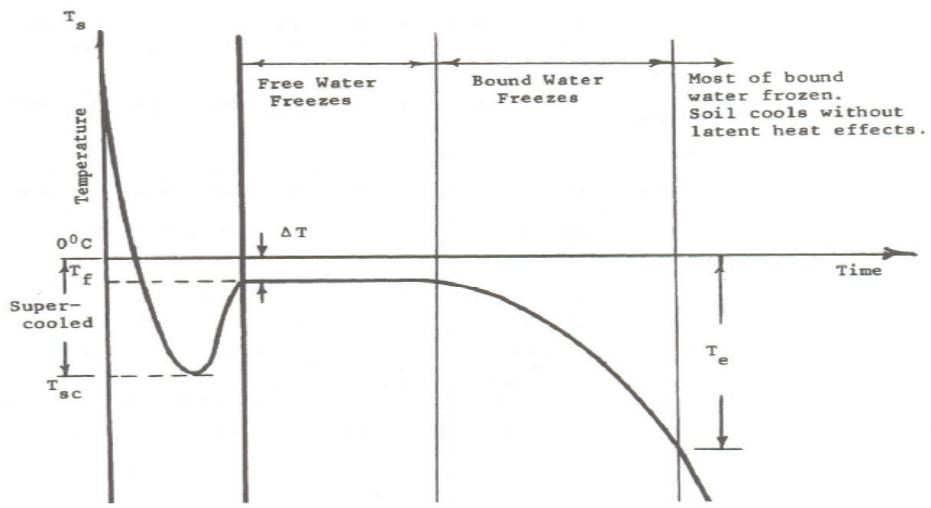


Figure 3.2: Cooling Curve for soil water and ice (Lunardini, 1981)

3.3 Mechanical Properties of Frozen Soils

It is very important to understand the mechanical properties of soils. Because of these both on natural and artificial frozen soil samples, many experiments are carried out. It should be noticed that the structure of frozen soil in site is different from artificial soil samples.

The mechanical properties of the frozen soils are as follows:

- Compression strength and tensile strength
- Shear strength
- Creep strength and long term strength
- Compressibility

The strength of frozen soil is highly dependent on temperature and time and is governed by cohesion of soil and internal friction of soil-ice matrix system.

(Harris, 1995)

3.4 Thermal Properties of Soil

The basic thermal properties of soils are thermal conductivity, heat capacity, diffusivity and latent heat. These properties can vary with phase compositions.

Temperature, soil type, water content, degree of saturation, dry density, organic contents are some of the most effective factors on soils thermal properties.

3.4.1 Thermal Conductivity:

Thermal conductivity is defined as the rate at which the amount of heat is transferred upon the existing temperature gradient in a unit time through a unit cross sectional area. Thermal conductivity is given as:

$$K = \frac{q}{A(T_2 - T_1)/L} \qquad (\frac{cal}{s\ cm\ °C}) \qquad \textbf{(3.15)}$$

where the temperature drops from T_2 to T_1 over a length of L of the element. **(Jumikis, 1977)**

3.4.2 Thermal Diffusivity:

In unsteady state, thermal behaviour, also heat capacity C of the soil governs the thermal behaviour of the soil. And the ratio of the thermal conductivity over heat capacity is defined as thermal diffusivity

$$a = \frac{K}{C} \qquad (\frac{mm^2}{s}) \qquad \textbf{(3.16)}$$

The thermal diffusivity of ice is nearly 8 times of the water. So the frozen ground has more diffusivity than that of the soil in thawed position. **(Jumikis, 1966)**

3.4.3 Heat Capacity:

One of the important thermal properties of water is defined as the quantity of the heat energy necessary to change the temperature of a unit mass. This is called specific heat capacity. If it is expressed on a unit volume mass, it is known as the volumetric heat capacity.

$$c_m = \frac{Q}{\Delta T} \qquad (\frac{cal}{cm^3 °C}) \qquad \textbf{(3.17)}$$

The total heat capacity is the addition of the heat capacities of constituents like solid, ice, unfrozen water and etc.

3.4.4 Latent Heat:

The volumetric latent heat of fusion L is the amount of heat required to melt the ice or freeze the water in unit volume of soil with no change in temperature. This factor must be calculated for all thermal requirements of freezing program. Because latent heat is the most when compared to all other heat losses. It usually represents the most important factor in the freezing process. It is defined as:

$$L = \rho_d.\omega.(1-\omega_{uw}).L' \qquad \textbf{(3.18)}$$

$$L = \rho_d.\omega.L' \quad \text{for } \omega_{uw}=0 \qquad \textbf{(3.19)}$$

3.5 Hydrologic Properties

Hydrologic properties can be related with bulk thermal properties and have important influence on final design. Most important considerations are:

1. Moisture content
2. Subsurface flow rates and direction of flow
3. Permeability of soil
4. Pore water chemistry

4. BASIC PRINCIPLES OF ARTIFICIAL GROUND FREEZING

4.1 Method and Design

4.1.1 Soil Investigation

Soil investigation reports should include all data till below depths of the proposed excavation. Disturbed and undisturbed samples have to be taken and frozen and unfrozen soil tests should be done. Soil type, density, and water contents are needed to estimate the soil thermal properties. In-situ permeability tests and ground temperatures have to be taken regularly. In some cases multilayered soils thermal properties will be so important that all care should be taken for these regions. Also the bottom soil properties are so important like the water existence or permeability. For example free water will go upwards to the frozen earth barrier so that the barrier will grow and much more energy would be needed. Generally frozen earth barrier should be fixed to impervious layer.

4.1.2 Ground Water

If the rate of velocity is more than 2m/day the freezing columns will not merge and a complete wall will not be formed. For liquid nitrogen systems Shuster (1972) reported that flows as high as 50 m/day have been stopped. So the permeability of the soil should be investigated carefully. This can be done by some radioactive solutions or fluorine ion in boreholes.

The second issue is the contamination degree of the soil. The presence of these kind of constituents (organic, inorganic, radionuclide and bacteriological) will cause low freezing temperatures, reduced ice content and lower strength. Also the salinity of the water should be determined. **(Jessberger, 1980)**

4.1.3 Thermal Analysis

The thermal analysis is the most important component of designing the ground freezing system, during the construction of frozen earth barrier walls. Four independent parameters are based on the factors in completing the thermal analysis, assuming hydrostatic conditions are relatively static.

The four parameters are:

- Thermal properties of the soil;
- Required time to freeze;
- Coolant temperatures. and
- Freeze pipe spacing.

4.2 Theories

There are mainly 4 theories based on heat flow in the body. These are:

1. The steady state flow of heat
2. The unsteady state heat conduction
3. F. Neumann`s Theory
4. J. Stefan`s Theory

All these theories are based on

a. Heat flows from regions of higher temperature to regions of lower temperature
b. The amount of heat in a differential element of soil or other material is proportional to its mass and its temperature
c. The rate of heat flow across an area is proportional to its size and to the temperature gradient

4.2.1 The Steady State Flow of Heat

In this theory some assumptions are made as

1. The soil is homogeneous, isotropic material
2. The geotechnical and thermal properties of the soil are constant
3. Freezing in soil begins at 0 °C
4. The thermal conductivity of the soil is independent of the temperature.

The theory was given by Fourier as in the following equation:

$$\frac{dQ}{dt} = K.A.\frac{dT}{dx'} \qquad \textbf{(4.1)}$$

where $\frac{dQ}{dt}$ is the rate of heat flow, K is the coefficient of thermal conductivity $\frac{dT}{dx}$ is the rate of change of temperature with respect to thickness, x, of the plate.

When the flow of heat is constant then the temperature remains constant in each point. Hereby:

$$\frac{dQ}{dt} = q \qquad \frac{dT}{dx} = \frac{T_1 - T_2}{x} \qquad \textbf{(4.2)}$$

where T_1 and T_2 are temperatures at the top and the bottom of a plate.

Then equation becomes $q = K.A.\frac{T_1 - T_2}{x}$ **(4.3)**

4.2.2 The Unsteady State Heat Conduction:

In this phenomenon temperatures can vary with time and position. Also in this state the heat flow is uniformly distributed over the cross sectional area. According to the Biot-Fourier heat equation in this state is derived as:

$$dQ = -(K).(\frac{\delta T}{\delta x}).(dA).(dT) \qquad \textbf{(4.4)}$$

where $\frac{\delta T}{\delta x}$ temperature gradient at any point and x is the depth. The minus sign shows the decrease in temperature as the x in the direction of heat flow increases.

4.2.3 Franz Neumann`s Theory

The original theory was given for the formation of pure ice from water. But it can be used in geotechnical engineering by fitting soil constants into the equation.
Two equations for ice and water were given by Neumann. The assumption is the initial temperature of the soil is positive and constant.

For the frozen part of the soil ($0<x<\xi$):

$$\frac{\delta T_1}{\delta t} = \alpha_1 . \frac{\partial^2 T_1}{\partial x^2} \quad \textbf{(4.5)}$$

and for the unfrozen part ($x>\xi$):

$$\frac{\delta T_2}{\delta t} = \alpha_2 . \frac{\partial^2 T_2}{\partial x^2} \quad \textbf{(4.6)}$$

Boundary conditions for the equation are:

1. The initial conditions at $x>0$, $t=0$, $T_1=T_0=\text{constant}$
2. The fixed boundary conditions
    a) At $x=0$, $t\geq 0$, $T_2 = T_s$
    b) When $x \rightarrow \infty$, $t\geq 0$, $T_2 \rightarrow T_0 = \text{constant}$
    c) The condition at the advancing isothermal surface of the frozen layer downwards at $x = \xi$, $t>0$, $T_1 = T_2 = \text{constant } T_f = 0\,°C$ where T_f is freezing temperature

4.2.4 Stefan`s Solution

Stefan`s solution presents a simple formula for the formation of ice which takes the frost penetration depth as a function of time while computing.

The assumptions of Stefan`s solution are:

- T_0; The temperature of the water at the isothermal boundary surface, between the frozen and unfrozen zone is 32° F, and that this is so below the frost boundary.
- T_s; The surface temperature (air temperature) is constant.
- The temperature in the frozen zone varies linearly.
- The density of ice is the same as that of water.

The main differential equation is similar as Neumann`s Theory, but there is no second equation as in Neumann`s solution.

The main differential equation for the frozen soil is;

$$0 < x < \xi \qquad \frac{\delta T_1}{\delta t} = \alpha_1 \cdot \frac{\delta^2 T_1}{\delta x^2} \tag{4.7}$$

The boundary conditions are;

At $x = 0$, $T_1 = T_s$

At $x = \xi$, $T_1 = T_f = 32^0 F$

The continuity condition for the flow of heat reduces to;

$$\frac{d\delta}{dt} = \frac{1}{Q_L \cdot \rho_s \cdot w} \cdot \left[K_1 \cdot \frac{\delta T}{\delta x} \right]_{x=\xi} \tag{4.8}$$

$\rho_s \cdot w$ can be explained as the amount of moisture in the soil.

(Jumikis, 1977)

5. APPLICATION IN U2/1 VIENNA-AUSTRIA

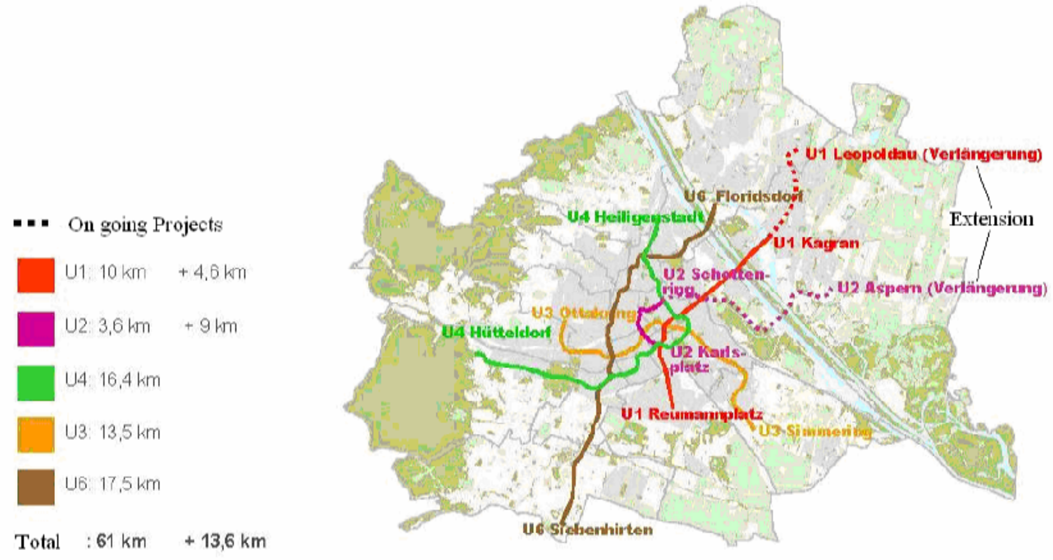


Figure 5.1: Vienna Underground Map

5.1 Definition of the Project

The subject of this project is the extension of the underground line 2 (purple line in Figure 5.1) from the Schottenring to the Aspern Street. Main Underground construction works are planned for the European Football Championship that will be held in 2008 both in Austria and Switzerland.

The route runs along the Maria Theresien Street and then turns to the northern, to the end of the existing station Schottenring. Afterwards the route drops rapidly, in order to make the under crossing of the Danube Canal. (www.magwien.at)

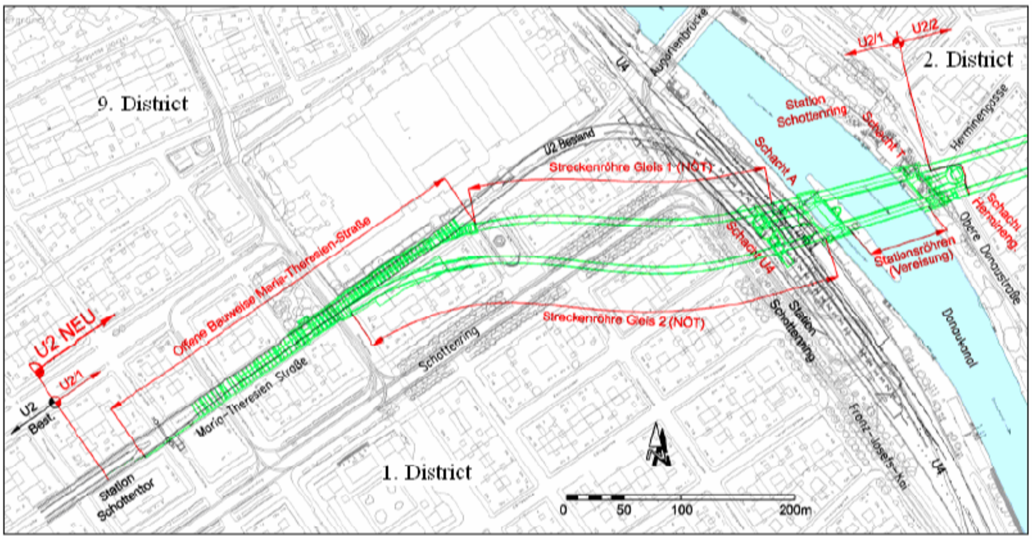


Figure 5.2: Plan of the Danube Crossing

The crossing under the Danube Canal represents the principal item of this underground section. The route takes place from the right Danube Canal bank in the protection of a building and drives under the former and Emperor Bath standing under protection, as well as the Contactor House designed by the architect Otto Wagner (Figure 5.2). The outbreak surface of the two tunnels is enough for approximately in each case 70 m - station pipes amounts to 76 m².

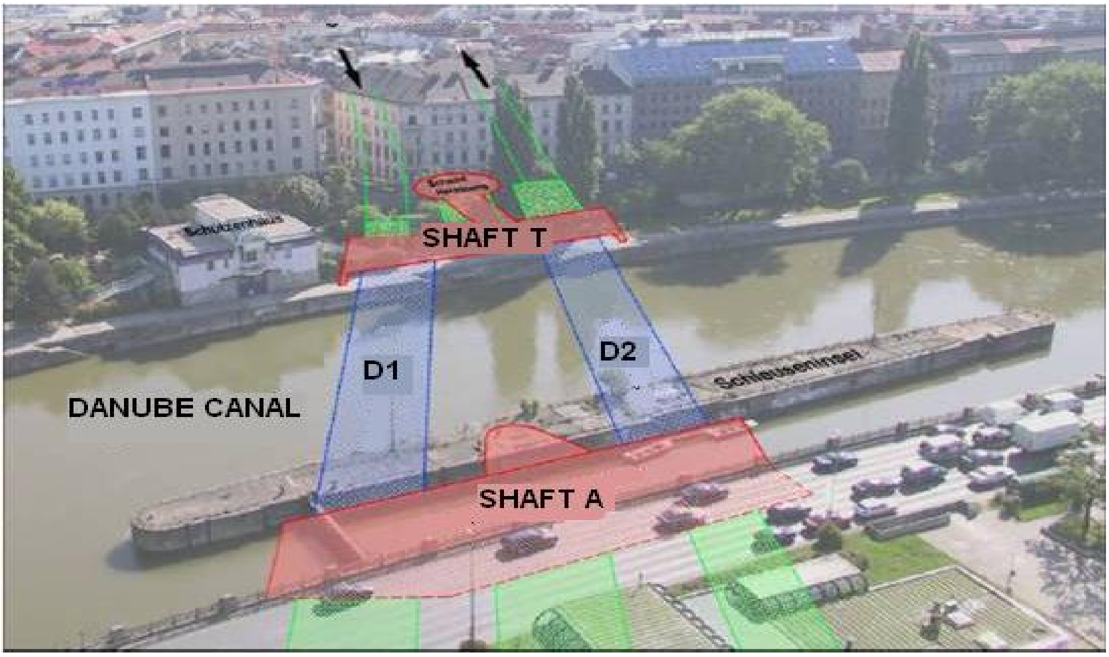


Figure 5.3: View from Ring Tower

Because of shaft construction works people can reach the first and second district from platforms of the new station Schottenring by means of escalator to the existing underground line 4, which will take place under the Danube Canal in the future.

5.1.1 Technical Data:

Freezing Project that lies in 1.District of Vienna, belongs to Viennese lines GmbH & CO and is constructed by ISP Monarth & Tatzber, Bilfinger Berger Bauges.m.b.H, Porr Technobau and Environment AG. The construction period has planned between 05/2003 to 08/2006, with the cost of 43.8 millions Euros. Some technical data about the project are:

First tube is 793.57 m and the second tube is 817.00 m. It was planned that 14,000 m^3 frozen soil will be obtained with brine and liquid nitrogen. The lengths of boreholes for freezing pipes are 9700 m. The construction site has been drained by means of 45 wells and 23 levels. **(Joestl etall., 2003)**

5.2 Geology

5.2.1 Geology of Vienna

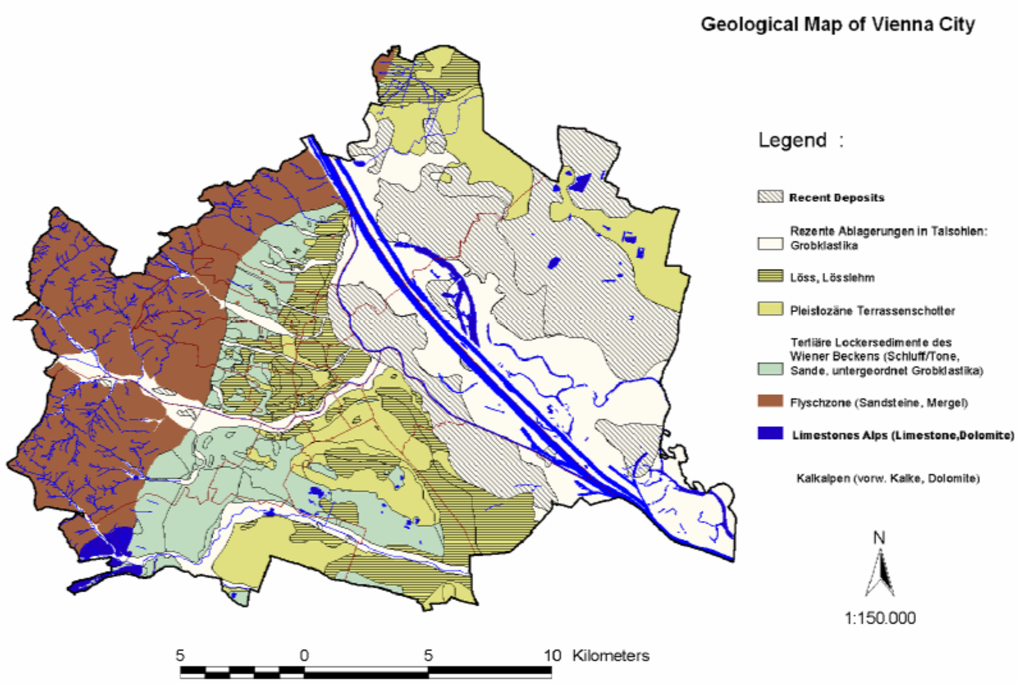


Figure 5.4: Geological Map of Vienna

Vienna lies at the eastern parts of the Alps, because of the west edge of the tertiary times Viennese basin is from the Pleistocene times to today. Viennese area landscape ends at Danube. Due to its geological history, natural soil profile can be seen. General scheme of the city is formed by different geological landscapes:

- Crushed stone and sand of the ice-age and the current unconsolidated Danube sediments of Quaternary
- Loose rocks - tertiary unconsolidated sediments of the Vienna basin
- Rocks of the Flyshzone and the Alps limestone in the western Viennese forest area

Powerful groundwater bodies can be found in the Danube crushed stones.

The Waldviertel belongs to the flattened mountains of the Bohemian Massif, the big crystalline area in the heart of Europe. There one finds crystalline and metamorphic rocks like granite, gneiss, slate etc.

In the south, the Danube embedded itself in the crystalline soil and divided the Dunkelsteiner Forest from the rest of the Bohemian Massif. Following this valley of beautiful scenery, the unique Wachau valley, the Danube enters the flat foothills of the Alpine upland.

On its further course, the Danube cuts through the easternmost foothills of the Alps (the Vienna woods) at Greifenstein-Klosterneuburg and crosses the Vienna Basin, that was formed in the Tertiary and filled with sediments.

The Vienna Basin contains Austria’s biggest oil and gas-fields. At its fracture zones ("thermal line") there are hot springs (Baden, Bad Vöslau). In the west the Vienna Basin borders the Vienna woods and the limestone Alps. In the south the basin borders the crystalline regions Semmering, Bucklige Welt, Rosalien- and Leitha mountains.

During the ice ages Lower Austria was mostly ice-free. Sediments from the detritus of the moraines, which were formed in front of the glaciers, were blown away by the wind. This fine sand covers today wide areas of the Weinviertel and forms the fertile soil for wine growing. (www.magwien.at, Pfleiderer and Hoffmann, 2004)

5.2.2 Geology of Site

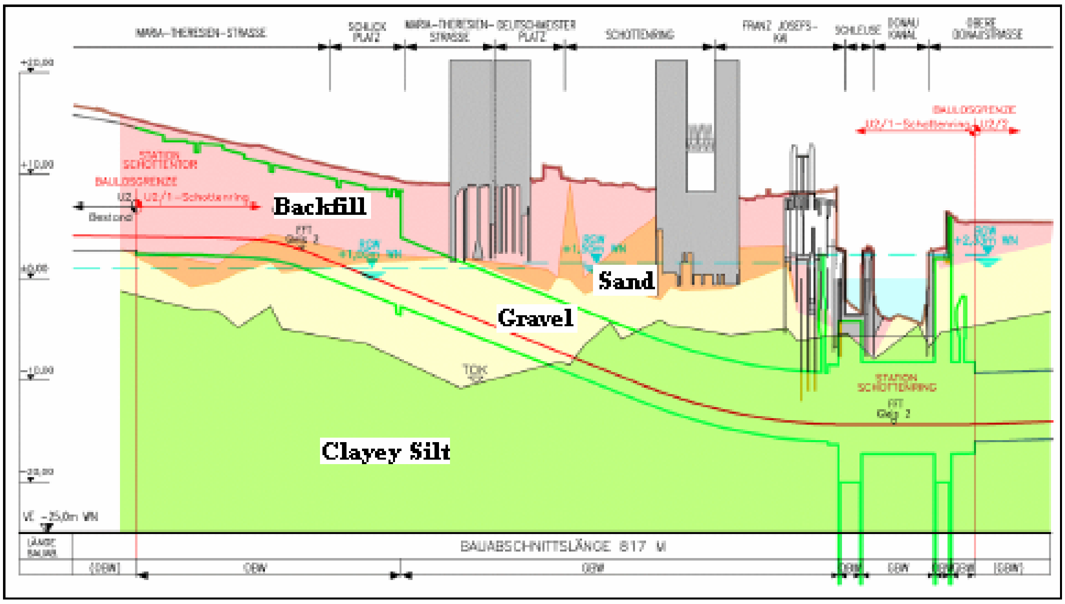


Figure 5.5: Geological cross section of the tunnel

Available favourable geological conditions are consisting of clay in accordance with silt and sandy gravel and the concrete sole of the emperor bath air-lock made the decision possible for a building ground freezing up from soil-mechanical view. The river bed is formed through sandy gravel for differently loose compaction and depths between 0.5 m and 1.5 m. River scours and bomb funnels from 2nd World War that were filled by debris can be found up to 5 m under the Danube Canal.

