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Analysis of Potential Problems Related to Water Inflow for TBM tunneling at Røssåga Hydropower Project

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MASTEROPPGAVEN

Kandidatenes navn:	Agnete Hoff Finnøy
Oppgavens tittel:	Analyse av mulige vannproblemer for TBM tunnel ved Røssåga kraftanlegg
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Utfyllende tekst:

1. Ved Nedre Røssåga er den ca. 7,5 km lange tilløpstunnelen planlagt drevet med TBM, og Røssåga prosjektet vil dermed få den første norske TBM-tunnelen siden 1993. Prosjektet ligger innenfor kambrosilur i Nordland, og i et område med mye kalkbergarter og mulighet for å treffe på karst. I denne masteroppgaven skal det foretas en ingeniørgeologisk analyse av mulige vannproblemer langs de planlagte TBM-tunnelene, med spesiell vekt på diskusjon og vurdering av faren for å treffe på karst og eventuelle problemer som dette vil kunne medføre for tunneldrivingen. Feltbefaring skal gjennomføres i den grad forholdene tillater det, men årstiden tatt i betraktning vil en stor del av arbeidet måtte baseres på gjennomgang av foreliggende grunnlagsmateriale og ved litteraturstudier. Følgende momenter fremheves som spesielt sentrale for oppgaven:

- Diskusjon av mulige metoder for avdekking av karst på forundersøkelsesstadiet og under driving.
- Vurdering av faren for å komme i berøring med karst ved Røssåga.
- Gjennomgang og diskusjon av erfaringer med tidligere tunnelprosjekter i karstområder i Nordland (både TBM- og konvensjonelt drevne).
- Innsamling og diskusjon av erfaringsdata fra internasjonale TBM-prosjekter i karstområder.

På grunnlag av datainnsamling og analyse av forholdene nevnt ovenfor skal det utarbeides en prognose for antatte innlekkasjeproblemer for TBM-tunnelen ved Røssåga. Mulige konsekvenser av karst for driving av TBM-tunnelen skal diskuteres og mulige tiltak i tilfelle karst påtreffes skal vurderes.

2. Oppgaven gjennomføres i samarbeid med Statkraft med byggeleder Erik Dahl Johansen som kontaktperson og Sweco Norge med ingeniørgeolog Bent Aagaard som kontaktperson.

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Bjørn Nilsen, Professor/hovedveileder

SKJEMAET TAS INN SOM SIDE 1 I MASTEROPPGAVEN
NTNU, 15 januar 2014

Foreword

This master thesis has been conducted at the Department of Geology and Mineral Resources Engineering at the Norwegian University of Science and Technology (NTNU) and it represents my final work in my master´s degree in Engineering Geology and Rock Mechanics. The thesis has been written in collaboration with Statkraft and Sweco.

I would in particular like to thank my supervisor Bjørn Nilsen for excellent guidance during the work with the thesis. I would also like to thank my co-supervisors, Erik Dahl Johansen from Statkraft and Bent Aagaard from Sweco, for providing information and answering my questions. I would like to thank Erik Dahl Johansen and rest of the crew at Røssåga for showing me around at the project area during my site visit at Røssåga.

Thank you to my classmates who have been keeping me with company at school and to my friends and family for support during the work with my thesis.

Trondheim, 10th of June 2014

Agnethe Hoff Finnøy

Sammendrag

Den nye tilløpstunnelen ved vannkraftverket ved Nedre Røssåga vil bli drevet med TBM og Røssåga prosjektet vil derved få den første norske TBM-tunnelen siden 1993. Røssåga ligger i Nordland hvor det er mye kalkbergarter og derved fare for å møte på karst og vannlekkasje under driving. Det er derfor foretatt en ingeniørgeologisk analyse av mulige vannproblemer i tilløpstunnelen. Analysen bygger på en vurdering av faren for å møte på karst ved Røssåga, gjennomgang og diskusjon av erfaringer med tidligere prosjekter i karstområder for både internasjonale og nærliggende prosjekter, og en gjennomgang av de ingeniørgeologiske forholdene ved Røssåga.

Mulige metoder for å avdekke karst under forundersøkelser og under driving er diskutert. Det er veldig vanskelig å forutsi karst på grunn av kompleksiteten til denne type berggrunn. Fokuset under forundersøkelsene burde derfor være på å fremskaffe nok informasjon til å kunne forutsi grunnforholdene slik at en grundig planlegging av tunnelen kan utføres. Kartlegging i terrenget av karst strukturer for så å utføre borehull i kritiske seksjoner kan brukes for å forutsi karst på tunnelnivå. Siden karst er veldig vanskelig å avdekke før driving er det anbefalt at man utfører undersøkelser underveis i drivingen. Sonderboring er en mulig metode for å detektere karst under driving, men kan heller ikke sies å være en sikker metode for å avdekke karst.

Det er gjort en vurdering av faren for å møte på karst i tunnelen. Karst kan forekomme i nesten alle kalkholdige bergarter. For tilløpstunnelen vil dette si at karst kan forekomme i kalkstein, marmor, kalkskarn, glimmerskifer og glimmergneis. Det er observert karst på overflaten i området og karst har forekommet i andre tunneler i det samme geologiske området, men det er ikke mulig å si sikkert om karst vil forekomme i tunnelen ut i fra informasjonen som til nå er tilgjengelig. For å ha en mulighet til å avgjøre dette er det nødvendig å utføre flere undersøkelser. Det som er sikkert er at mulighetene for å møte på karst er tilstede og at det er seksjoner i tunnelen som har større sannsynlighet for å møte på karst enn andre. Dette har blitt tatt hensyn til i prognosen.

En prognose har blitt uført for TBM-tunnelen for sannsynligheten for å møte på vannlekkasjer i tunnelen. Tunnelen er delt inn i seksjoner og vurdert med tanke på sannsynligheten for vannlekkasje på en skala: Veldig liten, Liten, Moderat, Stor og Veldig stor sannsynlighet. Vurderingen er gjort ut i fra sannsynligheten for karst og kunnskap om andre ingeniørgeologiske forhold som kan føre til vanninnlekkasje. Én seksjon med marmor er vurdert til å ha Høy sannsynlighet for vannlekkasje og en seksjon med glimmerskifer er vurdert til Moderat sannsynlighet. De to siste seksjonene går gjennom forskjellige bergarter og er vurdert til Lav og Lav-Moderat sannsynlighet for vannlekkasjer. Den største vanninnlekkasjen er forventet i sammenheng med karst.

Mengde vannlekkasje hvis karst blir påtruffet er veldig usikkert, men erfaringer tilsier at det kan forekomme *ekstremt høy* vannlekkasje. Små til moderate vannlekkasjer langs tunnelstrekningen kan forekomme i forhold til åpne sprekker og i svakhetssonene.

Mulige konsekvenser av karst for driving av TBM tunnelen er diskutert med hensyn til vannlekkasje. Det er en åpen TBM som brukes ved Røssåga. TBM-en tåler mye vann og det er en vannpumpe tilgjengelig. Den mest sannsynlige konsekvensen for vanninnlekkasje i tunnelen vil være dårlige arbeidsforhold med vann og løsmasser spredd utover i arbeidsområdet. Det kan også bli vanskelig å utføre sikring når bergmassen er våt og ustabil. En åpen TBM, slik som det er brukt ved Røssåga, er spesielt utsatt for slike forhold fordi den ikke har noe skjold og er åpen mot bergmassene i arbeidsområdet. Tiltak for å stanse vannlekkasjer, slik som forinjeksjon og lignende, vil gå utover produksjon og fremgang i tunnelen.

Tiltak for å unngå problemer med karst kan være sonderboring for så å utføre for-injekson hvis karst blir detektert. Å sonderbore og utføre for-injeksjon er anbefalt som den beste metoden for å hindre vannlekkasjer, også for TBM. Ved Røssåga vil kun ett sonderborehull bli utført på stuffen for hver runde. Mulighetene for å detektere karst foran stuff ved bare å utføre ett sonderborehull er små. Det anbefales at det utføres flere undersøkelser for å avdekke spesielt kritiske seksjoner med hensyn til karst og at det utføres mer grundig sonderboring gjennom disse seksjonene. På denne måten opprettholdes det en god balanse mellom fremdrift og undersøkelser. Også flere borhammere er mulig å installere for å øke effektiviteten til sonderboringen. Generelt er det viktig å ha tiltak planlagt for alle forventede eventualiteter, og at det er tilgjengelig nødvendig utstyr og folk med erfaring fra lignende forhold.

Summary

The new headrace tunnel at the water power station at Røssåga will be constructed with TBM and the project will thereby have Norway´s first TBM-tunnel since 1993. The tunnel is located in Nordland with a lot of calcareous rock types and the risk of encountering karst and water inflow is therefore present. It has therefore been performed an engineering geological evaluation of the possible water problems in the headrace tunnel. The analysis is based on a review of the possibility of meeting karst in the tunnel, review and discussion of experiences with former tunnel projects in karst areas for both international projects and projects from Nordland, and a review of the engineering geological conditions at Røssåga.

Possible methods for detecting karst during pre-investigations and during construction have been discussed. Karst ground is very complex ground to investigate. The focus during pre-investigation should therefore be on providing enough information to be able to predict the ground conditions so appropriate planning for the tunnel construction can be done. Surface mapping of karst features and boreholes in critical sections can be possible methods for predicting karst at tunnel level. Since karst is difficult to predict before excavation it is important that investigation during construction is performed. Probe drilling during excavation is a possible method for detecting karst, but karst can be difficult to reveal also with this method.

A review of the possibility of encountering karst has been performed. Karst can occur in almost all rock types that are calcareous. For the headrace tunnel this means that karst can occur in limestone, marble, calcareous skarn, mica schist and mica gneiss. Karst is observed in some places along the tunnel alignment and has occurred in tunnels in the same geological region, but from the information that is now available it is not possible to conclude with whether karst will occur or not in the tunnel. It is necessary to perform more investigations to give a more certain answer to this. What can be said for sure is that the possibility of karst in the area is present and some sections along the tunnel have higher probability of encountering karst than the others. This is taken into consideration in the prognosis.

A prognosis has been performed for the possibility of encountering water inflow in the tunnel. The tunnel is divided into four sections and evaluated on a scale of: Very small, Small, Moderate, Large and Extremely Large possibility of encountering water inflow. The evaluation is done with respect to the possibility of encountering karst and the other engineering geological conditions that can lead to water inflow. One section with marble is evaluated to have Large possibility of water inflow and a section with mica schist is evaluated to Moderate possibility. The two last sections consist of different rock types

and are evaluated to have Low and Low-Moderate possibility. The largest amount of water inflow is expected to be in relation to karst. The amount of water inflow that can occur if karst is encountered is very uncertain, but experiences indicate that extremely large water inflow can occur. Small to moderate water inflow along the tunnel alignment can occur though open joints and in weakness zones.

Possible consequences of karst for the construction of the TBM-tunnel are discussed with respect to water inflow. An open TBM is used at Røssåga. The TBM can stand a lot of water and it is a water pump available. The most likely consequence for water inflow into the tunnel will be poor working conditions with muck and water spreading out in the working area. Water inflow can cause instable conditions and difficulty with installing rock support. An open TBM, which are used at Røssåga, is particular sensible to these conditions because the machine does not have a shield and is open to the rock mass in the working area. Implementation of measures against water inflow is time consuming and will hamper the progress of the TBM.

Measures against karst and water inflow can be probe drilling and implementing pre-injection if karst is detected. This is recommended as a method for stopping water inflow, also for TBM. In the TBM-tunnel at Røssåga only one probe hole will be drilled in the tunnel face at the time. The possibility of detecting karst with one probe hole is small. It is recommended that more investigations is performed to reveal critical sections with respect to karst and that more thorough probe drilling will be performed in these sections. In this way a good balance is maintained between progress of the tunnel and the investigations. Also more drill hammers can be installed, to be able to perform probe drilling more efficiently. Generally, it is important to have planned measures against all possible eventualities, and that the necessary equipment is available and staff with experience from similar conditions

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1. Introduction

This chapter will give an introduction to the hydropower plants and the rehabilitation in progress at Røssåga. It will state the scope and limitations of this master thesis and give a quick summary of the background material that is used and investigations that is performed during the work with the thesis.

1.1 Introduction to Røssåga hydropower plants

The Røssåga hydropower plants are situated in the municipality of Hemnes in the county of Nordland in Norway. They were constructed in the 1950´s due to the need for power to the ironworks in Mo i Rana, regular power-supply to the municipalities and the electrification of the Nordlandsbane. Between Røssvatn and the ocean the fall of the water is utilized in two power plants: Upper and Lower Røssåga. “Stormyrbassenget” is an artificial basin that is constructed in between these two power plants and it works as an intake reservoir for the Lower Røssåga hydropower plant (Statkraft, 2010). An overview of the Røssåga hydro power plants is shown in Figure 1.

When Lower Røssåga was constructed it was installed six aggregates with capacity of 43,5 MW each. In 2010 Statkraft decided to rehabilitate three of the existing aggregates at Lower Røssåga. In 2012 it was decided to take out of operation the remaining three aggregates and to build a new underground power station with an aggregate of 225 MW and new water tunnels. In this way the capacity of Lower Røssåga will increase with approximately 100 MW. For Upper Røssåga it was decided to build a new tailrace tunnel with a length of almost 5 km.

The rehabilitation of both Upper and Lower Røssåga includes in total 19,5 km of tunnels. Statkraft invited for tenders of the rehabilitation of Upper and Lower Røssåga in spring 2012. It was planned to excavate the tunnels with the traditional drill and blast method, but Statkraft was also open for alternative solutions. The headrace tunnel of Upper Røssåga and the tailrace tunnel of Lower Røssåga were planned to have a cross section of 50 m^2. The entrepreneur LNS (Leonard Nilsen & Sønner) considered the project as suitable for use of TBM. They contacted the firm “The Robbins Company” which had a suitable TBM for the project with diameter of 6,7 m. Because of smoother surface of the tunnel and less friction the cross-sectional area of the tunnel can be reduced in TBM-tunnels (Nilsen & Log, 2013).

