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Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 856 8

Storebælt Eastern Railway Tunnel, Denmark – Ground freezing for tunnels and cross passages

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ABSTRACT: The Storebælt glacial tills have proven to be very problematic tunnelling media and in order to overcome the difficulties, various ground treatment techniques were employed. In particular ground freezing has been used on four of the most critical Cross Passages and on the two Junction Caverns in the main tunnels. The paper covers both the design and the implementation of the freezing. The design has involved laboratory tests on frozen soil to provide input parameters for stability analyses. Numerical modelling was used to evaluate the duration of freezing required to achieve the target temperatures. Four 100,000 kCal /hour Brine Freeze Plants have been employed on the project and the freeze pipes were typically installed using lost bit drilling techniques due to the adverse ground conditions. The use of ground freezing has allowed excavations to be performed in a safe manner in otherwise hazardous ground conditions.

1 INTRODUCTION

The Storebælt Eastern Tunnel has been built to provide a twin sub sea rail connection between the two islands Zealand and Sprogø in Denmark. The two bored tubes, each 7.4 km long, performed by four TBMs, are connected every 250m by a total of 31 cross passages. The distance between the two parallel tubes, external diameter of 8.5m, is 25m from center to center. The cross passages are approximately 17m long with an external diameter of 4.75m.

Roughly the middle half of the tunnels has been excavated in marl which in general proved to be a good and stable formation both for excavation of main tunnels and cross passages.

The glacial tills proved to be more problematic tunnelling media and particularly the variability of the lower till unit caused great difficulties for the main tunnel drives with frequent collapses of the overlying formation leaving large depressions in the sea bed.

For the hand excavated cross passages special precautions in the form of ground treatment had to be taken. As described by Doran et al., 1995, a number of ground treatment methods including Tube–A–Manchette grouting, vacuum dewatering, electro–osmosis, spile bars and ground freezing were applied. Ground freezing was used on four of the most difficult cross passage excavation as well as on the two TBM junctions. The freezing was used where major depressions had been observed above the tunnels during the main tunnel drives and the frozen zone was applied to provide a protective arch above the area to be excavated. This paper presents both the design and the implementation of the ground freezing.

2 GEOLOGY

The geological sequence consists of a Danian limestone overlain by a Palaeocene marl which again is overlain by a quaternary sequence.

The geological profile for the project is displayed in Figure 1.

The quaternary sequence is composed of glacial, late glacial and post glacial deposits with the glacial deposits dominating the sequence.

As can be observed in Figure 1 the glacial deposits are subdivided into an upper and a lower till unit based on lithological and geotechnical properties.

The upper till is a fairly homogenous clay till with clay content about 15% and an undrained vane shear strength typically greater than 200 kPa.

The lower till unit is dominated by compact clay till with an undrained vane shear strength greater than 500 kPa. Transitions to sand till (clay content < 12%) are often observed and meltwater sediments have been observed to in some areas form a considerable part of the lower till sequence alternating with the tills. The meltwater deposits vary from clay and silts to highly permeable sands and gravels. For further details on the geology reference is made to Foged et al., 1995.