The freezing up attempts at altogether 8 soil samples from the tertiary, tonus towards and sandy Silt became for the temperatures -5°C, -10°C, -15°C and -20°C accomplished. Freezing and thawing out procedures were simulated. In the unloaded condition the freezing attempts resulted in after 6 frost rope cycles elevations between 12.86 mm and 17.48 mm. In the thawing out procedure, the elevations decreased with all samples to values between 9.65 mm and 14.56 mm.

(Martak and Herzfeld, 2005)

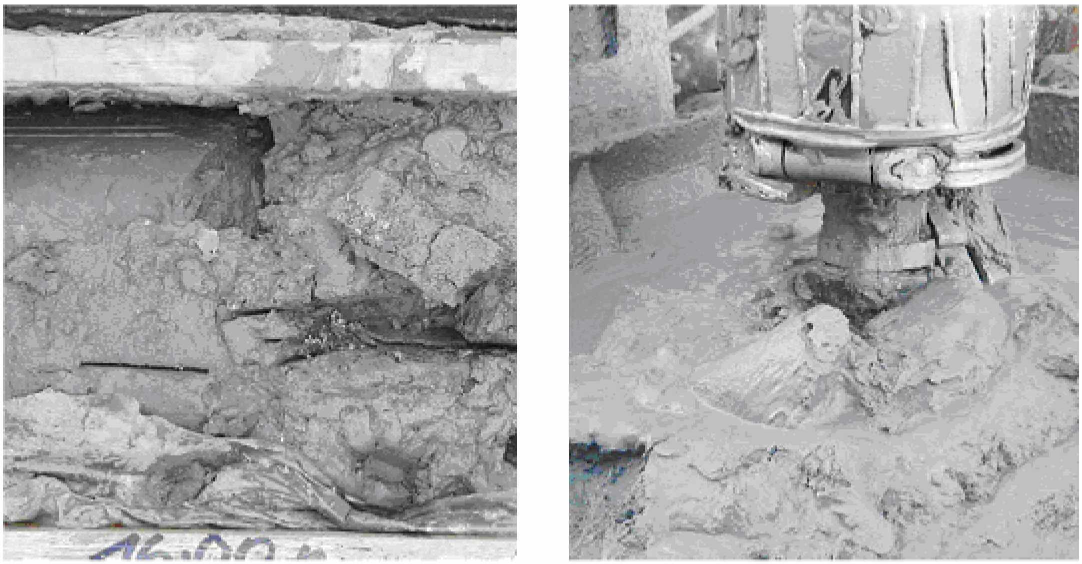


Figure 5.6: Soil Samples taken from boreholes and silty soil in piling bucket.

The creep deformations were appropriate 2.2 % and 9.8 %. The values showed a clear increase as a function of dropping the temperature. As maximum strengths with the fine to medium silts values between 1.99 N/mm² and 3.31 N/mm² are resulted. **(Jöstl etall, 2003)**

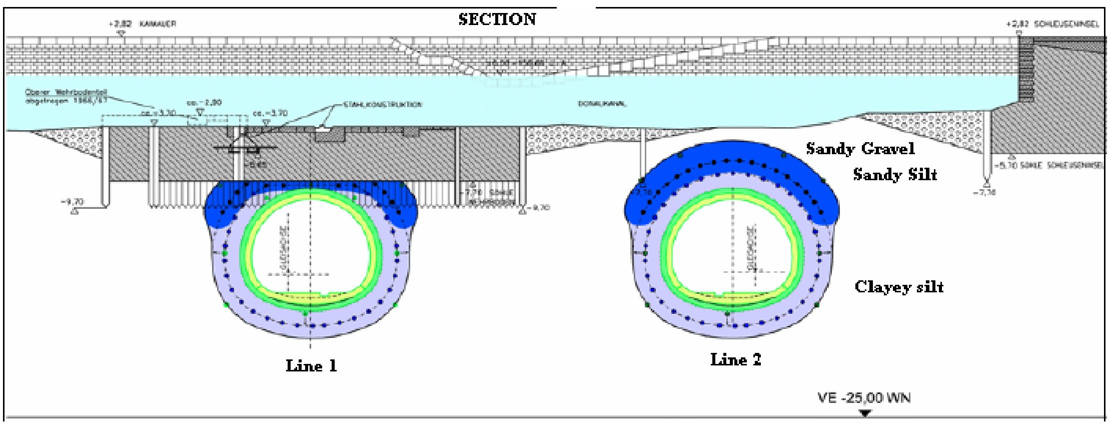


Figure 5.7: Cross section of the tunnel with planned freezing parts

5.3 Before Construction

As it is seen in Figure 5.7, two sections were planned and in progress. But in one tunnel because of the old foundation of the Emperor Bath freezing is bordered with that concrete structure. Both tunnels are driven through Clayey Silt soil and in order to prevent the water from Danube all section is planned to be frozen. But especially for the top parts, the freezing power and distribution of pipes are high with correspondence to bottom parts. Dark Blue Parts are planned to be frozen with LN_2, on the other hand light blue ones are planned to be frozen with brine.

From the first week of December 2001, the temperature variation under the Danube has begun to be measured in 7 boreholes. Seasonally the temperature gradient were measured between +9˚ C to +19˚ C. The seasonal variations of the Danube Canal are measurably, but for the cost of the project and construction period, winter term is selected. The deeper soil strata under Danube gave +12˚ C at average.

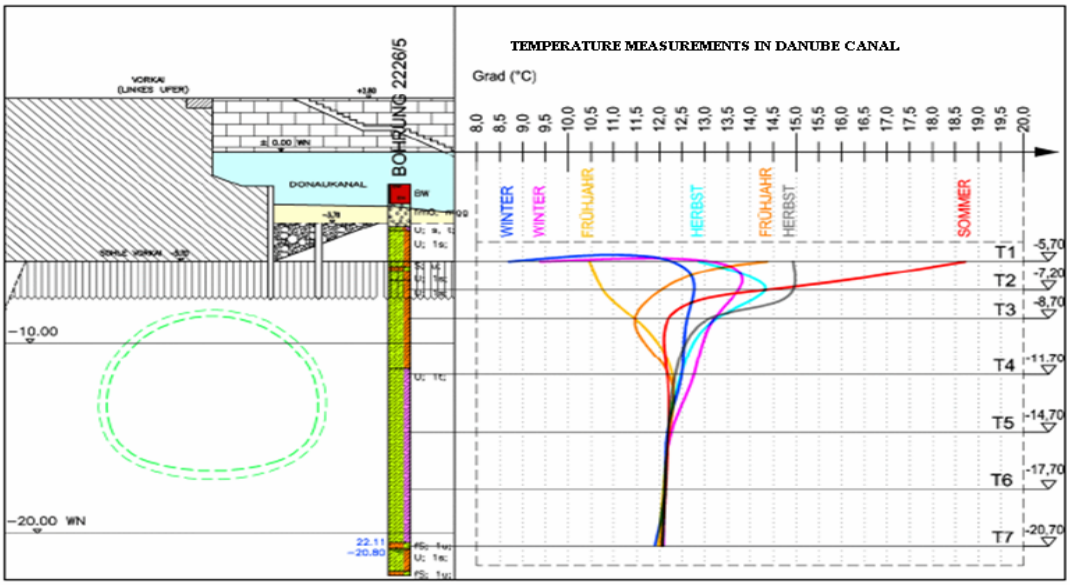


Figure 5.8: Temperature along borehole in Danube Canal (Jöstl etall., 2003)

The current consumption during the freezing-up period of approx. 43 weeks with approx. 3 million kilowatt-hours calculated. Nitrogen consumption (LIN) is proposed as 3 million liters. The soil volume which will be frozen amounts to approx. 14,000 m^3, the entire freezing period is approx. 10 months. **(Jöstl etall, 2003)**

5.3.1 Freezing Pipe Installation and Monitoring

The drillings of pipes are parallel to the tunnel axes. Per axes three drillings are to be implemented for temperature measurement due to of defaults from pit geometry diagonally to the tunnel axes. The drilling pattern and the respective drilling lengths are different due to the boundary conditions in tunnel 1 and tunnel 2. The drilling lengths for the production of the circular freezing up body amount to up to 40 m with an overall length of the drillings of approx. 9,700 m, the drilling work was started in June 2004, and the beginning of the freezing up was started at 13^{th} January 2005.

Figure 5.9: Drilling of boreholes of freezing pipes

Temperature measurements of frozen body over the cross section of tunnel-1 are done with 12 temperature measuring tubes, and in cross section of tunnel-2 are done by 9 distributed temperature measuring tubes. In the temperature measuring tubes as rule stationary temperature primary detectors at a distance of 2.0 m are arranged, thus an accurate determination of the temporal development of the frost body thickness and temperature distribution can be accomplished. At the beginning and at final range of the temperature measuring tubes, the distance of the temperature primary detectors is reduced to 0.5 m at connection to the Emperor Bath. The temperature measuring tubes were filled with a $CaCl_2$ solution, in order to guarantee a good heat transfer between soil and measuring level.

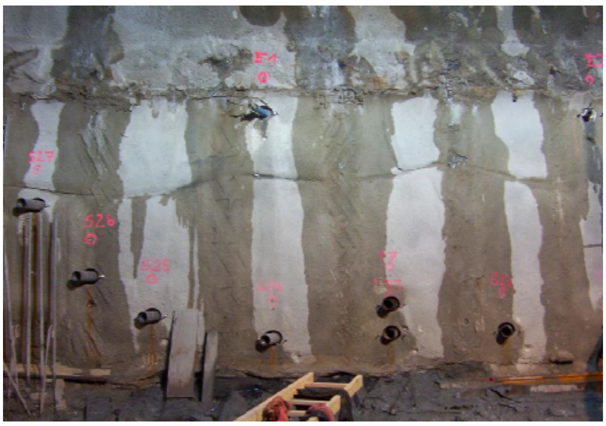


Figure 5.10: Freezing pipes and temperature pipes

The freezing procedures were already used several times in the past 25 years successfully in the Viennese underground constructions. For example as sealing and improvement of the soil in construction of the line U3 under the department store Herzmansky in the Mariahilfer road and Telecommunication Center in Meidling freezing process was done in success . **(Martak and Herzfeld, 2005)**

The same process was decided to be used in U2/1 Project passing through Danube Canal with common view of Municipality and TU Vienna and BOKU Vienna. The heart of the section U2/1 - Schottenring represents the station Schottenring within the range of the monument-protected emperor bath air-lock, which can be established again under the Danube Canal.

When planning of the under crossing of the Danube Canal a solution with half lateral excavations in the river was examined. It was however denied, because it would have stopped the trip traffic of the navigation at the Danube Canal over two seasons. This would have meant a substantial decrease of incomes for the trip navigation, whose subsequent costs could be calculated for the tourism only inaccurately.

The Danube Canal has its main arm in the northeast of the city in the last centuries. Vienna Municipality shifted Danube to the east districts several times. Today's situation goes on adjustments at the beginning 19^{th} Century.

Figure 5.11: Freezing process has begun

5.3.2 Considerations in Freezing

In this freezing project it is decided to use the system as mentioned above combined method as third solution. For the choice of the combined freezing process the following considerations were decisive:

- High cooling capacity would minimize the effect of thermal supply from the Danube Canal flowing which can not be excluded, fast up to 2 m/s
- The route of the tunnel drives under the air-lock island and the military field diagonally with the station pipes, whereby the monument-protected hydraulic engineering calculations are to be done;
- The clayey silt soil around the tunnel driving is inclined approximately with the freezing procedure over the duration of several months to intensive elevations,
- If soil ranges with the tunnel driving, which are unexpectedly seem to be soft during the work, a fast possibility high-energy of the post freezers, in particular toward the river bed of the Danube Canal is of planning;
- It is necessary, around the entire edge of tunnel outbreak a freezing up ring of approx. 2 m thickness, so that the hydraulic lift security of the station tunnel outer shells is given

These will be realized with liquid nitrogen (LN_2) for the shock freezing and full rings forward around and afterwards the brine ($CaCl_2$) will push freezing process.

With this procedure the rapid formation of a high-energy cooling potential (LN_2) can be fulfilled for the minimization of the ice lens formation. Nitrogen has a temperature of -196 as liquefied gas at 1 bar °C. **(Martak and Herzfeld, 2005)**

With the brine procedure an economical and reliable freezing process is available over several months, which can hold the circular frost body after some weeks and can enterprise the desired temperature range between -10 °C and for -20 °C. The brine used in the brine freezing process has a maximum freezing carrier temperature of approx. -40° C. The freezing procedure in the soil lasts however clearly longer than with the LN_2 (according to experience 5 to 7 weeks), which will particularly lead in clay towards silt to ice lens formation and elevations, which can be kept relatively small however by the load soil already frozen with LN_2 and the present buildings over it. Geotechnical control of the current temperatures in the freezing bodies is guaranteed by temperature measuring probes over the entire length of the frost bodies and distributed over the extent.

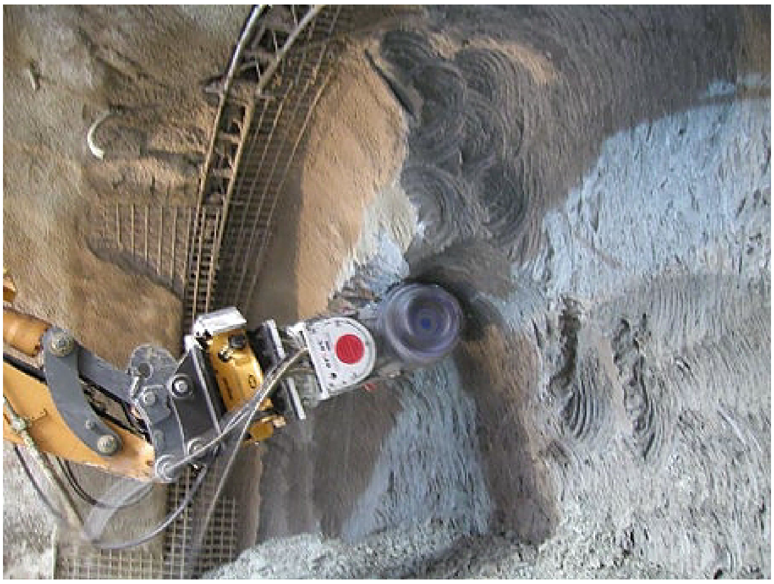


Figure 5.12: Excavation in the Tunnel

It was necessary to manufacture purposeful freezing drillings for the filling with brine/nitrogen from both sides of the Danube Canal which is 40 m long. From both sides pipes will be parallel to the tunnel axes and will be intersected in the center approximately 4 m. Their distance between each other are 1.1 m to 1.4 m. Drilled pipes diameters were 139 mm and exhibit in individual cases vertical deviations to 0.70 m and horizontal deviations to 0.12 m. According to plan altogether 4 x 55 drillings (30 drillings for the brine freezing ring, 13 drillings for the liquid nitrogen partial drillings for each tunnel driving) only few auxiliary drillings had to be implemented.

The regular thickness of the freezing up body amounts to within the range of the brine freezing 2.0 m, within the range of the combined freezing process (in the roofridges) up to 3,5 m with station pipe D1. With the station pipe D2 the freezing up body is enough to the military soil near and has thereby a smaller thickness. As edge of frost body the extent is defined, that exhibits a temperature of -2° C. For the formation of the frozen body with liquid nitrogen is estimated one period of 15 days, for the employment of the brine 50 to 55 days. Current control of the air conditioning probes supplies the information about the development of the frost body thickness.

After the frost body achieved the intended thickness, further increasing is no longer desired. The phase of the frost attitude begins. The frost border is to be kept constant by intermittent brine.

Figure 5.13: General view of excavated part from Shaft1

5.4 Costs

The offered total sum amounts to approx. 48 millions Euro. 50 % of the costs to the open building method and 50% are allotted to the closed building method. The distribution of the costs of the closed building method shows that approx. 50% of the costs on the building remedial measures (ground freezing, ground-water lowering, jet procedure, etc..) not applicable. The offered costs of the building ground freezing up, based on the mass determination of the advertisement, amount to approx. 4.6 millions Euros. During a theoretical building ground freezing up of approx. 14,000 m^3 and means 329 Euro/m^3. Related to the section length of approx. 817 m means amount to the net costs 55,080 Euro/m. **(Joestl etall, 2003)**

6. LABORATORY WORK

6.1 Freezing and Thawing Experiments

6.1.2 Laboratory Soil Parameters:

The aim of these experiments was to define the theoretical action of the frozen ground. The site behaviour was tried to be related with the results taken from experiments. Furthermore phenomena as Frost heave, thawing process of the site were tried to be understood.

Due to the experiments carried out on two type of silt in both TU Vienna and BOKU Vienna, soil parameters written below were found.

Table 6.1: Soil parameters (TU Vienna)

Parameters		Silt 2	Silt 3
Bulk Density	ρ_s [g/cm^3]	2.73	2.74
Density	ρ [g/cm^3]	1.96	2.02
Dry Density	ρ_d [g/cm^3]	1.61	1.63
Water Content	w [%]	22	24
Porosity	$n = 1- \frac{\rho_d}{\rho_s}$	0.41	0.41
Liquid Limit	w_L [%]	50	46
Plastic Limit	w_P [%]	28	27
Plasticity Index	$I_p = w_L - w_P$ [%]	22	19

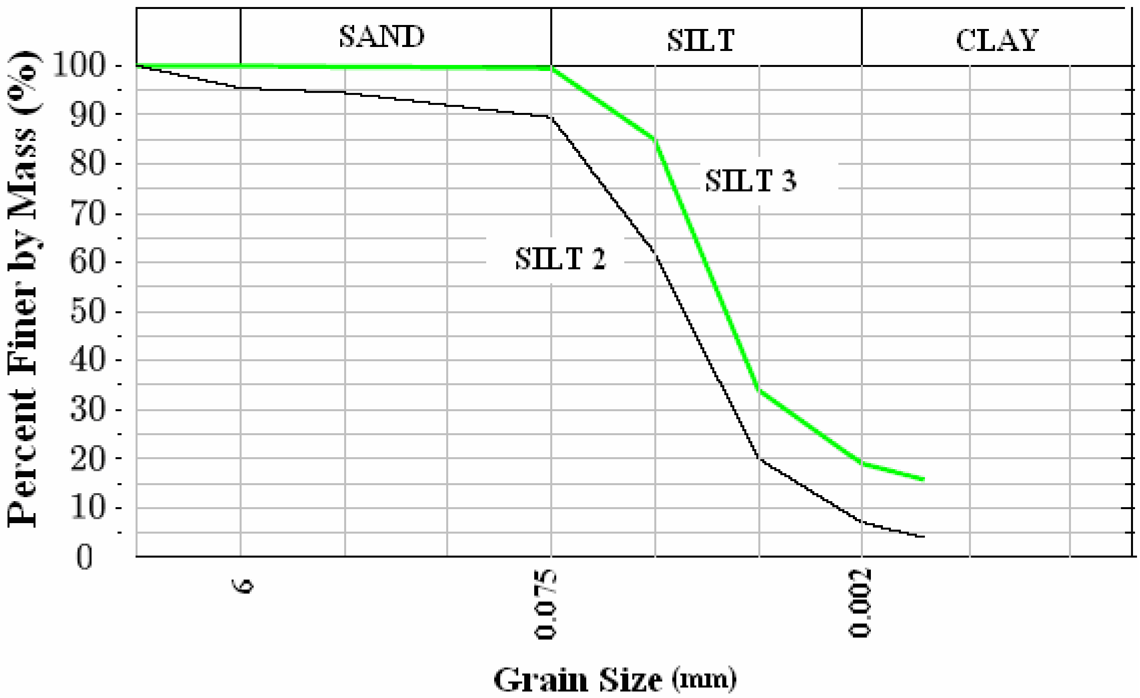


Figure 6.1: Grain size distribution of the soil

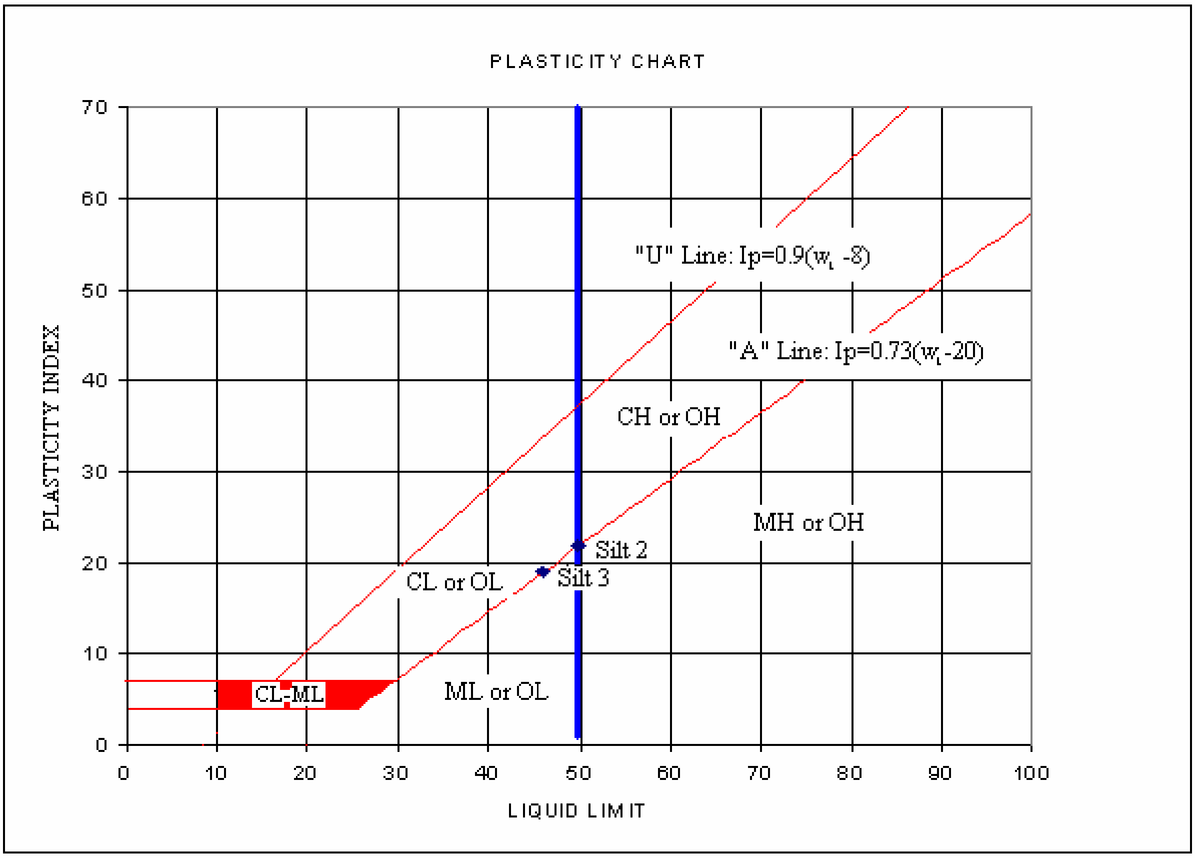


Figure 6.2: Plasticity Card: Samples can be seen as ML

According to the plasticity chart Figure 6.2, it can be classified as Clayey silt (ML). It is seen that it is poor graded silt with 27% of clay from grain size distribution Figure 6.1. Also the mineralogy of the clay was examined. As can be seen from table 2, 16 % of the all sample is with smectite group of clay minerals.

Table 6.2: Mineralogy of the clay in samples (Potz, 2005)

Silt	**Percentage in Clay**				**Total Clay**	**Percentage in Total**			
	Smectit	Illite	Caolinite	Chlorite	**%**	Smectit	Illite	Caolinite	Chlorite
2	58%	22%	10%	10%	26.8%	16%	6%	3%	3%
3	62%	20%	9%	9%	26.7%	17%	5%	2%	2%

6.2 Experiments

All the experiments were carried out by Institute of Material Science in Vienna University of Technology. 29 experiments were done on three different specimens taken from site, and these are classified as Silt 1, Silt 2, and Silt 3. There were 5 specimens from Silt 1, 10 specimens from Silt 2 and 14 specimens from Silt 3.

