The Joberg tunnel. Successful tunnelling in moraine.

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ABSTRACT: A 100 meter long tunnel in mixed face moraine and rock, below the ground water level, was the solution when the southern end of the Joberg tunnel had to be located in an area with high risk of rock fall. The rock fall risk from the steep mountain side was evaluated to be so high that an open cut was not an option. Since a rail tunnel in clay was constructed in Trondheim in 1942, using overpressure chambers, this is the first planned soil tunnel in Norway ever built.

The soil tunnel has been successfully completed in May 2016 by using typical soft ground tunneling methods. The excavation required a good cooperation between contractor, client and designer at site. Tunnel face and profile stability have been maintained by a steel pipe umbrella and immediately applying sprayed concrete and installing additional spiles, anchors etc. as required. Water ingress have been controlled by using drainage pipes in the tunnel and pumping wells drilled from the terrain along the tunnel alignment. Deformations have been measured by convergence profiles in the tunnel and terrain, and an 30 meter horizontal inclinometer installed above the tunnel. Deformations in the tunnel have been within the range of 12-27mm.

1 INTRODUCTION

The Norwegian Public Road Administration (NPRA), Statens vegvesen Region Vest, is constructing a road tunnel in Granvin county, Rv13, southeast of Voss, see Figure 1. The Joberg tunnel, length of 2032 m, shall make it safer and faster to pass below the Joberget mountain, where the road is quite often closed due to rock fall.

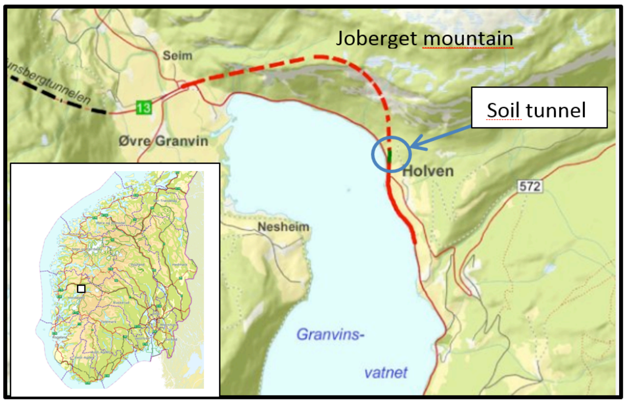

Figure 1. Project area

The main part of the tunnel is conventional drill & blast in gneiss type of rock with a contact into the underlying phyllite in the southern part, including also the soil tunnel part. The cross section is T9,5, appr. 71 m^2.

The decision to construct a soil tunnel in the southern entrance was made to avoid that open pit excavation in thick soil overburden in an area of scree material and high risk of rock fall during the construction phase.

Planning of a soil tunnel is not common in Norway. When Sweco Norge AS was awarded the contract, it was required that a company with experience from soil tunnels came in as sub consultant. iC Consulenten of Austria was selected for this task.

The construction works started in fall 2015 and the breakthrough was in summer 2016. The Contractor is a joint venture between Metrostav from the Czech republic and Bertelsen & Garpestad AS. Opening of the tunnel is planned in summer 2017.

2 PRE INVESTIGATIONS

When Sweco/iC entered into a contract with Statens vegvesen Region Vest early 2013, pre-investigations for the southern entrance had been ongoing since January 2009.

