Tunnels and Underground Cities: Engineering and Innovation meet Archaeology, Architecture and Art – Peila, Viggiani & Celestino (Eds)

Design of artificial ground freezing for an access tunnel of a railway station in Switzerland

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ABSTRACT: The paper presents the static and thermal design of an artificial ground freezing (AGF) measure with brine for an access tunnel of a railway station in central Switzerland. The geology of the first 45 m of the access tunnel consist of sediments (silty sand to sandy silt) followed by sandstone. The tunnel crosses under the railway lines and train platforms of one of the train stations with the highest traffic. At any time during construction of the tunnel traffic of trains cannot stopped. Due to the sediments aforementioned and since the water table lies slightly above the crown of the tunnel, AGF is the only suitable auxiliary measure for the tunnel. For achieving an adequate frozen body above the crown water table must be raise up by watering the ground locally with drains. The paper presents also the dimensioning and optimization of these drains. The strength parameters were estimated from literature and verified with uniaxial tests with frozen soil samples under constant temperature. Due to project requirements the core of the tunnel is also frozen. Implementation of the AGF measures starts from a shaft. Due to spatial conditions given by the location of reinforcement of the diaphragm wall of the shaft and of a reinforcement beam the distribution of the freeze pipes was demanding. The goals of the thermal design were the generation of a frozen body with uniform thickness around the tunnel and to avoid generation of excess pore pressure in the core of the tunnel during freezing.

1 INTRODUCTION AND SITE CONDITIONS

In order to increase the capacity of a train station with the highest traffic in Switzerland two caverns will be constructed beneath the existing station. The access tunnel to the west side of these caverns is located under the railway lines and train platforms and will be excavated conventionally from a shaft located outside the area of rail traffic (Figure 1).

The ground in the area of the shaft consists of sedimentary deposits. In the excavation direction (towards the caverns) a moraine layer followed by sandstones appears with an inclination of about 15°–20°. The cross section of the access tunnel is located completely in the sedimentary layers in the first 26 m (starting from the shaft) and completely in sandstone after approximately 40 m (Figure 2).

The sedimentary layers are heterogeneously distributed. According to the borehole profiles of exploratory drillings close to the shaft, silty sand and sandy silt layers are expected. In the excavation direction, the layers become coarser, changing to coarse sand and gravely sand. The ground is water bearing.

As the available geological-geotechnical data were incomplete in the design stage of the project, design was based upon working hypotheses, which were afterwards verified by field and laboratory investigations.

The design presented in this paper is the basis for the tender. Construction of this part of the project will start in spring 2019.

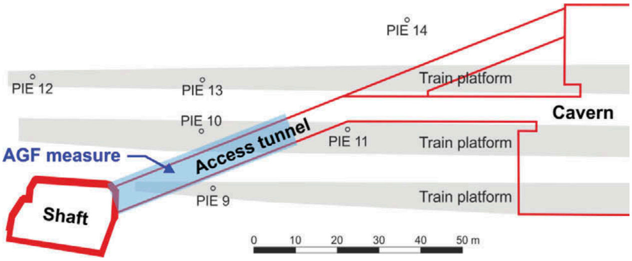

Figure 1. Top view of the access tunnel.

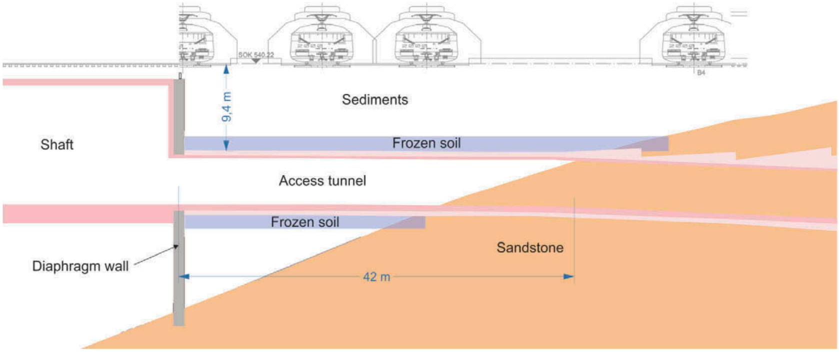

Figure 2. Longitudinal profile of the access tunnel.