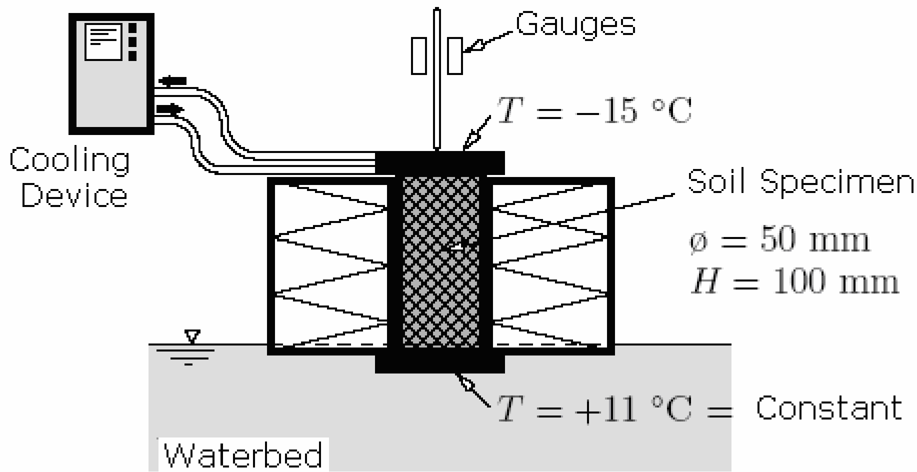


Figure 6.3: Schematic description of freezing

As shown in the figure 6.3, a cooling device is connected to the soil specimen with dimensions of the radius 50mm and height 100 mm. In order to reflect the site parameters bottom side of the specimen is filled with water at +11˚ C and freezing and thawing behaviour of the soil was tried to be understand. From the top of the soil is chilled to -15 degrees with a cooling device. In order to insulate the samples, all samples are prevented in insulation foam to get better results. (Figure 6.4)

Figure 6.4: View from experiment, insulation foams are used to prevent heat loses.

As can be shown in the figure 6.5, sample is monitored by 4 temperature gauges directly connected to the computer. All data from the soil was gained at every 2 minutes. All gauges have same gap between each other with 2 cm.

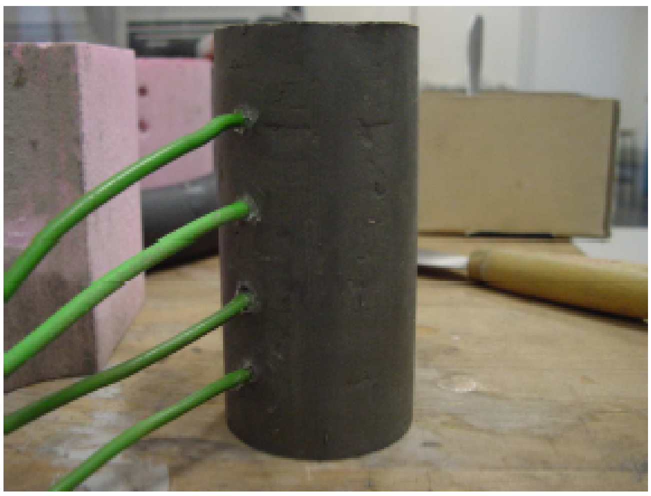


Figure 6.5: Specimen with temperature gauges

Experiments were done in 4 ways. Nine samples were examined with cooling rate -50˚C/h, three samples were examined with cooling rate -0.4˚C/h, seven experiments were done with thawing cycle after freezing, and four samples were examined with two step freezing.

6.3 Results and Discussion

6.3.1 Frost Action

After the experiments data the real ice can be seen at freezing point near 0 ˚C. This kind of behaviour is same as figure idealized frost heave soil column of Nixon 1992. Nixon has explained this phenomenon as the rate of heat extraction exceeds the rate of heat supply (water flow) to freezing point. The frozen fringe is defined as the region of the impeded flow caused by partial filling of soil pores by ice (Nixon, 1991). If the pressure in the ice exceeds the pressure of overburden material and other pressures, the soil begin to expand. When there is sufficient ice pressure, the new ice lenses will be formed. This upward movement in the site is called Frost Heave. In slow freezing process the temperature gradient will be formed near freezing fringe and unfrozen zones. In the fast freezing process the permeability of the frozen fringe will decrease and this causes the less ice lenses in the body. As a result the relation of the water and heat flow would cause thinner ice lenses upwards and thicker ice lenses in downwards away from the supply of cooler. **(Andersland, 2004)**

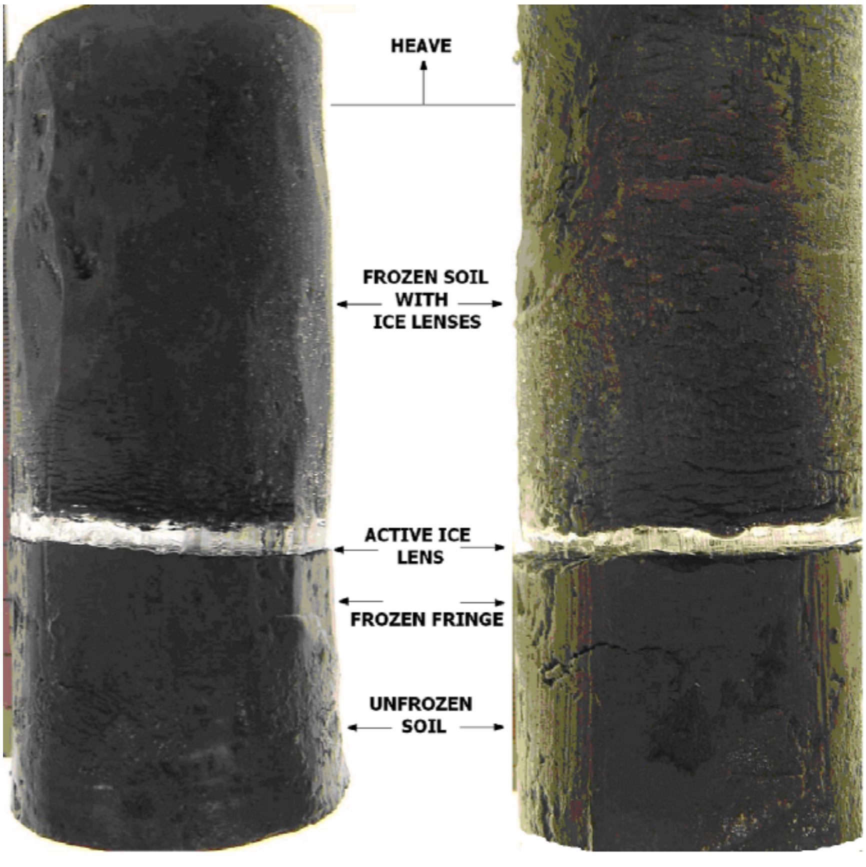


Figure 6.6: Frozen Soil Samples, Ice can be seen by naked eyes

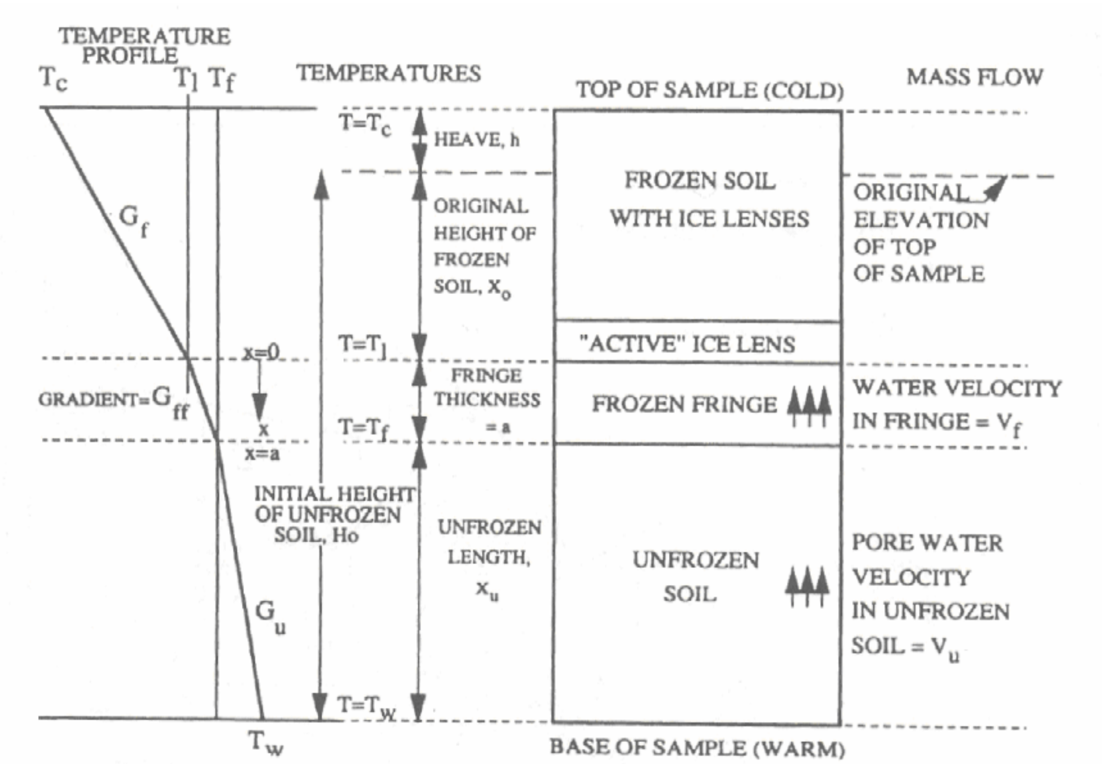


Figure 6.7: Frost heave in idealized one dimensional soil sample (Nixon, 1992)

6.3.2 Correlations and Statistical Evaluation of Results

Table 6.3: Correlation of heaving and water contents

Sample	Heaving (mm)	Δω (%)	x^2	y^2	xy
1 (F)	12.5	6	156.25	36	75
2 (F)	10	4	100	16	40
3 (F)	20	12	400	144	240
4 (F)	10	7	100	49	70
5 (F)	19	14	361	196	266
6 (F)	14	9	196	81	126
7 (S)	17	10	289	100	170
8 (S)	18	14	324	196	252
9 (S)	20	17	400	289	340
10 (S)	23	20	529	400	460
11 (S)	22	16	484	256	352
Total	Σ x = 185.5	Σ y = 129	Σx^2 = 3339.25	Σy^2 = 1763	Σ xy = 2391
Mean	16.86	11.73			

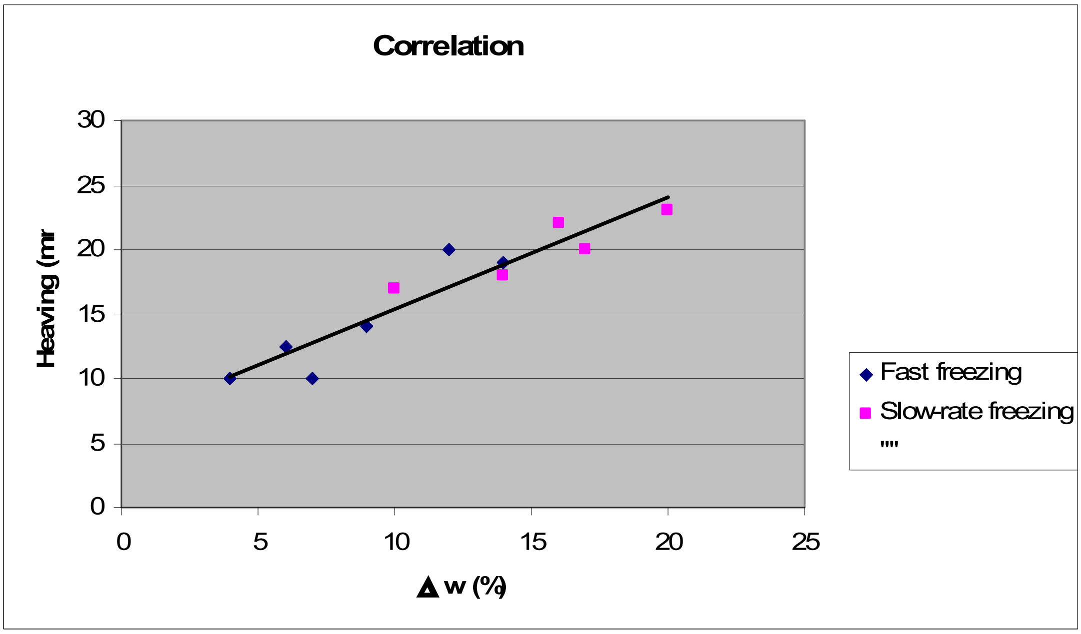


Figure 6.8: Correlation of Heaving with Δw (water contents)

$\Sigma d\, x^2$ =	211.05
$\Sigma d\, y^2$ =	250.18
Σd xd y =	215.59

$r_c = 0.938$
$r = 0.602$ From "r" Table; p=0.05
$r_c > r$

The calculated value (0.938) does exceed the tabulated value (0.602) at p=0.05 value. It also exceeds the tabulated value for p = 0.01 and for p = 0.001. So it can be said that 99.9% confident that heaving and increase in water content are positively correlated in experiments.

Table 6.4: Mann Whitney U Test data

Heave (mm)	10	10	13	14	17	18	19	20	20	22	23
Process	F	F	F	F	S	S	F	F	S	S	S
Row no	1.5	1.5	3	4	5	6	7	8.5	8.5	10	11

According to nonparametric alternative test (The Mann-Whitney U Test), null hypothesis of equal means is rejected at 95th percentile level.

$R_x = 5+6+8.5+10+11= 40.5$
$R_y = 5 \times (6+5+1)-40.5 = 19.5$
$R = 20 > R_y$ (rejected; from U Test table, n=5 m=6 R can be found at α=0.05)

This means the heaving ratio at slow rate freezing is more than that of fast freezing.

Table 6.5: Change of Consistency Index in Freezing Thawing cycle in specimen [before freezing/After experiment upper part/ After experiment bottom]

Sample Nr	Water Content (%)			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Consistency Index (I_c)
	Before	After Upper	After bottom				
13(2)	28.0 /	32.7 /	34.1	49.5	28.0	21.5	1.00 / 0.78 / 0.72
14(2)	26.3 /	30.3 /	31.0	49.5	28.0	21.5	1.08 / 0.90 / 0.86
15(2)	27 /	30.6 /	33.3	49.5	28.0	21.5	1.04 / 0.88 / 0.76
16(2)	27.4 /	32.4 /	32.9	49.5	28.0	21.5	1.03 / 0.79 / 0.77
17(2)	27.7 /	33.9 /	32.9	49.5	28.0	21.5	1.01 / 0.72 / 0.77
22(3)	24.7 /	30.3 /	36.6	46.0	27.0	19.0	1.12 / 0.83 / 0.50
31(3)	26.8 /	36.2 /	31.2	46.0	27.0	19.0	1.01 / 0.52 / 0.78

As it is seen from the Table 6.5 the consistency indexes of the specimen are above 1 before freezing. After freezing, the amount became smaller than “1”. This show the properties of the soil is physically changed and the soil become more plastic.

Table 6.6: Water contents in specimen [Before freezing/After experiment upper part/ After experiment bottom/ Overall]

Specimen no	Silt		Cooling Rate [°C/h]	Heaving (mm)	After Thawing (mm)	Water Contents (%)			
						Before	After Top	After Bottom	Overall
8	1	Freezing	-50	21		26	43	32	39
9	1	Freezing	-50	14		27	40	36	38
10	2	Freezing	-50	12.5		26	36	27	32
11	2	Freezing	-50	10		27	34	26	31
12	2	Two Step Freezing	-50	11		27	34	28	32
13	2	Freezing/Thawing	-50	11	6	28	33	34	33
14	2	Freezing/Thawing	-50	14	2	26	30	31	31
15	2	Freezing/Thawing	-50	13	2	27	31	33	32
16	2	Freezing/Thawing	-50	14	3	27	32	31	32
17	2	Freezing/Thawing	-50	15	3	28	34	33	34
18	2	Freezing	(0.4)	17		28	42	31	38
19	2	Two Step Freezing	-50	9		27	33	27	30
20	3	Freezing	(0.4)	18		26	46	34	40
21	3	Two Step Freezing	-50	26		28	58	30	43
22	3	Freezing/Thawing	-50	9	-0.5	25	30	37	33
23	3	Freezing	(0.4)	14		25	52	37	44
24	3	Freezing	-50	20		26	45	30	38
25	3	Freezing	-50	10		24	31	30	31
26	3	Freezing	-50	19		23	48	29	37
27	3	Freezing	-50	14		25	37	28	34
28	3	Freezing	(1.0)	20		26	47	36	42
29	3	Freezing	(1.0)	23		27	60	37	46
30	3	Freezing	-25	17		26	41	29	35
31	3	Freezing/Thawing	-50	20	3	27	36	31	34
32	3	Two Step Freezing	-50	30		25	56	28	43
33	3	Freezing	(1.0)	22		27	47	35	42

7. CONCLUSION

Artificial ground freezing is one of the most reliable methods in stabilizing the soft ground tunnels where the other methods cannot be effective or more expensive and where the ground water is concern. In this thesis the main idea for artificial ground freezing techniques and the design considerations with NATM tunnelling were discussed. All cases were considered from a geotechnical view and tried to be explained by principles of soil mechanics. Additionally one case study in Vienna Underground system has been given in details. Finally, the experiments carried out by Institute of Material Science at TU Vienna were given and some comments about the freezing process were tried to be explained. Cylindrical soil samples were tested in one-dimensional case and the relationship of heaving in slow-rate freezing and fast freezing and behaviour of soil during freezing were examined. The results can be summarized as below:

1- Heaving amounts for both slow rate and fast frozen samples are significantly different if p is set at 0.05 level (95% percentile level). This means the heaving ratio in slow rate freezing is more than that of fast freezing. There will be more heaving if the process is slow. This is because; growing of ice lenses in slow-rate freezing is more than that of fast one.

2- The means of water content values at slow rate freezing and fast freezing are giving reasonable differences. It can be said that the difference in two experiments is significant at the 0.05 level (95% percentile level) statistically. This means the water content ratio in slow rate freezing is more than that of fast freezing.

3- If the correlation of water contents and heaving values are studied statistically, it can be said 99.9% confident that heaving and increase in water content are positively correlated in experiments. So when the water content of the soil increases, the heaving ratio would increase. (Figure 7.1)

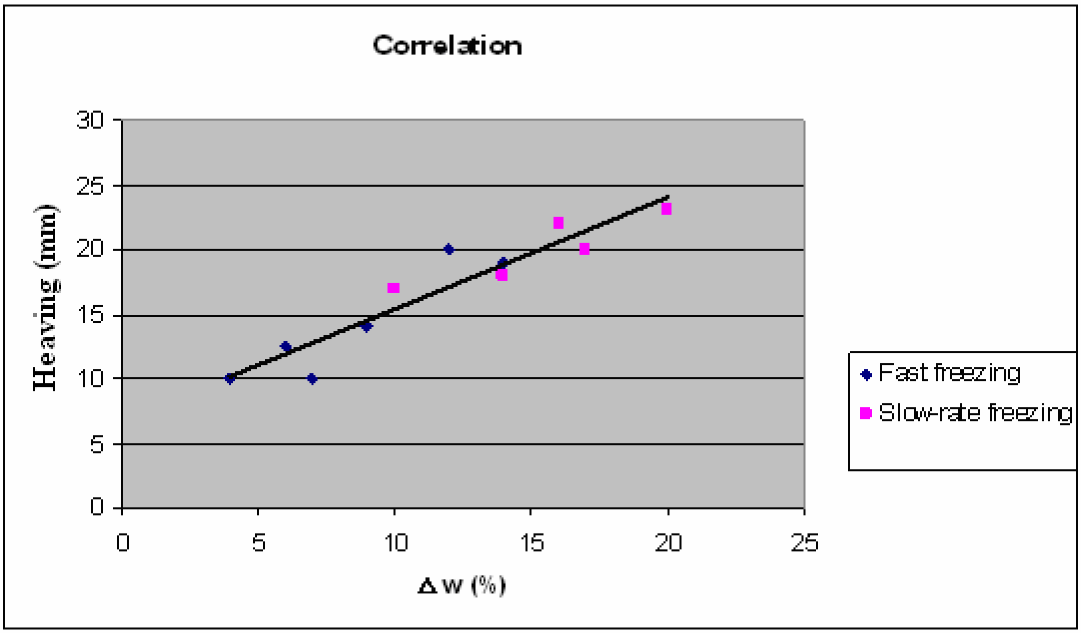


Figure 7.1: Correlation of water content versus heaving ratio.

4- The behaviour of heaving phenomena is related with the increase of water content in the soil. The water content increases with the decrease of temperature under freezing point. With the increase of water content, the heaving ratio increases. There is a physical relationship. When the water goes into ice phase, there would be 1.09 times volume change. So the difference between the last and the first water content ratio in frozen part is the change in volume.

5- The main process is the heat and water flow in the body. The heat is in water in soil. So the heat by means of water is transported to the cold and at 0º C the water is transformed into the ice. With the continuity of the heat transfer, the ice lens is formed and grown. So the heaving phenomenon occurs.

6- After thawing of the soil, due to changes in water content ratio the physical properties of the soil changes. If consistency indexes are studied, it can be seen that in silty soil, the soil becomes more plastic compared with the situation before freezing. This would be because of high percentage of clay minerals such as smectite.

7- Growing of crystals in slow reactions is regular and bigger than that of fast ones. So the more bigger crystals means the more volume deformation, if the freezing phenomenon is realised. This leads to the more heaving ratio in slow rate freezing.

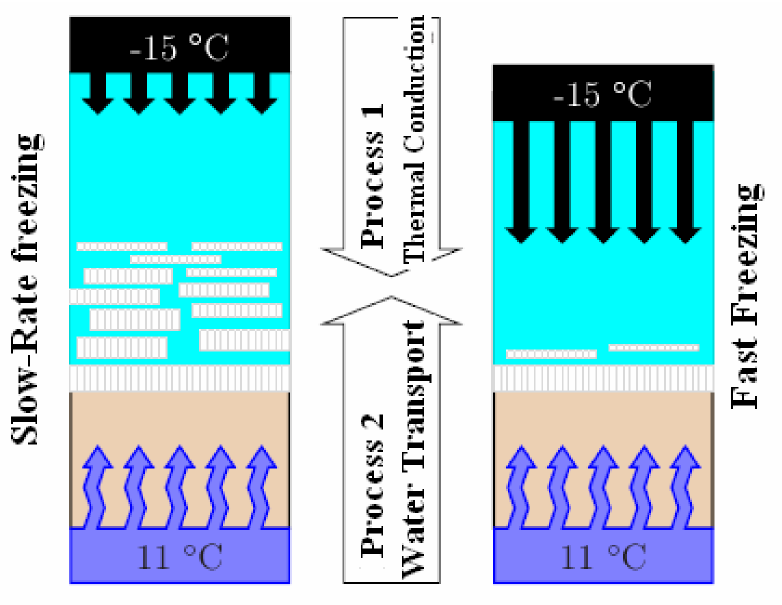


Figure 7.2: Behavior of one dimensional soil column during freezing (Martak, 2005)

8- The aim of this study to be used in practise was that; during lengthening of U2, the under crossing of the Danube Canal, artificial ground freezing method was chosen due to the circumstances in the site and its benefits given in section 2.

Moreover while chilling the ground, according to the results in slow-rate and fast freezing, first it was decided to freeze fast in order not to have much heaving amounts (Figure 7.2). After having the desired strength and thickness of the frozen wall, just to keep the amount of thickness, slow rate freezing would be applied. Because, there are some monumental buildings adjacent to the proposed project site and no more heaving is desired.