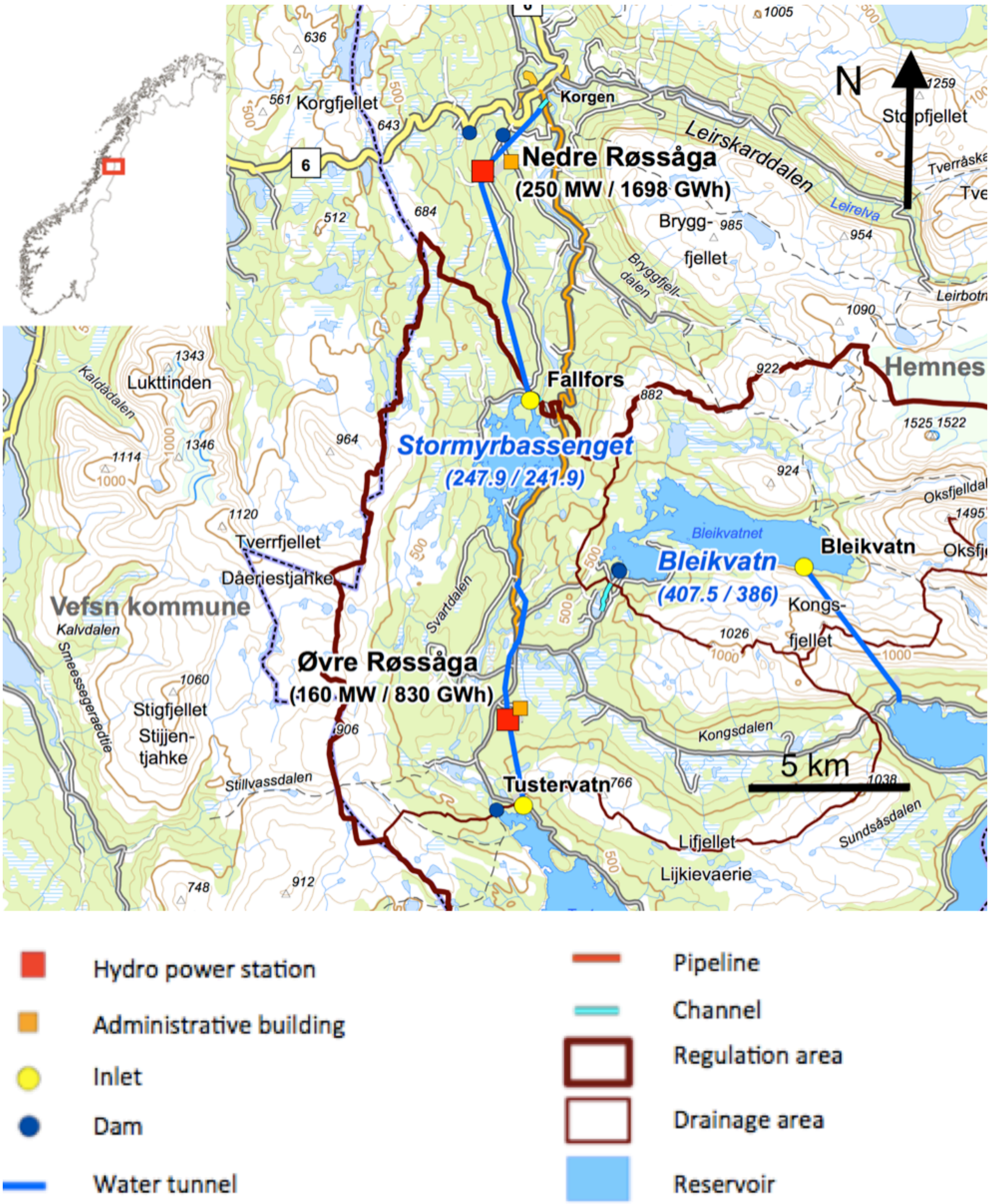


Figure 1. Overview of the hydro power plants at Røssåga (Statkraft, 2010). Modified by author.

The Norwegian expertise for full face boring has decreased over the years. To give a good alternative for use of TBM, LNS engaged AMH Consult, one of the leading

Norwegian TBM expert consultant companies, to be a part of the calculation team. LNS gave an alternative solution for using TBM in the headrace tunnel at Lower Røssåga and for the tailrace tunnel at Upper Røssåga. Statkraft did some changes during the negotiations and wanted the prize for the expansion of the tunnel cross sections to 65 m^{2}. LNS introduced a new TBM with a diameter of 7,23 m to meet this demand. In late fall of 2012 Statkraft and LNS signed an agreement. The contract was prepared in such a way that TBM was an option (Nilsen & Log, 2013). It was then decided to use TBM in Lower Røssåga and during fall of 2013 the plans for use of TBM in Upper Røssåga were rejected. The new tailrace tunnel of Lower Røssåga will be 3,6 km when completed and the construction will be done by drill and blast method. TBM was chosen in front of conventional method, due to among other things savings of tunnel meter, reduction of the need for rock support and to bring TBM-competence back to Norway (Nilsen & Log, 2013).

The construction started in March 2014, and by May 2014 (during site visit) the TBM was located 370 m into the surge chamber and 60 meters from where it will start to excavate the TBM-tunnel. The first 60 meters of the surge chamber was excavated with drill and blast method. The TBM machine was then established outside of the surge chamber and brought into the tunnel to start to excavate the rest of the surge chamber and from there continue with the headrace tunnel. The completed headrace tunnel of Lower Røssåga will have a length of 7560 m, and a cross section of 41 m^2 (Nilsen & Log, 2013).

Figure 2 shows the locations of the new and old headrace and tailrace tunnels at Lower Røssåga and for the water power stations.

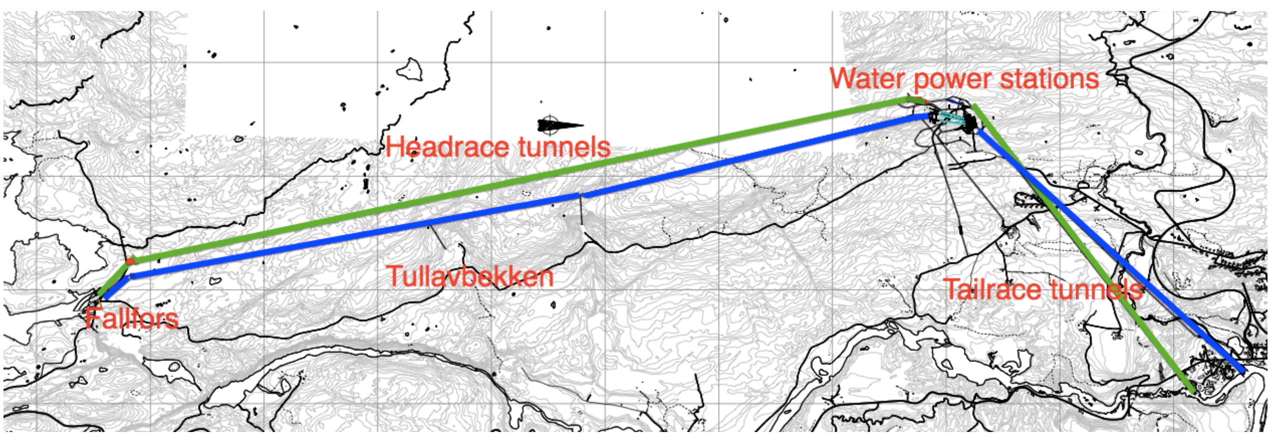


Figure 2. A set up for the water power plant of Lower Røssåga. The new headrace tunnels and the tailrace tunnel are marked in green, while the old tunnels are marked by blue. The map was provided by Statkraft and is modified by the author of this thesis. Scale of the map is not available.

The TBM that will be used at Røssåga is an open TBM, which means that the progress is achieved by pushing the gripper shoes to the tunnel wall and not against segments. The TBM has only a front shield and is elsewise open to the rock mass in the tunnel.

Conventional rock support is therefore used in higher degree than with shielded TBM´s. Open TBM has been used in all former TBM projects in Norway (Nilsen & Log, 2013).

1.2 Objective and scope of the study

The Røssåga hydropower station is located in Cambrosilurian in Nordland, in an area with a lot of calcareous rocks and possibility of encountering karst. Karst can among other problems cause major water leakage when excavating tunnels. It is therefore important to take this into consideration when planning a tunnel in this region. In this master thesis a prognosis for the assumed water leakage problems for the TBM-tunnel, meaning the headrace tunnel, at Lower Røssåga will be prepared. The thesis will in particular focus on the following:

- Discussion of possible methods for detection of karst during the pre-investigations and during excavation: Relevant literature and conversation with professionals on this topic will be used to gather information on this issue.
- Evaluation of the risk of encountering karst at Røssåga: Information from the pre-investigations for the TBM-tunnel, experiences from site visit, experiences from relevant tunnel-projects and other relevant literature will be used to evaluate the risk of encountering karst in the TBM-tunnel at Røssåga.
- Review and discussion of experiences with former tunnel projects in karst areas in Nordland (for both TBM and conventional excavated tunnels). A review based on the literature study of the excavation of the tailrace tunnel at Upper Røssåga and the Staupåga diversion tunnel will be performed. These projects are both chosen because they encountered water problems due to karst during the excavation and the experiences from these projects will be used to answer the issue of the master thesis. It has not been possible to find any other reports of tunnel-projects with karst-problems in Nordland.
- Collection and discussion of experiences with international TBM-projects in karst areas. A literature study has been conducted regarding the Kuhrang water transmission tunnel and the Alborz service tunnel, both situated in Iran, and the SMART-project in Malaysia. They are all constructed in karst-areas. The tunnels of Kuhrang and Alborz both faced problems with water inflow during construction due to karst. The SMART-tunnel faced concerns regarding karst and water leakage prior to and during construction, and had to be carefully planned and excavated to avoid incidents with karst and water inflow. Together these projects give useful experiences on the issue of water inflow when tunneling in karst.

A prognosis for the assumed water inflow into the TBM-tunnel will be prepared on the basis of the analysis above, data collection and the engineering geological conditions along the tunnel alignment. The possible consequences for encountering karst in the TBM-tunnel and possible measures if karst is encountered will be considered.

1.3 Limitations of the master thesis

The literature study of the tunnel projects in Nordland and the international tunnel projects are limited by the information that is available from the sources. It has been challenging to gather information regarding former tunnel projects in karst areas and some information that would be natural to include regarding the chosen tunnel projects are therefore missing.

It has not been possible to do an extensive field mapping due to the seasonal conditions at the site and time limitation. This will of course be a limitation to the prognosis for water inflow. However, experiences and observations from the personnel in Statkraft and Sweco, regarding the geology and karst, were gathered during this site visit to supplement the information from the background material. Still, some of the information regarding the geology, which would have been useful, has not been possible to gather.

1.4 Background material

The following background material are used in the work with the master thesis:

<u>Papers and reports on the engineering geological conditions of Lower Røssåga:</u>

The geological report on Røssåga is given in the tender documents (Statkraft, n.d.). For planning of the new tunnels and power station at Lower Røssåga it was performed engineering geological field mapping by Sweco during October and November 2010. Seismic refraction measurements was also performed downstream of the intake and by the outlet of the tailrace tunnel to get more information about the geological conditions like the depth to bedrock and rock mass quality. It is also performed two core drillings from the surface nearby the power station. The results from these pre-investigations are summarized in the geological report and are applied in this master thesis to make an engineering geological description of the tunnel alignment. The geological maps and profile of the tunnel alignment, which is used in the thesis, is provided from this report and is given in Appendix 5 and in figure 4. Some papers on the construction of the existing tunnels at Røssåga are also included in this report.

A report with description of the incident with karst at Fallfors is given written by Aagard (2013).

<u>Maps from NGU:</u>

Geological map from NGU (1:50000, 1:250000) are used to provide a more extensive description of the rock types in the area.

<u>Conversations:</u>

Conversations with Bjørn Nilsen, Bent Aagard, Erik Dahl Johansen and other personnel of Statkraft and Sweco.

Various materials:

Reports and papers from the construction of existing tunnels in Nordland.

1.5 Experiences from site visit

During 7th and 8th of May 2014 a site visit to Lower Røssåga hydropower station was done by the author of this thesis. The visit was mainly done to do an inspection of the TBM-tunnel, but also to talk to key-people and to do some field-investigations if possible.

A guided tour on the site was done to ensure a good overview of the project. Karst had already been experienced by the new intake to the tunnels at Fallfors, and a short tour was done to see this. During the site visit the TBM-tunnel was situated in the surge chamber on its way down, about 60 meters from where it will start to excavate the tailrace tunnel. The visit in the TBM-tunnel was done with the guidance from the construction manager at the site (Erik Dahl Johansen) and an engineering geologist from Sweco. The geology of the tunnel was inspected and discussed and some Q-values were set as well as some strike and dip measurements. Mapping of karst features and geology in the field were not done due to snow and other practical reasons.

The geology and possibility of encountering karst at the site was discussed with the construction manager and engineering geologist. They had some useful experiences and observations of the geology in the area and in particular regarding karst. The influence of water regarding TBM and the possibility of detecting karst during construction as well as pre-injection from the TBM was discussed with the operation manager of the TBM from LNS. He has experiences from other TBM-projects like Hallandsåsen, which experienced problems with water inflow during construction.

2 The engineering geological conditions of Lower Røssåga

To be able to give an engineering geological analysis of possible problems with water inflow along the TBM-tunnel it is necessary to give a presentation of the expected engineering geological conditions. The experiences from the site visit are also implemented in this chapter. The regional geology of Nordland will first be presented to provide an overview of the geology in this region.

2.1 Regional geology

The major part of Nordland consists of Caledonian rock types. Characteristic of the area is a complicated tectonic with several nappes and several phases of folding. A typical feature for this region is the thick layers of marble where karst occurs frequently. As seen in Figure 3, the nappes of Nordland is divided into three main units:

- The Köli Nappe included the Gasak nappe and the Fauske nappe**:** igneous and sedimentary rocks dominate the Köli Nappe.
- The Rödingsfjället Nappe Complex: It is situated above the Fauske-, Gasak- and the Köli Nappe and extends in a belt from Røssvatnet in the south to Fauske in the north. In Rana the Rödingsfjäll Nappe consists of the Beiarn Nappe, which is mostly composed of granitic gneiss above seven smaller nappes of calcite- and dolomite -marble. Mica schist also occurs, for example in Rana, which locally contains sedimentary iron ores. The hydropower plants of Røssåga are situated in the The Rödingsfjället Nappe Complex.
- The Helgelands Nappe Complex: This Nappe is the upper layer of the Nappe sequence in the southwestern part of Nordland and Nord-Trøndelag. The Helgelands Nappe consists of mica schist and mica gneiss in addition to huge granitic intrusions. The large proportion of intrusive rocks like granites and granodiorite is characteristic for the Helgelands Nappe (Bryhni, Nøttvedt, & Ramberg, 2006).