2 SPECIAL PROJECT REQUIREMENTS

2.1 *General aspects*

Train traffic and station operation must not be interrupted at any time during construction of the access tunnel; therefore, excavation methods like cut and cover or top-down construction were discarded. Due to the presence of water up to the crown level of the access tunnel jet grouting can also not be applied. Grouting or drainage were also excluded due to environmental aspects. The quality of the groundwater must be preserved and a draw-down of the water table was also not an option. Therefore, artificial ground freezing (AGF) as auxiliary measure was selected in order to impermeabilize, stiffen and strengthen the ground around the tunnel and thus mitigate the potential hazards of water inrush or inadmissible settlements of the railway lines.

2.2 *Water table*

Due to the uncertainty concerning the elevation of the water table and a potentially significant seepage flow, piezometers were installed close to the project area (PIE9 to PIE14 in Figure 1). According to the measurements, the water table varies seasonally and is in average about 0.6 m above the crown of the tunnel. For the application of AGF, it is mandatory to have a degree of saturation of minimum 50–70% (Andersland & Ladanyi, 2004). Thus, the water table should be raised up artificially during the freezing stage (Section 4). According to the measurements, there is no evidence of seepage flow in the area of the planned access tunnel.

2.3 *Tunnel face stability*

Ground reinforcement by bolts was not considered adequate for stabilizing the face under the expected conditions (sandy silt to silty sand). Thus, great part of the face should also be frozen for ensuring its stability.

2.4 *Location of the freeze pipes*

In the AGF method, pipes are installed in the ground, and a liquid refrigerant circulates through the pipes and extracts heat from the surrounding soil (Figure 3). After a certain time period, a frozen soil body develops. The freeze tubes used in the AGF method consist of two concentric pipes (Harris, 1995). The outer pipe is closed at the freeze end; the inner one is open.

Implementation of the AGF measures starts from a shaft. The shaft is supported by a reinforced diaphragm wall and two horizontal beams. The lower beam is located beneath the crown of the access tunnel and the upper one approximately at the highest point of the planned frozen body (Figure 4). The horizontal beams cannot be perforated for installing freeze pipes. The reinforcement of the diaphragm wall was distributed in such a way that its perforation is only possible along specified vertical lines (turquoise dot-dashed lines in Figure 4). Thus the boreholes for the freeze pipes can be located only along these vertical lines and, since the boreholes must be drilled using a preventer in order to avoid drainage of the ground at the crown area of the tunnel, not closer than 30 cm to the horizontal beams.

3 DESIGN OF THE AGF MEASURE

3.1 *Static design*

The following hazard scenarios were considered for the static design of the frozen body:

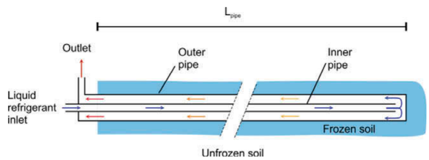

Figure 3. Freeze pipe.

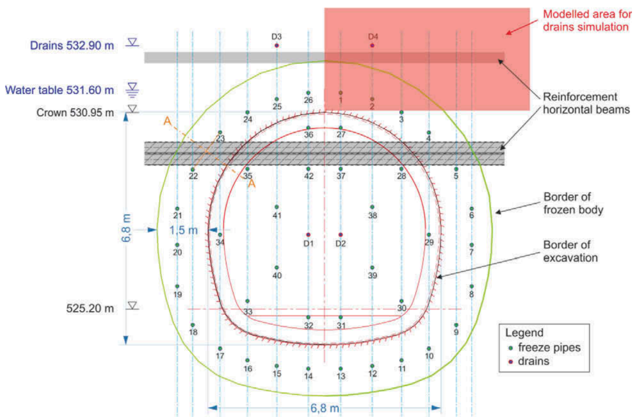

Figure 4. Location of the freeze pipes.