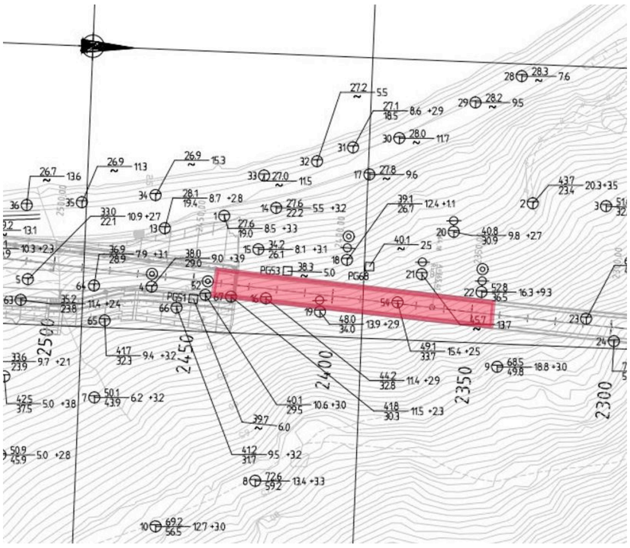

Figure 2: Pre-investigation drillings

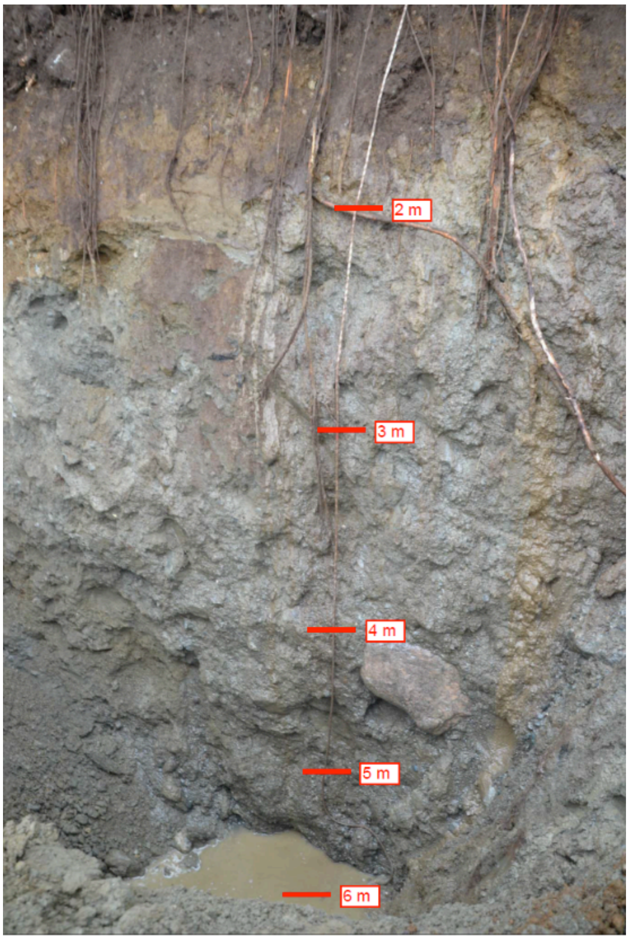

Figure 3: 6 m deep test pit in moraine

Soil overburden was soon found to be thick all over the area, so that a tunnel entrance in rock could not be easily established. 50 total soundings were executed (Figure 2), 7 soil test series, 5 wells and also a number of refraction seismic profiles. Probably most important for the understanding of the soil conditions and properties, was excavation of three deep pits in the moraine material, see Figure 3.

Hydrogeological testing was also performed in some of the bore holes to know more about permeability of the soil.

3 GELOGY, ROCK AND SOIL

The southern entrance at Holven, is located next to Granvinsvatnet, in a valley side with thick moraine deposits. Ground investigations has registered deposits with thickness from 12 to 16 meter along the soil tunnel alignment, see Figure 4. The rock surface is found in the tunnel floor at the entrance point. Approx. 75-80 meter inside the tunnel, the entire tunnel profile consists of rock. Both the terrain and rock surface is inclined (approx. 30°) as illustrated in the cross section in Figure 4.

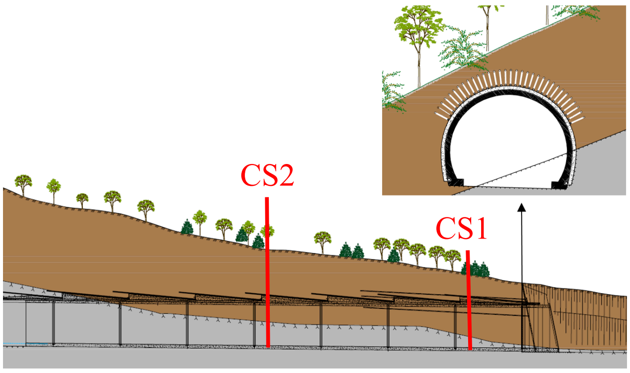

Figure 4: Longitudinal profile and cross section of the entrance point for the soil tunnel at Holven

The deposit is part of the Hollve-deposit consisting of a well graded ground moraine. The moraine is divided in three layers; a bottom layer consisting of a clay rich moraine, a middle layer consisting of a sandy moraine and a top layer of weathered soil and blocks from rock falls. Both the bottom and middle moraine layers are typically defined as gravelly, sandy, silty and clayey material.

Density of the moraine is between 23 and 24 kN/m^2, which in itself indicate over-consolidated material. Water content in the moraine is measured to be around 8-9%, which is considered low. Variations of the tests are between 6 and 16%. In the modelling that was made by iC, friction angle in the moraine between 35 and 38 degrees was used. It has not been possible to get undisturbed samples for laboratory measurements, but prepared samples gave friction angle around 39 degrees under saturated conditions.

The ground water table is typically located 5-10 meter beneath the terrain surface. Hydrogeological investigations indicated local permeable parts in some of the test wells, but no permeable layers in the moraine were discovered during excavation.