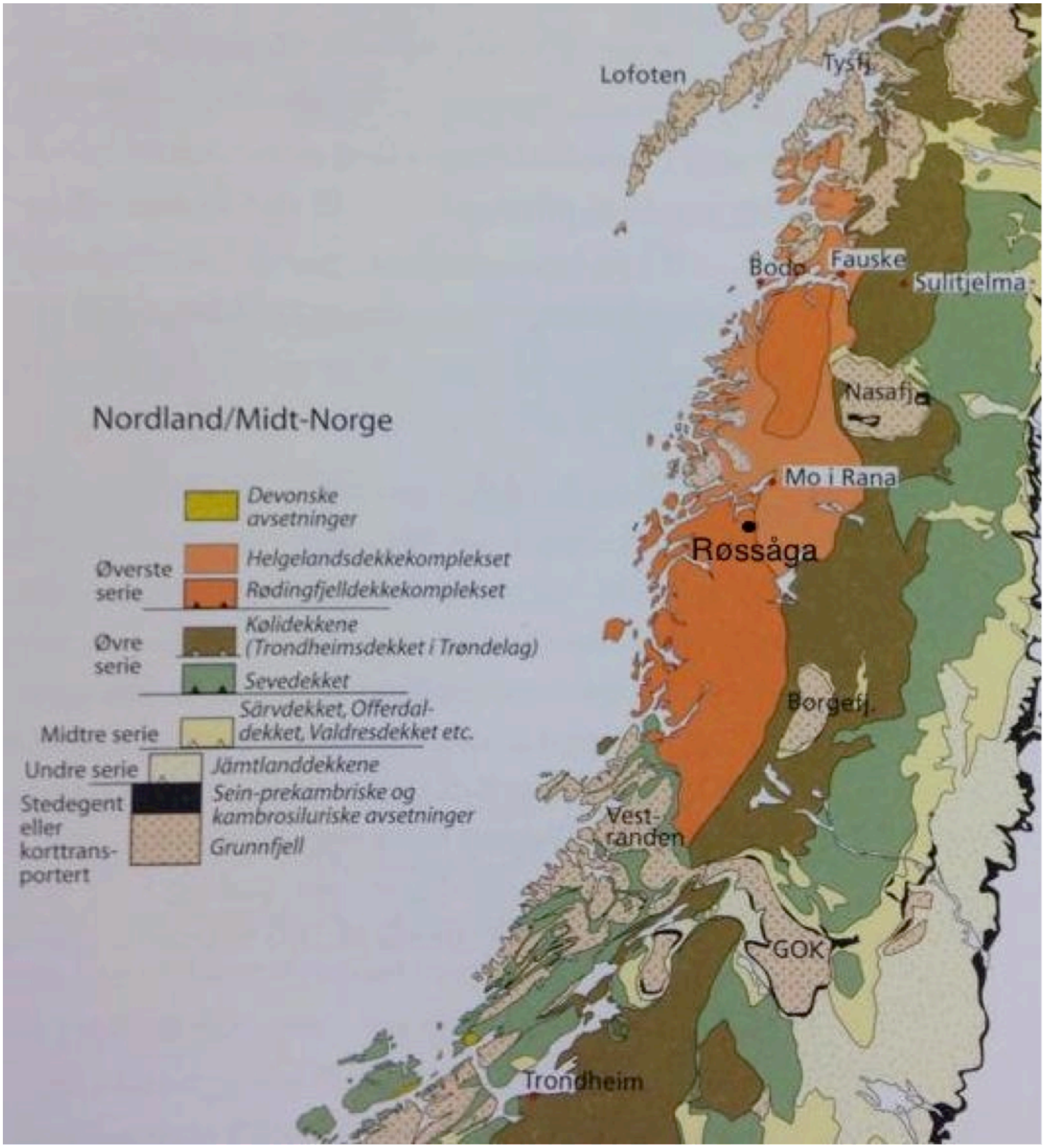


Figure 3. Regionl geology of Nordland/Mid-Norway (Bryhni, Nøttvedt, & Ramberg, 2006). The location of Røssåga is marked on the map. Modified by the author.

2.2 Rock types

The geological map of the area and geological section of the tunnel is given in the Appendix 5 and in Figure 4. All the rock types in the area have been folded during the orogeny of the Caledonides and because of this they will some places occur in thin layers overlaying each other. The rock types can therefore vary more frequently than what is given on the map. The main rock types in the area are as follows:

- Limestone/marble: Limestone is a rock composed principally of calcite. The marble in the area is calcitic and has been formed by metamorphosis of limestone. These rock types can be prone to karst (Goodman, 1993; NGU, n.d.a). The rock types and their characteristics are thoroughly described in Chapter 3.
- Mica schist/mica gneiss: The mica schist is a metamorphic rock that is mostly comprised of quartz, feldspar, muscovite and biotite among others. The mica gneiss is a rock type that is formed during high regional deformation and contains more mica than regular gneiss (>40 % mica) (NBG, 1985a). The mica gneiss in the area is finely grained with grey stripes and varying amounts of mica (Statkraft, n.d.). There are sections with mica gneiss and mica schist in the area that can be calcareous. In the southern section of the tunnel the mica schist is reported as calcareous (NGU, n.d. a)), but since the rock types are varying more frequently than given on the map, calcareous mica schist and mica gneiss can not be excluded in rest of the tunnel alignment either. These rock types can therefore also be prone to karst.
- Granite/granodiorite: Granite and granodiorite are plutonic rock types that are comprised of quartz, alkali feldspar, plagioclase and mica. The granodiorite has usually more plagioclase then alkali feldspar (NBG, 1985a). In the area the rocks are partly foliated and some places the rock is banded (NGU, n.d.b)).
- Quartzite: Quartzite is a strong, hard rock consisting almost entirely of quartz crystal in a dense mosaic texture. Quartzites originate from metamorphism of quartzose sandstones, siltstone and chert (Goodman, 1993).

As it appears from the pre-investigations the southern part of the headrace tunnel will be constructed in mica schist/mica gneiss and limestone/marble. To the north the tunnel is expected to cross granite, mica schist, limestone and possibly smaller zones of quartzite and calcareous skarn (Statkraft, n.d.). Skarn is formed by metamorphosis of calcareous rocks and is typical a white, coarsely crystalline rock with well-formed crystals of the calcium silicate mineral wollastonite and perhaps garnet (Goodman, 1993). The rocks in the area have fold axis in direction N-S. The strike directions of the rocks are mostly N-S with dip of 10-25° to the West (Statkraft, n.d.).

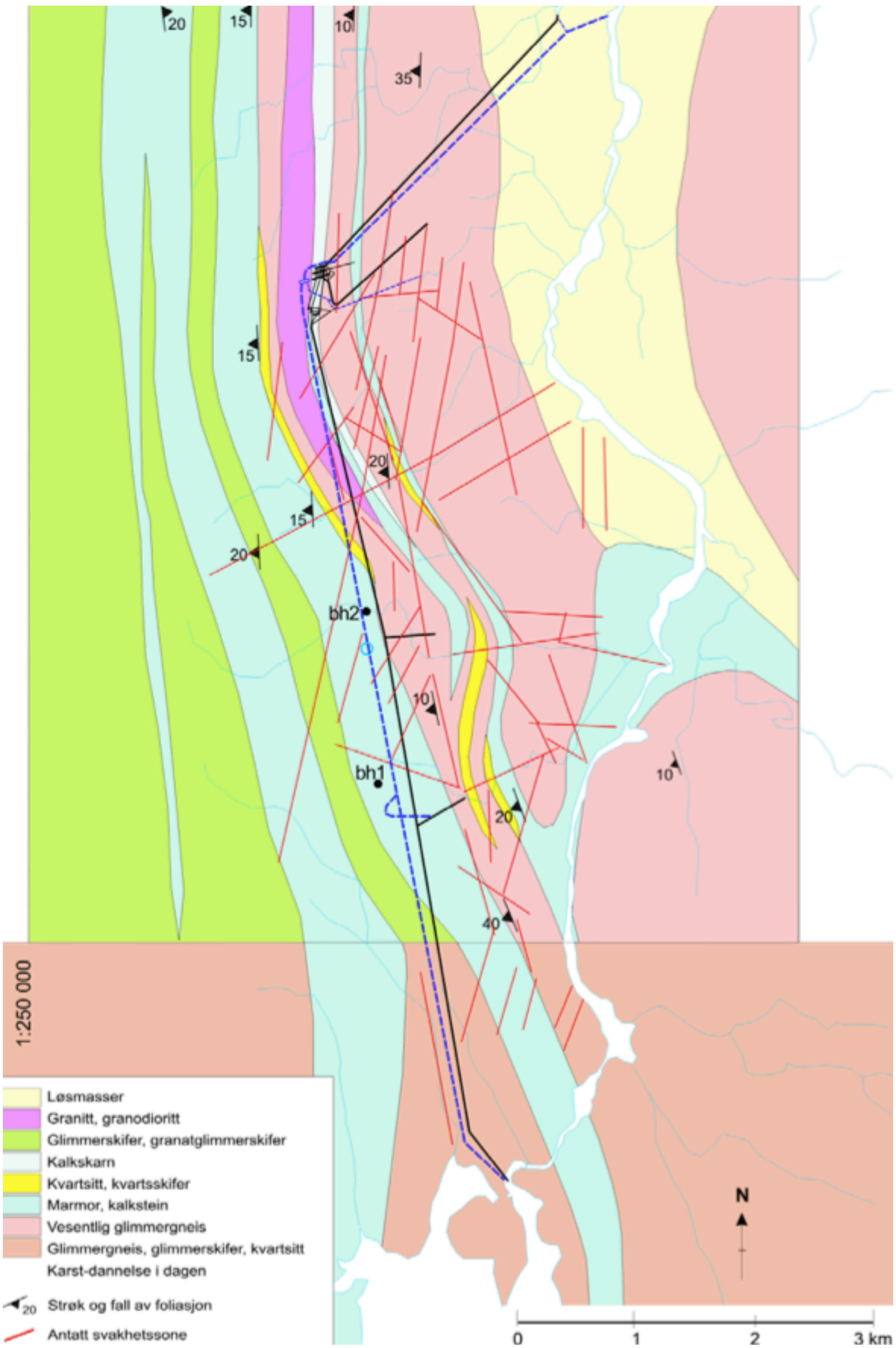


Figure 4. Geological map of Lower Røssåa (from NGU and modified by Statkraft (n.d.)).

2.3 Surficial deposits

The surficial deposits at the site of Lower Røssåga are shown in Figure 5. There are large areas covered with thin layer (0,2-0,5 m) of humus and peat. The bedrock outcrops frequently in this area. There are also areas with thick deposits like river-, ocean/fjord- and glaciofluvial -deposits in addition to small areas of thin moraine, weathering material and landslide material. It is mostly humus and peat that is covering the TBM-tunnel alignment.

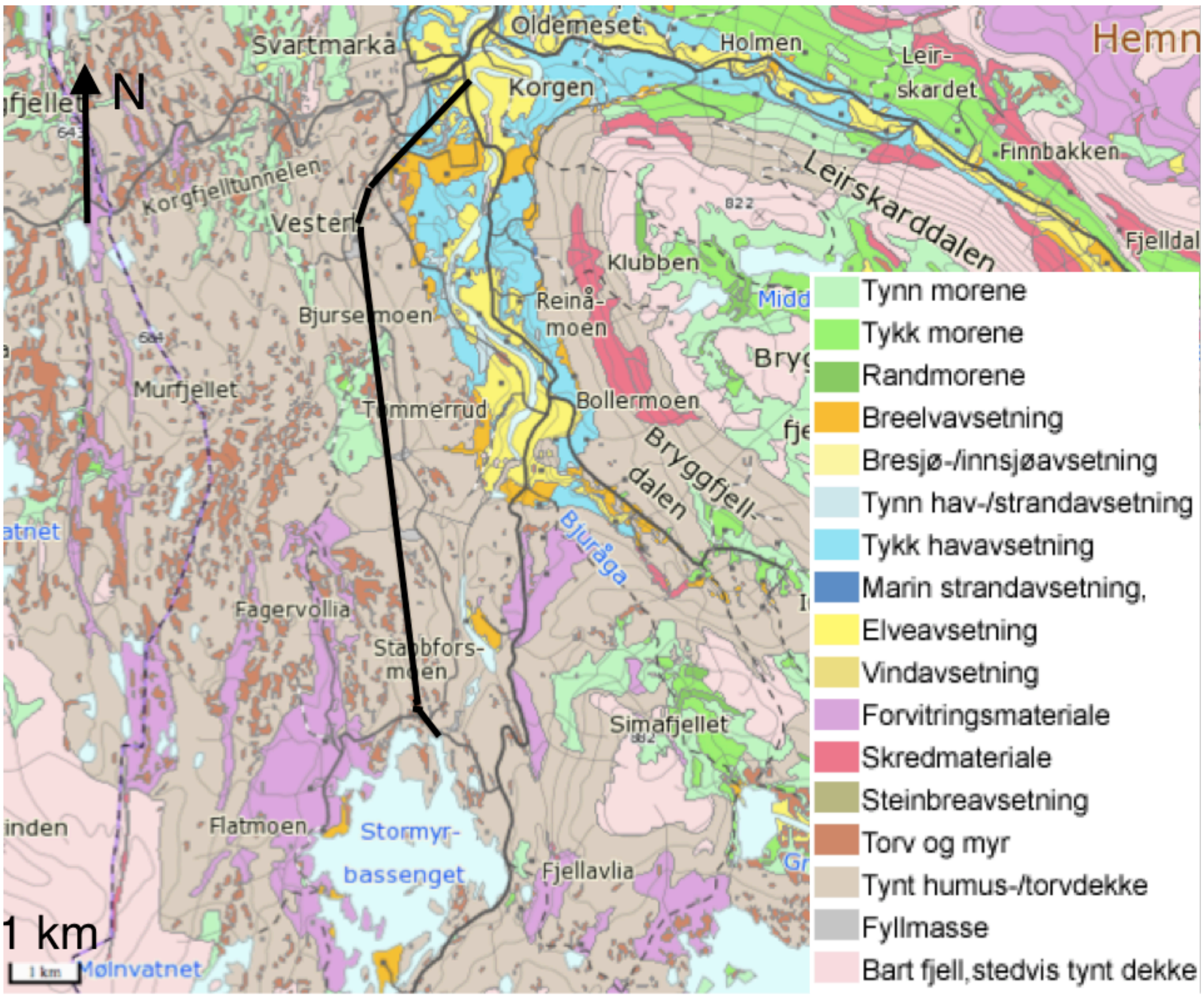


Figure 5. Surficial deposits at the site of Lower Røssåga (NGU, 2011) with the tunnel alignment of the tailrace and headrace tunnels. Modified by the author of this thesis.