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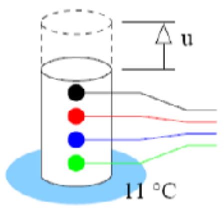
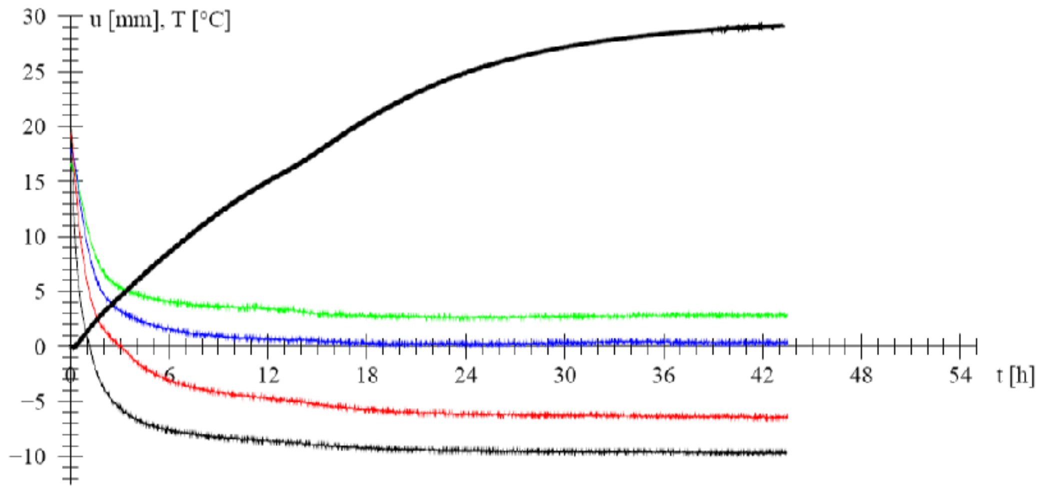
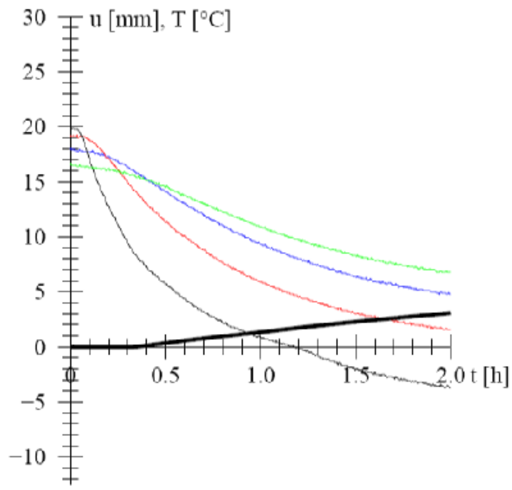
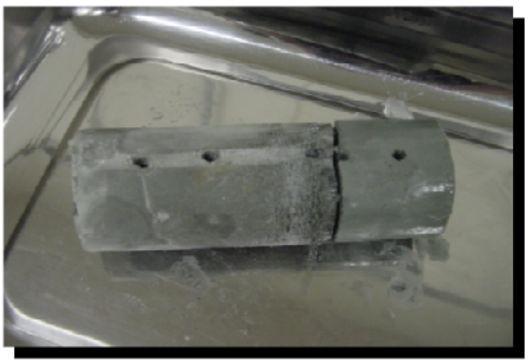
APPENDIXES

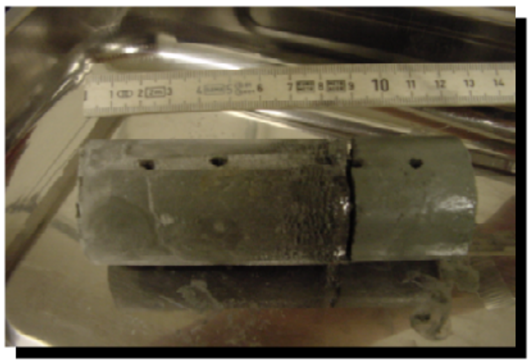
sample: 005 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one–step freezing (cooling rate: 50.0 °C/h)	27.7 % (before)
remark: ––	42.8 % (after – overall) –– (after – frozen) –– (after – unfrozen)



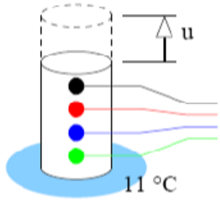


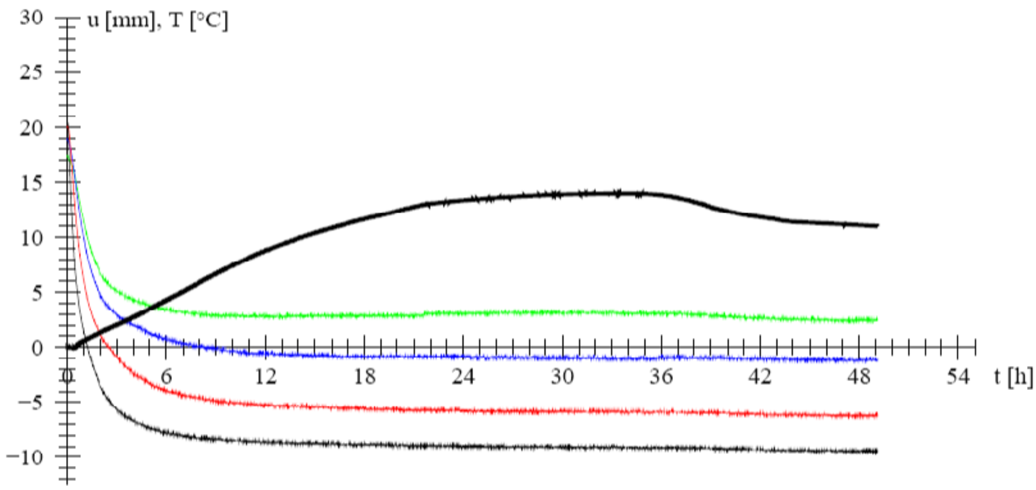


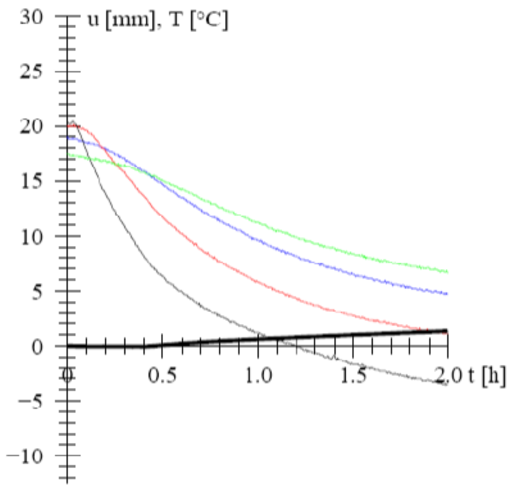


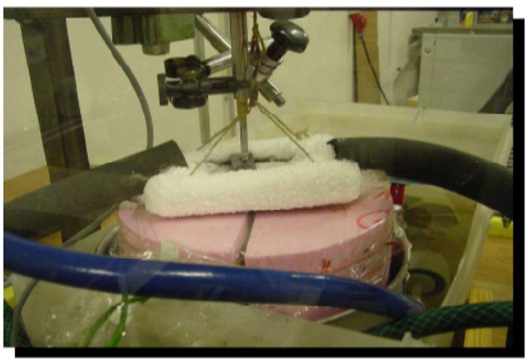


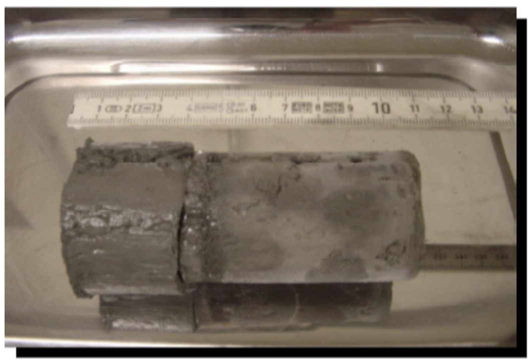
sample: 006 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.7 % (before)
remark: −−	37.3 % (after − overall)
	−− (after − frozen)
	−− (after − unfrozen)



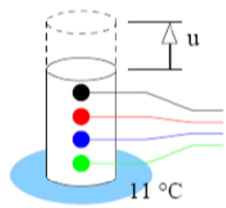


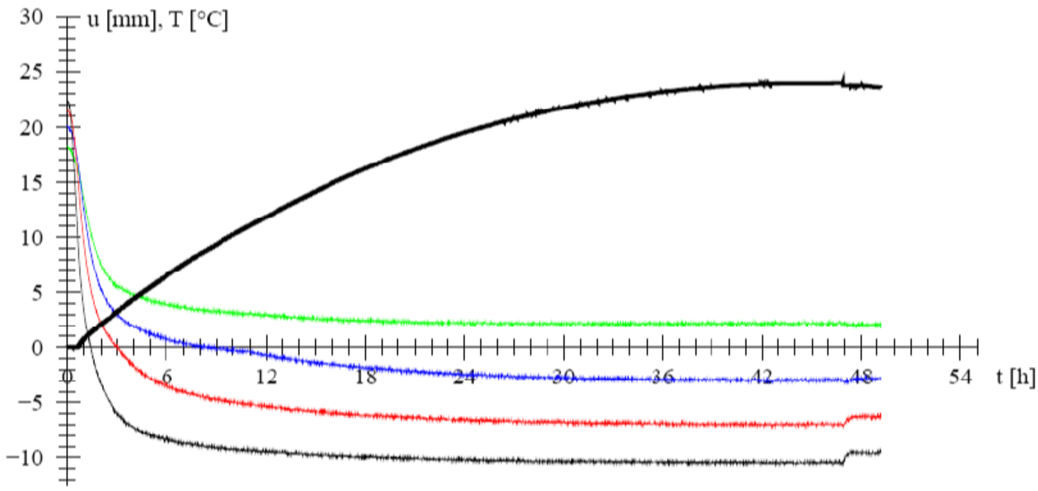


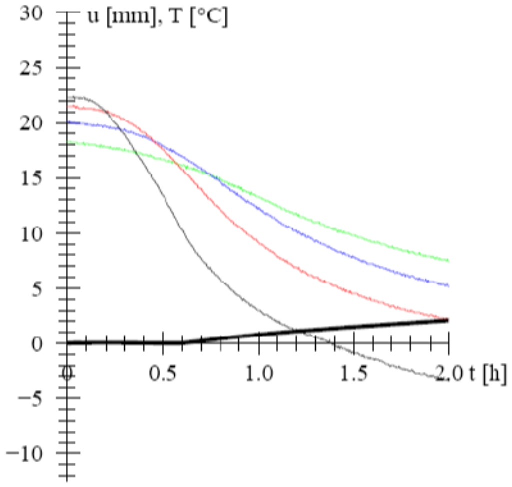


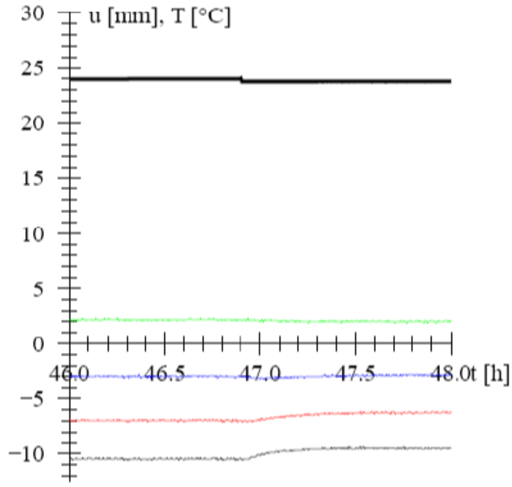


sample: 007 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one–step freezing (cooling rate: 50.0 °C/h)	26.2 % (before)
remark: ––	40.4 % (after – overall) –– (after – frozen) –– (after – unfrozen)

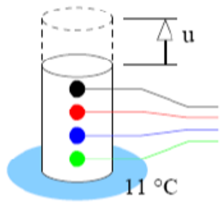


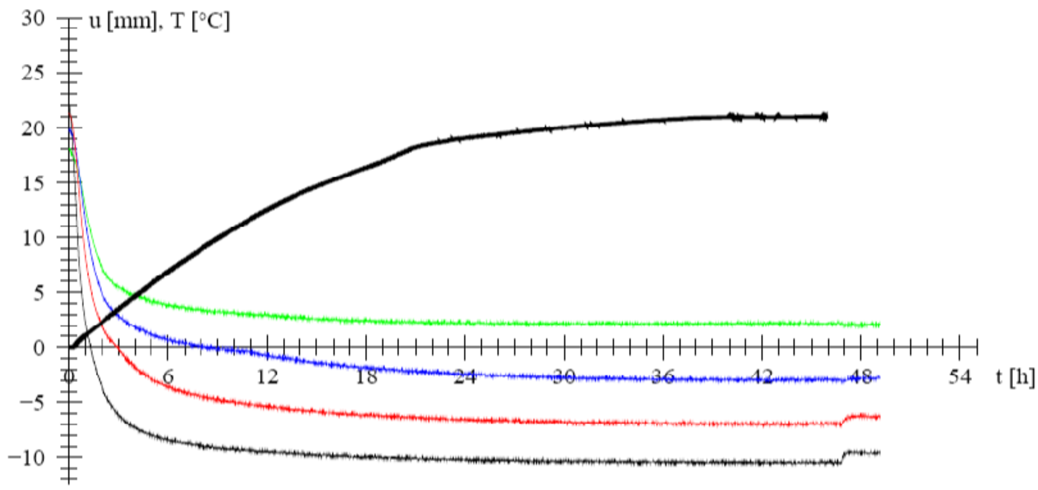


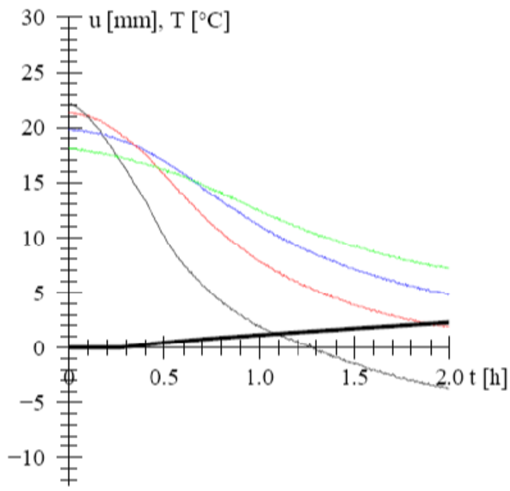


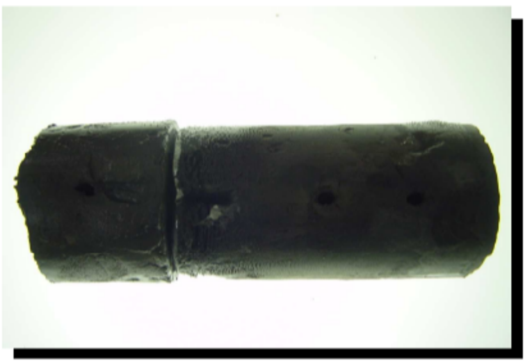


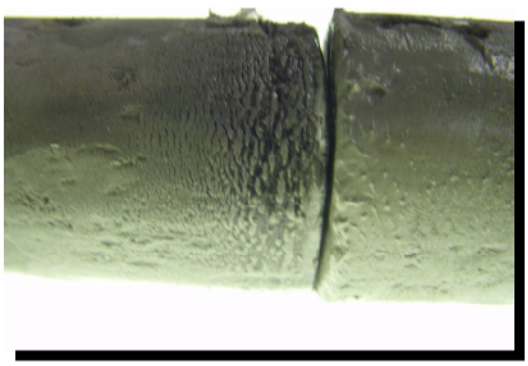
sample: 008 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.4 % (before) 39.2 % (after − overall) 43.4 % (after − frozen) 31.9 % (after − unfrozen)
remark:	



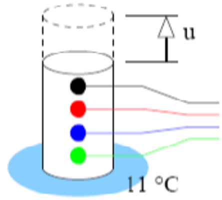


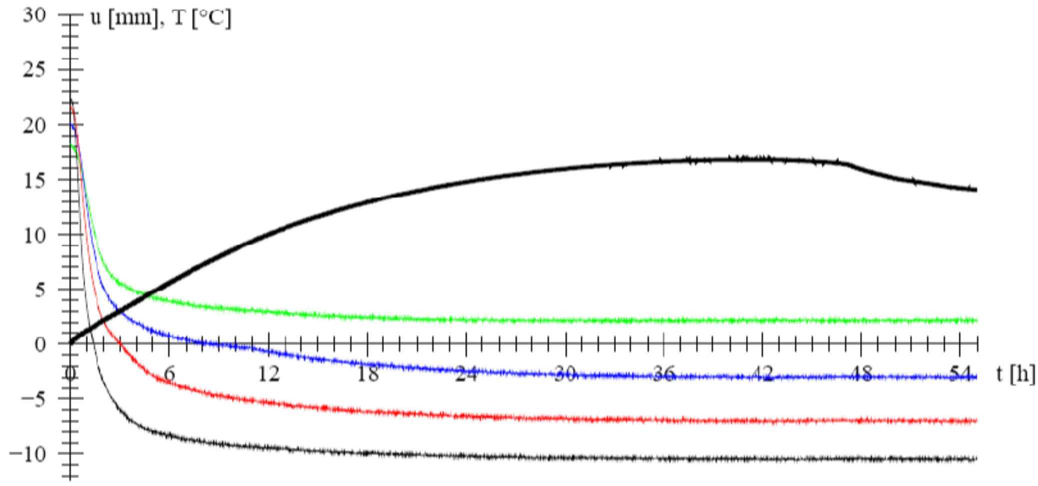


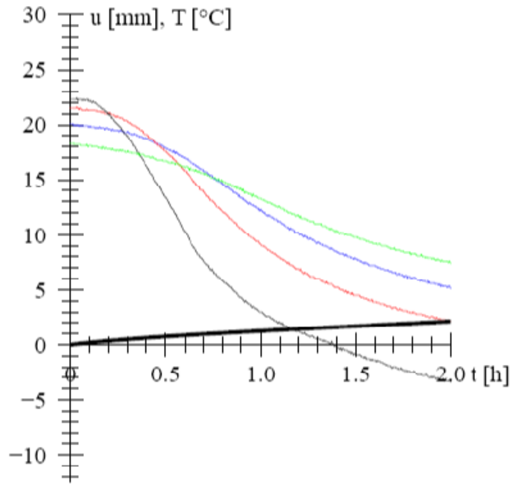


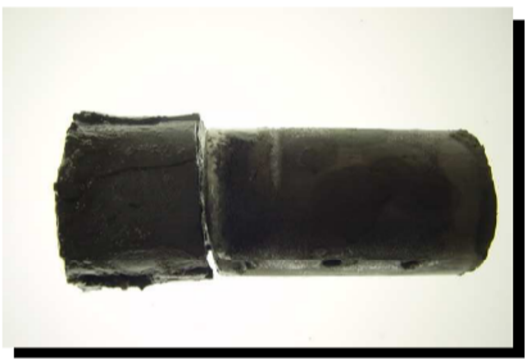


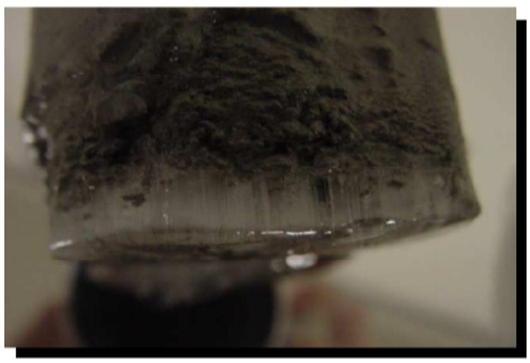
sample: 009 (silt 1) dimensions (H/D): 100/50 mm	water content:
experiment: one–step freezing (cooling rate: 50.0 °C/h)	27.1 % (before)
remark: ––	38.2 % (after – overall) 40.0 % (after – frozen) 35.5 % (after – unfrozen)



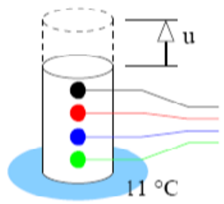


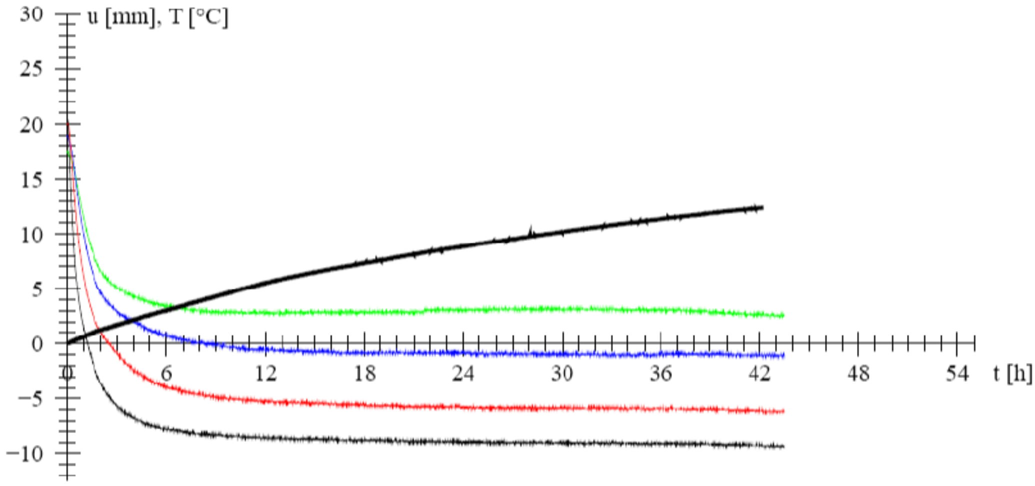


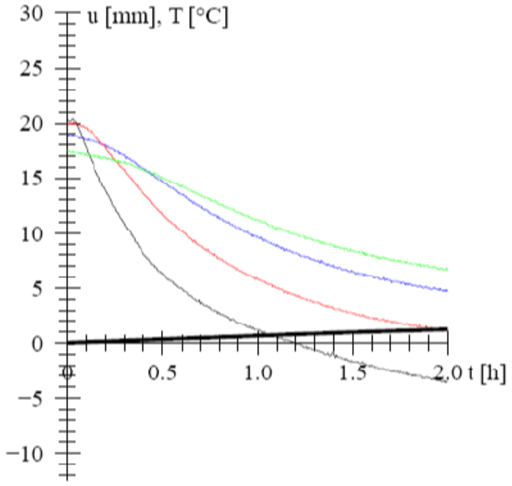


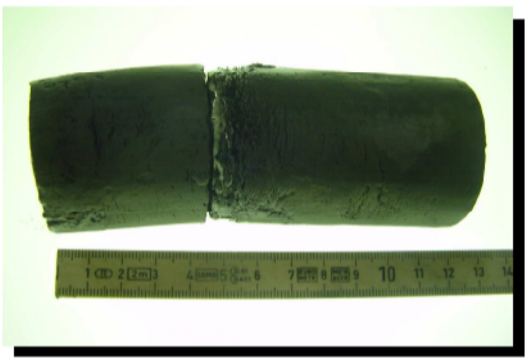


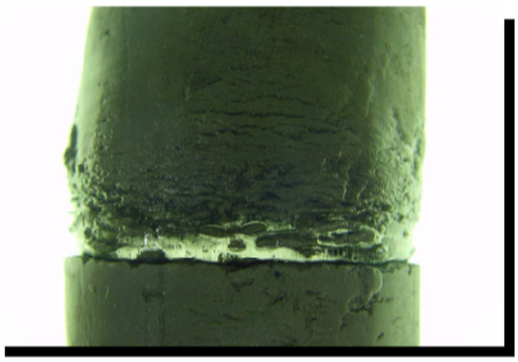
sample: 010 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	26.2 % (before)
remark: −−	32.2 % (after − overall)
	35.7 % (after − frozen)
	27.1 % (after − unfrozen)



