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RAILWAY TUNNELLING IN FROZEN GROUND ON BOTHNIABANA

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ABSTRACT

A new railway line was under construction in North-eastern Sweden, along the shoreline of the Gulf of Bothnia. The terrain consisted of Postglacial clay and silt valleys, and ridges of Precambrian bedrock, crossing the railway line. The railway level was about 20m below the ground surface.

For the design of freezing, a freezing analysis was carried out to determine the pipeline depths and distances, to estimate the necessary time for freezing and to determine temperatures within the frozen zone. To estimate the deformations, displacements and safety level, a mechanical analysis of the tunnel cross-section was carried out. As input, the geometry of the soil layers, bedrock and temperature zones was given. Time- and temperature-dependent mechanical parameters were estimated for different soils and temperatures. The analysis was carried out for 12 phases simulating different phases during tunnelling. The estimated displacements for the tunnel arch were negligible, if the temperatures were at or below -15oC.

The freezing was started in May 2002, and completed in September 2002. The tunnelling was started in September, and it was completed in November 2002, following with the casting of final liner. According to the experiences until now, the work has progressed successfully.

INTRODUCTION

The application of ground freezing in deep excavations and tunnelling in soft soils is not a new idea. It has been widely discussed in Ground freezing symposia since 70'ies. There are certain reasons, why ground freezing has proved to be recommendable: the applications do not cause any environmental impacts on the site, it can be rationally planned, designed and controlled, and it uses the ground itself as a structural element. Ground freezing causes, of course, certain permanent changes on the freezing soils after thaw: frost action causes ice segregation in the frost-susceptible soils, and results in consolidation settlements after thaw. These can be pre-estimated and evaluated in relation to neighbouring structures, if needed. As a geotechnical procedure, ground freezing has been economically used as an alternative strengthening method, or as the only possible method on many sites.

The frozen soil has dramatically higher strength and stiffness than the unfrozen soil. The mechanical behaviour is highly dependent on the temperature below freezing point. In frozen soils, the strength and deformations are also time-dependent, which means that they have clearly plastic properties that become more and more dominating when frozen temperature rises. Experience, anyhow, has showed that these characteristics allow in normal cases the engineering application of frozen ground temporary support in geotechnical engineering, if the specific properties and conditions are properly verified and controlled for the site.

From safety reasons, the frozen ground, as a tunnel support is a challenging structure. The displacements must be in given limits, and the overall safety must be high enough to maintain the continuous construction procedure until the final support has been completed.

The Stranneberg tunnel was located in Sweden, at the western coast of the Gulf of Bothnia, five kilometres along the Bothniabana railway line from Ornskoldsvik towards Umea (Figs. 1 and 2). Construction planning for the freezing started in November 2001. Freezing took place in summer 2002 and excavation in the autumn of the same year. The tunnel (100 metres long, 10 metres high and 10 metres wide) required 40 000 cubic metres of soil and weak rock mass to be frozen. Freezing work on the tunnel started in early 2002 with the boring of freezing pipes and the tunnel broke through the frozen section in December 2002.

Fig. 1. Location of Bothniabana.

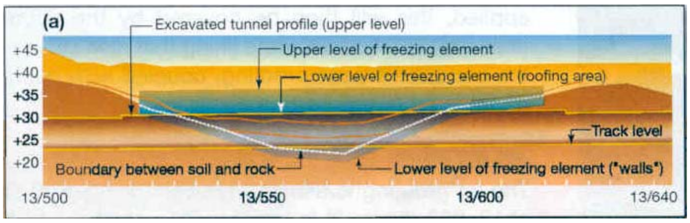

Fig. 2. Longitudinal profile of the ground freezing site.

SITE CONDITIONS

The railway was constructed at a depth of 20 metres in the muddy ground of Stranneberg. Open excavation was not an option for the tunnel because the heaping of excavated soil would have caused environmental problems. Owing to the size of the tunnel's cross section, reinforcing the soil by freezing was considered to be the technically best and safest option.

Along the coast of the Bothnian Gulf, on the Precambrian bedrock is overlain by glacial till containing stones and boulders, and glaciofluvial sediments, gravel and sand. In valleys and on the lowland, upper sediment strata consist of soft postglacial organic clay/silt. Groundwater level is normally a few meters below the ground level. The organic clays have a high sulphide content, and the tunnelling in the soft ground was seen environmentally better than deep excavation and covering, resulting in expensive dumping of large soil volumes.