The rock mass in the soil tunnel section consists of a cambro-silurian phyllite.

4 EXCAVATION TECHNIQUE AND ANALYSIS

4.1 *Norwegian experience with tunnelling in soil*

Many Norwegian tunnel projects have experienced sections of the tunnel with large weakness zones (“earthlike”) or limited or missing rock mass overburden. Norwegian tunnelers have managed these situations by use of different methods as; pregrouting ahead of face, spiling bolts (normally 6 m long Ø32 mm bars), short excavation rounds and immediate rock support (normally sprayed concrete and bolting).

The first and only(?) tunnel that was planned and excavated in soil in Norway, is a part of the Tyholt rail tunnel in Trondheim. Excavation of around 100 m at the Lerkendal side was excavated in marine clay by use of a closed system of air overpressure (3 bars). The tunnel was opened in 1957. At Eidsvoll a 500 m long rail tunnel in clay was started in 1995, but after a collapse the construction was completed as cut and cover.

4.2 *Tunnelling technique for the Joberget tunnel*

The alignment of the tunnel was more or less fixed due to the connection to the existing road south of the tunnel. A conceptual design was performed where different methods of steel pipe umbrella were considered for establishing a temporary stable situation during excavation. Ground freezing was also considered in an early stage, but since a simpler method seemed to be more promising, the alternative was never evaluated in full.

Contrary to rock tunnelling, certain aspects need particular consideration in soil tunnelling:

- The ground frequently needs some treatment in advance of the excavation process, such as de-watering, grouting or support measures ahead of the tunnel face.
- It is common practice to subdivide the excavation section into top heading, bench and invert instead of a full face excavation. In case of limited stand-up capacity (and time) of the soil, a further subdivision of the top heading face may be necessary (so-called pocket excavation).
- Normally the tunnel lining is structurally designed as a closed ring. In our specific case at the Joberget tunnel the invert is in rock, therefore no structural invert is required.
- The advance length is a very important parameter to control the short-term stability. The advance length is connected with the stand-up time of the soil and the pre-support measures used. Common advance length in soil is some 1.0 m to 1.5 m.
- The perimeter of the excavation advance mostly required additional support measures, such as spiles, forepoles, pipe umbrella or grouting. In case of very cohesive soil it may also be possible to excavate a limited length without such measures.
- The tunnel face may require systematic support, such as face bolting in combination with a layer of mostly reinforced shotcrete. This is necessary to prevent face collapse in major scale and to avoid detrimental effects of moisture causing progressive loosening of the soil in the face.
- The tunnel lining is considered as a structural 3-dimensional support system; therefore, appropriate reinforcement connections at construction joints are considered.

Although tunnelling should take place below the normal ground water level, it was found that the concept with steel pipe umbrella or even the traditional method of 6 m spiling bolts could be used. A prerequisite for a successful performance was to keep control of water seepage into the tunnel. Several ways of draining the moraine were therefore implemented.

4.3 *Numerical modelling*

At the stage of designing this tunnel there was relatively little information available regarding the properties of the moraine. Therefore, a range of properties was estimated by the involved experts and the analyses for the tunnel were consequently with the perspective of a parametric study. The parameter sets included two sets of moraine parameters and two scenarios with gravel lenses contained in the cohesive moraine matrix, see tables 1 and 2.

Table 1: Ground parameters, unfavourable

	Unit weight [kN/m^3]	Poisson`s ratio [-]	Young`s modul [MPa]	Cohesion [kPa]	Friction angle [°]
Moraine	20	0,35	80	15	38
Gravel lenses	20	0,35	80	0	35
Rock	25	0,25	3000	1000	35

Table 2: Ground parameters, favourable

	Unit weight [kN/m^3]	Poisson`s ratio [-]	Young`s modul [MPa]	Cohesion [kPa]	Friction angle [°]
Moraine	21	0,35	200	100	35
Gravel lenses	21	0,35	200	0	35
Rock	25	0,25	3000	1000	35

Two calculation cross sections were chosen to represent typical geological and geometric conditions. Both are selected at positions of the maximum cross section resulting from the pipe roof construction. The positions of the sections are shown in Figure 4.