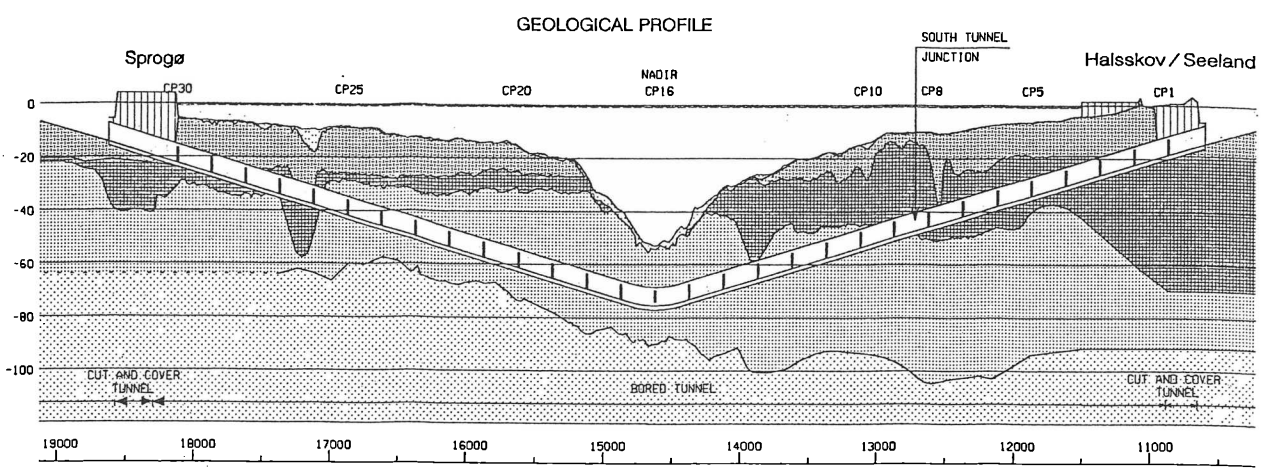

Fig. 1. Geological cross section

3 LABORATORY TEST

To develop the design of the temporary frozen ground support, knowledge of the strength and deformation characteristics of the soil were required. A first assessment based on tests on North German tills was given by Jessberger et al., 1990. To further investigate the frozen soil properties a series of unconfined compression tests on frozen clay till samples were performed in the on-site soil laboratory in accordance with the recommendation given by Sayles et al., 1987.

A series of tests, at a constant strain rate of 1%/min were made on 42mm diameter specimens with a height to diameter ratio of about 2.0. Two series, each including 6 nos of tests, were performed at −10°C and at −20°C. These tests determined the unconfined compression strength q under rapid loading conditions.

The strength of frozen soil is however dependant on the loading rate and up to a certain limit it is valid that the higher the loading rate the higher the apparent strength. Creep tests are often carried out at stress levels of 50–100% of q with long term failures occurring at stress levels significantly below 100% as described by ISGF Working Group 2, 1991.

Based on the experience by Jessberger et al., 1990, on North German tills no failure occurs if the stresses in the frozen soil are less than about 50% of q. Accordingly, a number of long term sustained loading tests were made on soil samples at 50% of the ultimate strength observed in the UCT tests. The results of the creep tests performed on the Danish tills confirmed Jessberger's observations.

Based on the tests performed the following unconfined strength design parameters were used in the design calculations:

q_d=1.25 Mpa for T=−10°C

q_d=3.00 Mpa for T=−20°C

Tests were performed on undisturbed clay tills with a natural water content of 10–12% and on remoulded "muddy" clay tills with a water content around 20%, just about the liquid limit. This was done to take into account the variability of material left behind by the TBM mining process. Apart from the water content the samples were similar with respect to grading curves, mineralogy, etc.

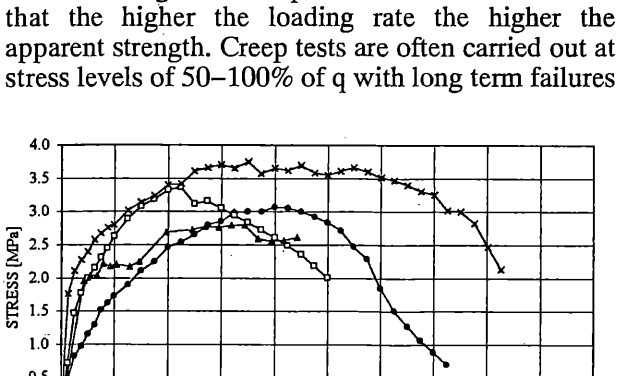

Fig. 2. Unconfined compression tests at −10°C

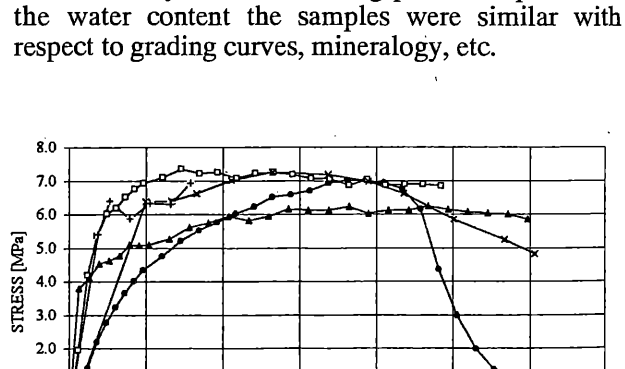

Fig. 3. Unconfined compression tests at −20°C

Based on the difference in consistency it was expected that the two types of samples, undisturbed and remoulded, would show a variation in strength and behaviour in the tests but as can be observed in Figures 2 and 3, there proved to be little difference in strength and no marked difference in the stress strain curves for the two groups of samples. During the testing "normal" shear failures were frequently observed for the undisturbed samples, whereas the remoulded samples tended to reach a maximum stress level and then fail in a plastic type deformation. Ultimate failure resulted from the eccentric loadings caused by pivoting of the loading head at the high strains.

4 DRILLING AND INSTALLATION

The field operations of establishing a ground treatment by freezing can be divided into the following four steps:

1. Pregrouting of ground to be frozen.
2. Drilling and installation of freezing pipes and temperature control pipes in the ground.
3. Installation of the brine distribution circuits between freezing pipes and freezing plant.
4. Freezing and maintenance of the frozen soil mass during the service period.