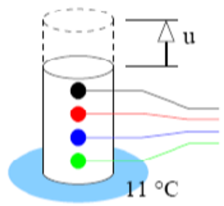


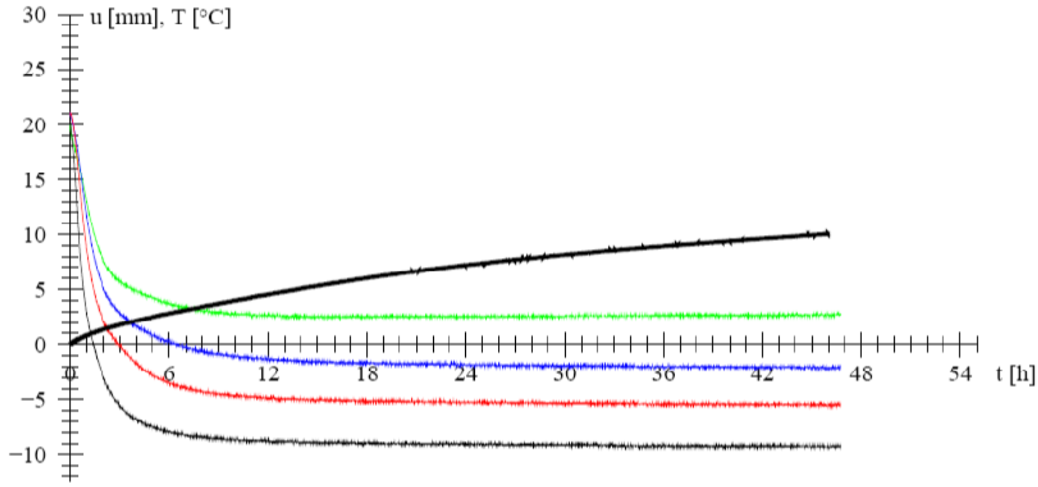


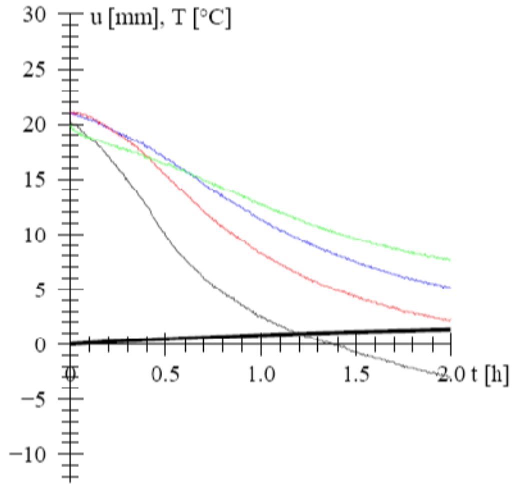


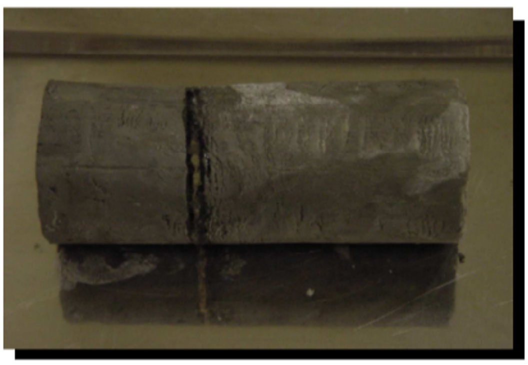


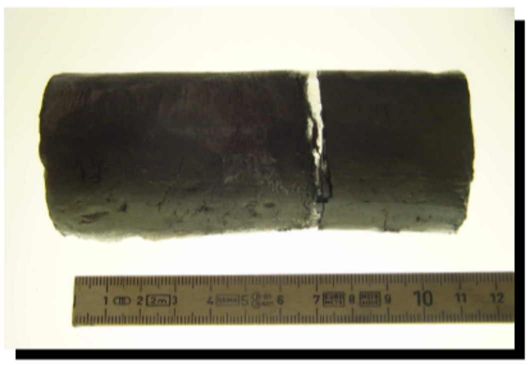
sample: 011 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	27.1 % (before)
remark: −−	30.8 % (after − overall)
	33.6 % (after − frozen)
	26.1 % (after − unfrozen)



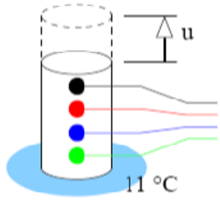


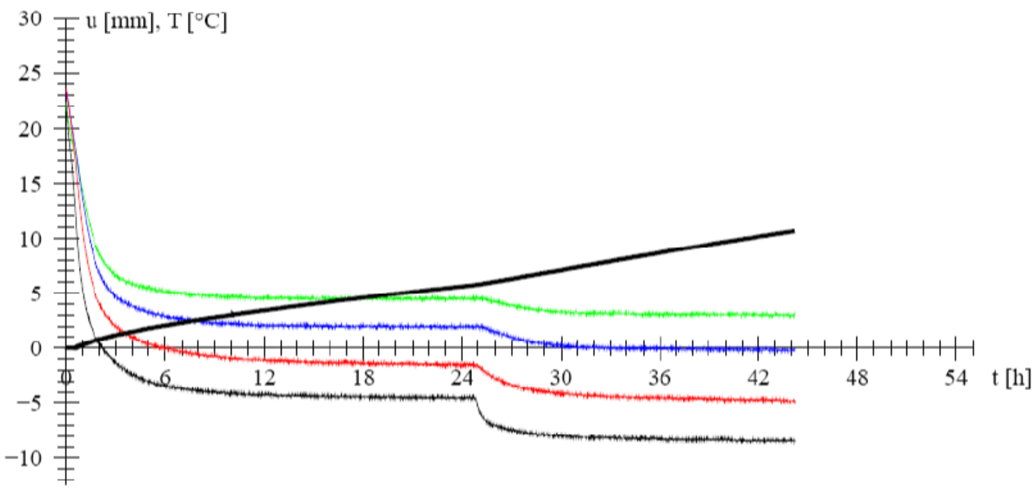


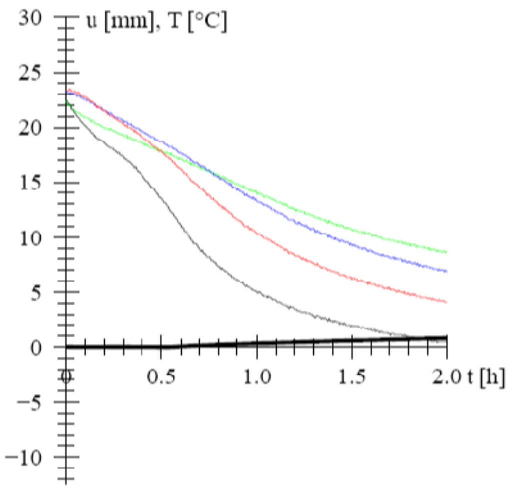


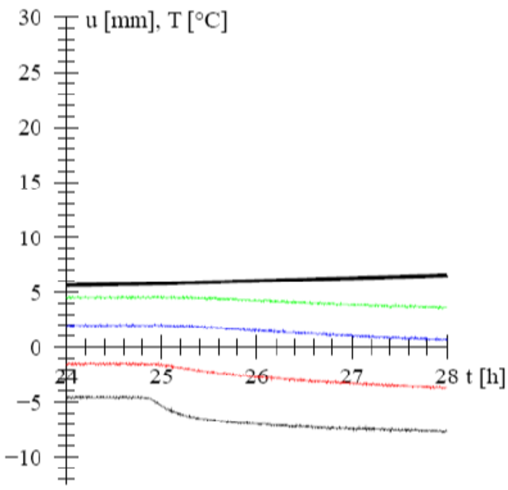


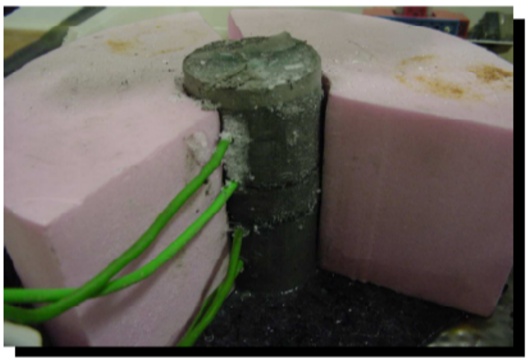
sample: 012 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.3 % (before)
remark: −−	31.5 % (after − overall)
	34.4 % (after − frozen)
	27.5 % (after − unfrozen)

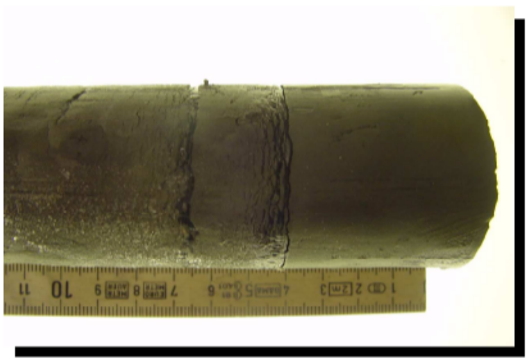




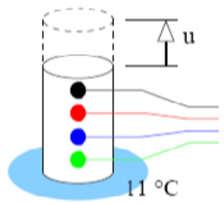


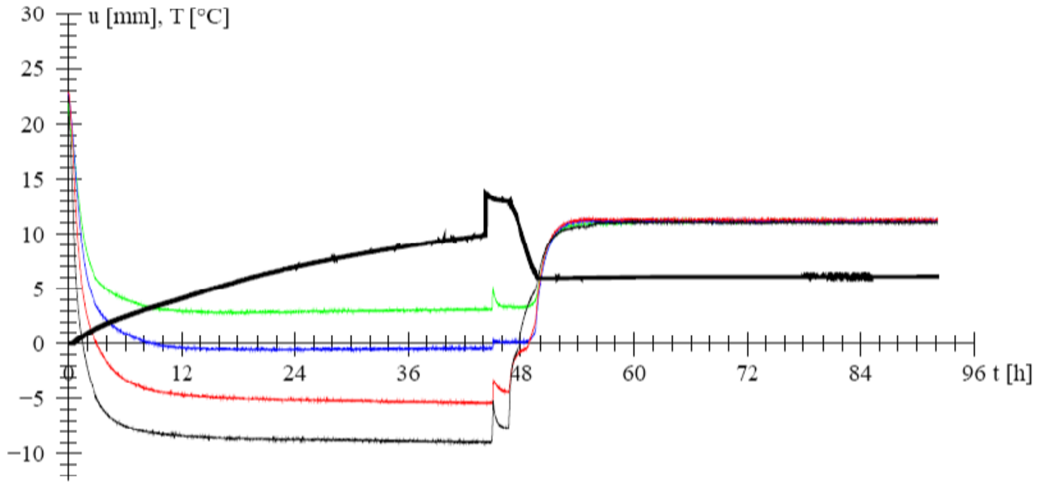


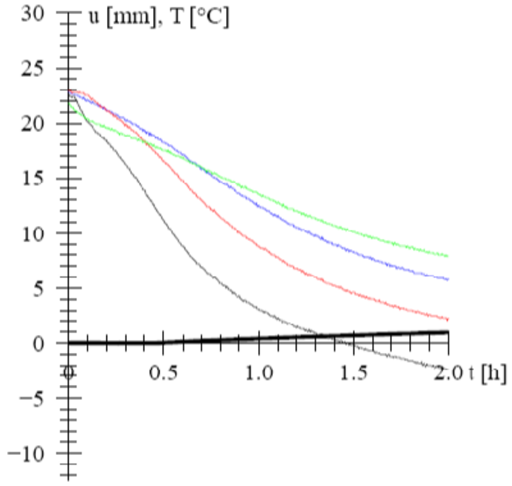


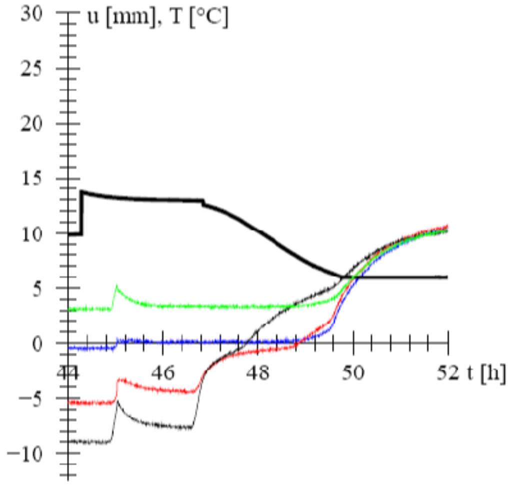


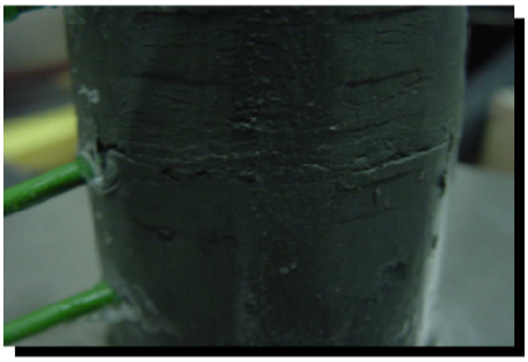
sample: 013 (silt 2) dimensions (H/D): 100/50 mm	water content:	
experiment: freez/thaw (cooling rate: 50.0 °C/h)	28.0 % (before)	
remark: sample was uninstalled after freezing and reintegrated for thawing to get the weight after the freezing process	33.2 % (after − overall) 32.7 % (after − frozen) 34.0 % (after − unfrozen)	



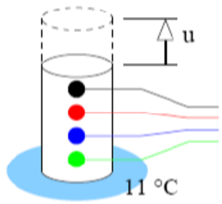


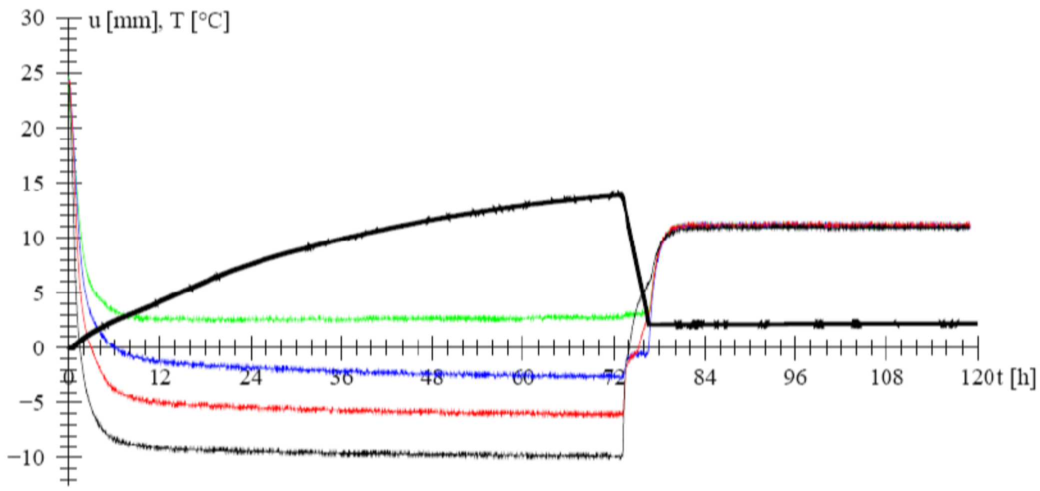


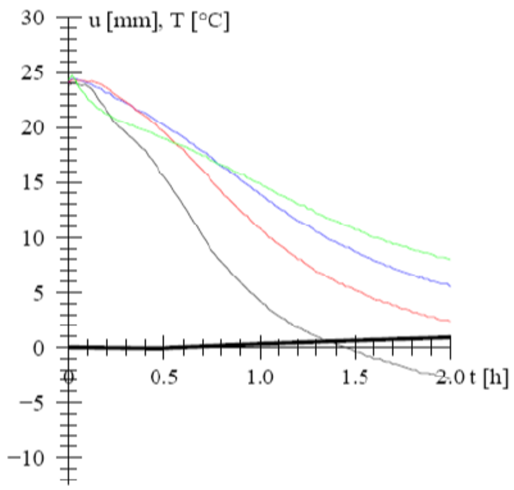


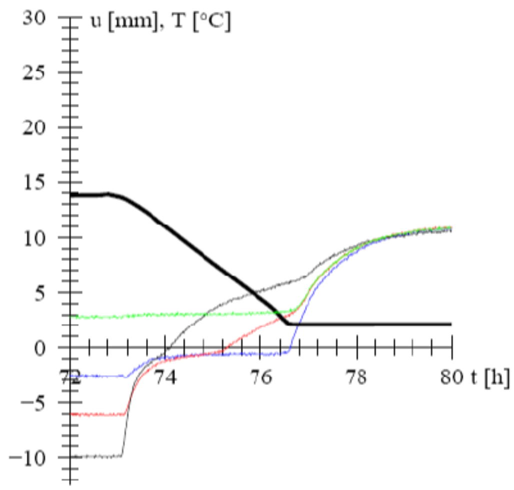


sample: 014 (silt 2) dimensions (H/D): 100/50 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	26.3 % (before)
remark: electrical power outage for one hour and ten minutes during the thawing process, measurement continued after this	30.6 % (after – overall) 30.2 % (after – frozen) 31.0 % (after – unfrozen)

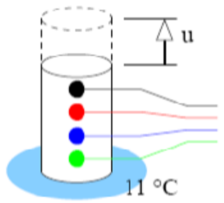


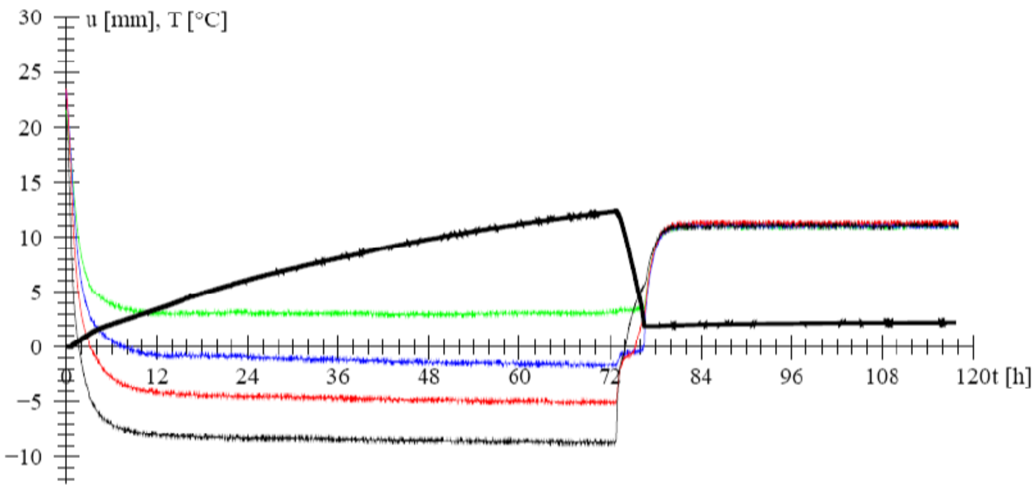


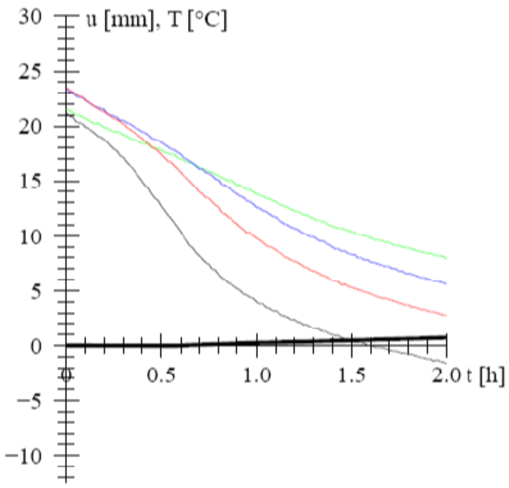


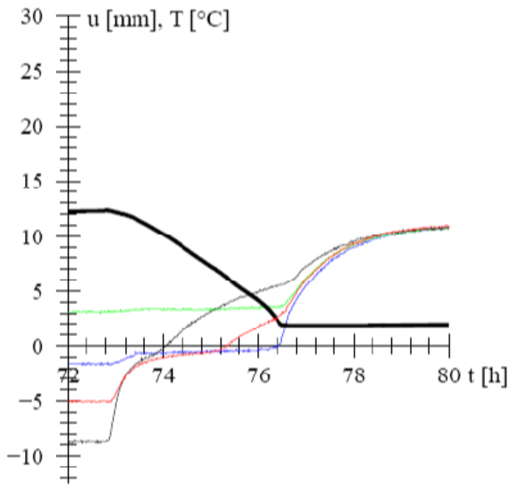


sample: 015 (silt 2) dimensions (H/D): 100/49.5 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.2 % (before)
remark: —	31.7 % (after – overall)
	30.6 % (after – frozen)
	33.3 % (after – unfrozen)

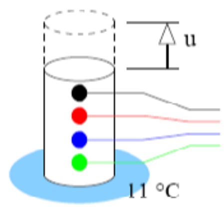


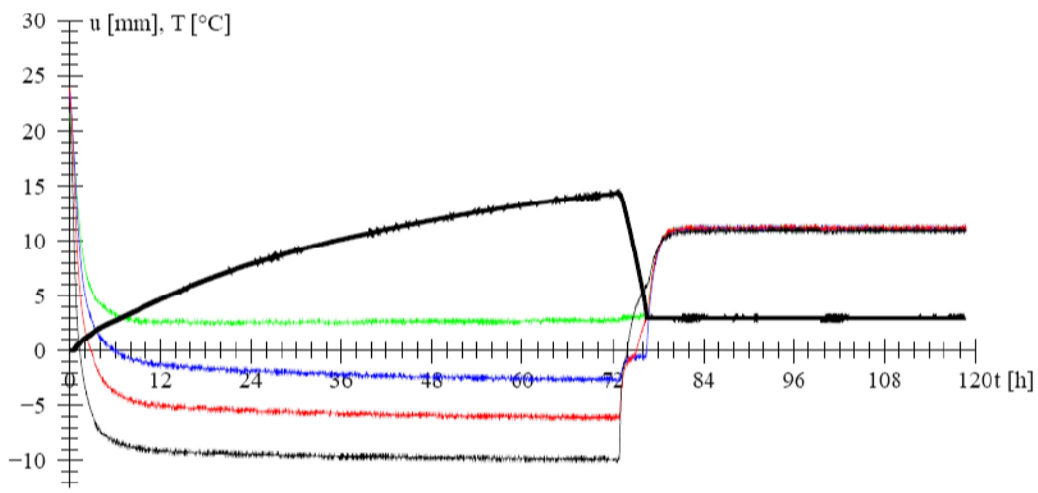


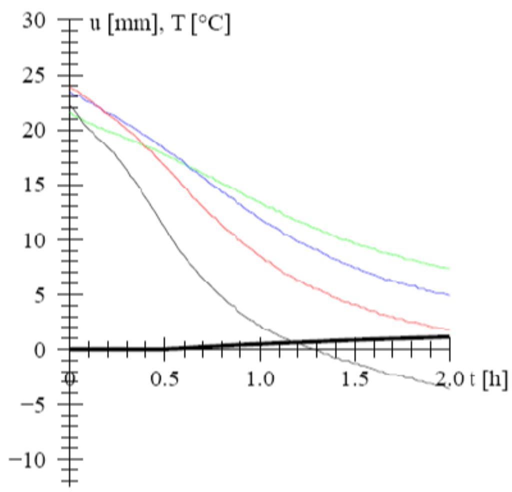


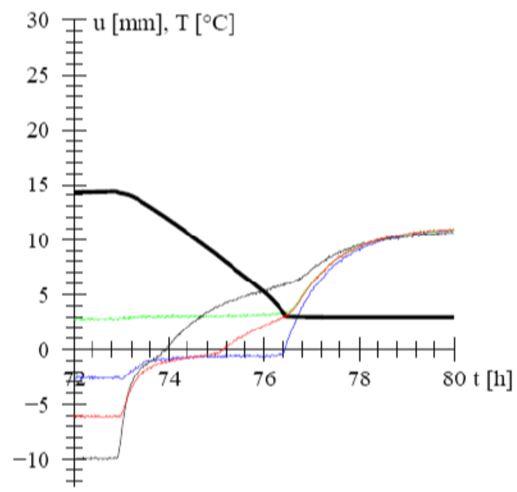


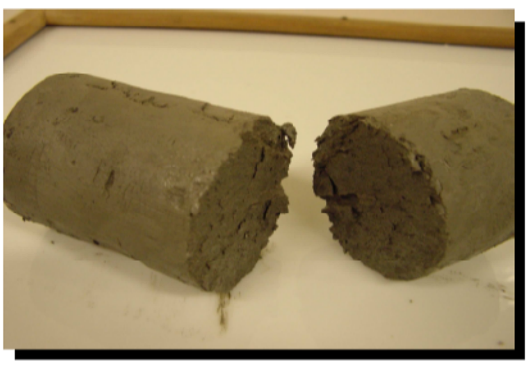
sample: 016 (silt 2) dimensions (H/D): 100/49.9 mm	water content:	
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.4 % (before)	
remark: ––	32.1 % (after – overall) 32.4 % (after – frozen) 31.2 % (after – unfrozen)	