2.4 Jointing

It is anticipated low to moderate fractured rock mass in the tunnel. The observed joint sets in the area are as follows:

1. Joints along the foliation with strike N-S and dip of 10-25° to the West.
2. Joints approximately parallel to the foliation with a steep fall in direction of both east and west.
3. Joints with strike N110°Ø and dip of 80-90° to the North (Statkraft, n.d.).

During the site visit some strike and dip measurements were performed outside of the portal of the TBM-tunnel, along the construction road from the portal to the new hydro power station and also at the portal to the new hydro power station. The compass was irrupted by the equipment in the tunnel so measurements performed in the tunnel was not correct and is not included in the results. The locations for the measurements were chosen because the same rock types also occur further up the tunnel alignment and the locations where considered to be representative for these rock types. However, more measurements should have been done to get values from the whole tunnel alignment and the other rock types. The strike and dip measurements were measured in the mica gneiss nearby the portal of the TBM-tunnel, but further down the construction road the rock type changed to mica schist with grenade. The strike and dip measurements are given in Appendix 2.

The measurements are plotted in a rose diagram in Figure 6 and in a pole plot in Figure 7. Joint set 1, which are the joint set along the foliation are distinct in the plots and are oriented in an unfavorable direction compared to the tunnel direction. Fractures parallel with the tunnel must be avoided because this poses very difficult conditions for water sealing (Klüver, 2000). Joint set 2 is not distinct in the plots, and only one joint is represented from this joint set. Joint set 3 is more apparent in the figures and is oriented in a favorable direction compared to the tunnel direction.

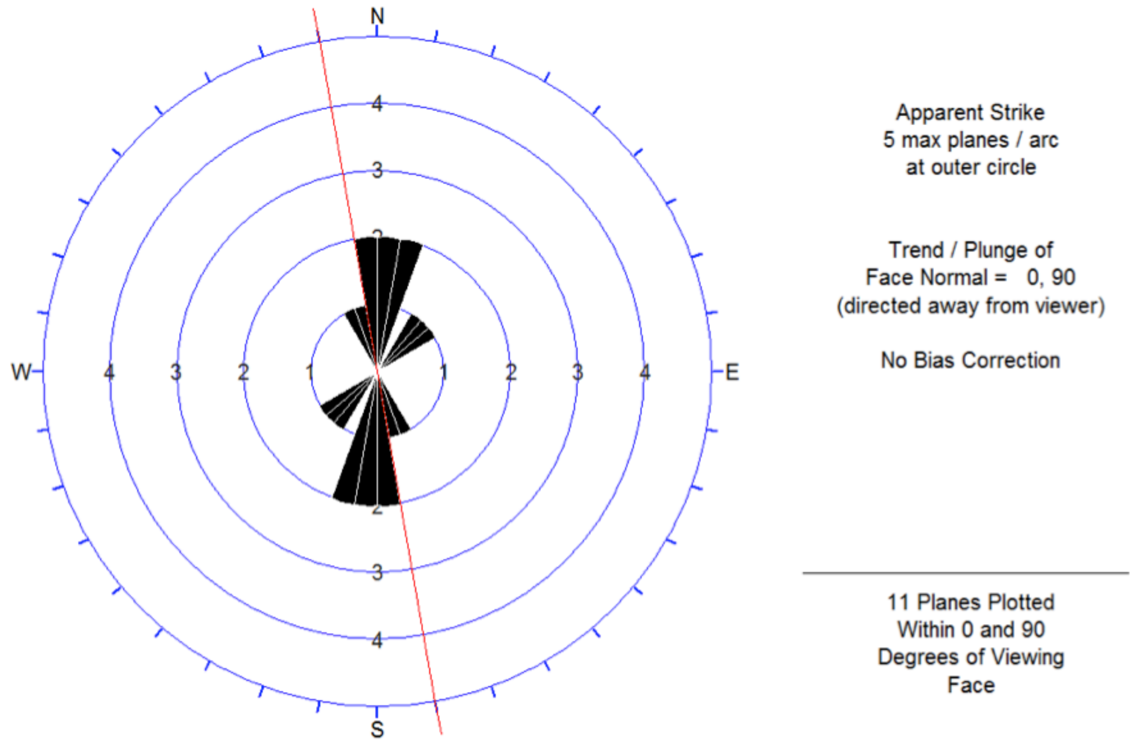


Figure 6. A rosette plot for the strike and dip measurements performed during the site visit. The red line shows tunnel-orientation.

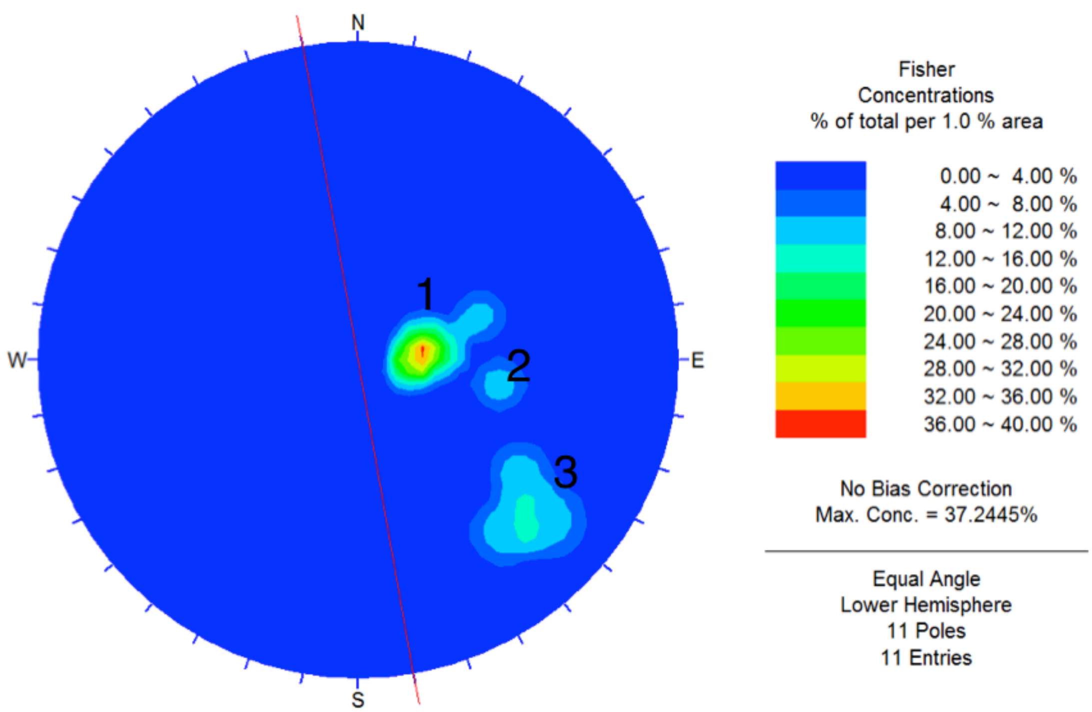


Figure 7. A pole plot for the strike and dip measurements that were performed during the site visit. Red line shows tunnel orientation.

2.5 Weakness zones

At Lower Røssåga it was experienced stabilization problems during excavation of one of the adit tunnels, probably in connection with a weakness zone. Except for some concrete casting in the existing headrace tunnel, probably because of weakness zones, there is no other information of significant weakness zones in the headrace tunnel. The weakness zones observed during the engineering geological field observations are given in Appendix 5. They are small to moderate in size. Possible weakness zones are oriented in three main directions:

- NNE-SSV: in the northern area
- NV-SE: in the southern area
- ENE-VSV: 2-3 zones along the alignment

2.6 Rock stresses

The overburden over the TBM-tunnel is mostly moderate between 100-200 m. The occurrence of some small joints and fissures in the power station and transformer room could indicate high stresses in the area (Statkraft, n.d.).

According to Myrvang (2001) will the vertical stress component mostly correspond to the gravitational value, where the vertical stress component (σv) is dependent on the density of the rock mass ρ, the overburden (h) and the acceleration of gravity (g) according to the following formula:

$$\sigma v = \rho * g * h$$

The horizontal stress component will often be influenced by stresses dependent on the geology. This will often result in larger horizontal stresses than vertical stresses and the horizontal stresses will often be much higher than the gravitational horizontal stresses. The horizontal stresses are often anisotropic, which means that the difference between the smallest and largest horizontal principal stresses will be significant.

It seems like the Caledonian rock types generally has low horizontal stresses, which probably is due to the fact that the rocks are often very fractured, but there are some exceptions. At a quarry in Fauske, which is also situated in the same geological region as Røssåga, it was experienced large horizontal stresses in the magnitude of 15 MPa less then 5 meters underneath the surface. From the map in Figure 8 the direction of the horizontal stresses in Norway can be seen. In Nordland it is observed that the horizontal stresses are somewhat anisotropic and the major principal horizontal stress near Røssåga is mainly oriented in direction of NV-SØ. (Myrvang, 2001).

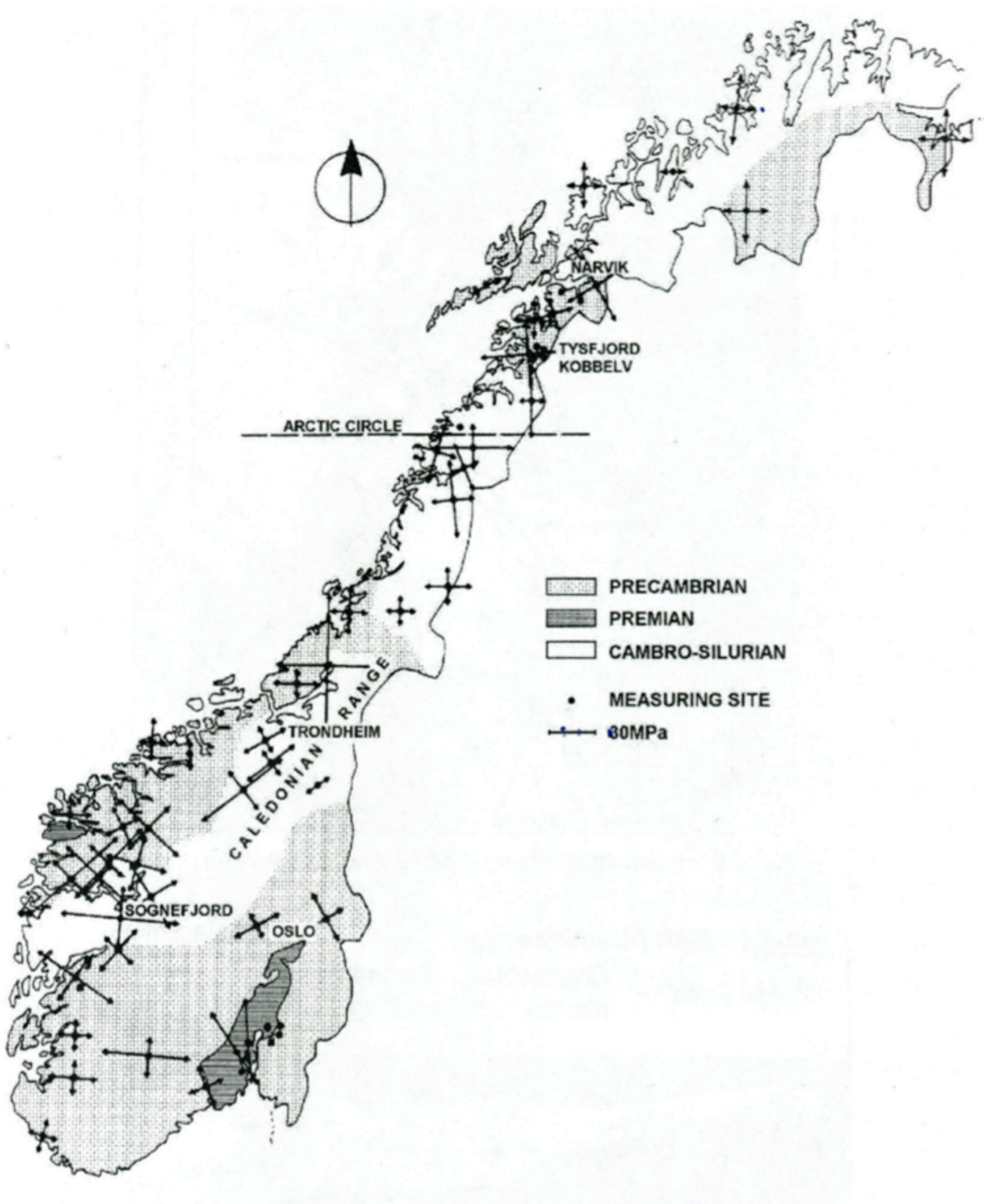


Figure 8. The direction of the horizontal stresses in Norway (Myrvang, 2001).

2.7 Rock mass quality

The Q-method is a system for classification of rock masses related to stability in a tunnel. From six different parameters a Q-value can be calculated using the following formula:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

The six different parameters are:

- RQD=Rock Quality Designation
- Jn=Joint set number
- Ja=Joint alteration number
- Jr=Joint roughness number
- Jw=Joint water reduction factor
- SRF=Stress reduction factor

The Q-values are estimated during geological mapping in the tunnel according to Appendix 1, but can also be estimated during pre-investigations, like field mapping or core logging. Q-values from pre-investigations can be used to give an indication of the rock mass quality one can meet during excavation, but it is important to have in mind that the conditions at tunnel-level can be very different than what is observed at the surface above the tunnel. The parameters can also be difficult to estimate during such investigations. High Q-values indicates good stability and a low Q-value indicates poor stability.

The Q-system was originally based on data from tunnels excavated by drill-and-blast method, but later on data from TBM-tunnels has also been included in the system. Engineering geological mapping is more difficult in a TBM-tunnel than in a tunnel excavated by drill and blast methods since the walls in a TBM-tunnel are quite smooth and it is therefore difficult to study the joint faces. In a TBM-tunnel the loose blocks will not fall down in the same degree as during drill and blast, so potentially unstable blocks may be found in TBM-tunnels even if the Q-values are high (NGI, 1997).

Q-values were estimated during site visit in the tunnel, at the tunnel portal and in the area around the tunnel portal. The rock type in the tunnel will be described in Chapter 2.8. The tunnel walls were very dusty so the Q-values could only be estimated properly nearby the TBM and at the tunnel portal. The rock was more fractured in the section near the cutterhead. One Q-value was estimated in the less fractured section behind the TBM backup and one Q-value in the more fractured section near the cutterhead. The jointing in the tunnel was parallel with the foliation and occurred in some of the weak layers that contained mica. The rest of the locations for the Q-values were chosen because they were easily available at the tunnel-portal and were considered representative for the conditions in the section. The parameters in Table 1 are estimated according to the descriptions given in Appendix 1

The ESR=1,3 for the TBM-section because the Q-value was estimated in a surge chamber. The span=7,35 and is the diameter of the TBM. According to the Q-method (referring to Appendix), the rock quality in the section excavated with TBM is classified as very good The section by the portal is excavated with conventional method and the span for this section is estimated to be approximately 8 m and ESR=1,3. The rock mass quality for this section is therefore classified as good (Appendix 1) (NGI, 2013).