- large deformations due to creep of the frozen body;
- overstressing of the frozen body due to high shear stresses; and
- overstressing of the frozen body due to high normal stresses and bending moments.

The first hazard scenario can be excluded if the stresses in the frozen body are low and the loading period (*i.e.* the time between excavation and development of the temporary support resistance) short. In the present case the advance per round is one meter and temporary support consists of steel arches and 40 cm reinforced shotcrete that is applied immediately after each excavation round.

The other two hazard scenarios were evaluated by means of numerical stress analyses of a plain strain model, considering the frozen body as an elastic material and checking the elastically determined stresses. The strength of frozen soil depends strongly on temperature and is very low for temperatures higher than -2 ºC. Therefore, only the region of the frozen body with temperature lower than -2 ºC was considered as bearing in the numerical analyses (the thickness of the frozen body was taken equal to 1.25 m instead of the actual 1.50 m). For the sake of simplicity, a homogeneous frozen body was considered, with parameters corresponding to the mean temperature of -10 ºC. Based on experience of projects in similar ground, conservative parameters for the frozen body were selected (uniaxial compressive strength of 2.3 MPa and shear strength of 1 MPa). The results of the stress analysis indicate that:

- the normal stresses are lower than 50% of the uniaxial compressive strength (global safety factor ≥ 2);
- tensile stresses do not develop; and
- shear stresses are lower than 16% of the shear strength (global safety factor ≥ 6).

The assumed strength parameters were verified later by means of laboratory tests on remoulded samples obtained in the area close to the shaft. The samples were compacted to the same density as in-situ, saturated and frozen to temperatures of approximately -10.5 ºC. The results of the first two samples show uniaxial strength higher than 5.3 MPa (Figure 5). Although these values are twice as high as the values that were adopted in the stress analysis, the thickness of the frozen body was not reduced.

3.2 *Thermal design*

3.2.1 *AGF method and goals*

Taking the costs into consideration and the absence of relevant seepage flow, AGF with brine instead of liquid nitrogen was chosen. Thermal design was based upon typical parameters for this method (freeze pipe temperature of -30 ºC and freeze pipe diameter of 4 inches). The thermal design aims to determine: the number and distribution of the freeze pipes; the required refrigeration time for the growing stage; and the necessary cooling power. In AGF

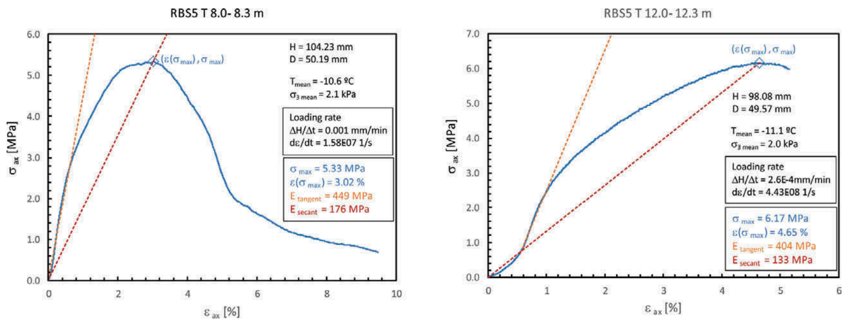

Figure 5. Results of uniaxial tests on frozen samples.

the growing stage denotes the refrigeration stage where the frozen body increases its dimensions continuously. After having achieved the planned frozen body size, excavation of the tunnel starts and refrigeration of the pipes outside of the tunnel continues but with reduced cooling power just in order to maintain the thickness of the frozen body (holding phase). The goals of the AGF measure are, as mentioned above, achieving a thickness of the frozen body around the tunnel of at least 1.5 m and freezing a great part of the face within a reasonable refrigeration time. An additional goal of the thermal design is to find a freeze pipe distribution that allows dissipation of excess pore pressures especially during freezing of the face area. Specifically, due to the volume expansion of frozen pore water, excess pore pressures develop, which reduce the resistance of the ground to shearing. Therefore, the design must allow dissipation of the excess pore pressures. For this purpose two drains are installed in the centre of the access tunnel. In order to avoid drainage of the tunnel area to be frozen these two drains are connected to vertical tubes with a spillover at the level of the drains for watering (Section 4). Prior excavation the vertical tubes are removed so that drainage is allowed. Thermal design was carried-out using the 3D numerical code FREEZE (Sres, 2009), which was developed at the ETH Zurich for simulating coupled thermohydraulic processes and verified against the results of large scale model tests (Pimentel *et al.* 2007). As mentioned above there is no evidence of seepage flow in the area of the project and thus only thermal simulations were performed.