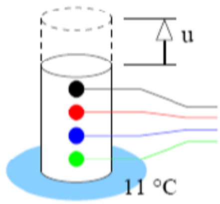


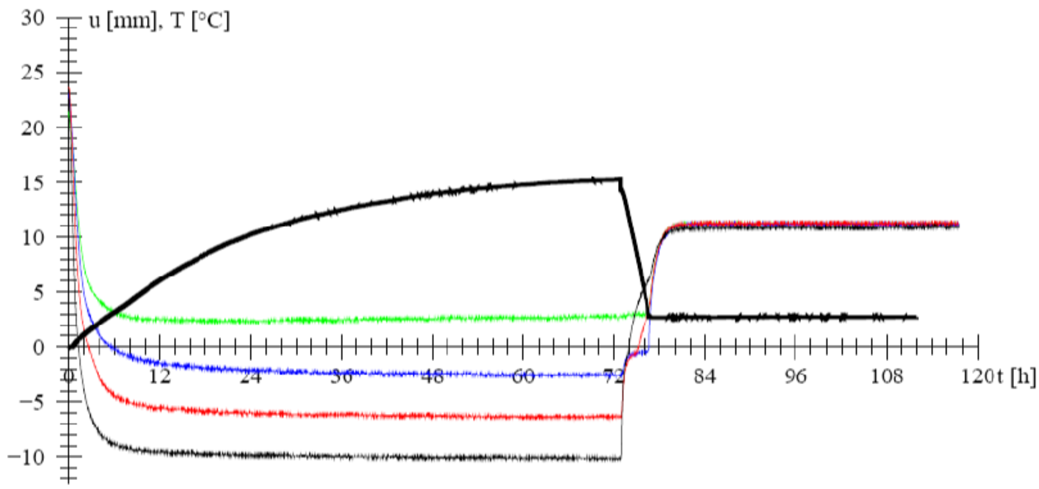


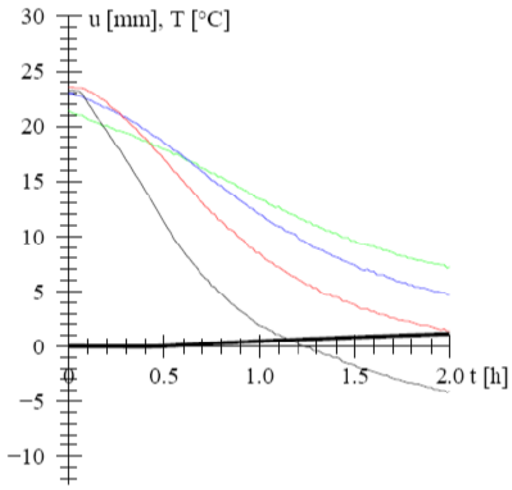


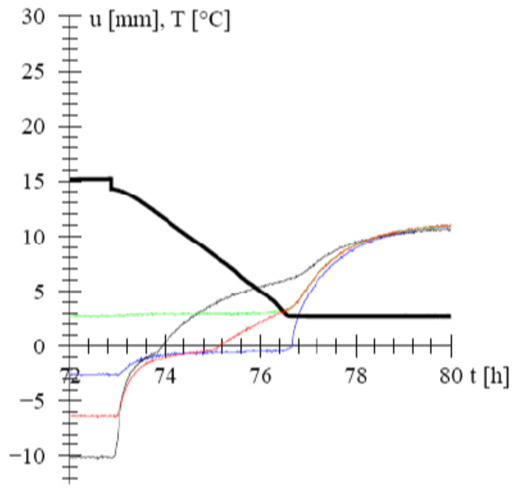


sample: 017 (silt 2) dimensions (H/D): 100/49.9 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	27.9 % (before) 33.6 % (after – overall) 33.9 % (after – frozen) 32.9 % (after – unfrozen)
remark: —	

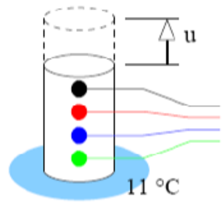


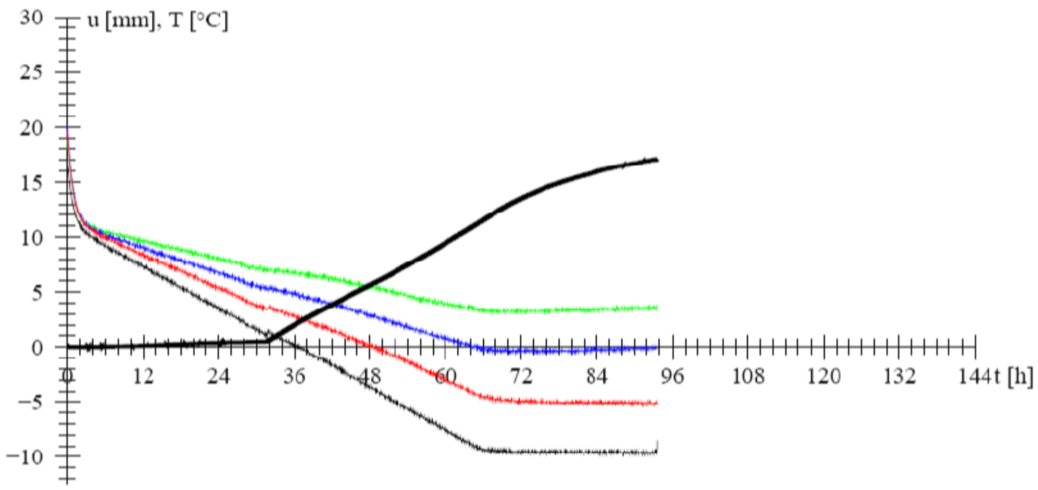


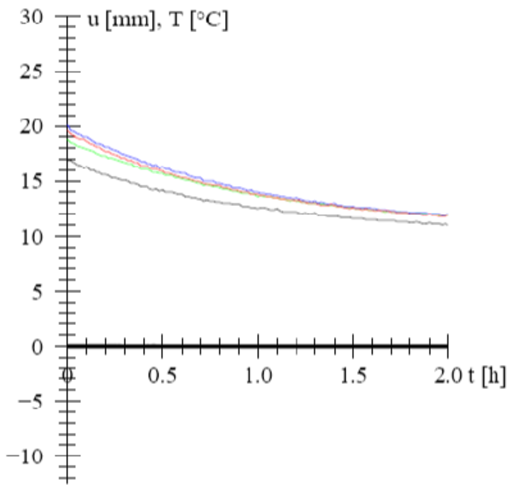


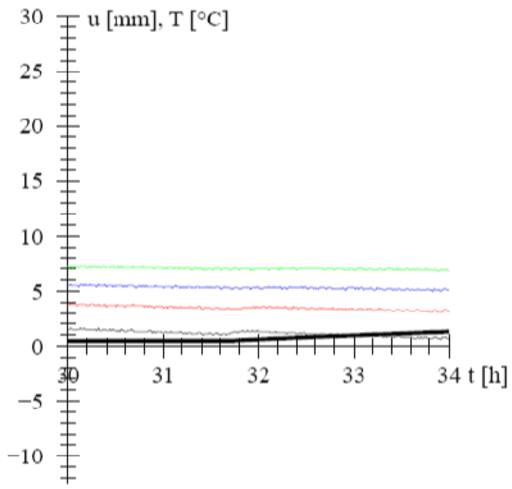


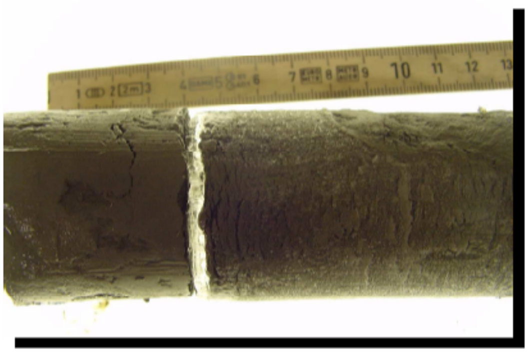
sample: 018 (silt 2) dimensions (H/D): 100/49.6 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	27.8 % (before)
remark: −−	37.6 % (after − overall)
	42.4 % (after − frozen)
	30.5 % (after − unfrozen)

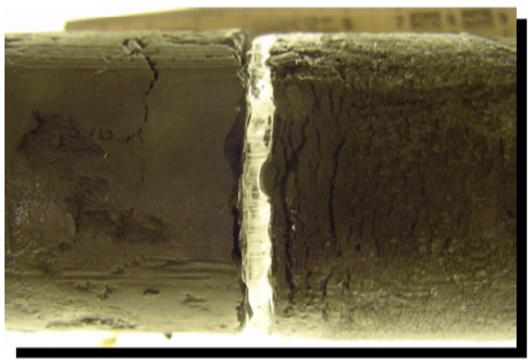




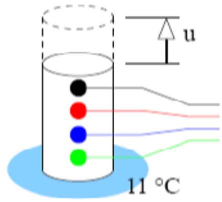


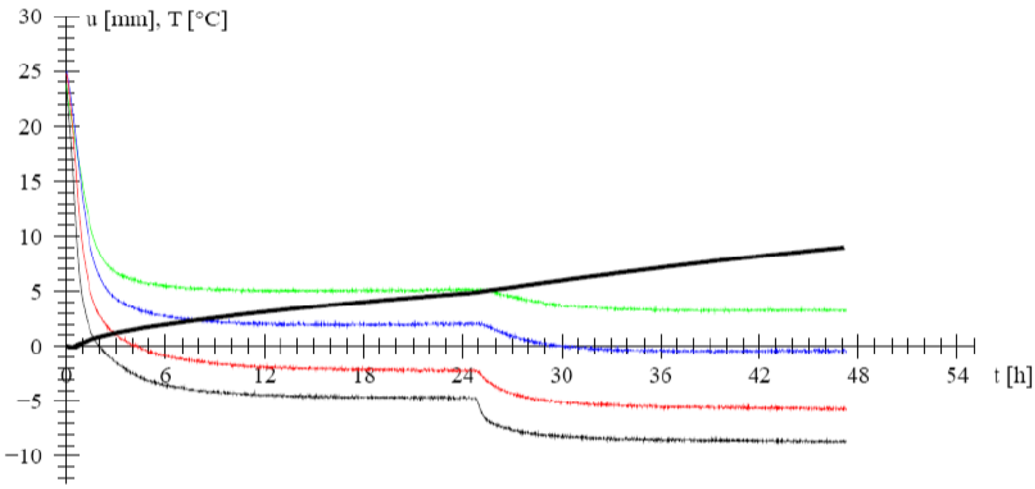


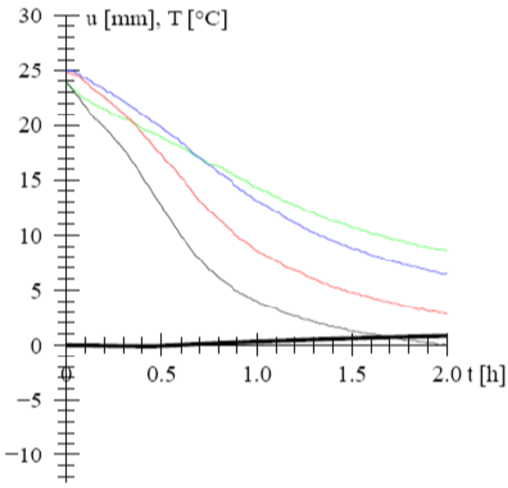


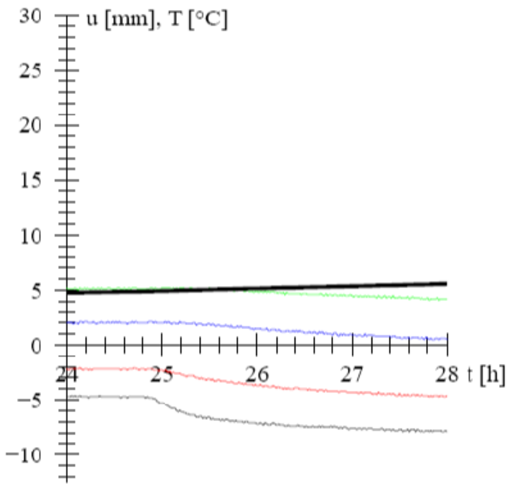


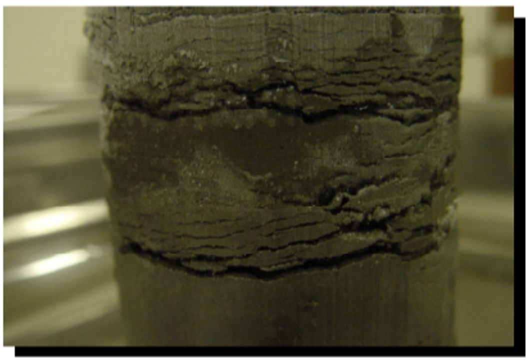
sample: 019 (silt 2) dimensions (H/D): 100/49.7 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.1 % (before)
remark: ––	30.3 % (after − overall)
	33.1 % (after − frozen)
	26.9 % (after − unfrozen)

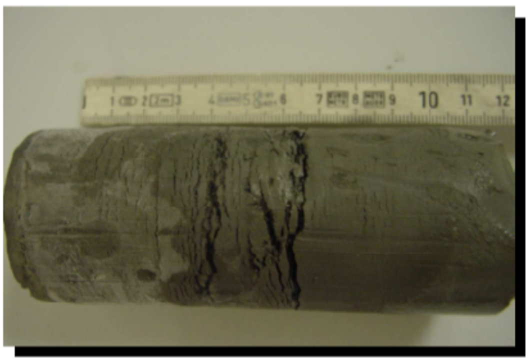




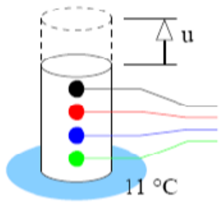


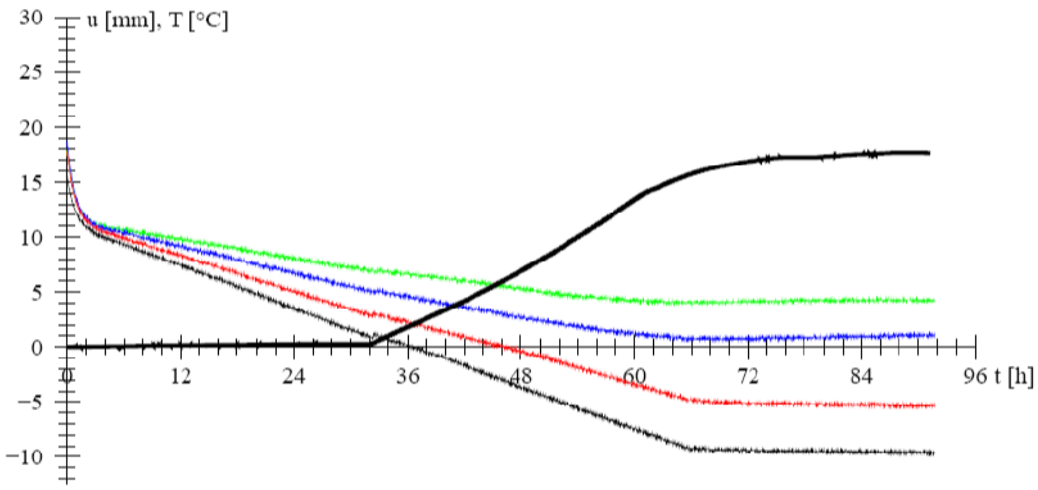


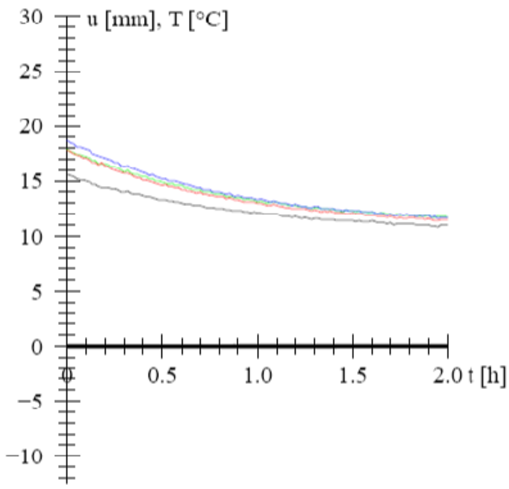


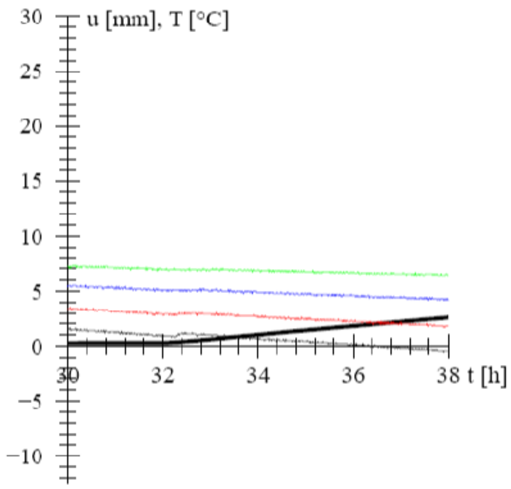


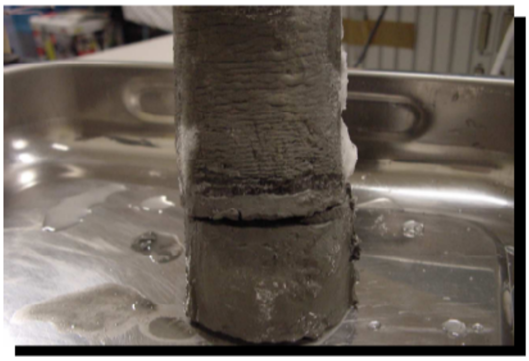
sample: 020 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	26.1 % (before)
remark: −−	40.3 % (after − overall) 45.9 % (after − frozen) 33.7 % (after − unfrozen)



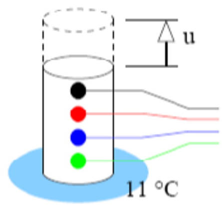


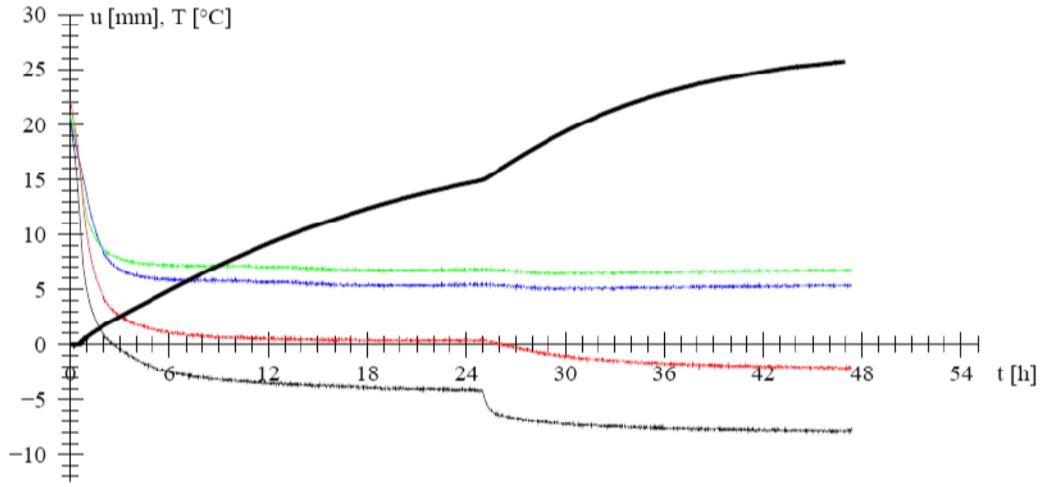


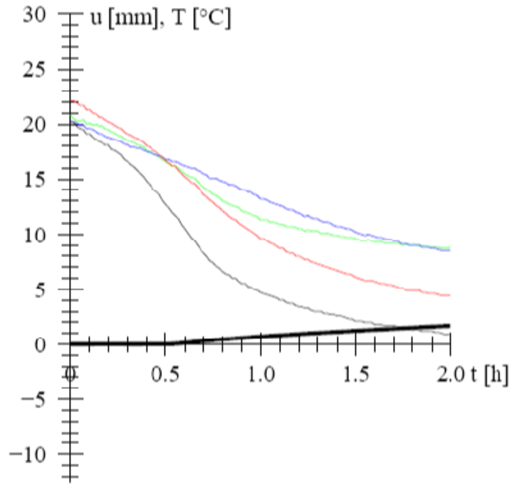


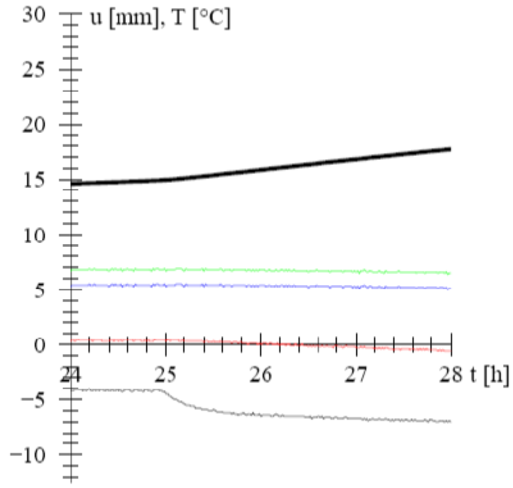


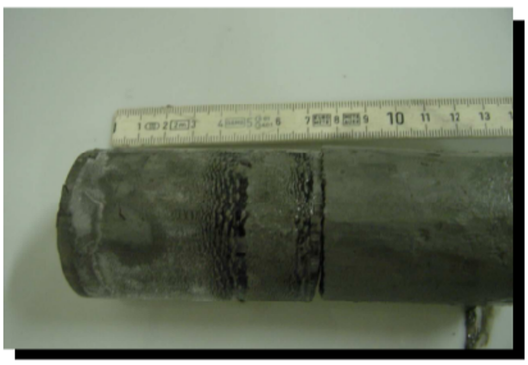
sample: 021 (silt 3) dimensions (H/D): 100/49.8 mm	water content:	
experiment: two−step freezing (cooling rate: 50.0 °C/h)	27.5 % (before)	
remark: −−	42.8 % (after − overall) 58.0 % (after − frozen) 30.2 % (after − unfrozen)	

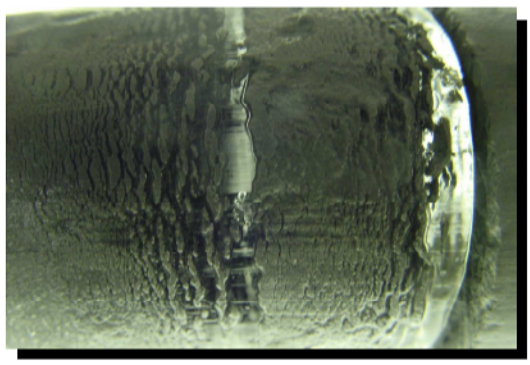




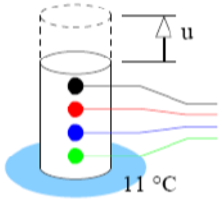


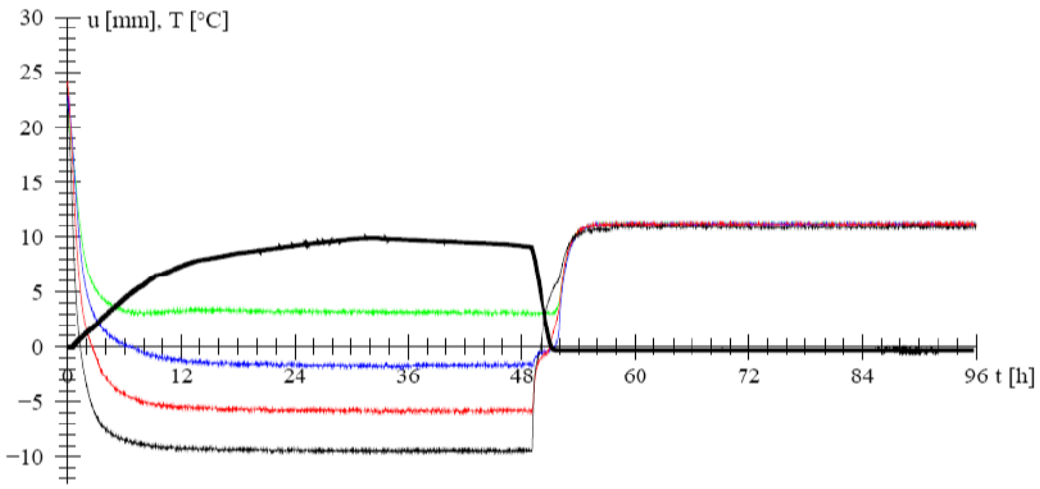


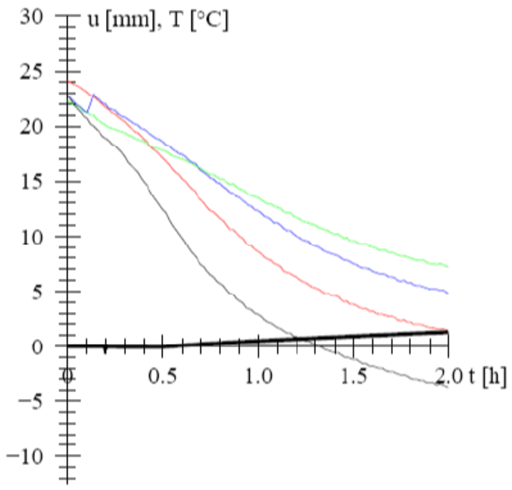


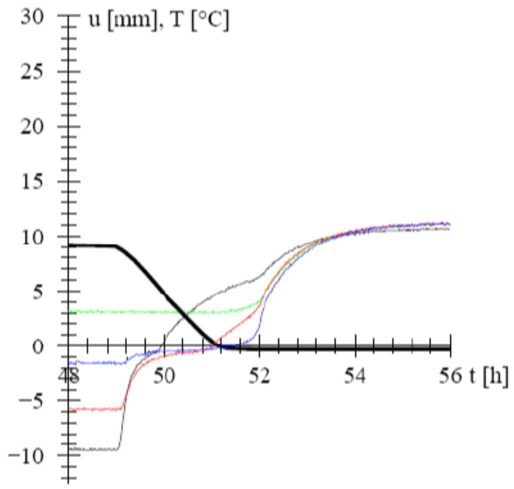


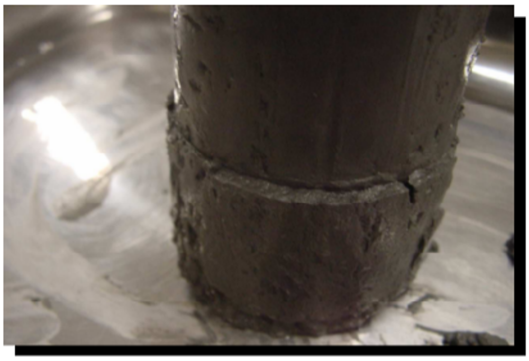
sample: 022 (silt 3) dimensions (H/D): 100/50 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	24.7 % (before)
remark: sample stressed with extra load of 1300g on the top	32.8 % (after – overall) 30.3 % (after – frozen) 36.6 % (after – unfrozen)