The grouting in the first phase was carried out using cement bentonite grout with grouting being performed through PVC tube–a–manchette 37/48mm pipes. This exercise was performed to grout up water bearing strata thus reducing the risk of potentially dangerous water movements in the zone to be frozen.

The length of the freezing pipes was usually around 10m but on a few occasions were up to 20m long pipes installed, but to maintain drilling accuracy it was preferable to limit the length. The freezing pipes were typically installed using lost bit drilling technique due to the adverse ground conditions, with the 60/75mm steel freeze pipe rods being used as drill rods and water as drilling fluid. On completion of drilling the drill bit was sealed and the freeze pipe pressure tested to 10 bars to ensure complete watertightness. This step was essential as the consequences of leakage of brine into the formation could be disastrous. In case of leakage a 48/50mm copper pipe was installed and sealed inside the steel pipe by grouting. To enable circulation of the brine a 25mm steel pipe was introduced inside the outer pipe thus allowing the coolant to be led in through the center pipe and flow out through the annular space between the two pipes or vice versa.

Copper–Konstantan thermocouples were used for temperature control and they were normally installed in the grout holes used for pregrouting. In some cases the thermocouples were simply grouted into the tube–a–manchette pipes using inexpensive PVC lined thermocouples. In other cases the grout holes were redrilled and steel pipes similar to the freeze pipes were installed and more rugged metal lined thermocouples were placed in alcohol inside the pipe. The advantages of the latter system were the easy replacement of thermocouples and the possibility of reusing the thermocouples. The thermocouples were usually placed with a spacing of 2m to 3m inside the pipe.

The freeze pipes were divided into groups of 3 to 6 pipes connected in series by a 38/48mm rubber hose and all groups were then connected in parallel to two 100mm Victaulic distribution pipes, one leading the brine to the freeze pipes and one collecting the return flow. As the freeze plant was often placed at some distance from the freeze pipes 150mm pipes were used to convey the brine to and from the location of freezing thus minimizing the pressure loss. All pipes and hoses were insulated to avoid heating of the brine.

The brine used was a solution of 25% to 30% by weight of calcium chloride in water lowering the freezing point to between –30°C and –50°C.

The four freeze plants employed on the project were electrically powered plants with a maximum cooling capacity of about 100,000 kCal/hour using Freon R22 as the primary cooling agent. The combination of brine and freon was preferred for safety reasons.

Temperature monitoring of the thermocouples was carried out by dataloggers which were connected to telephone modems allowing off–site monitoring of the ground temperatures. In addition to the ground thermocouples a number of thermocouples were installed into the brine distribution system, making it possible to check the performance of the individual group of freeze pipes, and ensuring that all groups were receiving sufficient flow of brine.

5 DESIGN

The ground freezing structures were used in two ways, as impermeable water barriers preventing

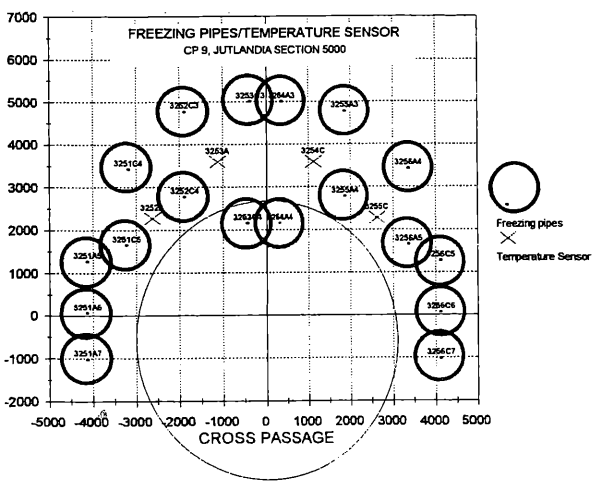

Fig. 4. Cross section at cross passage ground freezing

ingress of water into the excavation and/or as structural member providing a protective hood over the excavation.