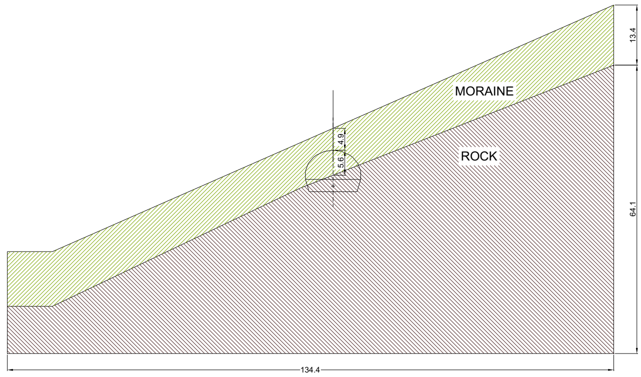

Figure 5: Cross section 1, model geometry

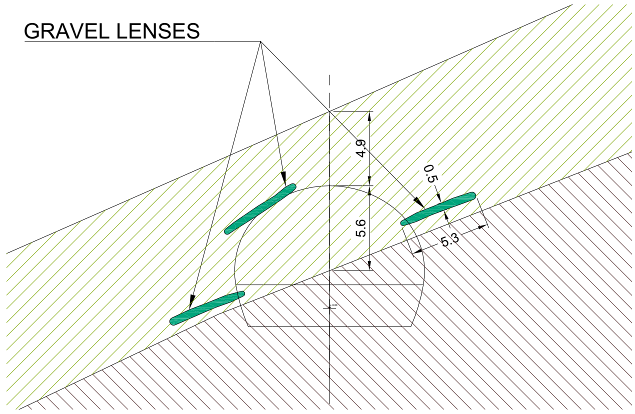

Figure 6: Gravel lenses at CS 1

Cross section 1 (CS1) represents the situation with the tunnel top heading lying almost fully in the moraine layer, see figure 5 and 6. The bench is situated partly in the moraine not allowing for a stiff support from the underlying ground at the top heading excavation step. The overburden is app. 5 m.

Gravel lenses of app. 5 m width and 50 cm thickness were assumed to be located on the top heading section. The lenses are considered with zero cohesion.

In general the required reinforcement for the outer shotcrete lining is determined by linear shotcrete design. Bending moments are overestimated in this procedure. At the worst case scenario with gravel lenses in unfavourable moraine parameters the linear calculation showed critical moments, but a non-linear modelling of the shotcrete lining confirmed the design.

The calculations provided displacement values up to max. 40mm and acceptable lining forces. The more unfavourable scenario was found at the cross section near the portal with low overburden. The assumed gravel lenses did not affect the stability of the lining, however led to higher sectional forces. The applied reinforcement was still sufficient.

4.4 *Face stability*

Face stability was investigated by calculating the stability of potential failure wedges ahead of the tunnel face, see figure 7. Conservatively, the face of the top heading was assumed to consist fully of soil type material. In unfavourable geological conditions, additional support measures such as face sealing with 10cm shotcrete + 1 layer wire mesh and face bolting was considered. The required number of face bolts was calculated within the face stability check. The favourable impact of the pipe roof on the face stability was not taken into account. The calculations showed that the inclination of the rock level does not have an influence on the face stability.

Additional support measures at the face were only required under the most unfavourable conditions. According to the calculations 4 face bolts in the top heading were sufficient.

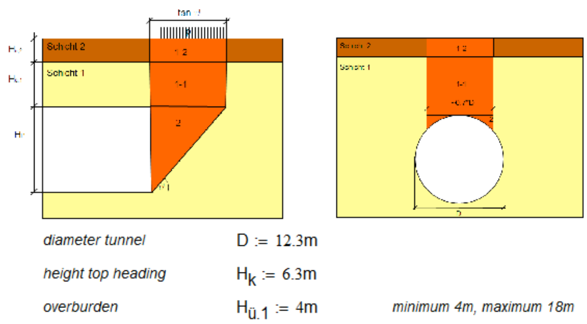

Figure 7: Geometry of failure wedge

4.5 *Design of pipe roof support*

Considering the three-dimensional effects of load distribution ahead of the face, the loads for the pipe roof support were calculated using the equation for the silo earth pressure by Terzaghi. The size of the silo is dependent on the tunnel diameter and the round length. The design loads for the pipe roof are calculated using a beam with rigid connections as static system. Due to the inclination of the pipes, normal forces are causing compressive stresses in longitudinal direction. This favourable stresses are neglected in the design. The partial safety factor for actions is chosen to g_G=1.20, since the pipe roof only needs to act in a temporary construction state. With advancing excavation, the loads from the pipe roof are distributed to the shotcrete lining.

The calculated bending moments were below the bearing capacity of the pipe roof system and the adequacy of the chosen pipe roof system was proved.

4.6 *Inner lining load analysis*

The inner lining was carried out without invert and on both sides the arch is supported by concrete foundations. The inner horizontal diameter of the tunnel is 10 m and the height of the arch is 7.92 m (6.42 m above the gradient). The thickness of the inner lining is 40 cm.