The tunnel was a single-track tunnel with the profile 9.6m high and 8m wide. The contract covered excavation and final lining of the tunnel. Track and other installations will be done later. The tunnel strengthened by freezing was stretched from rock to rock, with a total length of 100 meters. The freezing, totally about 40 000m^3, was carried out in one phase, and the frozen soil was loosened by blasting. The blasted soil, about 7000m^3, was then transported out through a 200m long access tunnel and piled on the ground in a temporary basin. The blasted surface was immediately covered with a polyethylene insulation mat and shotcreted for temporary support. The tunnel was opened in 3m progress per blast. After completing the excavation, the final lining of watertight, reinforced concrete was cast and finished.

DESIGN

To solve basic problems dealing with the freezing and excavation, ground-freezing analysis was carried out to estimate the freezing time and temperatures in the frozen arch and sidewalls.

The freezing was carried out using vertical columns, in which brine was circulated. The freezing was extended at least 5m above tunnel ceiling, 5m outside the sidewalls and 2m in the base rock below the tunnel. The diameter of the column was 200mm. Distance between individual columns was about 2m in a diagonal network. The freezing of the ground was analysed assuming column surface temperatures of -15, -20 and -27oC (Fig.3). The temperatures of the arch region and wall region were estimated with time. The freezing time necessary was found to be 2-3 months with the stated temperatures. The cooling device was designed to ensure the needed temperatures. Resulting from the analysis, temperature zoning was established for the arch and vertical walls for the estimation of mechanical parameters in the mechanical analysis.

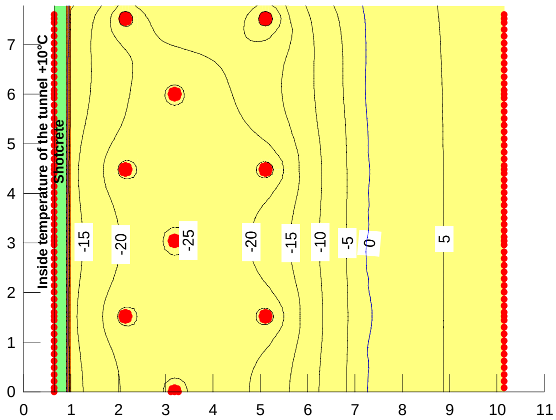

Fig. 3. Temperature zones in Temp/W -analysis. Horizontal section on the frozen wall with shotcrete and thermal insulation (R= 1m^{2}K/W). Phase: 90 days after shotcreting, column temperature -27°C.

The mechanical behaviour was analysed applying creep strength vs. temperature and time, and creep deformation modulus vs. temperature and time for the frozen soil horizons. The basic parameter models were established from literature (Zhu & Carbee 1987, Zhu et al. 1988), and controlled with the laboratory testing data from Technical University of Lulea. The characteristics were controlled. Examples of the design characteristics for stress-strain analysis for the loading time of 90d are illustrated in Figures 4 and 5.

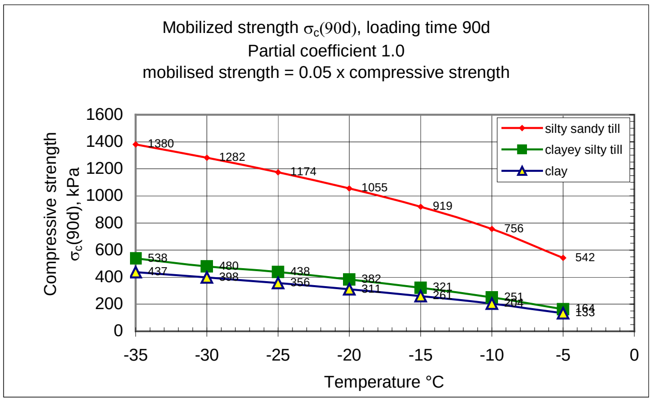

Fig. 4. Design strength vs. temperature and loading time (degree of strength mobilisation according to the technical description of Golder Associates AB).

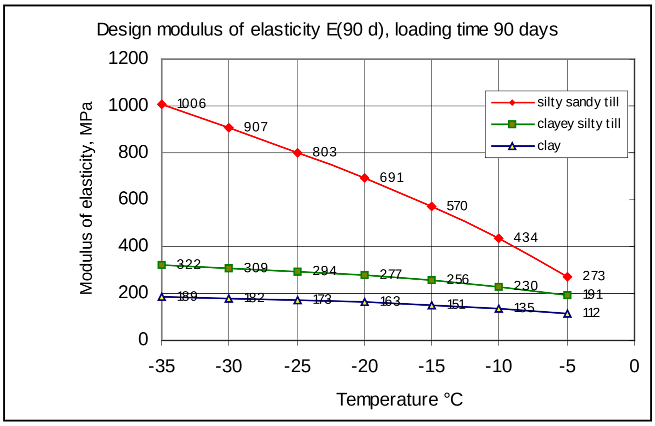

Fig. 5. Design deformation modulus vs. temperature and loading time. Modulus reduction for clay and clayey silty till (Zhu & Carbee 1987), modulus reduction for silty sandy till (Zhu et al. 1988).