The geometrical restraints of drilling an adequate pattern of freeze pipes from within a circular tunnel often presented a major challenge. Therefore the design was always a compromise between structural desirability and practical geometrical restraints. The geometry of freeze structure was carefully studied by building a 3D model in a spread sheet environment and cutting sections at 2m intervals. Figure 4 shows a typical section. As can be observed in this figure a radius of influence of 0.6m for the individual freeze pipe was used at this stage of the design. The static calculation was carried out either by simplified hand calculation or in more difficult cases a finite element program was used to analyse the stress state in the frozen structure. The input for the FEM analysis was obtained from the laboratory testing of the frozen soil samples and the result of the analysis would be a temperature requirement for the central core of the frozen structure. When the freeze pipe layout was defined the duration of freezing to achieve a certain target temperature had to be estimated. Due to the latent heat of water when changing from liquid to solid state the calculations are not as straight forward as ordinary thermo–dynamic problems. Closed form solution is available for the simpler cases such as individual or rows of freeze pipes in a homogenous infinite space, as presented by Khakimov, 1957, and these are quite useful in the planning stages. However once the design has been prepared a more sophisticated method is required to take into account the complexity of the individual freeze pipe pattern and local variability of ground and boundary conditions. For this purpose a finite element program was employed. This program allowed modelling of variable boundary conditions and thermal properties of the soil as well as the geometry of the zone to be frozen and of the freezing pipes. The further advantage of the FEM program is that boundary conditions can be changed at a certain time step which provided the flexibility to model changes in brine temperatures, use of intermittent freezing and alternative excavation sequences. Another advantage was that any point of weakness in the freeze pattern would be highlighted by the output from the program. An example of the output from the program is displayed in Figure 5.

Usually freeze pattern designs were prepared to allow for a 3 to 4 weeks period to achieve the target temperatures.

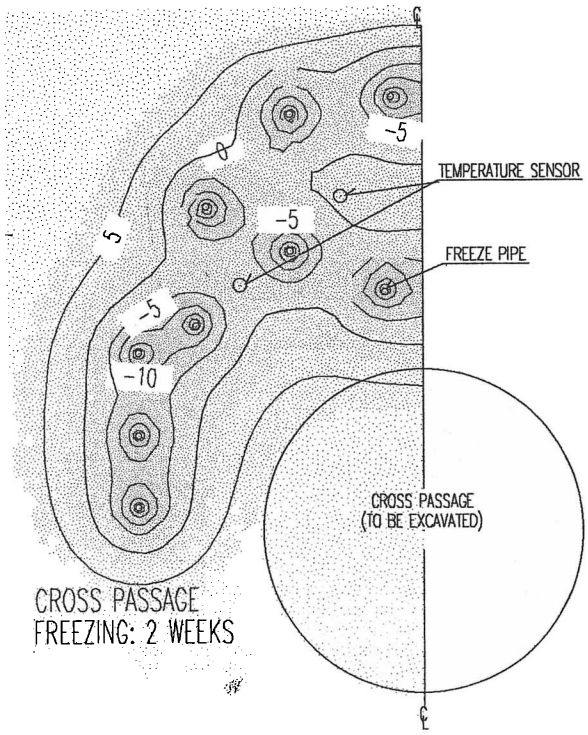

Fig. 5. Ground temperature profile at cross passage ground freezing

6 FROST HEAVE PRESSURES

When planning ground freezing near existing structures, the effect of frost heave pressures should always be considered.

When the pore water within frost susceptible soil freezes and the frost heave is restricted, heaving pressures are generated primarily in the direction of the freezing front. One of the several controlling factors for the frost heave pressures is the pore size distribution which can be related to the permeability of the soil. The relationship between permeability and frost heave pressures has been presented by Jessberger et al., 1990, and is shown in Figure 6. As can be observed considerable frost heave pressures can be built up depending on the permeability of the soil.

When ground freezing was used to protect cross passage excavations, frost heave pressures caused some concern in relation to loading of the main

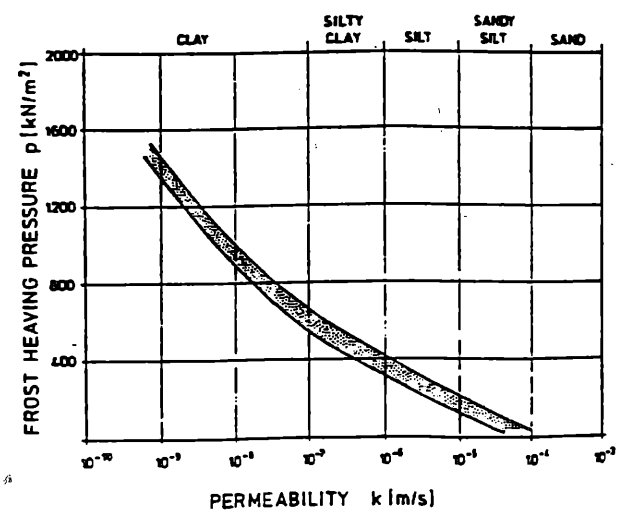

Fig. 6. Frost heave pressures as a function of permeability. After Jessberger et al., 1990.

tunnel lining, but as displayed in the case study, the freeze pipes were placed more or less radially out from the main tunnel lining, and therefore only secondary heave pressure components were expected to load the main tunnel lining. To further reduce potential frost heave effects intermittent freezing was used on a couple of occasions. To monitor any deformation of the lining, survey targets were mounted on the lining and surveyed throughout the period of freezing. Also glass tell–tales glued onto the tunnel lining were used to monitor movement and development of any observed cracks.