The inner lining is modelled through structural elements (beams) in an integrated Finite Element model comprising the moraine, primary lining and inner lining. A non-linear material was chosen to examine the stresses developed in the unreinforced concrete sections. The contact between primary and secondary lining was modelled by radial elastic springs. The tangential bedding is set to zero due to the fact that the presence of the waterproofing membrane between the inner and outer lining allows no transfer of tangential forces.

In the model the installation of the inner lining takes place at a point of time when the outer lining is carrying the loads. It is assumed that in long-term, the decay of the outer lining might lead to a redistribution of stresses which are then being transferred from the outer to the inner lining. Therefore, in the model the outer lining is deactivated at a certain stage and the transition of all loads to the inner lining is achieved. After that, different load combinations are applied for the definition of loads for the inner lining design. At the present study only loads from rock mass pressure and temperature were considered. Load combinations without rock mass pressure acting on the inner lining had also to be taken into account.

The temperature loads were determined according to Norwegian standards. In the project location a temperature range of max. +31°C and min. -26°C needed to be considered.

The maximum axial forces in the inner lining were found at the connection point between vault and foundation.

In conclusion, the compressive strength of the concrete was not exceeded in any case, while the developed tensile stresses lead to limited crack zones. Therefore, according to the above mentioned calculation analysis an unreinforced lining was considered feasible.

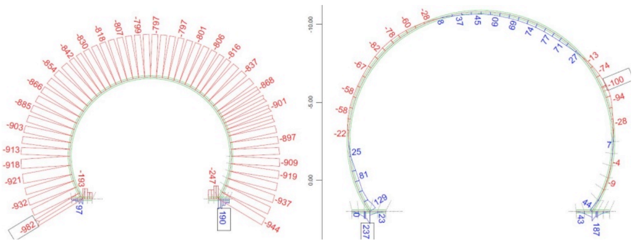

Figure 8: Critical load case, normal force left, bending moments right.

4.7 *Description of excavation method*

The tunnelling method that was finally implemented in the soil section of the tunnel follows the common principles of soft ground tunnelling with sequential excavation methods. The tunnel consists of an outer lining (reinforced shotcrete, 30 cm thick) and a reinforced cast-in-situ final lining 40cm thick, separated by a waterproofing system.

The excavation cross section is split into two main parts: top heading and bench (Figure 9). Following the geotechnical conditions predicted by a considerable number of boreholes, the invert section of the tunnel lies in rock, except a short length at the portal zone where the rock is slightly below invert level. Therefore, no closed invert was foreseen in general.

Considering the expected stiff properties of the moraine, the design concluded that the top heading could be advanced to the end of the soil section (**Feil! Fant ikke referansekilden.** 9). A short bench or even invert follow-up was not considered necessary. As a general rule, the advance of the face was foreseen in short sections, i.e. 1 m each. Lattice girders are installed at each round of advance near the tunnel front. After each excavation step, the primary lining is installed in two layers, the second layer being applied one round behind the first layer. The shotcrete is reinforced by 2 layers of wire mesh.

Although the moraine was considered to be of good strength and stiffness, certain risks had to be dealt with, such as low overburden at the portal area, sand or gravel lenses with poor cohesion and pockets of ground water. To cope with these aspects, a pipe umbrella was foreseen with a length of 15 m and overlap of 3 m. The pipe umbrella in built a 120° angle of the tunnel crown. For geometrical reasons of making a pipe umbrella, a conical section of the primary lining is produced with this method (Figure 9). The excavation and support has to follow the conical position of the pipes. This over-excavation is later filled with sprayed concrete.

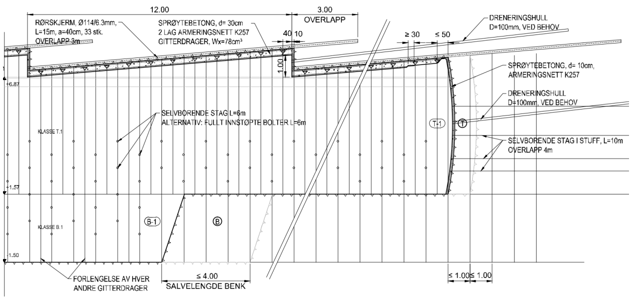

Figure 9: Typical excavation and cross section with top heading and bench

For the scenario of poorly consolidated soil layers in the tunnel face a sub-division of the top heading face excavation in so-called excavation pockets was foreseen (Figure 10).

The excavation pockets are combined with immediate face support using reinforced shotcrete and face bolts with load distribution plates. This method is an adequate tool to increase the stability of the excavation step and to reduce the time lap between opening the excavation and applying the support. The latter is particularly important at water saturated ground conditions, where negative pore pressure provides a short-term stability.