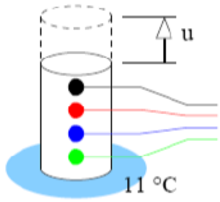


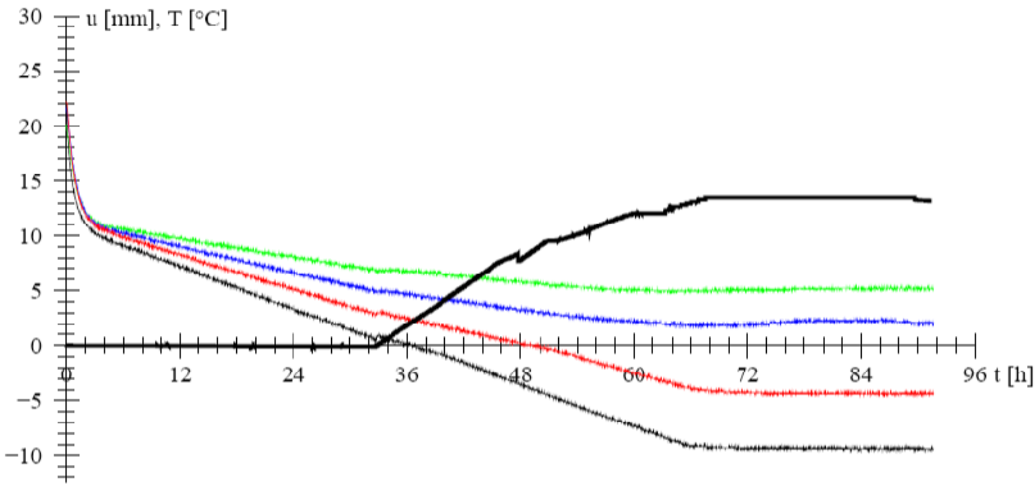


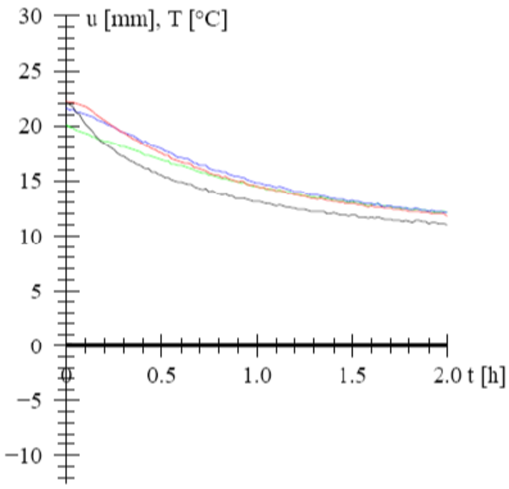


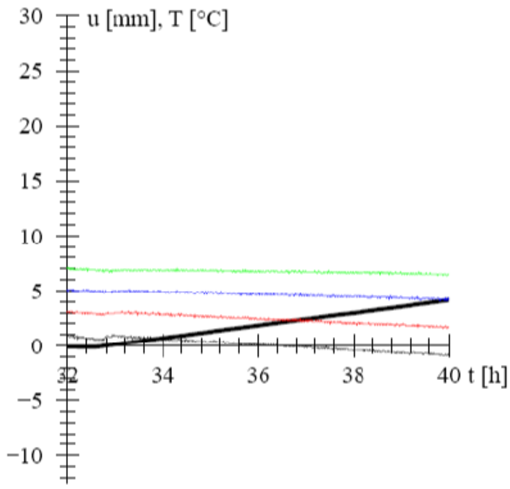


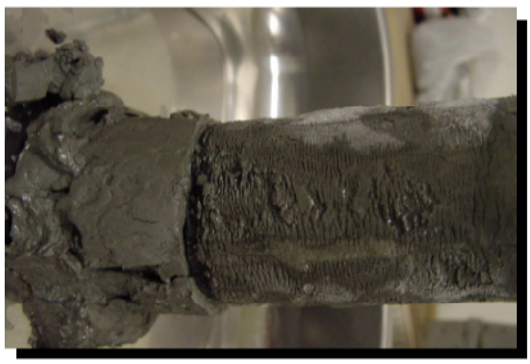
sample: 023 (silt 3) dimensions (H/D): 100/49.6 mm	water content:
experiment: one−step freezing (cooling rate: 0.4 °C/h)	24.6 % (before)
remark: sample stressed with extra load of 1300 g on the top	44.1 % (after − overall) 51.6 % (after − frozen) 36.9 % (after − unfrozen)



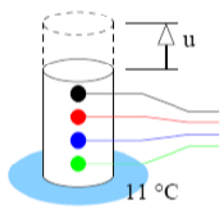


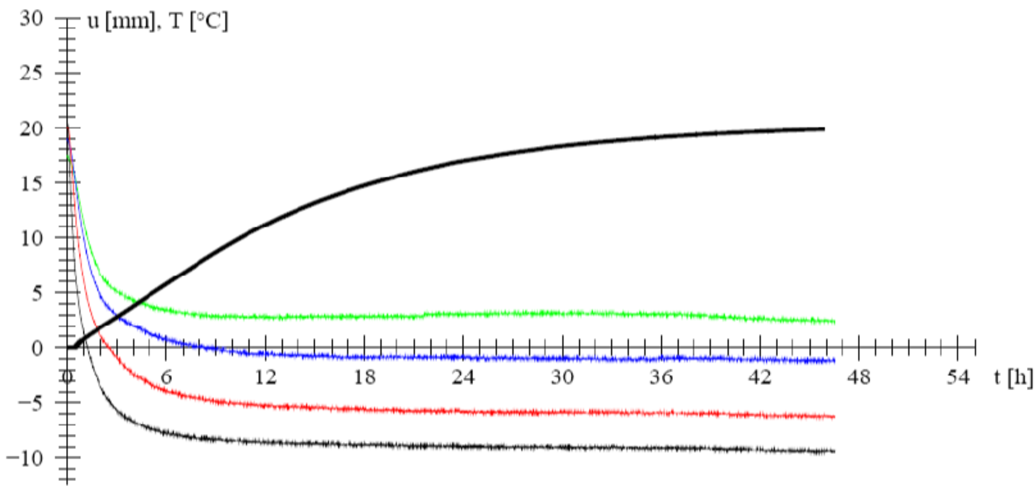


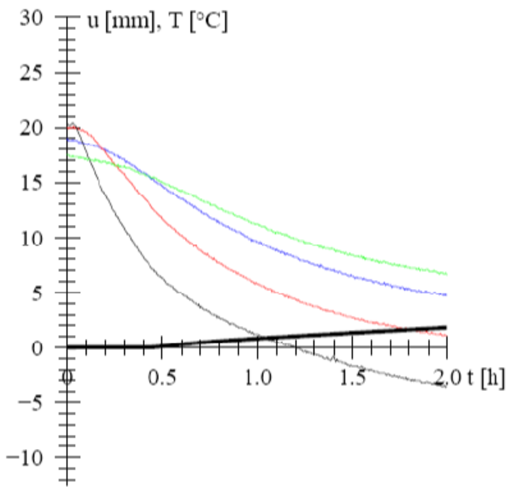


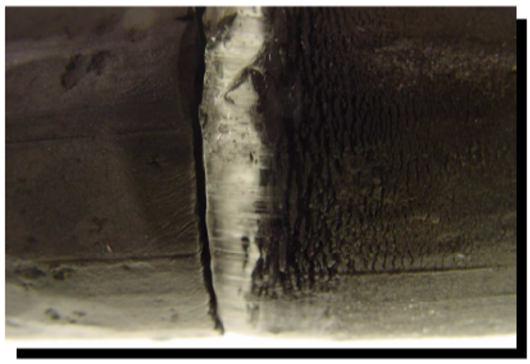


sample: 024 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	25.9 % (before)
remark: −	38.2 % (after − overall) 44.9 % (after − frozen) 29.8 % (after − unfrozen)

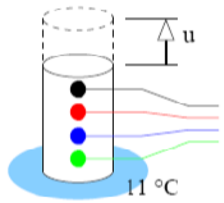


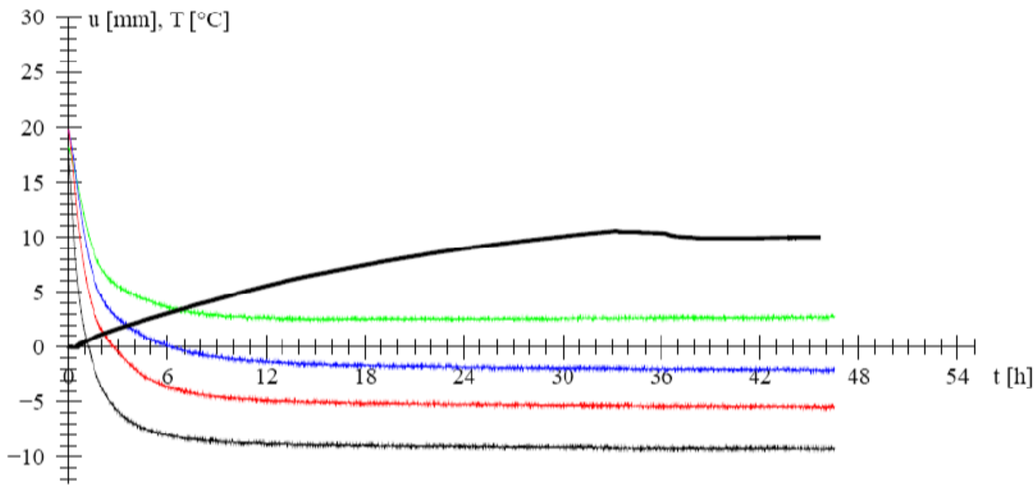


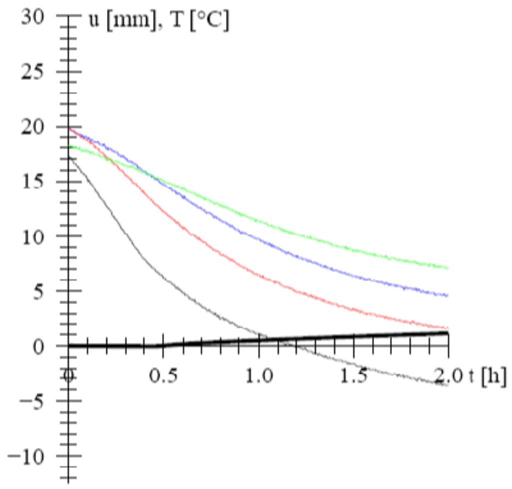


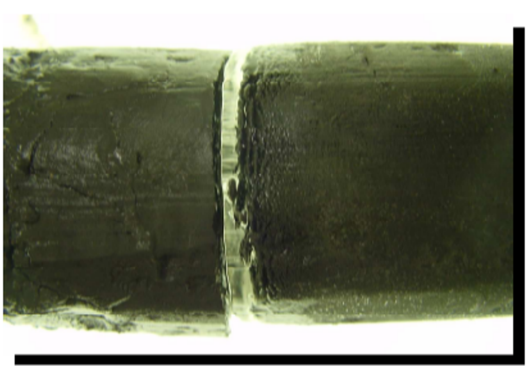


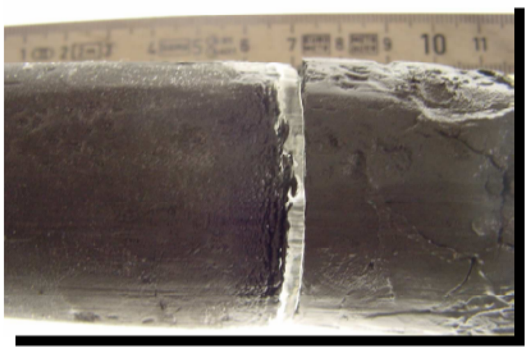
sample: 025 (silt 3) dimensions (H/D): 100/49.8 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	23.9 % (before) 30.6 % (after − overall) 30.9 % (after − frozen) 30.0 % (after − unfrozen)
remark: −−	



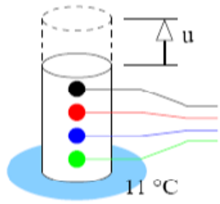


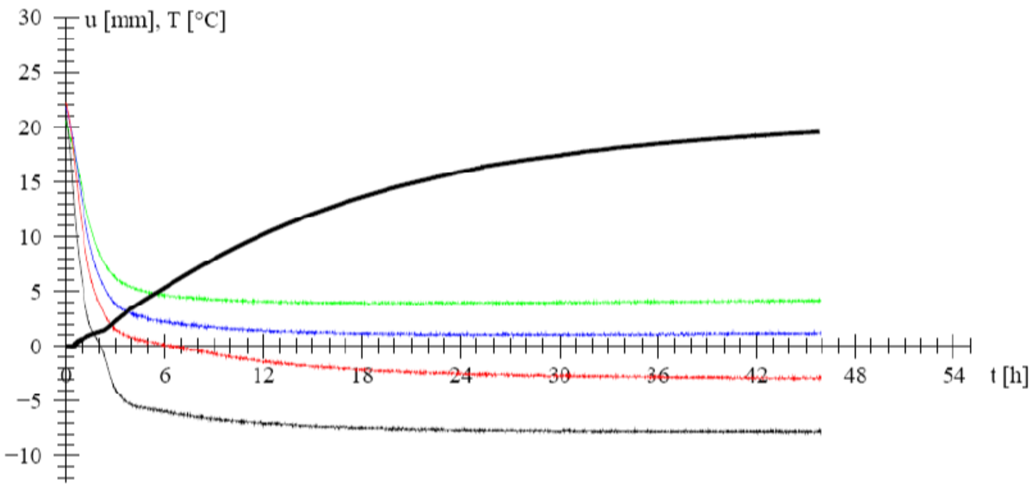


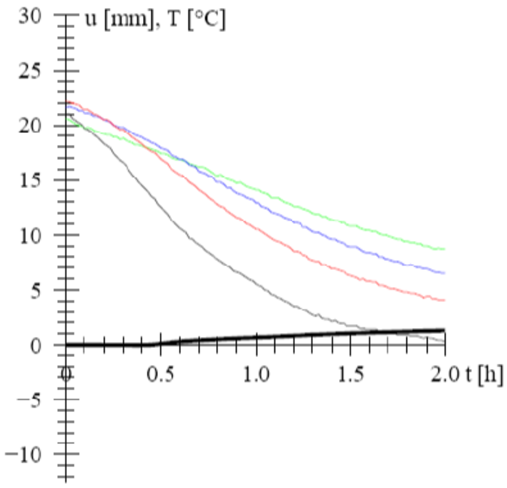


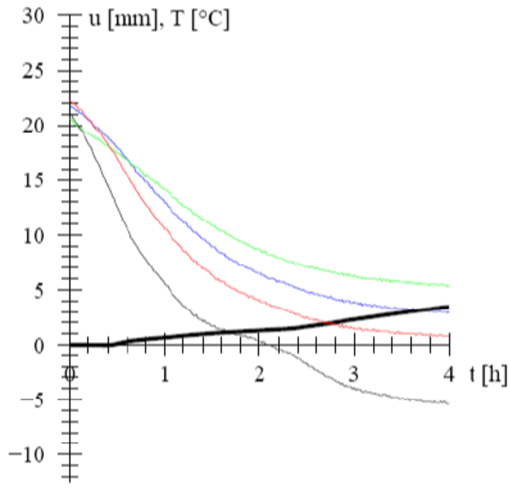


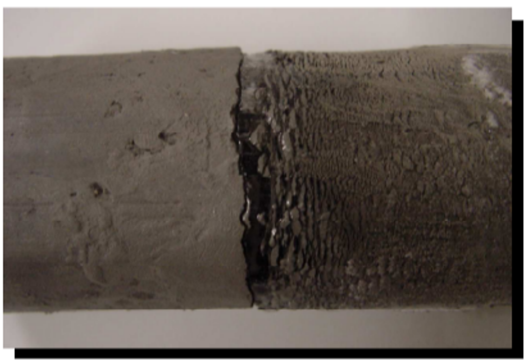
sample: 026 (silt 3) dimensions (H/D): 100/50.2 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	22.8 % (before)
remark: ––	37.2 % (after − overall)
	48.4 % (after − frozen)
	28.6 % (after − unfrozen)

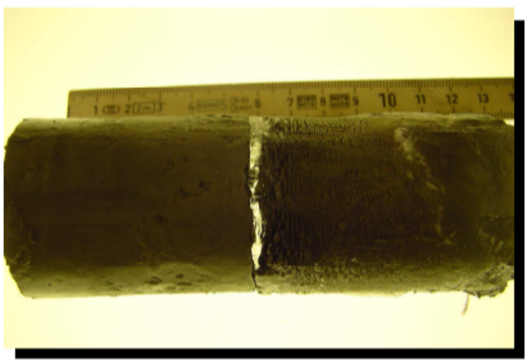




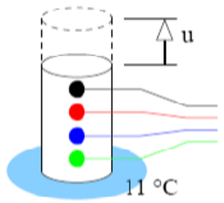


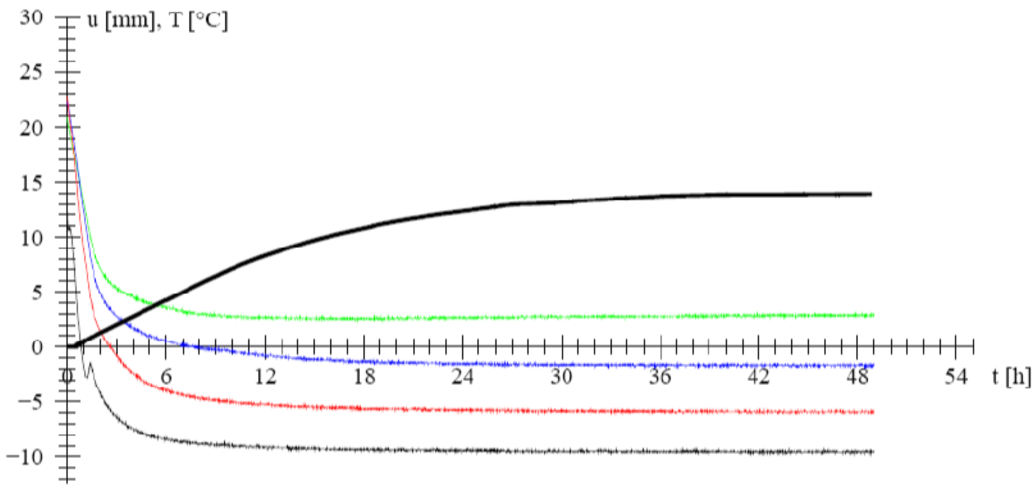


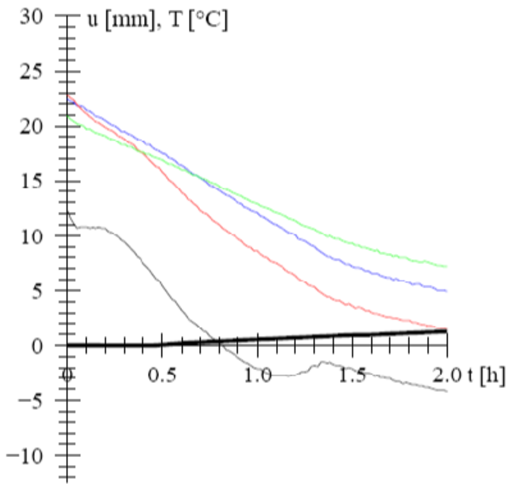


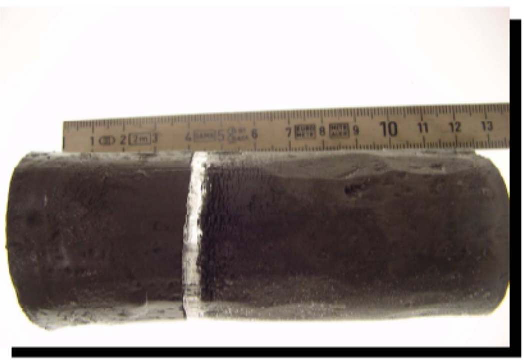


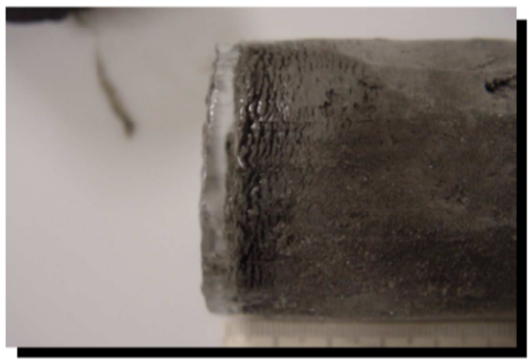
sample: 027 (silt 3) dimensions (H/D): 100/49.3 mm	water content:
experiment: one−step freezing (cooling rate: 50.0 °C/h)	24.6 % (before)
remark: −−	33.6 % (after − overall) 37.4 % (after − frozen) 28.1 % (after − unfrozen)



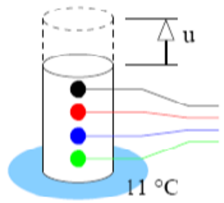


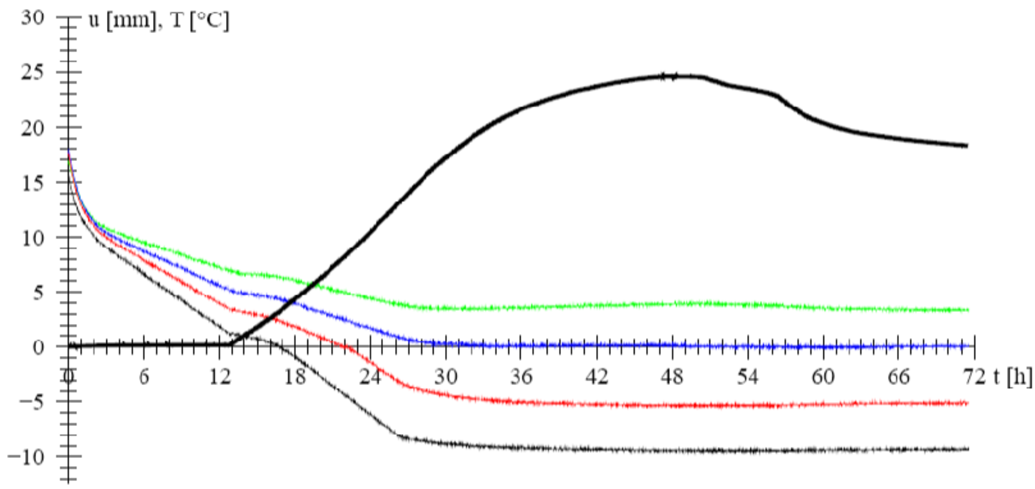


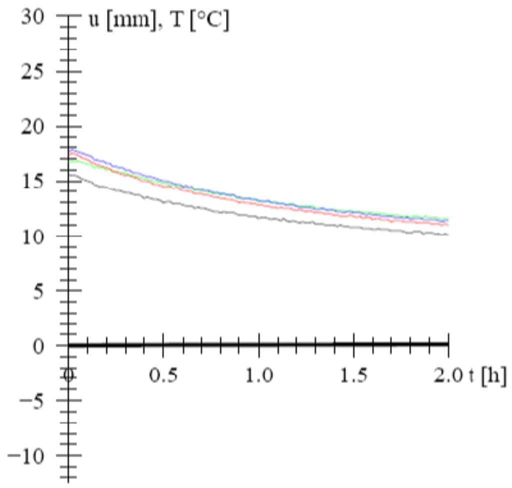


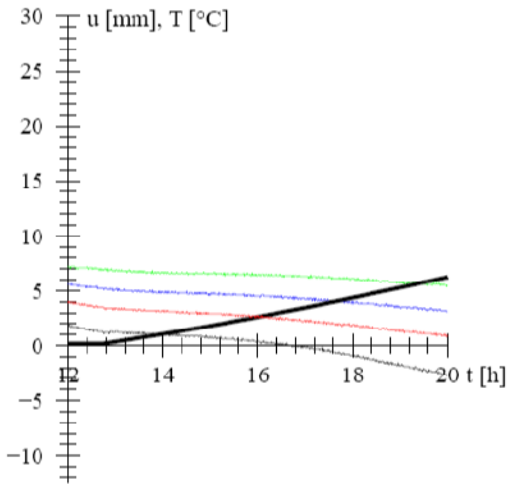


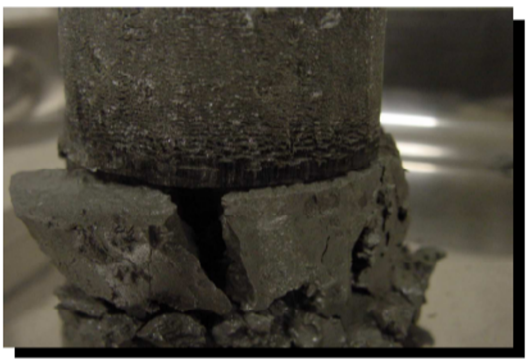
sample: 028 (silt 3) dimensions (H/D): 100/49.9 mm	water content:
experiment: one−step freezing (cooling rate: 1.0 °C/h)	25.5 % (before) 42.0 % (after − overall) 47.1 % (after − frozen) 35.7 % (after − unfrozen)
remark: −−	