3.2.2 *Thermal parameters*

In order to estimate more realistically the thermal parameters, the mineralogical composition of the remoulded samples (Section 3.1) was determined with X-ray diffraction. The average values of the mineral contents from the three samples are listed in Table 1. The samples consist mostly of quartz and calcite, which are minerals with high thermal conductivity and thus advantageous for the application of AGF.

The unfrozen water content for temperatures below the freezing point (T_0 = 0 ºC) was determined after Anderson & Tice (1972) and the corresponding parameters were taken from literature for a silty sand to sandy silt (α = 0.016 und β = -0.608; Frivik, 1981) (Figure 6 left). The soil to be frozen is considered as saturated. The heat capacity of the soil is determined as the arithmetic mean of the heat capacity of its constituents, weighted by their fraction. Analogously the thermal conductivity of the soil is calculated as the geometric mean of the thermal conductivity of its constituents weighted by their fraction. The parameters for the mineral fraction are evaluated in the same way, *i.e.* as arithmetic or geometric mean of the different mineral constituents weighted by their fraction. The temperature dependency of the thermal parameters of the mineral fraction is neglected, what is not the case for water and ice. In Figure 6 the heat capacity and thermal conductivity of the soil over the temperature are plotted.

3.2.3 *Required refrigeration time for the growing stage*

The optimum distribution of the freeze pipes was determined iteratively by means of successive simulations. The optimization goal was to find the minimum number of freeze pipes which allow: (a) achieving an approximately uniform frozen body with a thickness of 1.5 m; (b) freezing of the ground inside the tunnel cross-section; and, (c) rapid dissipation of the excess pore pressures that develop due to volume expansion of frozen pore water during the freezing stage. These simulations were performed taking additionally into consideration the required cooling power (Section 3.2.5).

Figure 7 shows the temperature field after 9.33, 11, 13, 15, 17 and 19 days of refrigeration for the optimized freeze pipe distribution. The isotherms for T = 0 ºC (border of the frozen

Table 1. Mineralogical composition.

Mineral	quartz	calcite	plagioclase	K-feldspar	muscovite	dolomite	chlorite
Content [%]	42.0	33.27	9.83	4.40	3.57	3.20	2.73

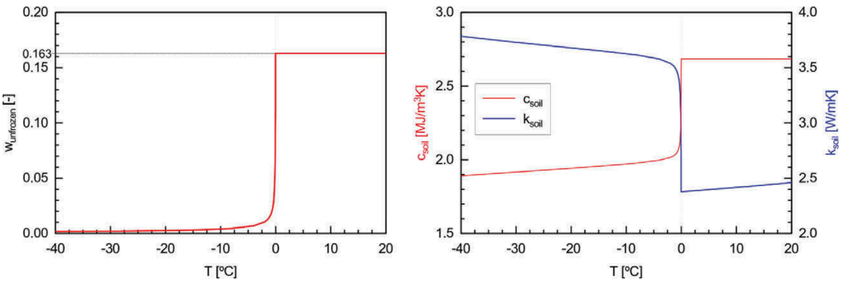

Figure 6. Thermal parameters of the soil (l.h.s.: unfrozen water content; r.h.s.: specific heat capacity and thermal conductivity).