The results showed that only minor deformations in the order of a few millimeters occurred during freezing and the deformations related to the freezing was not experienced to be a problem for the main tunnel lining.

7 CASE STUDIES

In the following two case studies are presented. The South Tunnel TBM Junction Cavern and Cross Passage 8 have been selected as structures excavated under a protective hood of frozen ground. In the two cases the frozen hood functioned both as an impermeable water barrier and as a reinforcement of the ground.

7.1 *South Tunnel TBM Junction Cavern*

The south tunnel junction is located in the transition zone between the till formation and the marl formation as displayed in Figure 1.

Due to relatively high ground water pressures in the order of 4 to 4.5 bar frequent problems with instability of the overlying till formation were experienced in this area. The high water pressure in combination with the significant difference in material properties of the two formations caused this zone to be one of the most difficult for the tunnel boring. The presence of highly water charged sand bodies within the till formation frequently resulted in loss of ground, thus providing a direct hydraulic connection between the sea bed and the tunnel crown. The residual strength of the material in these zones was commonly little more than a slurry and obviously did not provide any suitable protection for

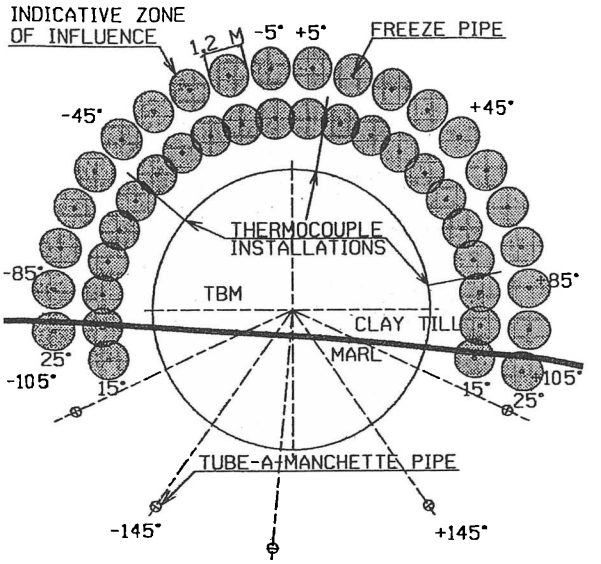

Fig. 8. Cross section at TBM junction displaying Jutlandia freeze pipe.