The strength and deformation properties of the weakest soil horizon, consisting of organic clay, were later checked with laboratory testing. Unfrozen, undisturbed samples were taken, and with specimens, frozen in the laboratory. Standard compression tests at temperatures -10°C and -20°C as well as creep tests at -10°C with two constant axial load levels were conducted. The results proved that the pre-estimated characteristics were slightly on the safe side.

Quasi-elastic analysis, applying time-dependent strength and deformation characteristics, was carried out using the geomechanical computer program PLAXIS. To control displacements and safety level, analysis was carried out in stages that were seen critical for the construction work. These stages were:

Phase 0: Initial stress state before opening
Phase 1: Short term state before shotcrete (parameters for 3 days)
Phase 2: Short term state before shotcrete, higher inside temperatures (parameters for 3 days)
Phase 3: Short term state after shotcreting (parameters for 3 days)
Phase 4: Long-term state after shotcreting (parameters for 90 days)
Phase 5: Long-term state after shotcreting (parameters for 90 days). With a weakened interface between shotcrete and frozen ground
Phase 6: Same as Phase 5, to estimate the factor of safety, applying parameters for 90 days
Phase 7 Long-term state without shotcreting (parameters for 90 days)
Same as Phase 4, but without shotcreting
Phase 8: Same as Phase 7, to estimate the factor of safety, applying parameters for 90 days
Phase 9. Long-term state after shotcreting (parameters for 90 days). With a weakened interface between shotcrete and frozen ground, higher temperatures than in Phase 5
Phase 10: Same as Phase 9, to estimate the factor of safety, applying parameters for 90 days
Phase 11: Same as Phase 2, to estimate the factor of safety, applying parameters for 3 days

An example of the results of strain analysis for the tunnel section for 90days is illustrated in Fig. 6.

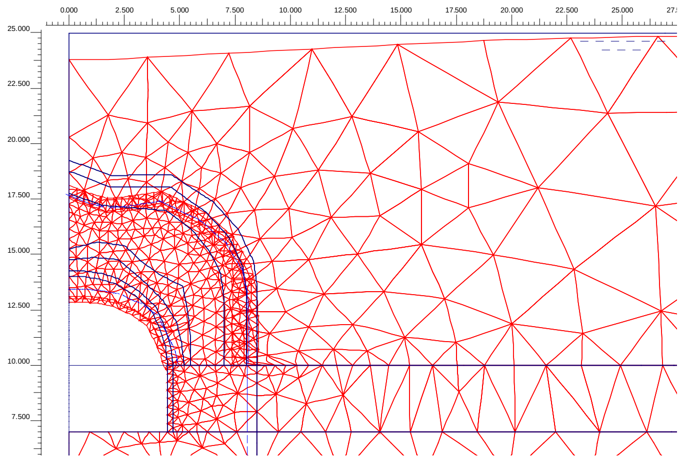

Fig. 6. Phase 5. Deformed tunnel section with a circular arch. The estimated maximum displacement was 17.1mm.

According to the analysis, temperatures at the highest -15°C were seen necessary to ensure reasonable safety and small displacements in the tunnel. The estimated overall safety level was about 2.5 and maximum displacements less than 30mm in the long-term (90d). Intermediate phases were safer.

FREEZING

Freezing columns were installed in the spring of 2002. The cooling facility and main pipelines were installed in the late spring (Fig.7), as well as thermocouple profiles in small-diameter casing tubes.

Fig. 7. Freezing installation on the ground surface, summer 2002.

During installation, the bedrock surface within the central, deep part of the tunnel line was met 1-2m deeper than presented in bidding documents. This caused a need to install a set of extra cooling columns to ensure the necessary safety for tunnelling. Monitoring was installed in 5 sections to control the frozen sate of the ground support. They were read individually on the ground surface. An example of temperatures in one of monitored sections is illustrated in Fig. 8.