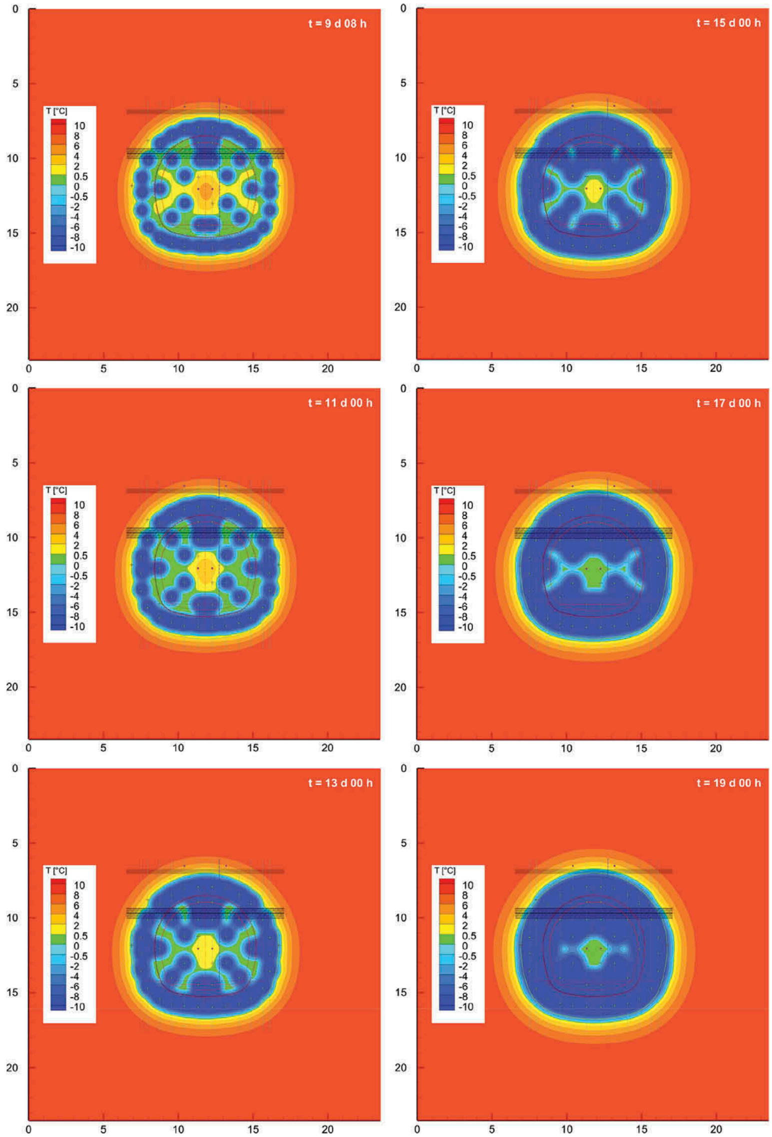

Figure 7. Simulation results after 9.33, 11, 13, 15, 17 and 19 days refrigeration.

body), -2 ºC and -10 ºC are marked. The green and red lines represent in all diagrams of Figure 7 the border of the planned frozen body and the border of the tunnel excavation respectively. The freeze pipes and the two drains in the area to be excavated are plotted as filled green and blue circles.

Closure of the frozen body is achieved after 9.33 days refrigeration. After further 4 days, *i.e.* 13 days of refrigeration in total, the frozen body has reached the planned thickness of 1.5 m except at the area around the reinforcement beam; radially draining paths are formed and remain open during two further refrigeration days. After 17 days refrigeration, the frozen body has reached the planned thickness everywhere. Finally, after 19 days refrigeration, only a small area (about 2 x 2 m) of the tunnel cross section remains unfrozen and its temperature is not higher than 0.5 ºC. Since this small area of the face is considered to remain stable due to the apparent cohesion of the ground, the growing stage can now be finished, taking 19 days in total. In order to evaluate the effective thickness of the frozen body (region with $T \leq$ -2 ºC; *cf.* Section 3.1) and its mean temperature, the temperature distribution along the most critical line crossing the frozen body (line A-A in Figure 4) is evaluated for different refrigeration times. The results are plotted in Figure 8, whereby the origin of the x-axis is located at the intersection of line A-A with the line connecting the adjacent freeze pipes nr. 22 and nr. 23 (Figure 4). The area marked blue represents the extension of the planned and effective frozen body. The temperature at the outer and inner border of the frozen body after 19 days refrigeration amounts to -3.2 ºC and -19.4 ºC, respectively, and is thus lower than -2 ºC. The average temperature is equal to -14.3 ºC, which is lower than the -10 ºC assumed in the static design.