Table 1. Estimation of the Q-values from the site visit, along with the respective parameters.

Location	RQD	Jn	Jr	Ja	Jw	SRF	Q	Comment
1	87	2	2	1	1	1	87	This was estimated in the section to the left of the TBM-backup.
2	95	2	2	1	1	1	95	In the section directly behind the TBM backup.
3	87	3	2	1	1	3	23	At the tunnel-portal
4	83	3	2	1	1	3	22	At the tunnel-portal
5	60	4	2	1	1	3	12	Outside of the tunnel, in the area directly to the left of the tunnel portal.

2.8 Observations of the geology in the tunnel

During the site visit the TBM-tunnel was situated in the surge chamber on its way down, about 60 meters from where it will start to excavate the tailrace tunnel. According to the geological map the surge chamber is expected to go through calcareous skarn. The rock type that is observed in the tunnel is highly folded mica gneiss with elements of eclogite, and layers and lenses of pegmatite containing large elements of mica. The geology of the tunnel at the section with the TBM-backup can be seen in Figure 9 and Figure 10. Observation of pegmatite can be seen in Figure 11. The tunnel was generally dry with moisture in some places, except by the tunnel entrance where there could be observed some droplets from the roof.

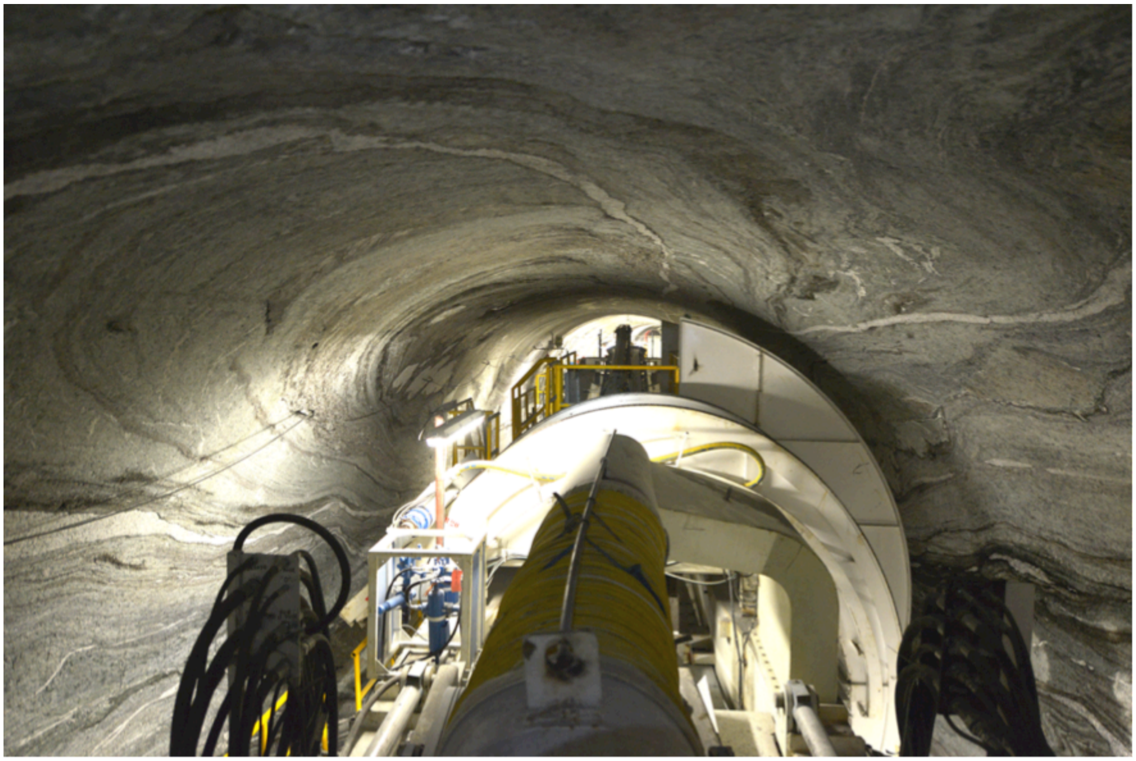


Figure 9. The geology in the TBM-tunnel. The picture is taken in the TBM-backup and backwards, during site visit.

Figure 10. The geology in the TBM-tunnel in section with TBM-backup. Taken during site visit-

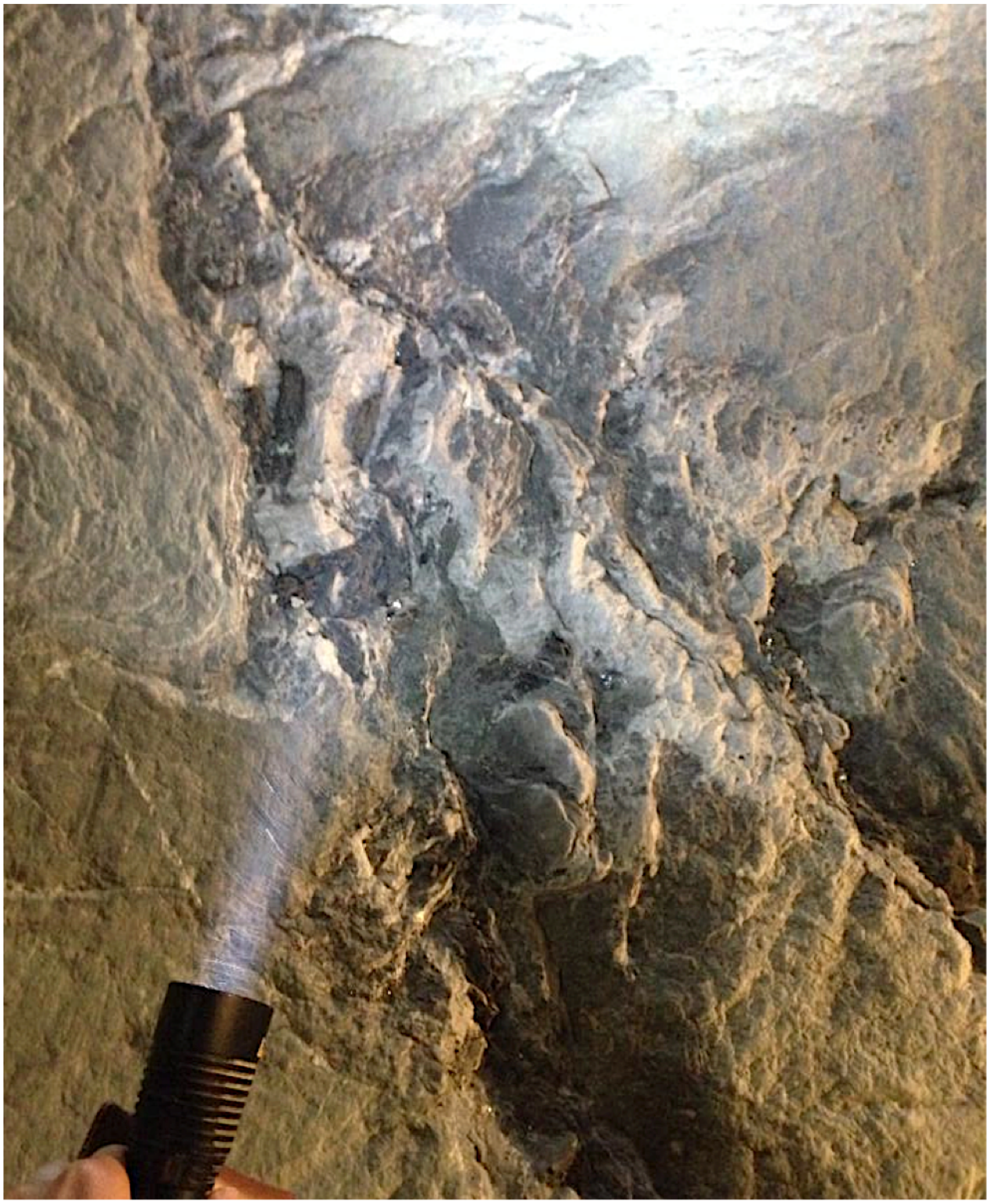


Figure 11. Observation of pegmatite with large elements of mica, Picture taken in the TBM-tunnel during site visit.

3 Karst
The subject of this chapter will be the minerals that can be dissolved in water and the rocks that they form. Calcite, dolomite, gypsum and salt are minerals that can all be dissolved in water. Some of the rock types that they form are limestone, marble, rock gypsum, rock salt, anhydrite and chalk (Goodman, 1993). The main focus in this chapter will be on the carbonates since these are the rock types that are abundant at Røssåga and may contain karst. Carbonate is the term for the rock types that are composed of minerals containing carbonate ions (CO_3^{2-}). For Norwegian conditions these minerals are mainly calcite and dolomite (NGU, 2008c).

3.1 The formation of karst
Calcite, $CaCO_3$, is a very common rock-forming mineral. Limestone is a sedimentary rock that consists of more than 50 % calcite. Pure limestone contains more than 95% $CaCO_3$, but all gradations of limestone exist (NGU, 2008c). When the proportion of argillaceous material increases it becomes argillaceous limestone, and it grades into calcareous shale or calcareous mudstone as the argillaceous material becomes dominant (Goodman, 1993). Metamorphic and recrystallized limestone is called marble or limestone marble. In dolomite the mineral dolomite, $CaMg(CO_3)^2$, is the dominating carbonate mineral. Metamorphosed dolomite is called dolomite marble. The dolomite is formed from limestone by addition of magnesium rich solutions, either immediately after the rock formation or later. Other carbonates are chalk, shellshand, carbonatite and magnesite (NGU, 2008c).

Carbonate rocks are soluble in water through a chemical process. For example in limestone the water reacts with CO_2 and becomes dilute carbonic acid (H_2CO_3), which makes the water an agent of solution. Carbonic acid attacks limestone by stripping of the Ca^{2+}, which are then carried off in solution. The solvent then picks up bicarbonate (HCO_3-). This process is given in this equation:

$$CO_2 + H_2O + CaCO_3 \rightarrow Ca^{2+} + 2HCO_3{}^{-}$$

Equation 1. The chemical formula for solution of limestone with acidic water.

Continued removal of the rock by water over time causes a special type of rock mass that is pierced by caves and passageways with depressions on the surface. This type of landscape is called karst. Karstification is the term for the process where karst is developed (Goodman, 1993). According to Goodman (1993) the development of karst topography in limestone advances through stages as illustrated in Figure 12, which may be identified as young, mature and old age karst:

- In a young karst landscape the land surface has not been lowered and it retains normal surface drainage, except that some stream discharge is lost to underground passageways and there are some springs where these flows rejoin the surface.

- In early maturity, vertical joints in the ground have been enlarged by solution to form narrow vertical gaps and caverns. The collapse of some of these caves has led the surface to sink in to form sinkholes.
- In late maturity there is a well-developed and integrated underground runoff system such that all surface streams have a complex hydrology. The land surface is irregular and covered with red clay soil in various thicknesses. Rainwater is transported easily through closely spaced fissures in the soil. The top of the rock is pinnacled, but this cannot be seen because it is covered in soil.
- In old karst landscape the limestone is virtually leveled or almost or completely removed. The insoluble residue from the limestone is left as residual clay. Some rock outcrops as knife-edged ridges and represent the remains of walls between adjacent caverns where the roof has disappeared. In this type of landscape there is no surface stream, the water filters down through the soil to join the ground water.

This is of course a simplified model; the different stages may coexist due to climatic variations and different kinds of contacting rock formations. Time for maturation of the karst landscape may vary. In hard limestone the maturation takes a very long time and one cannot expect to see any changes between each visit to a site. In porous, soft limestone, this process can proceed rapidly (Goodman, 1993).

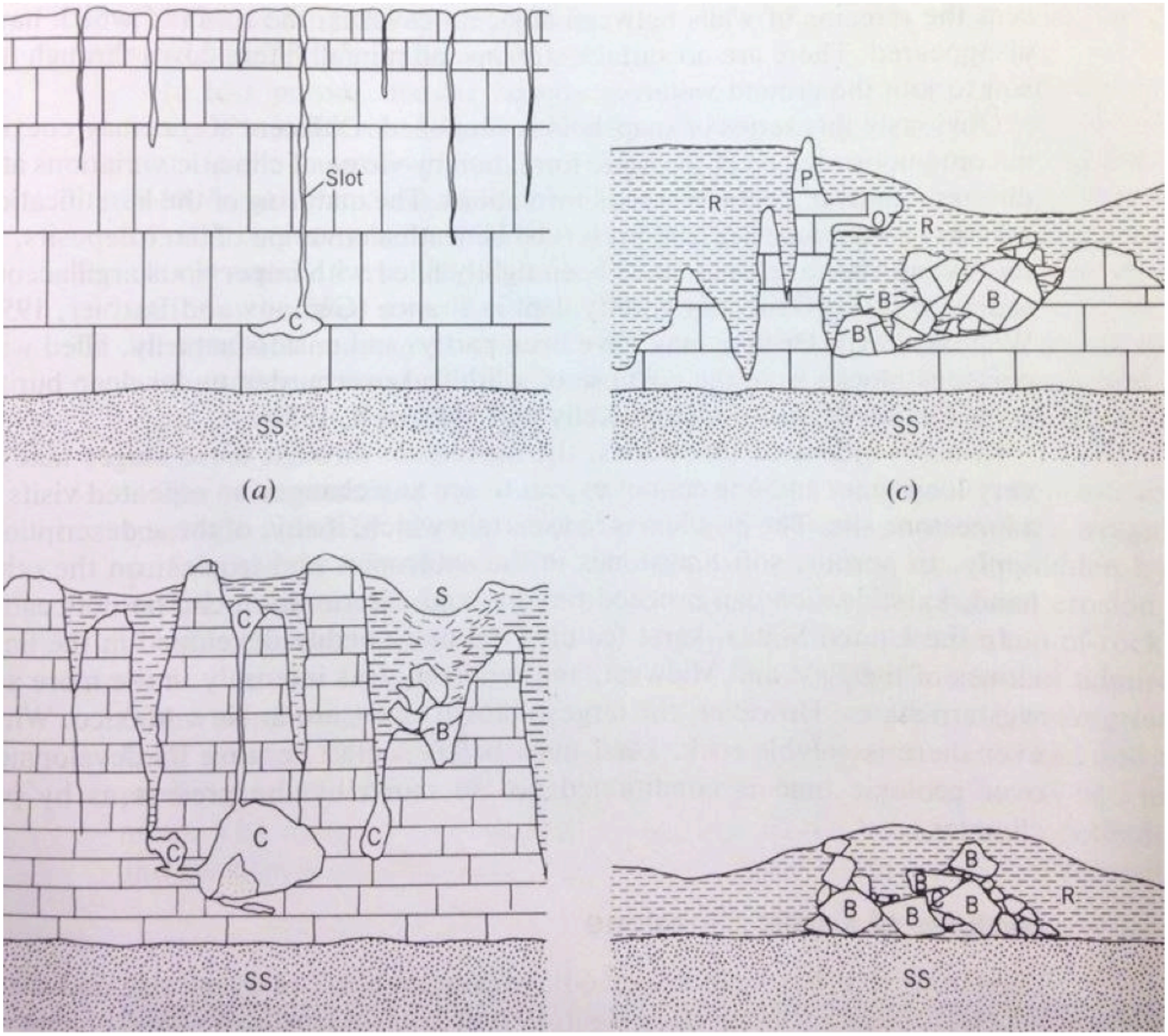


Figure 12. Development of solution cavities and karst features. a) youth; b) early maturity; c) late maturity; d) old age. C, cavity; S, sinkhole; SS, sandstone; B, block; R, residual soil; P, pinnacle; O, overhanging pinnacle (Goodman, 1993).