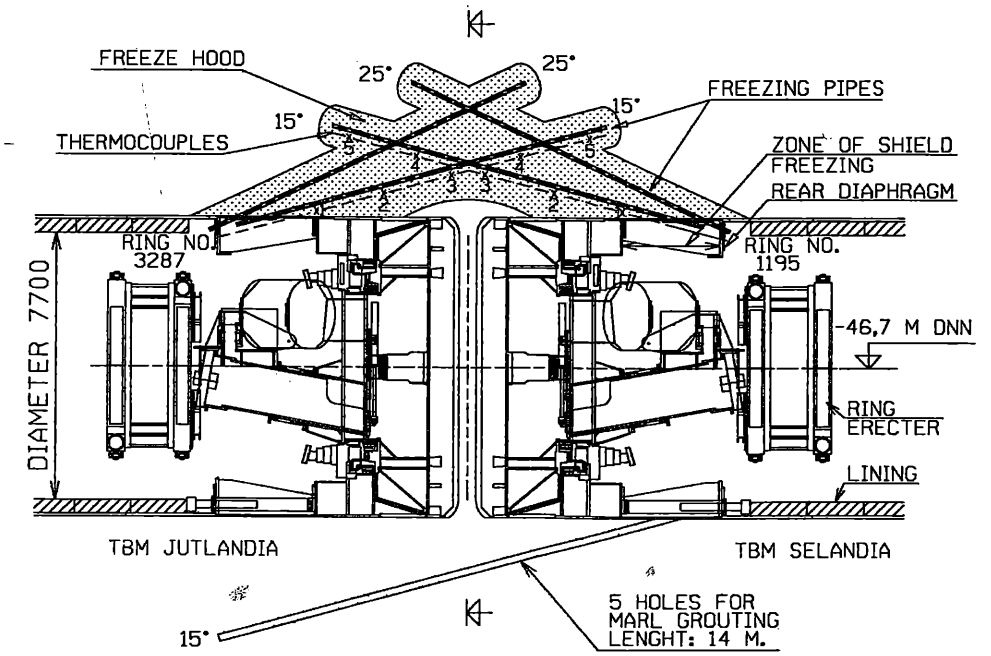

Fig. 7. Longitudinal section at TBM junction

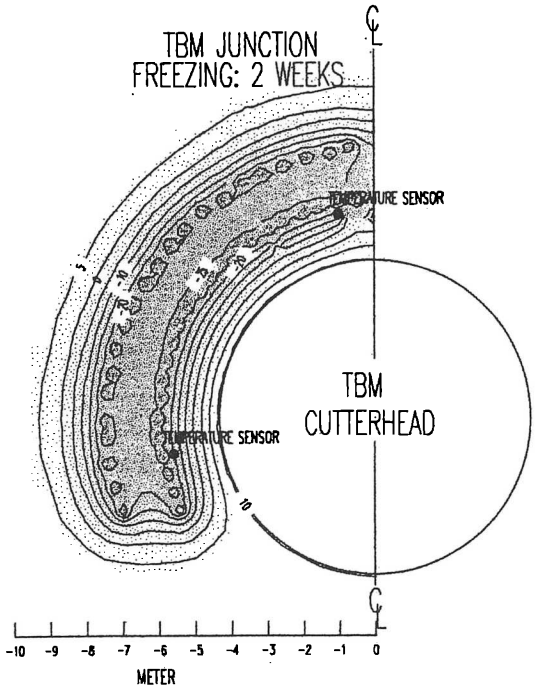

Fig. 9. Predicted ground temperature profile after two weeks of freezing at South Tunnel junction

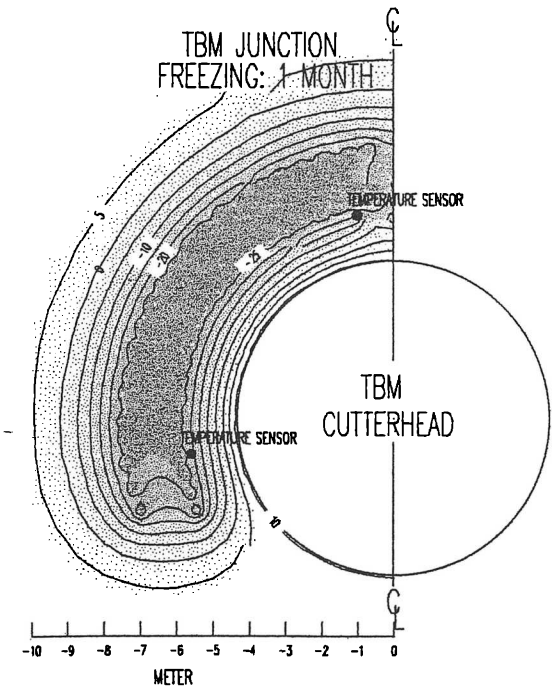

Fig. 10. As Fig. 9. after one month of freezing

the hand excavated junction cavern. The observation of depressions in the sea bed therefore mandated the protection of the junction cavern excavation by a freeze hood. The freeze pipes were installed through penetrations in the TBM shield, using blowout preventers bolted onto the shield to limit water

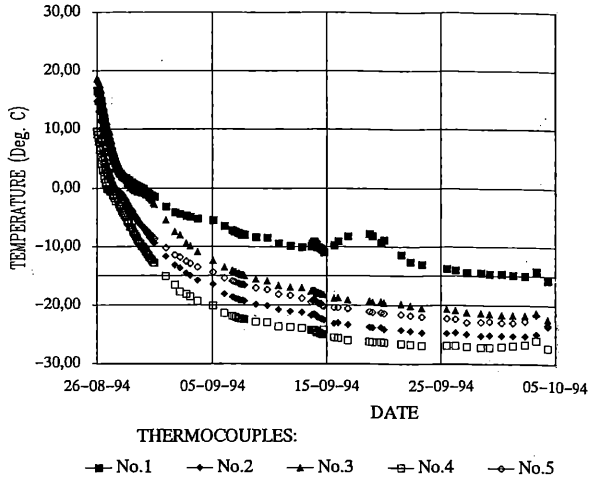

Fig. 11. Monitored temperatures at the South Tunnel junction

ingress. The freeze pipes were installed by a small drilling machine mounted on the segment erector such that the drilling machine could easily be moved between holes just by rotating the erector. The installation of freeze pipes was preceded by a grouting exercise. The grouting was performed because of the severely disturbed soil in the area with a purpose of void filling, compaction and limiting water flow across the zone to be frozen. During the first phase of grouting a cement bentonite grout, c/w=0.5, was used and during the second phase a microfine cement grout, c/w=0.4, was injected. The total quantities used for the grouting exercise added up to 270m^3 cement–bentonite grout and 55m^3 microfine cement grout, which gives a good indication of the ground disturbance in the area.