3.2.4 *Influence of borehole deviations on the refrigeration time*

All boreholes must be drilled by means of horizontal directional drilling. This technique limits, but does not eliminate the deviations completely. The prescribed tolerance is 20 cm at any point of the boreholes. In order to quantify the influence of possible borehole deviations on the necessary refrigeration time, the most critical freeze pipes (nr. 22 and nr. 23 in Figure 4) were moved vertically by 20 cm in the opposite direction. These pipes (as well as pipes nr. 4 and 5 due to symmetry) are critical because they are located close to the horizontal beams, where drilling restrictions (Section 2.4) lead to a relatively big spacing of 1.35 m (nominal position). It is assumed that this spacing increases to 1.76 m due to drilling inaccuracy. The numerical simulation was performed for the same time-development of the freeze pipe temperature as in Section 3.2.5). The frozen body reaches the planned size (green line in Figure 9) after 19.5 days refrigeration (Figure 9 shows the temperature field). Thus the refrigeration time needed for the growing stage increases due to borehole deviations only slightly. Analogously to the case without borehole deviations (nominal position), the temperature distribution along the line A-A (Figure 4) was examined (Figure 10). After 19.5 days refrigeration the temperature at the outer

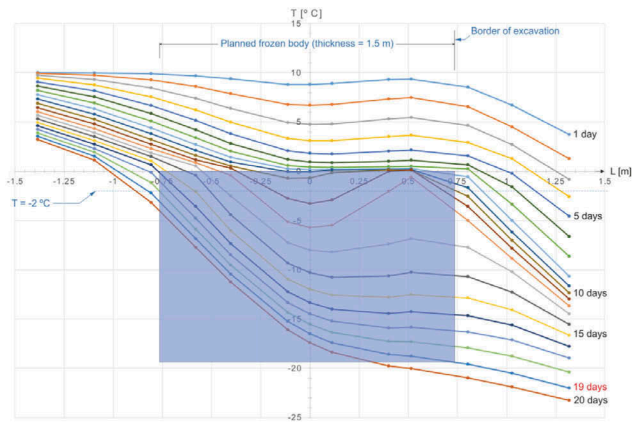

Figure 8. Temperature distribution along line A-A (*cf.* Figure 4).

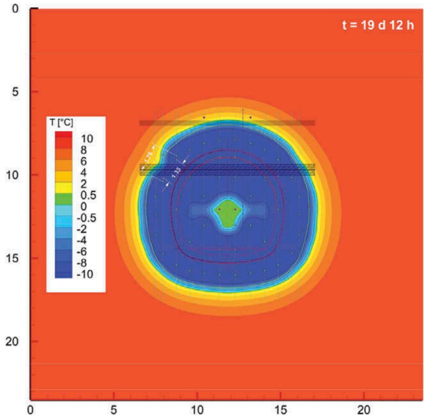

Figure 9. Simulation results after 19.5 days refrigeration time considering a possible deviation of the boreholes.