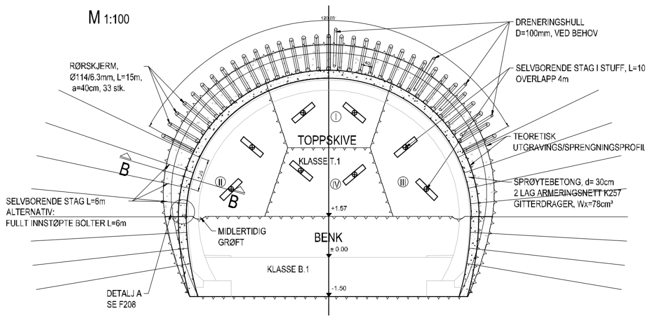

Figure 10: Support Class T.1 / B.1

5 RISK ANALYSIS AND USE OF REFERENCE GROUP

5.1 *Risk analysis*

During the conceptual design phase, a risk analysis was carried out for the construction phase and for the life time of the project. The basis for the analysis was the conceptual design.

The findings showed that water would be the dominating factor related to the risk of flowing ground, wash out of material and subsequent outfall and possible collapse of the tunnel. Even if the concept had implemented a lot of precautions related to the excavation and support sequence, it was found necessary to establish well points from surface in the "upstream" area close to the tunnel.

5.2 *Reference group*

Excavation of a soil tunnel was considered a high risk, in particular since NPRA and Norwegian tunnel engineers have limited experience with this type of tunnelling. A reference group was therefore established already in the conceptual design phase, spring 2013. Members of the group were:

Professor Bjørn Nilsen, NTNU
M.Sc. Anders Beitnes, WSP Norge AS
Ph.D. Roger Olsson, NGI.

The reference group followed the design and in particular the start-up of construction work. They gave valuable input to design, risk assessment and stressed the importance of the possibility of taking the right decisions at the right time during construction.

6 CONSTRUCTION PHASE

The excavation works in the open cut at Holven started in the September 2015 and was finished in February 2016. Soil tunnel excavation started in February 2016 and was successfully finished late May 2016. The total length of the soil tunnel is 100 meters.

6.1 *Ground conditions*

The ground conditions were as expected. In general, the phyllite was of good quality and little affected by weathering. The contact between moraine and rock showed up to be very good, with no weathering of the rock and with no permeable layers in the moraine. The contact was therefore found to be without leakages in all sections. The moraine consists of two types, see Figure 11:

- Grey moraine: Bottom layer of consolidated and clayrich moraine.
- Brown moraine: Upper layer of sandy and less consolidated moraine. Present in the upper left side of the tunnel profile in the first 50 meter of the tunnel.

During excavation, only minor leakages have occurred at face, typically in the brown moraine.

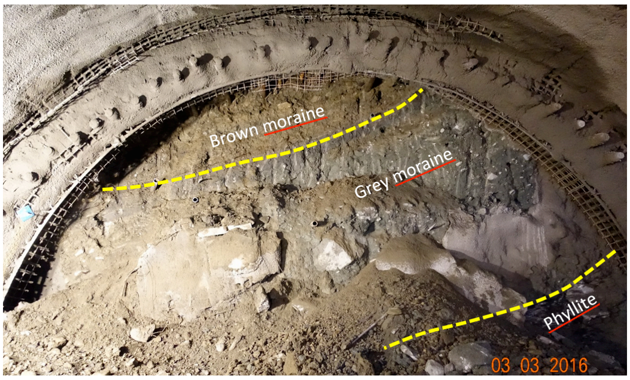

Figure 11: Face approx. 10 meter inside the tunnel.

6.2 *Excavation and equipment*

For excavation, the contractor has used a regular excavator with different types of equipment (shuffle, ripper and hammer).

Figure 12: Drill head for steel pipes. Only the yellow drill part is retracted

Rig for spraying concrete and concrete car have always been present at the face during excavation in case of stability problems. A regular tunnel jumbo has been used for drilling & blasting and installation of rock/soil anchors. Only small modifications on the jumbo were necessary to make it able to drill steel pipes. The drilling equipment for pipes is shown in Figure 12.

6.3 *Deformation measurements*

Two methods have been used to monitor deformation inside and outside the tunnel:

1. Convergence measurements:
    a. Tunnel profile. An example of deformation plot is illustrated in Figure 14.
    b. Terrain along the tunnel axis
2. Horizontal inclinometer (30 meter long) installed from the portal wall, above the tunnel along the tunnel axis. Since the inclinometer was installed before the tunnel excavation, the readings show the total deformation.