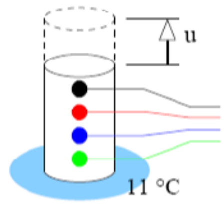


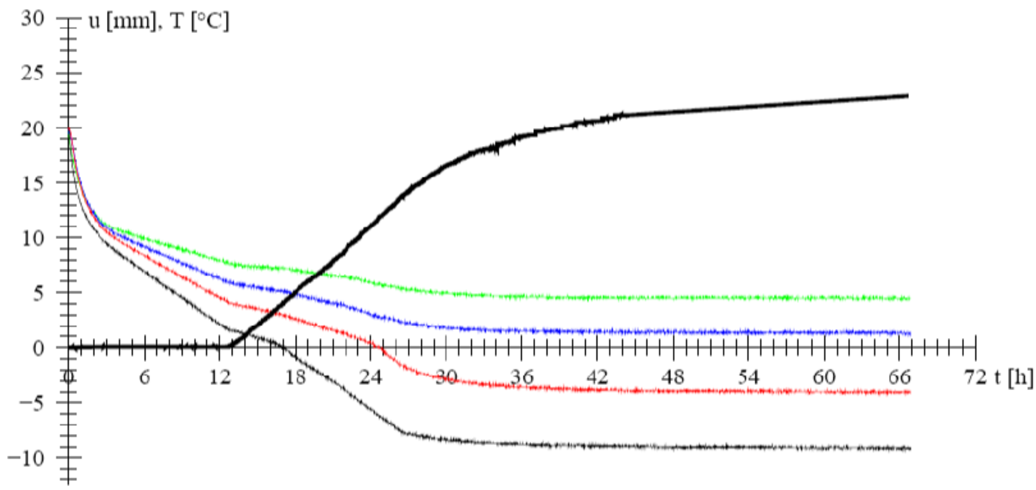


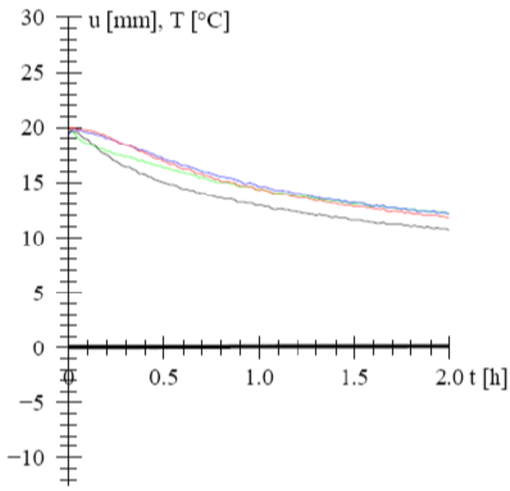


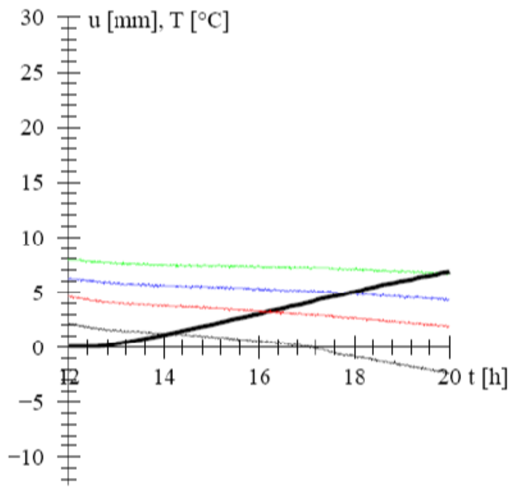


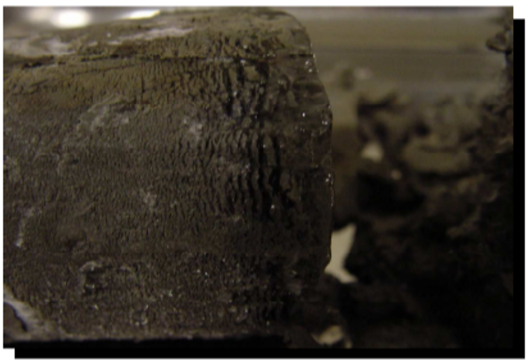
sample: 029 (silt 3) dimensions (H/D): 100/48.8 mm	water content:
experiment: one−step freezing (cooling rate: 1.0 °C/h)	26.7 % (before)
remark: ––	46.4 % (after − overall) 59.9 % (after − frozen) 36.6 % (after − unfrozen)

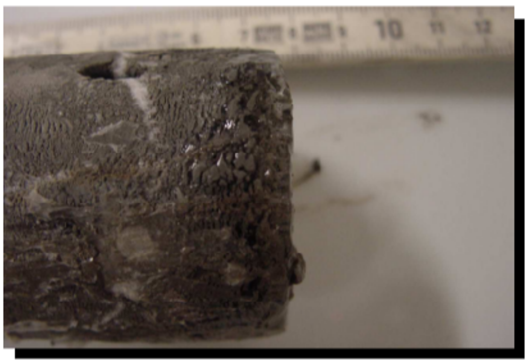




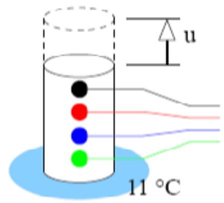


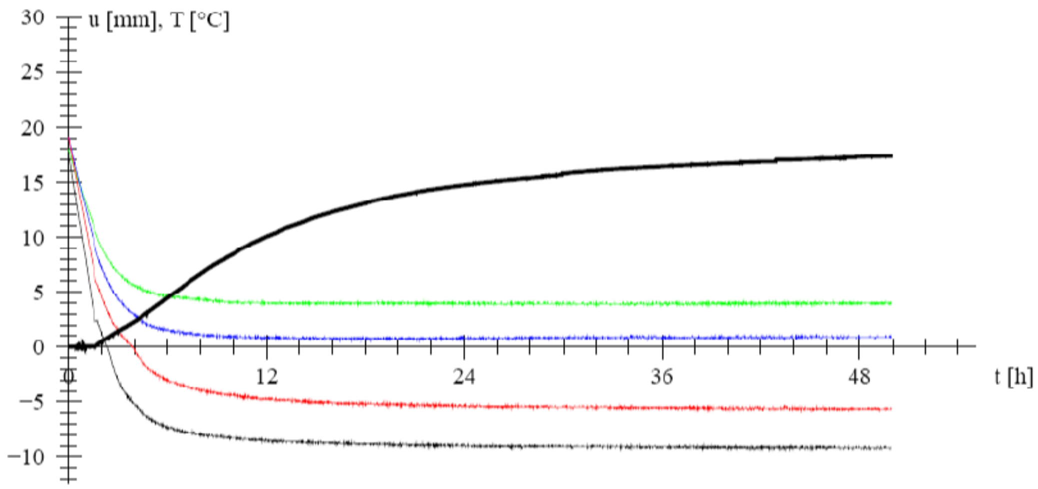


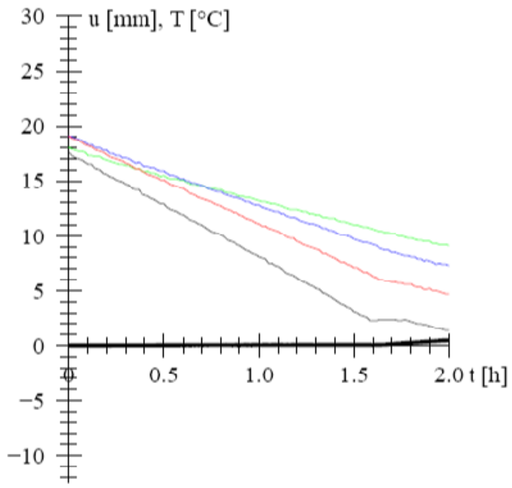


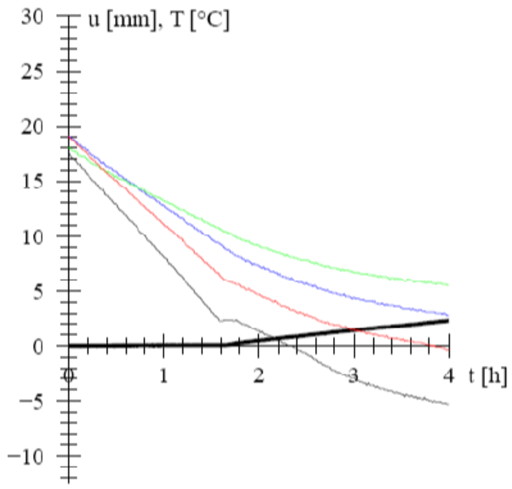


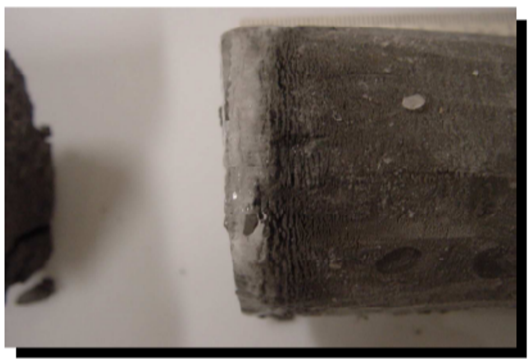
sample: 030 (silt 3) dimensions (H/D): 100/50 mm	water content:
experiment: one−step freezing (cooling rate: 25.0 °C/h)	25.8 % (before)
remark: ––	35.1 % (after − overall) 40.7 % (after − frozen) 28.8 % (after − unfrozen)



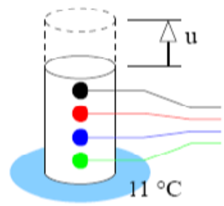


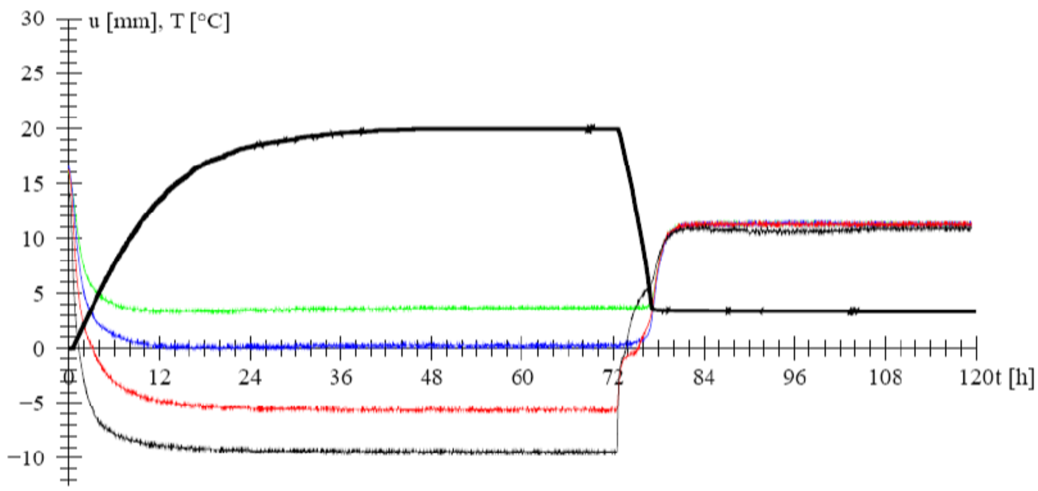


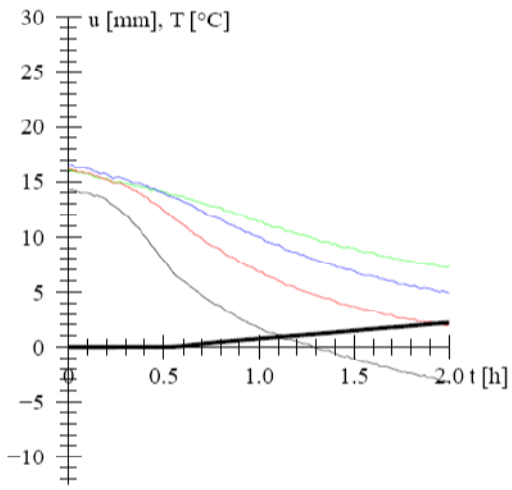


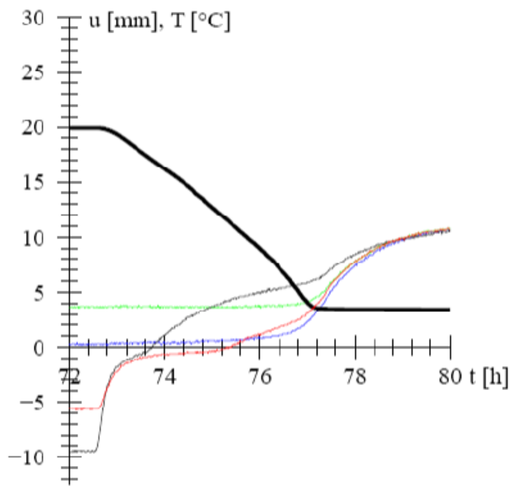


sample: 031 (silt 3) dimensions (H/D): 100/50.2 mm	water content:
experiment: freez/thaw (cooling rate: 50.0 °C/h)	26.8 % (before)
remark: ––	34.1 % (after – overall) 36.2 % (after – frozen) 31.2 % (after – unfrozen)

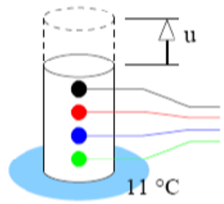


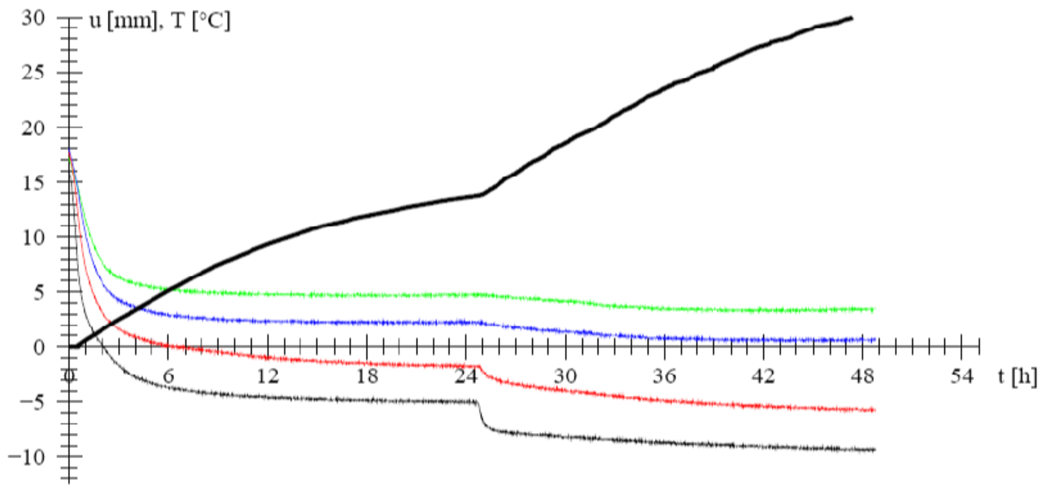


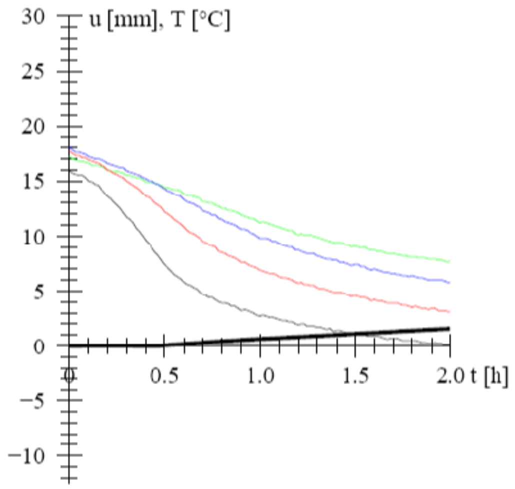


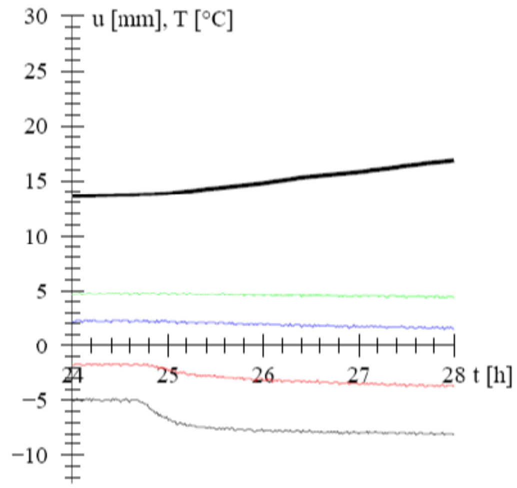


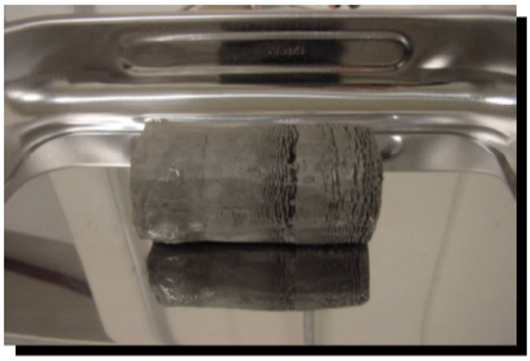
sample: 032 (silt 3) dimensions (H/D): 100/48.5 mm	water content:
experiment: two−step freezing (cooling rate: 50.0 °C/h)	25.1 % (before)
remark: —	42.6 % (after − overall)
	56.0 % (after − frozen)
	28.2 % (after − unfrozen)











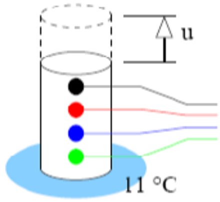
sample: 033 (silt 3) dimensions (H/D): 100/50 mm

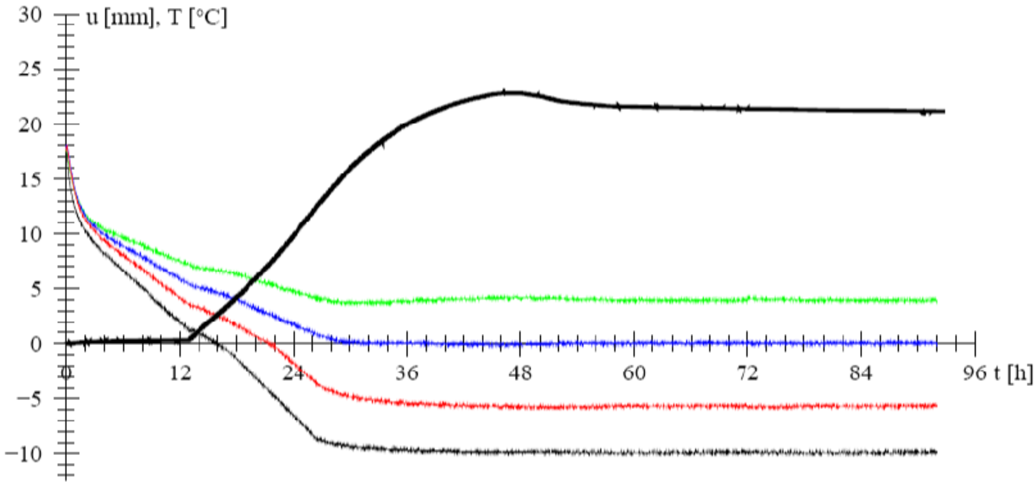
experiment: one−step freezing (cooling rate: 1.0 °C/h)

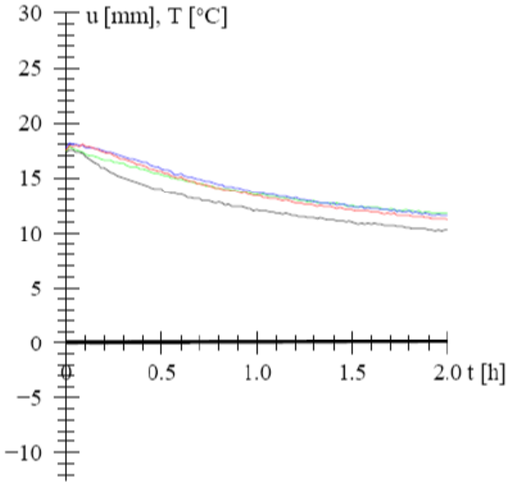
remark: −−

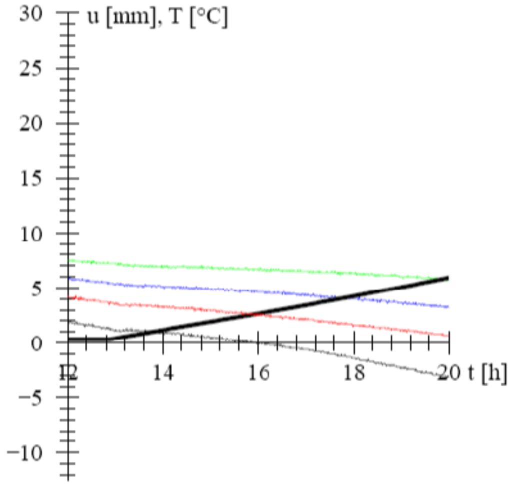
water content:

26.6 % (before)
42.3 % (after − overall)
47.3 % (after − frozen)
35.4 % (after − unfrozen)









ÖZGEÇMİŞ

Mustafa KOÇ 1979 yılında İzmir'de doğdu. İlk öğrenimini Altinordu İlköğretim Okulu'nda, liseyi Sekine Evren Anadolu Lisesi'nde tamamladı. 1998 yılında girdiği İstanbul Teknik Üniversitesi İnşaat Mühendisliği Bölümü'nden 2003 te mezun oldu. 2001 yılında başladığı Çift Anadal Programı dahilinde Jeoloji Muhendisliğinden 2004 yılında mezun oldu. 2003 yılında İstanbul Teknik Üniversitesi Fen Bilimleri Enstitüsü'ne bağlı Zemin Mekaniği ve Geoteknik Mühendisliği programında Yüksek Lisans öğrenimine başladı.

Mustafa KOÇ, 2004-2005 yılı Eylul –Temmuz ayları arasında Erasmus Socrates burslu öğrencisi olarak, TUWIEN (Technical University of Vienna), Avusturya bünyesinde çeşitli araştırma çalışmalarında bulunmuştur ve Yüksek Lisans tez çalışmasını adı geçen üniversitede yürütmüstür. 2005 yılı Ağustos ayında başladığı Geomar adlı şirket ile Türkiye ve Dubai'de Geoteknik Mühendisliği alanında çalışmıştır.