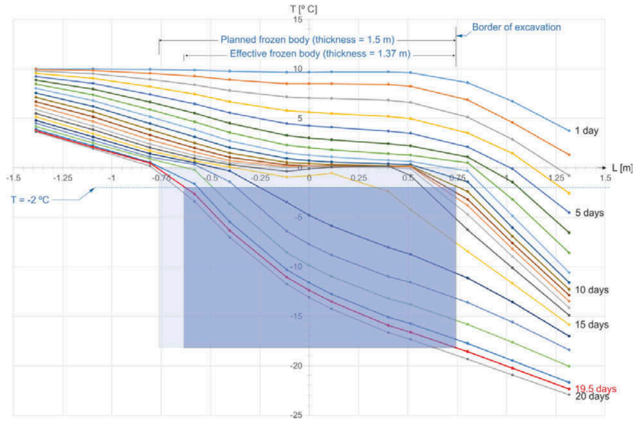

Figure 10. Temperature distribution along line A-A (*cf.* Figure 4) considering a possible deviation of the boreholes.

border of the frozen body amounts obviously 0 °C, while the temperature at the inner border reaches -18.2 °C. The effective thickness of the frozen body for this refrigeration time and its average temperature amounts to 1.37 m and -11.9 °C. The assumptions of the static design (1.25 m and -10 °C) cover thus also the case of drilling inaccuracies.

3.2.5 *Required cooling power*

Due to the big temperature gradient between freeze pipe and surrounding soil the maximum cooling power is required immediately after starting refrigeration with a constant very low temperature of the freeze pipes (*e.g.* T_{pipe} = -30 °C). With increasing refrigeration time the required cooling power for maintaining this low temperature at the freeze pipes decreases continuously. Thus, one possibility for reducing the cooling power that must be installed is to start at a moderate low freeze pipe temperature (*e.g.* T_{pipe} = -5 °C) followed by a continuous decrease until reaching the final lowest freeze pipe temperature (-30 °C in the present case). Three cases were simulated (reaching the lowest temperature after 5, 7 or 8 days refrigeration time). For each of these cases the cooling power was read out at each time step and for each freeze pipe. The total net cooling power is determined by multiplying the sum of cooling power of all freeze pipes by their average length of 40 m. Figure 11 shows the total net cooling power (l.h.s. axis) and the freeze pipe temperature (r.h.

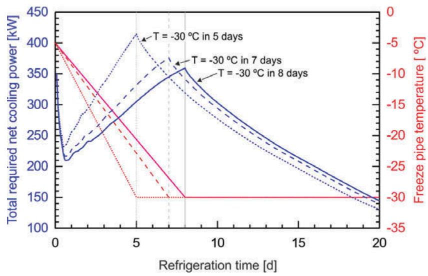

Figure 11. Total required net cooling power for the AGF.

s. axis) over time for the three analysed cases. Cooling down in 8 days results in the lowest power requirements; the required net cooling power amounts to 350 kW in this case.

4 RAISING UP OF THE WATER TABLE ABOVE THE TUNNEL CROWN

The performed piezometric measurements show that the water table is located 0.9 m lower than the highest point of the planned frozen body. The water content in the crown area of the planned frozen body should be increased artificially by watering the ground locally with two horizontal drains. The horizontal drains should be placed about 25 cm above the highest point of the planned frozen body (Figure 4). The effectiveness of this measure was checked numerically using the commercial FE-code Plaxis. Specifically, the 2D seepage flow through the partially saturated soil in the tunnel crown region was analysed considering an isotropic and homogeneous ground obeying the van Genuchten model. The soil parameters were taken from the literature for loamy sand (van Genuchten, 1980), whereby the computations were performed for two permeability values ($k_f = 10^{-5}$ m/s and $k_f = 10^{-6}$ m/s). Due to symmetry conditions only half of the crown region of the planned frozen body was modelled (red area in Figure 4). The drains were considered as line elements with constant water input quantity q. The length of the drain element corresponds to the perimeter of the real drain with diameter D = 102 mm. In the numerical computations, the location of the drains and the water input quantity were varied and the watering was simulated until achieving steady state, *i.e.* until the saturation degree reaches its final value. Figure 12 shows the contour lines of saturation degree at steady state for the relatively high permeability of 10^{-5} m/s and an input quantity per meter drain of q = 0.2 m^3/day. Steady state is reached in this case after less than 5 days watering. Consequently watering can start together with refrigeration. The results for the lower