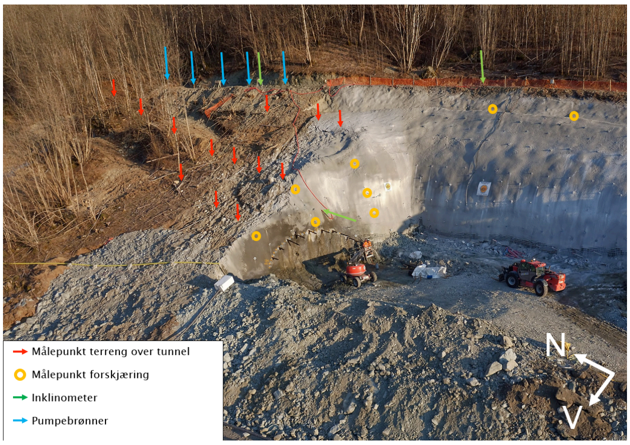

Figure 13: Location of measuring points, wells and inclinometers

Plots from the horizontal inclinometer show that the deformation typically starts 5-6 meter in front of the face, and stabilizes 5-6 meter behind the face. The total deformation registered by the inclinometer in the first part of the tunnel is around 28 mm. Convergence measurements in the same section show typically 13-14 mm vertical deformation. This indicates that half of the deformations already have occurred before the convergence profiles are installed.

Plotting of the deformations and construction stages together, is important when evaluating the results.

6.4 *Daily follow up and routines*

Personnel from Sweco and iC Consulenten have been responsible for the daily follow up on site during construction. An important part of this work has been a daily meeting with the contractor in the tunnel and close to the face. This made it possible to discuss and decide immediately the necessary additional mitigation measures based on the results from deformation measurements, stability at the face and general observations in the tunnel.

7 CHALLENGES AND SOLUTIONS

7.1 *Excavation of the open cut*

During excavation of the open cut, a trench and a small construction road was wrongly excavated in the terrain above the tunnel. This reduced the overburden from originally 2,5-3,0 meter to 2,0-2,5 meter in local areas. It also reduced the strength of the soil in the first part of the tunnel.

In addition to dividing the profile in several sections in this area during excavation, additional steel pipes and soil anchors was installed to support this part of the tunnel profile. The excavation went as planned without any major stability problems.

7.2 *Drilling of steel pipes*

The steel pipes in the umbrella are 15 meter long and consists of five pipe sections with length 3 meter. Except for the first pipe which has a sacrificial drill bit, each pipe is threaded in both ends and screwed together during drilling.

Seven umbrellas have been drilled in the moraine. The majority of pipes have reached 15 meter and several pipes have been drilled trough granite boulders. It was experienced that some of the pipes cracked in the connection point between the pipe sections during drilling in certain areas of the moraine probably due to reduced steel thickness in the connection point.

Where 15 meter length was not achieved, or where deviation during drilling caused pipes to either move in to or out of the tunnel profile, regular self drilling spiles have been used to maintain an even profile during excavation.

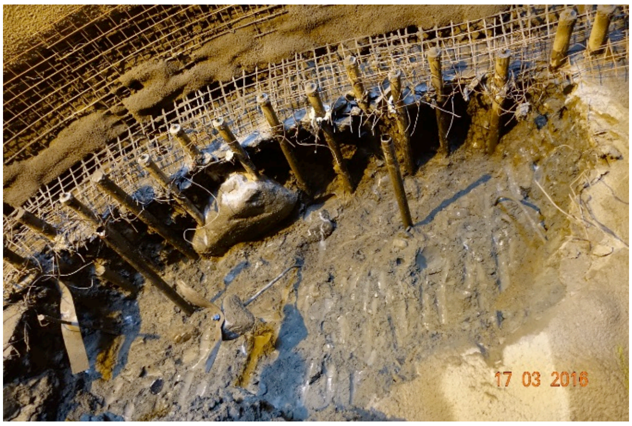

Figure 14: Steel pipes drilled through a boulder

7.3 *Face stability*

The moraine has consisted of a grey and brown layer as described above. The stability in the grey moraine can be characterized as good. Excavation of this part of the face has been more or less in one section and only supported by a thin layer of sprayed concrete and sporadic use of face bolts. The brown moraine was less cohesive and gave some water seepage. Therefore it has been necessary to divide the brown moraine into 2 to 4 sections during excavation. Face support in terms of wire mesh, sprayed concrete and face anchors have been more comprehensive in this part of the moraine. See Figure 15.

Figure 15: Grey moraine in the right side is excavated in one section. Brown moraine in upper left side is supported by sprayed concrete, wire mesh and face bolts.

Another important routine was always to have a spraying rig and concrete car present at the face during excavation. Within minutes, sprayed concrete could be applied, if necessary.