The layout of the junction cavern freeze pipes is presented in Figures 7 and 8. As shown only the till formation was frozen whereas the marl formation in the lower section of the tunnel section was treated by microfine cement grouting. A total of 89 freeze pipes were installed from the two TBMs adding up to a total freeze pipe length of 925m being serviced by two freeze plants. The freezing was monitored by 3 thermocouple installations from each TBM, with 5 thermocouples in each installation as shown in Figure 7. Figures 9 and 10 display the predicted temperature development at the center section between the two TBMs after 2 weeks and after 1 month of freezing. The positions of the thermocouples are indicated. Figure 11 shows the actual temperature development for one of the thermocouple installations. Thermocouple no. 3 is located in the central zone of the freezing as shown in Figure 7, and can be compared to the temperatures at the location of the sensor points in Figures 9 and 10. Good agreement is observed with a predicted temperature just below −20°C after 2 weeks and around −25°C after 1 month of freezing.

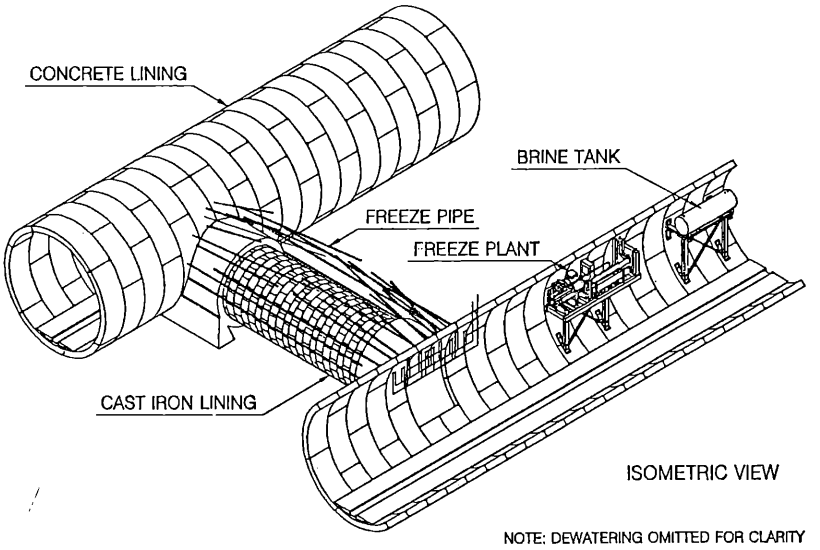

Fig. 12. Layout of cross passage ground freezing

Based on FEM stability analysis the target temperature for a 2.5m wide core of the frozen body was set at −15°C, which was achieved after approximately one month of freezing.

A special problem in relation to the TBM junction freezing was the heat being transmitted from the main tunnel through the TBM shield into the ground, preventing an efficient freezing of the ground close to the shield. This effect can be observed in Figure 11, where thermocouple no. 1, positioned near the shield, is cooling down much slower than the other 4.

To overcome this problem square pipes were welded to the internal face of the shield as indicated on Figure 7. By circulating brine through these pipes it was possible to chill the shield skin and ensure that the freeze body had good contact with the skin, but as displayed even with this precaution the cooling down was still slower.

The junction cavern was successfully excavated by hand and built over a period of 2 weeks under the protection of the frozen hood in essentially dry conditions apart from condensed water. No problems with ground stability was experienced. It should be noted that traditional hand excavation in frozen soil is extremely hard work in an often narrow, damp and cold working environment.

7.2 *Cross Passage 8 Excavation*

Cross Passage 8 (CP8) is located entirely within the lower till unit approximately 44m below sea level as displayed in Figure 1.

The primary elements of ground treatment on the majority of cross passages in till comprised Tube–A–Manchette grouting and vacuum dewatering by well points as described by Doran et al., 1995. By this treatment the ground water pressure could normally be eliminated and stable ground conditions could be achieved. The hand excavation could then be carried out safely with a combination of timber and shotcrete as temporary excavation support.

On CP8 however a major water charged silty fine sand lens was located a few meters above the main tunnel which had led to a number of contacts being made to the seabed as previously described for the southern junction cavern. During flow tests on 60mm diameter well points installed in the upper sand layer initial flows above 100 l/min and up to 800 l/min were recorded from individual well points. If these flows could have been maintained the ground water pressure could have been lowered significantly, but due to the fineness of the sand the filters clogged, and flows reduced dramatically preventing an effective dewatering. Initially a minor freeze hood above the cross passage collars near the main tunnel lining was foreseen. Due to the ground water pressure, it was decided to extend the freeze hood to cover the whole length of the cross passage, and design it to carry the additional water pressure. Due to the strength of the clay till in the lower section of CP8 and because it was possible to dewater the formation in this section, the freeze arch was only extended to the knee level of the cross passage.