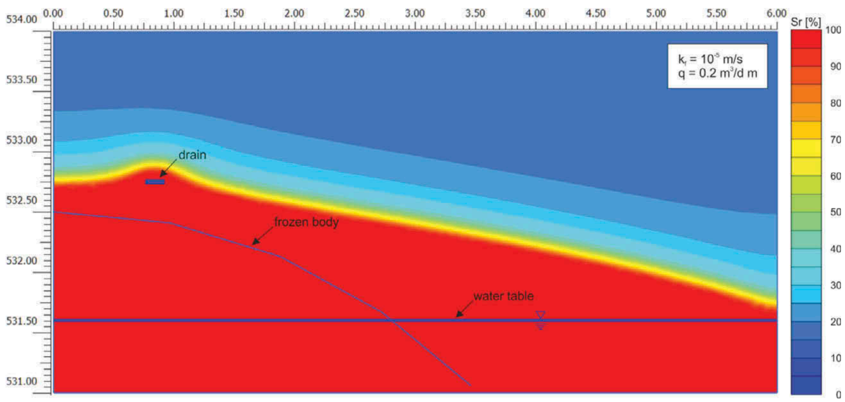

Figure 12. Saturation degree in the crown area of the frozen body at steady state.

permeability ground are about the same, the only difference being that the input quantity should be one order of magnitude smaller (the required amount of input water increases linearly with the permeability of the ground).

5 SUMMARY AND CONCLUSIONS

Artificial ground freezing with brine as an auxiliary measure fulfils the project requirement of constructing the access tunnel under the railway lines and train platforms without causing interruptions of the train traffic and station operation. The ground is water bearing but the water table in the crown area of the planned frozen body must be raised up with two drains. The effectiveness of these drains could be proven by unsaturated seepage flow analyses. The strength and deformability parameters were estimated for the static design and successfully verified with laboratory strength tests on frozen samples. Due to face stability requirements, the major part of the ground inside the tunnel cross section must also frozen. For the chosen thermal parameters (freeze pipe temperature and diameter of T_{pipe} = -30 ºC and D_{pipe} = 102 mm) the required thickness of the frozen body amounts to 1.5 m. The project conditions impose limitations to the the location of the freeze pipes. An optimized distribution of the freeze pipes could be achieved after several iterations. With this setup, consisting of 42 freeze pipes and 2 drains in the tunnel, excess pore pressure due to volume expansion of frozen pore water can be dissipated and a uniformly frozen body can be created. The refrigeration time needed for creating a frozen body according to the project requirements (thickness of 1.5 m and freezing of the tunnel cross-section) amounts to approximately 20 days even if considering deviations of the boreholes within the allowed tolerances. For an optimized operation of the cooling units the required net cooling power amounts to 350 kW.

REFERENCES

Andersland, O. & Ladanyi, B. 2004. Frozen Ground Engineering – SecondEdition. American Society of Civil Engineering, John Wiley & Sons Inc., Hoboken, New Jersey.

Anderson, D.M., & Tice, A.R. 1972. The Unfrozen Interfacial Phase in Frozen Soil Water Systems. Physical Aspects of Soil Water and Salts in Ecosystems, pp. 107–124.

Sres A. 2009. Theoretische und experimentelle Untersuchungen zur künstlichen Bodenvereisung im strömenden Grundwasser. Veröffentlichungen des Instituts für Geotechnik (IGT) an der ETH Zürich, Volume 234.

Pimentel, E., Sres, A. & Anagnostou, A. 2007. 3D-Modellierung der Frostkörperbildung beim Gefrierverfahren unter Berücksichtigung einer Grundwasserströmung. Proc. 22. Christian Veder Kolloquium, Graz, No. 30, 2007, pp.161–176.

Frivik, P. 1981. State-of-the-Art Report. Ground Freezing: Thermal Properties, Modelling of Processes and Thermal Design.

Harris, J.S. 1995. Ground freezing in practice. Thomas Telford, London.

van Genuchten, M.TH. 1980. A closed-form equation for predicting the hydraulic conductivity of unsaturated soils. Soil Sci. Soc. Am. J. doi:10.2136/sssaj1980.03615995004400050002x.