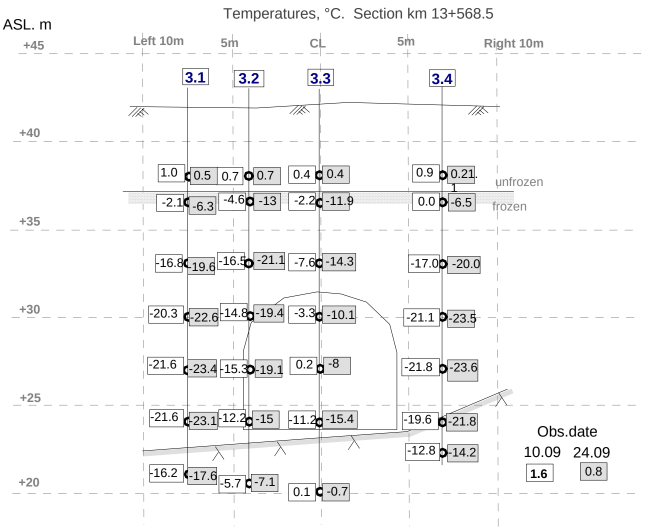

Fig 8. Observations 24.09.2002 and 10.9.2002, Section km 13+568,5.

Freezing was started in May 2002, and the temperatures necessary for excavation were reached in September 2002.

EXCAVATION AND TEMPORARY LINING

The freezing was also applied for the tunnel soil. This was seen necessary for several reasons:
- To create a tree-dimensional support for the temporary tunnel end,
- To enable the application of normal rock tunnelling procedure and machines in frozen ground tunnelling and
- To minimise the risk of ground water leakage to the tunnel through the rock floor.

The tunnel was blasted in 3m lengths, and the frozen soil was excavated and transported. The procedure was like in rock tunnels.

The charging, blasting and excavation was done as quick in order to enable supporting with shotcrete without delay. Normally one length was completed in 2-3 days. Blasting of frozen soil was found to be similar to blasting of tenacious rock, needing a higher rate of charging.

Fig. 9. Tunnel end after blasting. The cooling pipes for tunnel soil freezing were emptied before blasting.

After opening of the advance, tunnel walls and arch were lined with thermally insulating polyethylene mat, and covered with a 300mm thick layer of shotcrete that was anchored in the frozen soil (Fig. 10). After reasonable curing of the shotcrete, the next advance was opened, and the temporary supporting constructed.

Vertical and horizontal displacements were monitored after shotcreting at 5 sections with tachymeter measurements. Temperatures were also monitored at 5 sections to ensure safe temperatures. No notable displacements were observed during the tunnelling.

A reserve plan for unexpected occasions was done. A reserve facility for instant freezing of possible unfrozen bodies as well as ground water leakage with liquid-nitrogen, and a few large support frames in the case of excessive displacements were available at the site during tunnelling.

Fig 9. Shotcreting of the blasted tunnel end in October 2002.

FINAL LINING

The final, 0.5m thick concrete wall and arch were cast after the tunnel had been excavated in January-August 2003.

The active freezing was stopped in August 2003, after reaching the necessary strength of the final lining. The thawing started with warming of the frozen ground, resulting in slow transfer of earth pressure to the lining. The later thaw of the frozen ground will be slow, lasting at natural temperatures 5 a' 10 years.

The frozen soil was frost susceptible, and some frost heaving due to ice segregation was observed. At this site, however, frost heaving or later resulting thaw settlement was not considered as a problem, because the site was a sloping farmland with no sensitive structures.

DISCUSSION AND CONCLUSIONS

The experience gained in this project could be concluded as follows:
- The analysis on freezing, applying characteristics of local soils, proved to be realistic
- The mechanical behaviour, simulated for different phases in the framework of time-dependent mechanical characteristics, and changing loadings and geometry, was estimated with success
- The tunnelling process was controlled to ensure the safe construction work
- The project could be carried out in the planned schedule and costs, and applying standard rock tunnelling procedure.

In the future cases, development might be gained in minimising the construction time, in the automatic management of freezing process and in more sophisticated monitoring systems. The critical moments in the project costs are besides safety and quality, project time and energy consumption.

ACKNOWLEDGEMENTS

The client of this project was Bothniabanan AB, offering a good platform and support during the construction process. The project was based on the qualified preliminary investigations and design of Golder Associates AB, enabling the detailed design and construction with realistic principles and basic data

The professional enthusiasm and activity of the management as well as site team of Lemcon Ltd was key factor for the successful implementation. For VTT Building and Transport this was a challenging, multi-disciplinary geotechnical mission, which demanded application of sophisticated design tools and specific testing facilities.

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