Figure 12 shows the general layout of the ground freezing system with a freeze plant located in both the north and the south tunnel. Figure 13 displays a longitudinal section of the freeze pipe protection.

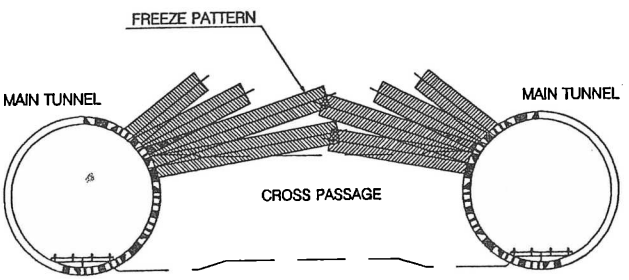

Fig. 13. Section at cross passage ground freezing

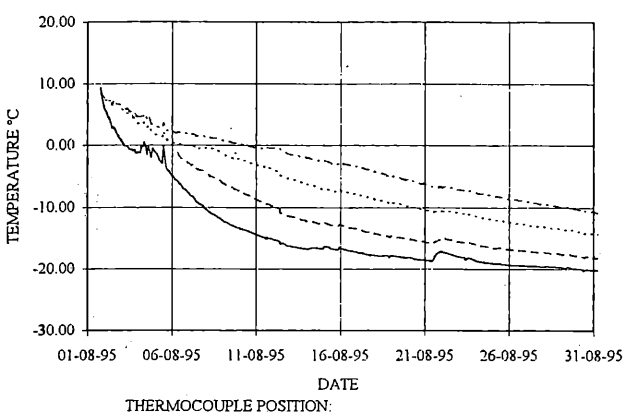

Fig. 14. Monitored temperatures at CP8 freezing

Based on static calculations the target temperature for the core of the frozen hood was set at −10°C providing a frozen arch, 1.4m in thickness, covering the length of the cross passage. The ground temperature was monitored by 7 installations in the south tunnel, and 4 in the north tunnel with each installation holding between 4 and 6 thermocouples typically placed at 2m intervals. Figure 14 displays the monitored temperatures for one of these installations and as can be observed temperatures are steadily dropping with the first thermocouple below the target after approximately 1 week and the last after approximately 1 month of freezing. This shows how important it is to know the exact position of the thermocouples and also that the positioning of the monitoring points should be well considered to enable the required information to be obtained.

Figure 14 displays the brine temperature at the inlet and at the outlet. The inlet temperature was held at about −30°C, and as can be seen, the outlet temperature is steadily dropping through the period, as a result of the ground cooling down. The brine flow circulated through the freeze system was usually around 40m^3/hr and in general a temperature

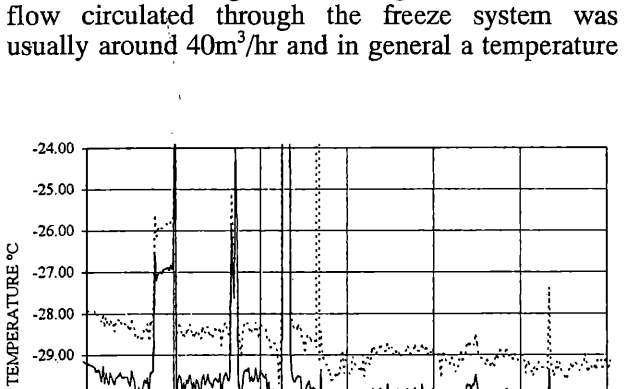

Fig. 15. Brine temperature at CP8 freezing

increase in the return flow of 1–3°C was observed after the first week of freezing. Due to the extensive frozen hood at CP8 approximately half of the excavation cross section was heavily frozen which lead to slow excavation progress but no problems with water ingress or ground stability was observed during the 4 weeks of cross passage excavation.

8 CONCLUSION

Ground freezing is an expensive ground treatment method. It required time to install the freeze equipment, to establish the frozen zone and approximately 50% additional time was required to excavate the protected structures.

Ground freezing is however unique for its ability to be able to monitor the effectiveness of the treatment and for the possibility to apply quality assurance procedures that ensure, with a high degree of certainty, the treatment design objectives have been achieved.

The use of ground freezing on the Storebælt Tunnel was employed where no alternative existed to the ground stability problems encountered and allowed excavations to be performed in a safe manner, in otherwise hazardous ground conditions.

9 ACKNOWLEDGEMENT

The authors thank A/S Storebæltsforbindelsen for their cooperation during the ground treatment work and in the preparation of this paper.

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