Cavities or enlarged voids can occur in almost all calcareous as well as gypsiferous and saline rocks. The styles and dimensions of these karst features are affected by the composition, texture, structure, strength and geologic history of the rock. Some examples are:

- Cavities in young porous limestone tend to be small. The rock often develops a spongy character near the surface.
- Dense pure limestone without layering can give rise to large openings of irregular shape.
- Dolomites tend to develop small holes, called vugs, but they can also contain large caverns.

- Limestone and dolomite with foliation or variable purity tend to develop slim but extensive openings along bedding planes, joints and faults.
- Marble often develops large caverns, which are usually elongated along the direction of foliation.
- Calcareous shales may contain bedding plane cavities where thin layers of limestone members have been dissolved.
- Cavities are clustered in zones of close jointing or fracturing.

In dense limestone, the rock between the voids may remain completely unaffected and give no hint of the proximity of even giant openings. The surface of a joint in a karstic rock has a characteristic roughness after the work of solution. Any calcareous rock that displays these surfaces has the potential for housing caverns and voids (Goodman, 1993). Caves and voids can contain water, air and/or infillings of different sorts like sand or soil (T&T, 2003).

Solubility and permeability are equally important factors in the process of karstification. The basic factor of permeability in the carbonate rock mass is jointing. Fragmentation of masses, resulting from tectonic processes, represents the most important factor in karstification. The groundwater movement through the rocks is dependent on the size of the channels and fractures and their degree of interconnection (Milanovic, 2004). The rocks, which contain silicates and fine-grained minerals, are less prone to karstification than pure and coarse-grained carbonates (Barla, Diederichs, & Loew, 2010).

It has been found evidence of the importance of water-bearing deposits in the development of karst. The carbonate rocks with no overlying water-bearing deposits is less prone to karst than one with such deposits. Rocks exposed to weathering without a cover of such deposits tend to become hardened and resistant to solution and erosion. Without soil-deposits above, the water will not receive carbon dioxide from a soil-zone before entering the rock and the water will thus be less effective in dissolving the underlying carbonate rocks (Herak & Stringfield, 1972).

3.2 The hydrogeology of karst

It is important to understand the hydrogeology of karst when tunneling in this type of ground. The hydrogeology of karst will therefore be described with focus on the risk of inflow into tunnels.

The water in a karst aquifer collects in networks of interconnected cracks, caverns and channels. The water table of the aquifer is not well defined throughout the aquifer, but has regional as well as local dips. The interconnection of the karst channels and high permeability of the karstic ground allows fast filling and nearly equally fast drainage of water, which means that the aquifer reacts quickly to for example high precipitation during rainy season. The level of the water table between different seasons can be great (Milanovic, 2004).

An aquifer can be divided into three zones as can be seen in Figure 13; the unsaturated zone, the transfer zone and the inundation zone. Each zone constitutes different characteristics regarding karst and different risks for tunnel construction.

- <u>Unsaturated zone:</u> This zone is located directly beneath the surface and possesses dry fractures, caves and channels. The water moves almost vertical through this zone towards the transfer zone. It is no risk of high water inflow in this zone; the rock mass and karst features are mostly dry. However, stability problems can occur in large caves etc.
- <u>Transfer zone:</u> This zone is located between the maximum and minimum groundwater level. The zone is characterized by highly karstic rock. The rock mass are mostly dry, but in periods with a lot of precipitation the karstic features may be filled quickly with water. This means that sudden inrushes and flooding can occur when tunneling in this zone. Stability problems and inflow of filling material may also occur.
- <u>Inundation zone:</u> This zone is always located below the groundwater table, characterized by continuously moving water. Tunneling in this zone will almost always be followed by permanent inflow of significant amount of water if karst is encountered. Stability problems may occur in this zone as well (Sharifzadeh, Uromeihy, & Zarei, 2012).

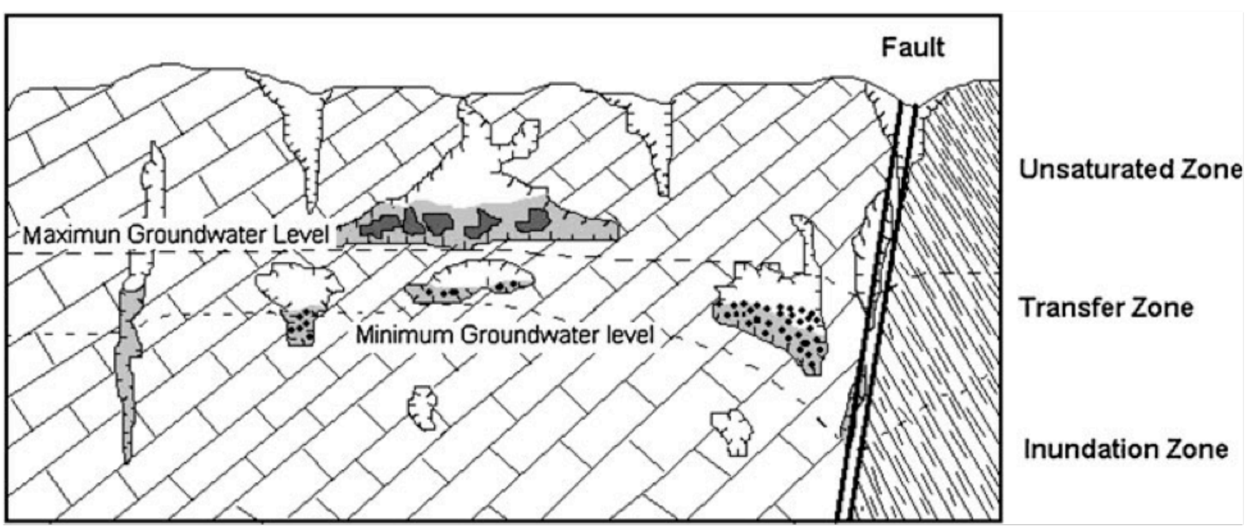


Figure 13. Model of karst development in an aquifer and production of different zones (Sharifzadeh et al., 2012).

The degree of karst decreases with depth especially below the lowest groundwater levels. The most active karst channels are located directly above the base level of erosion. The karst channels below this zone are a rare phenomenon, but they may pose risks of inflow under high pressure (Sharifzadeh et al., 2012). The highest inflows into tunnels are due to open karst conduits of large diameters (in meters scale) and/or network of joints widened by dissolution of the rock (Loew et al., 2010).

3.3 Karst in Norway

Norway has large and geologically very varied occurrences of carbonates:

- Low- and un-metamorphic limestone occurs in the Oslo field and was deposited during Ordovician and Silurian age.
- Precambrian carbonate rocks occurs in between Kvænangen and Repparfjord and in the area of Bamble.
- Metamorphic carbonates from Late Precambrian and Silurian age occurs along the whole Caledonian mountain range
- Magmatic carbonates in the Fensfelt in Telemark and in the Seilandsprovince in Western Finnmark (NGU, 2008c).

Karst is a normal phenomenon in Norway where limestone occurs, but it is best developed in marble. Nordland and Troms are the most important karst regions in Norway. In the Caledonides the limestones are often folded and stretched so they are in shape of long stripes that can reach up to several kilometers in length. Here “stripekarst” is formed as shown in Figure 14, which is a typical karst formation in Norway (Bryhni et al., 2006).

Figure 14. "Stripekarst" in Nordland (Bryhni et al., 2006).

3.4 Field observation of karst at Fallfors and Tullavbekken

Observations nearby Tullavbekken (location of Tullavbekken is marked on Figure 2) suggest that the terrain is karstic in this area, possibly in connection with marble (Johansen, 2014). The terrain is irregular and it is no surface streams. The area is very dry compared to the surroundings and it seems like the surface streams have been drained into the ground.

It was experienced problems with karst and water leakage when constructing the new intake to the headrace tunnels at Fallfors in October 2013. The observable karst features are solution channels in marble along the foliation and are primarily oriented in the same direction as the tunnel. The following is a description of the event made by Aagaard (2013):

The water in Stormyrbassenget was lowered 5 m to make it possible to construct the intake. This revealed a landscape of karst along the western part of the planned channel to the intake. The rock type was marble. The rock was dissolved along the layers forming water-bearing channels, with strike and dip N135°Ø/30°SV. This is shown in Figure 15. Channels perpendicular to the foliation also pierced the rock. Solution of a joint in marble is shown in Figure 16. The water leaked in and upwards from the fractures, causing too much water to gather in the construction area.

The location for the intake needed to be changed to enable the construction. It was decided to move the intake more to the east to avoid the most karstified area. The rock type in the new location is calcareous mica schist, but it was not observed karst or any wellsprings in this rock type. It was still expected that some leakage could occur through the joints (Aagaard, 2013). It was not met any further problems with karst during the construction of the intake after the change of location.

Figure 15. Solution of joints in marble at the originally planned intake at Fallfors. The photo is taken by Erik Dahl Johansen in October 2013

Figure 16. Solution of joint in marble at the construction area for the planned intake at Fallfors. The photo is taken by Erik Dahl Johansen in October 2013.

4 Prediction of karst

Karst areas constitute very complex ground to investigate due to unpredictable location, dimensions and geometry of the karst structure. Karst terrain is one of the most intricate grounds to be assessed for civil engineering purposes and it is more difficult to investigate the more mature the karst is (Kleb et al., 2004). This chapter will give a review and a discussion of possible methods to predict karst during pre-investigation and during excavation of a tunnel.

4.1 During pre-investigation

The term pre-investigation refers to the investigation for planning of the tunnel until tender. The purpose of performing pre-investigation is to provide sufficient information so that it is possible to plan and consider the consequences of constructing the tunnel (Statens Vegvesen, 2003). The aim of investigating karst during pre-investigation should not be on identifying all of the karst features along the tunnel alignment because this can be impractical, if not impossible, both because of time and money. The focus should be on providing enough information to develop reliable predictions of ground conditions and ground behavior during construction (Fischer et al., 2009).

4.1.1 Desk-study:

A thorough desk-study is important because it provides information for targeting and planning a site investigation efficiently and cost effectively (T&T, 2003). In karst areas it is therefore very useful to do a review of already existing information on the project area to provide an indication of existence of caves, sinkholes, disappearing streams and other features of the ground which might foretell the degree of dissolution or fracturing of the rock (Kleb, et al., 2004). According to Kleb et all. (2004) useful sources when looking for these features can be:

- Geological maps: When looking for karst it is necessary to consider which rock types karst can be developed in and use a geological map to find the distribution of these in the area of the project.
- Air photos: Vertical aerial photographs viewed stereoscopically are usually a good starting point for a site investigation if available. These photographs are taken with more than 50% overlap of the image area so that every point on the ground is photographed from two camera points. With the help from a stereoscope the land surface appears in exaggerated relief. Sinkholes, for example, appear as small, closed depressions, often with standing water or darker colour tones. Locations of springs where water returns from underground streams can often be identified from staining of the rock (Goodman, 1993).
- Hydrogeology reports: These reports can be useful to give information about the development and extension of karst in the area, because karst is highly related to the groundwater-system.

- A review of former projects in the area: Tunnel projects that have been excavated in the same area and under the same geological conditions can give useful experiences of which geological conditions one can expect. If karst has been encountered in the same geological area earlier, it is likely that this could happen again.

4.1.2 Observations in field

During a field inspection it is done a visual inspection of the site, either on the ground or from the air. A field inspection cannot be used to confirm the location of subsurface features, but can locate soluble rock types, sinkholes and other surface features like streams that disappear into the ground (Kleb et al., 2004). Mapping of these features can give an indication of the extent of karst for excavation of the tunnel and may give clues for areas that are important to investigate further. Figure 17 illustrates the karst features that can be observed on the surface in karst terrain.

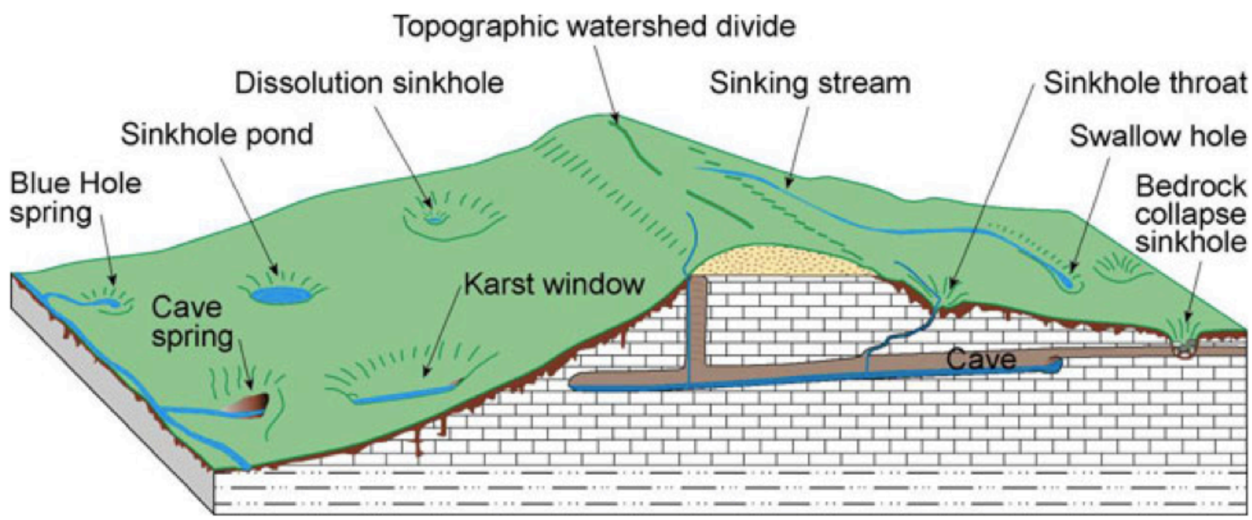


Figure 17. Illustration of karst landscape, with different karst features on the surface (Kentucku Geological Survey, 2012).