7.4 *Water leakage*

The biggest concern related to instability during excavation, was water leakage. Permeable layers and lenses in the moraine or leakage at the interface between rock and soil could potentially cause major stability problems. Of that reason, several measures to drain the moraine in front of the face were executed during the construction phase:

- Five pumping wells were installed from the terrain along the tunnel alignment. The wells were equipped with a pump, which was operative during the entire construction phase.
- Perforated drainage pipes with length 5-30 meters were installed from the open cut, face and between steel pipe umbrellas to drain the moraine in front of the face and above the tunnel.

In general, the moraine and rock mass in the tunnel have been dry. Only minor seepage in the brown moraine, drainage pipes and steel pipe in the umbrella have occurred.

7.5 *Blasting*

The top heading was excavated in 1-meter steps. This means that the rock also had to be blasted in 1-meter steps, preferably without getting overbreak. An overbreak into the face could potentially cause downfalls from the overlying moraine. This proved to be challenging, but was handled with careful blasting and in many cases by chiseling.

8 LIMITATIONS OF THE METHOD

Excavation of the Joberget soil tunnel has been performed without facing critical problems related to stability and inflow of water. Challenges for the performance were excavation below the ground water table, uneven overburden of soil, limited knowledge of soil and rock permeability and stiffness parameters of the soil. The construction phase revealed very good stability of the moraine and only few and minor seepages into the tunnel.

However, it cannot be concluded that we have found a general method that can be used for soil tunnels in Norway. The stability of the soil in the tunnel face and above the tunnel will remain the largest challenge, and only soil that can be kept stable long enough so that proper support can be installed, can be excavated with the steel pipe umbrella concept.

Sand/gravel and also silty material with low or no cohesion will easily run into the tunnel when it is exposed. If running water (or high water table) is combined with this type of material, steel pipe umbrella alone will not function.

9 INNER LINING

The normal solution that is used in Central Europe for the design and construction of soil tunnels is to have an inner lining of cast concrete with membrane/filter in direct contact with the primary support. This solution was also implemented here. The membrane is a type II membrane with minimum thickness of 2 mm according to table 8.2 in N510.

The design was to have a 400 mm unreinforced cast concrete placed on a concrete foundation. Due to regulation in the NPRA, a minimum amount of reinforcement had to be installed. The construction is without frost protection other than the concrete itself and the draining textile between the membrane and the sprayed concrete. To gain more experience related to how the frost goes into the construction, both Ulvin and Joberget tunnel are instrumented with temperature probes.

10 CONSTRUCTION TIME AND COST

10.1 *Construction time*

After excavation of the open cut and support of the walls with sprayed concrete and anchors, a concrete arch was installed above the tunnel entrance and the first steel pipe umbrella was installed. When the top heading started in mid-February 2016, it took 3 months for completion. The contractor worked 24/6 with 3 shifts of 8 hours. When excavating the first 10 meters, the cross section was divided into 4-5 sections and immediate support was installed after each excavation. Rate of excavation was approx. 0.5 m/day.

On the next 40 m larger parts of the face were excavated before support and rate increased to 1 m/day. Between chainage 50 and 100 m the stability of the soil allowed an increase of distance between lattice girders to 1.5 m. Excavation rate increased to 2 m/day.

Benching of the whole length of 100 m took approx. 2 weeks.

In total, the excavation of the top heading and bench took 3 months and 1 week. This equals an average construction time of 1 meter/day. This was according to the estimate made during planning of the work.

10.2 *Construction cost*

The actual cost for the excavation and primary support of the soil tunnel was NOK 300.000,-, which is less than the estimate.

The cost of the inner lining, cast in place concrete of 400 mm incl. membrane and reinforcement will be approximately NOK 100.000.-/m. For the complete tunnel with technical installations, the cost for the tunnel in soil is approx. NOK 425.000.-/m excl. of VAT.

The total amount of sprayed concrete used both at the face and for filling of the conical shape of the tunnel was about 34% above the calculated volume. Filling up of the conical shape of the tunnel due to the drilling of pipes was approximately 8 m^3/m. There could be large savings if the angle of the cone could be reduced and at the same time accuracy of drilling improved.

Due to good quality of the moraine during the whole drive, less bolts were used and the number of pipe for drainage was also reduced.

11 CONCLUSION

The first planned soil tunnel the last 60 years in Norway was successfully completed when the Joberg tunnel was excavated 2016. The success was due to extensive preinvestigations, good planning and design and also thank to a experienced contractor that followed the instructions given by the consultants.

The soil conditions proved to be better than expected, which meant that the possible precautions that were built in to the design, did not all come in to use.

A lesson learned must also be that the next soil tunnel in Norway cannot just copy the Joberg tunnel design, but take benefit from the good planning and execution of work.