4.1.3 Geophysical methods

Geophysical methods can be a useful tool for assessing of karst terrain for tunnel alignment. It is necessary to understand the nature of the target of interest to determine whether it will contrast from its surroundings or stand out in the geophysical survey data set. When planning karst detection survey it is important to consider the likely range of depth, lateral and vertical dimensions, nature of overburden, degree of infill etc. to be able to decide the best method to use under the conditions at the site. It can be useful to combine the results from several methods to distinguish between the karst features and other ground features (T&T, 2003).

Geophysical methods on karst have not produced consistently reliable results so far, however technology is advancing and there are some methods that can produce useful results in certain situations (Kleb et al., 2004). Some methods will be reviewed, which are all described as possible methods for detecting karst by Kleb et all. (2004). All of the methods have their strengths and their weaknesses for detecting karst.

4.1.3.1 Electrical resistivity measurements

Electrical resistivity surveys measures the ability of the ground to pass a current. The ability is dependent on the composition of the ground. The current is passed into the ground via two current electrodes and the potential are measured between two other electrodes. An apparent resistivity can be calculated on the basis of the measured resistance and a geometrical factor that is decided on the basis of the electrode placements. All of the resisivities that are in the reach of the measurements are represented in the apparent resistivity. The data is being inverted to find the specific resistivity in the different parts of the subsurface (NGU, 2008a).

The penetration depth is dependent on the distance between the current electrodes. By increasing the distance between them, the current will reach deeper and the measurements will get response from deeper areas. However, increasing the penetration depth will decrease the resolution (NGU, 2008a). A rule of thumb is that the depth of investigation is ¼ of the distance between the two end electrodes used for measurements. The presence of underground utilities, particularly metallic pipelines and electric lines will provide a significant interference in the data (Hoover, 2003)

An empty cave in the subsurface will represent large resistivity, and will increase the apparent resistivity. Caves and voids filled with mud and water will lower the apparent resistivity. Subsurface karst features may therefore be identified from anomalous resistivity values (Hoover, 2003). However, the resistivity in open channels or voids without water is not *always* large, so resistivity measurements can be a challenge if it is open channels or voids without water (Rønning, 2014). An example of an electrical imaging profile, where the variations in resistivity represents different features is shown in Figure 18.

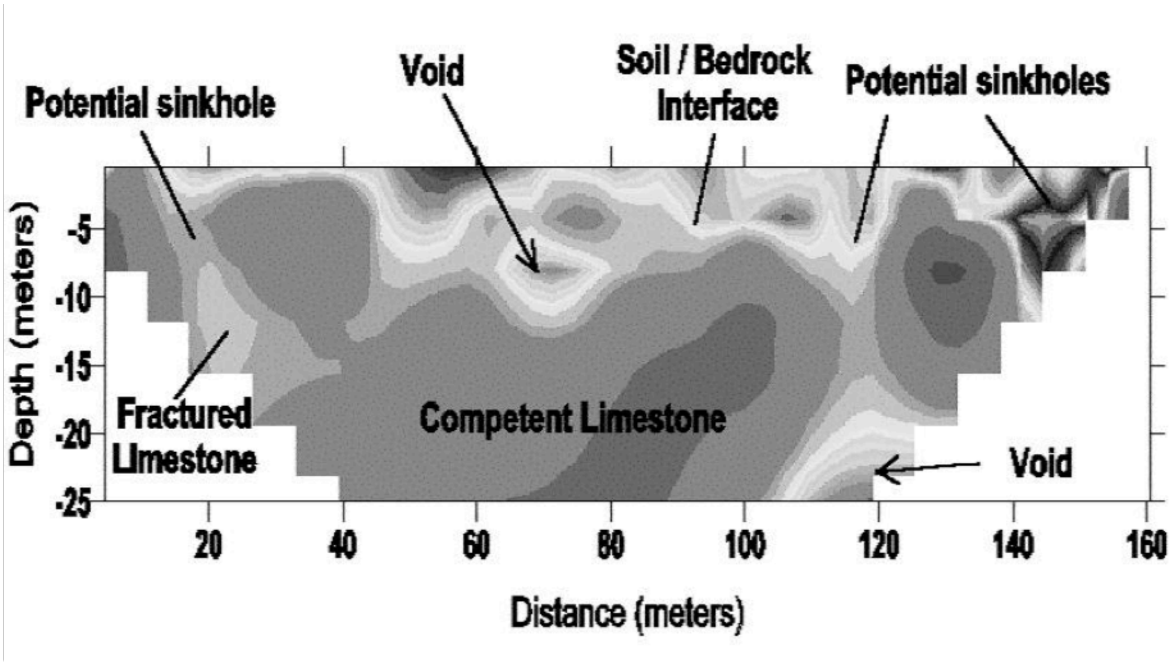


Figure 18. Electrical imaging profile developed from 56 electrodes spaced 1 meter apart. The variations in resistivity represent different features of the ground (Hoover, 2003).

4.1.3.2 Seismic methods

Seismic methods measure the velocity of compression waves, traveling through the ground. Wave velocity decreases in more fissured and more cavernous ground (Kleb et al., 2004).

The use of the seismic refraction method requires the ground to be described with more or less horizontal layers with homogenous parameters. By measuring the time it takes for the wave to travel from the energy source to the geophone, the materials P-wave velocity and the thickness of the layers can be calculated. Sometimes is a layer invisible in the measurements because the wave from the interface was arriving later than the wave refracted from the above layers, creating a so-called blind-zone. Another weakness with this method is that a layer will not appear in the measurements if it has lower velocity than the layers above (NGU, 2008d).

The use of seismic reflection method is dependent on good propagation of high frequency and is therefore best for measuring in location where there is fine-grained water saturated soils. When compared to refraction seismic the reflection seismic gives a more direct and detailed image of the layers in the ground, but gives poorer information about the layers seismic velocities and thickness. The depth range for seismic reflection method is from 10 m to several hundred meters (NGU, 2008d)

The seismic methods are so far restricted to detect boundaries between strata and the interface between soil and rock. However, with the use of cross-hole seismic methods in boreholes, it is possible to locate subsurface voids, but this should be restricted to critical location at the site due to high expenses (Kleb et al., 2004).

4.1.3.3 Microgravity

Microgravity measurements are based on the principles of mass and density. All objects attract each other with a force, which is proportional to their masses, and inversely proportional to the square of the distance between their masses. During microgravity surveying the gravity at different locations is measured. The changes in gravity at different locations are attributed to changes in the earth's mass. For example will a smaller gravity measurement be present if there is a subsurface cave present within the bedrock. This is because the density of a cave, either filled with water, soil or air, will be smaller than the density of the surrounding rock. This creates a negative anomaly in the measured gravity in the area. The measurements produce a set of number that can be interpreted to determine subsurface density of mass distribution. Gravity surveys are usually presented as a contour map, where the variation in gravity can be seen (Hoover, 2003).

Successful application of the gravity method requires high-density contrasts between the cavity and the surrounding matrix. Microgravity, conducted with close spacing and careful implementation in order to ensure high resolution and accuracy, remains one of the methods best suited to the detection of voids in the uppermost 20 meter, even when the voids are relatively small (Chalikakis et al.,, 2011).

4.1.3.4 Ground Penetrating Radar

The ground penetrating radar is an electromagnetic method that can be used for the investigation of the stratification and structures in the ground. With a special antenna, electromagnetic wave pulses are emitted into the ground. A part of the wave energy is being reflected back to the surface when the wave pulse meets a boundary that represents a change in the mediums dielectric properties (NGU, 2008b). The dielectric constants of a material are dependent on the ability of a material to store a charge when an electric field is applied (Hoover, 2003). The rest of the energy will continue downwards. In this way one can get reflections from several boundaries in the ground. The reflections are received with a receiver antenna at the surface. The depth to the different features can be found from the measurements (NGU, 2008b).

GPR appear to be a popular geophysical tool for identifying and locating subsurface karst features such as cavities, conduits and solutionally enlarged fractures (Chalikakis et al., 2011).

A slowly moved antenna can measure features that are centimetres or less apart, but the limitation of this method is the depth of penetration. Karst bedrock commonly weathers into a residual soil, which is conductive. This reduces the penetration of the electromagnetic radar pulse (Hoover, 2003). In good conditions the depth of penetration could be as deep as 20-40 meters (NGU, 2008b). This limits the karst investigation with GPR to identification of shallow karst.

4.1.4 Borehole investigations

The probability of a random borehole from the surface to intersect a karst feature is low; therefore a large number of boreholes are necessary to be able to reliably detect the karst phenomena at the site (Kleb et al., 2004). This can be both expensive and time consuming and should therefore be restricted to critical locations. There are several methods that can be used for karst exploration in boreholes from the surface and some of these methods will be described very briefly.

Drilling boreholes can be done either with or without core recovery. If a core is recovered from the borehole, logging to give a description of the rock type, degree of weathering, fractures and other geological features may be performed. To draw information from the borehole it is possible to perform photographing or televiewing in the borehole. These techniques can give information of the geological conditions like the placements of joints, faults, open cavities and sites where water is flowing into or out of the hole (Goodman, 1993).

Water-pressure testing can be done by pumping in water under pressure into a closed of section of the borehole and the resulting water flow is monitored. The results of this test can give an indication for the degree of openness of the joints and the placement of significant water conductors, like karst features (Goodman, 1993).

Most of the geophysical methods that are available as surface exploration have also been adapted to borehole investigation (Goodman, 1993). As previously stated, it is possible to use cross-hole seismic methods in boreholes for detecting subsurface voids and Goodman (1993) reports of successful use of cross-hole GPR for detecting subsurface voids.

4.2 During excavation

Limiting situation for a TBM is when and where a machine doesn't work for what it was designed and manufactured for and the advance is significantly slowed down or even obstructed. The most limiting situations for a TBM are among others the inflow of groundwater and the occurrence of karstic caves. Investigation ahead of tunnel face for detecting these conditions is therefore important (Peila & Pelizza, 2009). The concept of investigation ahead of face with respect to karst will be further discussed.

4.2.1 Investigation ahead of the tunnel face

Both direct and indirect investigation ahead of tunnel face reduces the progress of the TBM. It is therefore necessary to find a balance between the exploration costs for lost production and for the costs if the TBM-productivity is hampered by an incident (Peila &

Pelizza, 2009). According to Peila & Pelizza (2009) the direct investigation methods available for TBM tunneling are the following:

- Boreholes with core recovery: Horizontal boreholes are normally performed through TBM-cutter head; inclined boreholes are normally possible from behind the cutter head in an open TBM. The objective of the boreholes is to determine the ground conditions ahead of the face; among others the presence of water and karst can be determined from boreholes. However, boreholes with core-recovery are not commonly performed from the TBM because of the time and the drilling diameter required.
- Boreholes without core recovery (probe drilling): The registration of drilling parameters can be done using a data-logger: drilling rate, pressure on drill bit, pressure of the drilling fluid and torque can be registered. Probe drilling is often used for establishing the existence of water conducting features in front of face (Statens Vegvesen, 2003). The procedure for probe drilling in a TBM will be further reviewed in Chapter 7.7 as a measure against karst during excavation.
- Geological mapping of the face and/or the sidewalls to characterize and classify the rock/soil mass. This can be used to evaluate and update the prognosis for the remaining excavation of the tunnel (Statens Vegvesen, 2003).

There are also indirect investigations in the meaning of geophysical methods that can be used ahead of the tunnel face (Peila & Pelizza, 2009).

It can be difficult according to Klüver (2000) to detect water-conducting channels by performing probe drillings, because the chances of hitting a channel with one probe drilling hole is small. This means that the water inflow from a borehole is not always representative for the conductivity of the rock mass. It is therefore important to probe in several places at the face, to increase the chances of detecting the water-conducting channels.

4.3 Discussion of possible methods for predicting karst

Karst ground is very challenging to investigate due to the highly unpredictable nature of the karst features. During pre-investigation it should not be the intention to detect all of the karst features along the tunnel alignment, but it is important to be able to predict the ground conditions so appropriate planning for the tunnel construction can be done.

It is important to do a thorough desk-study to be able to plan an efficient site investigation. A desk-study can only provide an *indication* of karst in the area, but cannot give a concrete answer to if karst will occur during excavation of the tunnel. Observations in field should be performed to verify and extend the findings from the desk-study. However, to be able to predict karst at tunnel-level other methods are necessary to perform, like geophysical methods or borehole investigations.

Different geophysical methods to detect karst have been reviewed in this chapter. The different methods have their strengths and their weaknesses with respect to karst detection. In karst ground the seismic methods can only be used to map bedrock and the presence of sinkholes underneath residual soil, it is however possible to identify subsurface voids if using cross-hole seismic, but this should be limited to critical sections. Microgravity can detect even relatively small voids in the uppermost 20 m, if the contrast in density between the void and its surrounding is high. GPR provides very detailed information about the subsurface, but has a depth limitation of 20-40 m. Resistivity measurements can detect subsurface voids, but detecting empty voids is not always possible, so it is best suited to detect voids filled with water. From the review of the geophysical methods it can be stated that these methods can sometimes be useful to identify karst if the conditions are right, but the methods have not produced consistently reliable results so far. The technology is however advancing, and perhaps in the future it is possible to do more accurate geophysical investigation of karst.

The probability of detecting karst caves or voids with boreholes from the surface is small; therefore a large number of boreholes is necessary to be able to reliably detect these karst features. Boreholes should therefore be restricted to investigate critical sections determined from former investigations. It is possible to detect karst with several different techniques in boreholes.

Since karst ground is very complex and the pre-investigation of this type of terrain involves considerable uncertainty it is important that it is performed investigation ahead of face during construction to detect hazards in advance. This can allow the tunnel progress to stop in good conditions so measures can be performed to avoid incidents with karst. However, it can be difficult to detect a single karst channel with probe holes, so probe drilling should be performed in several places at the tunnel face.

5 Experiences from other tunnel projects in Nordland

When planning tunnels it is very useful to consider experiences from other tunnels nearby, since these can provide useful information of what condition one can expect when constructing the new tunnel. The experiences with water inflow from the construction of tunnels in the area are of great value. This is because it can give a clue on the severity of problems with karst and water inflow one can expect when excavating the TBM-tunnel at Lower Røssåga. Therefore two different tunnel projects will be examined in this chapter; the old tailrace tunnel of Upper Røssåga and the headrace tunnel in Staupåga at Svartisen hydropower plant. They are chosen because they are both in the same geological area as Lower Røssåga and both of them experienced water leakage during excavation due to the occurrence of karst.

The pre-investigations and geology of each project will be described as well as the problems with water inflow during excavation and how the issues were solved. The useful experiences from these projects related to the issue of the master thesis will be discussed.

5.1 The existing tailrace tunnel of Upper Røssåga

This chapter will focus on the problems with water inflow while excavating the tailrace tunnel of the hydropower plant at Upper Røssåga in the 1950´s. The planning of the project of Upper Røssåga started in 1949. In 1955 it was initiated necessary research to find the proper layout for construction. It was decided to go for an alternative, which included 2700 m of headrace tunnel from the intake above the Røssvass dam to the reservoir west of Halvardalen and a tailrace tunnel of 4500 m, which passes underneath of Bleikvasselv and culminates below of Kløftmyrfoss. A setup of the waterpower plant of Upper Røssåga is shown in Figure 19. The construction started in 1957 (Sørensen, 1957).

5.1.1 Geology and pre-investigations

After selecting the alternative for construction of Upper Røssåga it was desirable to quickly as possible identify the geological conditions to decide the tunnel alignment and the exact placement of the underground station. It was therefore performed core drillings. For the tailrace tunnel, core drillings were performed in those sections that were not covered by too much soil. A “Bergmester” was employed and he did the analysis of the drill cores and did a lot of site visits. In this way there was obtained a good overview of the geological conditions (Sørensen, 1957).

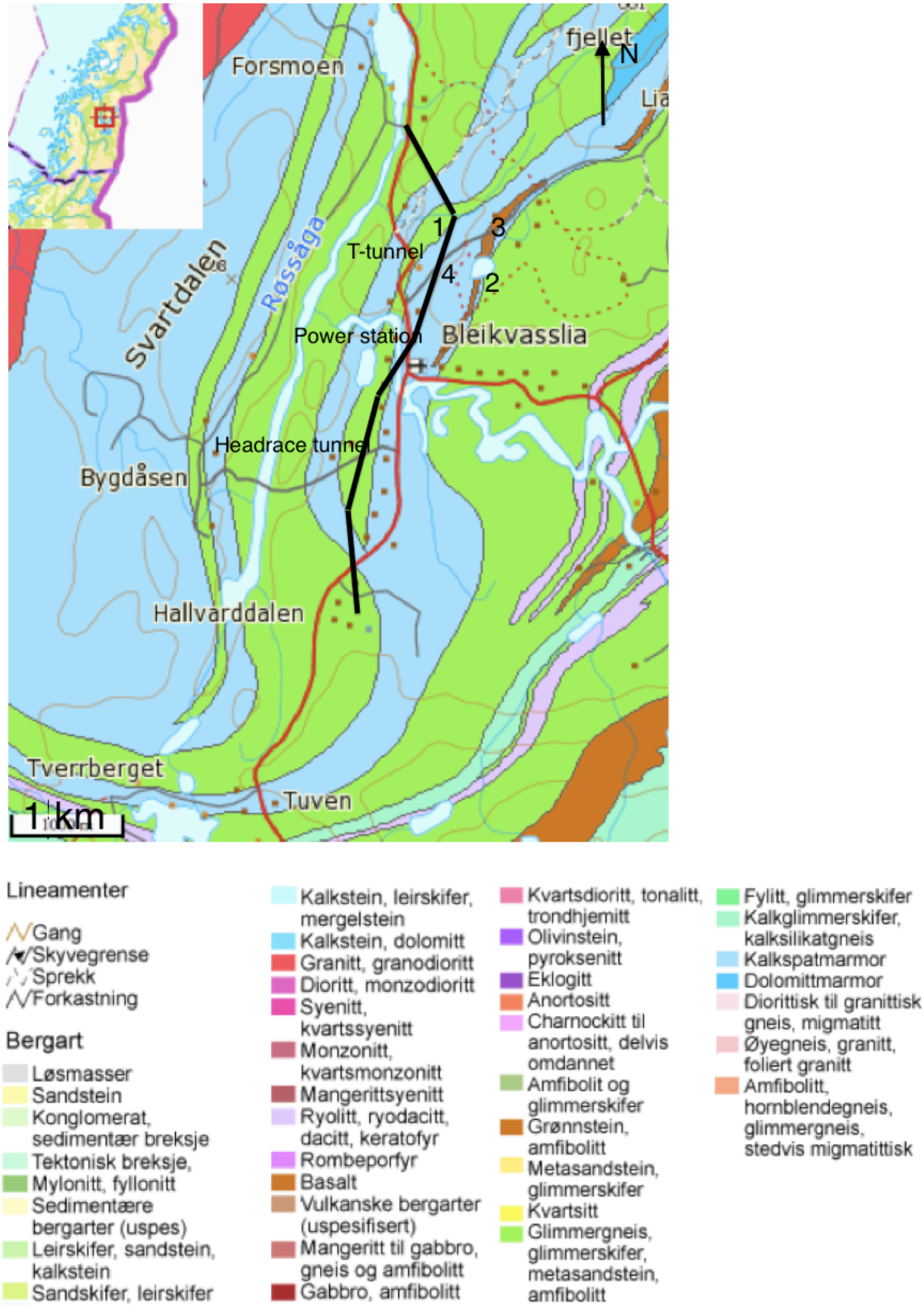


Figure 19. Geological map of the area of Upper Røssåga (NGU, n.d.b)), with the tunnel alignment. Modified by the author.

The geology of the area is shown in Figure 19. Between the tunnel entrance and the crossing of Bleikvasselv, the tunnel is situated in the border between silicate rocks and calcareous rock types. The dominant rock types in the tunnel are amphibolite and mica schist, but marble also occurred. The layers are mostly horizontal and in the transition zones the rock can be very fractured and water bearing (Simonsen, 1957).

5.1.2 Problems while excavating

The problems with water inflow in the tailrace tunnel were due to a water-conducting gap in a section with calcareous rock, probably marble. The course of events will be further described.

The tailrace tunnel was excavated from one access point in each end with the drill and blast method. The 10th of January 1958 the boreholes encountered large amount of water at point 1 (see Figure 19). The great water pressure caused a water jet with a length of 20 m from the top holes in the face to the invert of the tunnel. An open gap in the rock of 0.8 to 1.0 m was encountered after the next blast. The gap, with a dip of 60° to the south-east, cut straight through the tunnel.

Field investigation showed that two waters in Snelia were drained at point 2, and the river north of the waters was drained at point 3 (see Figure 19). It was believed that these watercourses were drained into the tunnel through the open gap. Also a wellspring that served as a water supply for the school in Bleikvassli (point 4 in Figure 19) suddenly dried out. It was decided to seal the cross section with concrete lining, and let the gap fill up with water behind it (Simonsen, 1957).

The work with the concrete lining started. In the end of January 1958 a heavy rainfall started which culminated the 31th of January. The leakage into the tunnel was larger than what the pumping system could handle (> 14000 l/min). They day afterwards the whole tunnel was filled with water. When the leakage into the tunnel decreased, draining by pumping was performed and the 12th of January the tunnel was dry again. The sole of the tunnel was now covered with mud, sand and gravel. Cleaning up of the tunnel and reparation of different equipment resulted in a downtime that lasted to the 28th of February. The work with concrete lining was finished the 18th of April (Simonsen, 1957).

The inflow from the waters in Snelia into the gap in the tunnel was estimated to normally reach a size of 1 m^3/s during flood in the rainy season. To prevent this it was decided to redirect the stream from Snelia away from the tunnel. Water pumps with a capacity of 28000 l/min were also installed in the tunnel. The rainfall during the following fall was unusually large, and the gap filled up with water. The 26th of October a rumble could be heard from the tunnel. Water flooded the sole of the tunnel and during one hour the water raised to a level of 2 meters. This corresponds to a total inflow of 3000 l/s into the tunnel through the gap. Fortunately the working team was not in the tunnel at the time of the incident. After 44 hours the water had inundated the tunnel in its whole length of 1320 m. It took six days before the water had been pumped out of the tunnel (Simonsen, 1957).

The following could be identified as the reason for the disaster: The rock in the tunnel sole next to the gap had given in for the water pressure and the baseplate of the concrete lining was broken of in its whole length along the ditch. It appeared as if the rock had a weakness zone 0.8-1m beneath the baseplate. The water had been filling up beneath the baseplate and when the plate gave in, fragments of rock up to 200 kg was thrown out. The rock fragments were rounded in shape and was weathered, similar to material one could expect to find in a karst cave. The tunnel was cleaned up and the normal working operation started the 28th of November (Simonsen, 1957).

After the last flood it was decided to not install a concrete lining over the gap. Instead it was installed 3 waterproof pumps with a capacity of 21000 l/min in addition to the already existing pumps. The redirection of the stream from Snelia was also thought to help the prevention of similar cases. During the further construction of the tunnel it was also used diamond bore holes to detect water in front of the working face (Simonsen, 1957). Besides these mentioned incidents there are no further records of difficult conditions during the excavation of this tunnel (Norconsult, 2012). The waterpower plants of Røssåga were finished in 1963 (Nilsen & Log, 2013).

5.2 Svartisen hydropower plant

The Svartisen hydropower plant is situated in Meløy municipality in Nordland County in Norway. The Svartisen Glacier is the second largest glacier in Norway and covers about 475 km^2. The construction of the Svartisen power plant started in 1987. The power plant utilizes the water draining from the western and northern parts of the Svartisen glacier. There were two construction sites at this project: Trollberget in the east and Holandsfjord in the west. Through a system of tunnels the water is channeled from the east to the main reservoir, Storglomvatn, for storage. A 7.3 km long headrace tunnel takes water from Storglomvatn to the Svartisen underground powerplant, which discharges into Nordfjord. A system of roof gutter tunnels collects water from the glacial regions on the west side and transfers it through a vertical shaft into the headrace tunnel. The setup for Svartisen waterpower plant is shown in Figure 20. Five TBM´s were used in this project, with four in use at the same time in one tunnel system (Water Power & Dam Construction, 1992). The incident that will be described in this chapter was situated at Trollberget in Staupåga diversion tunnel.

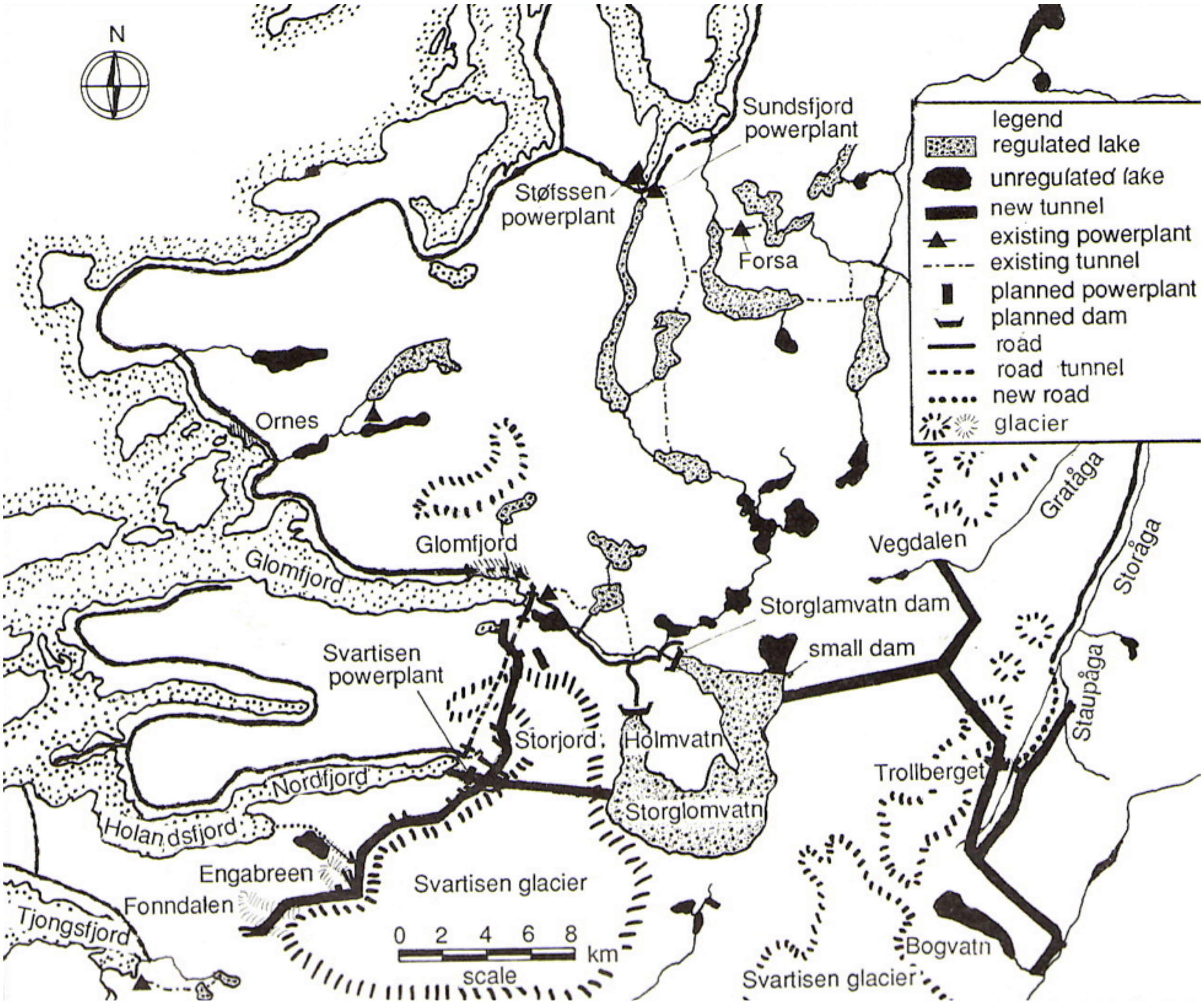


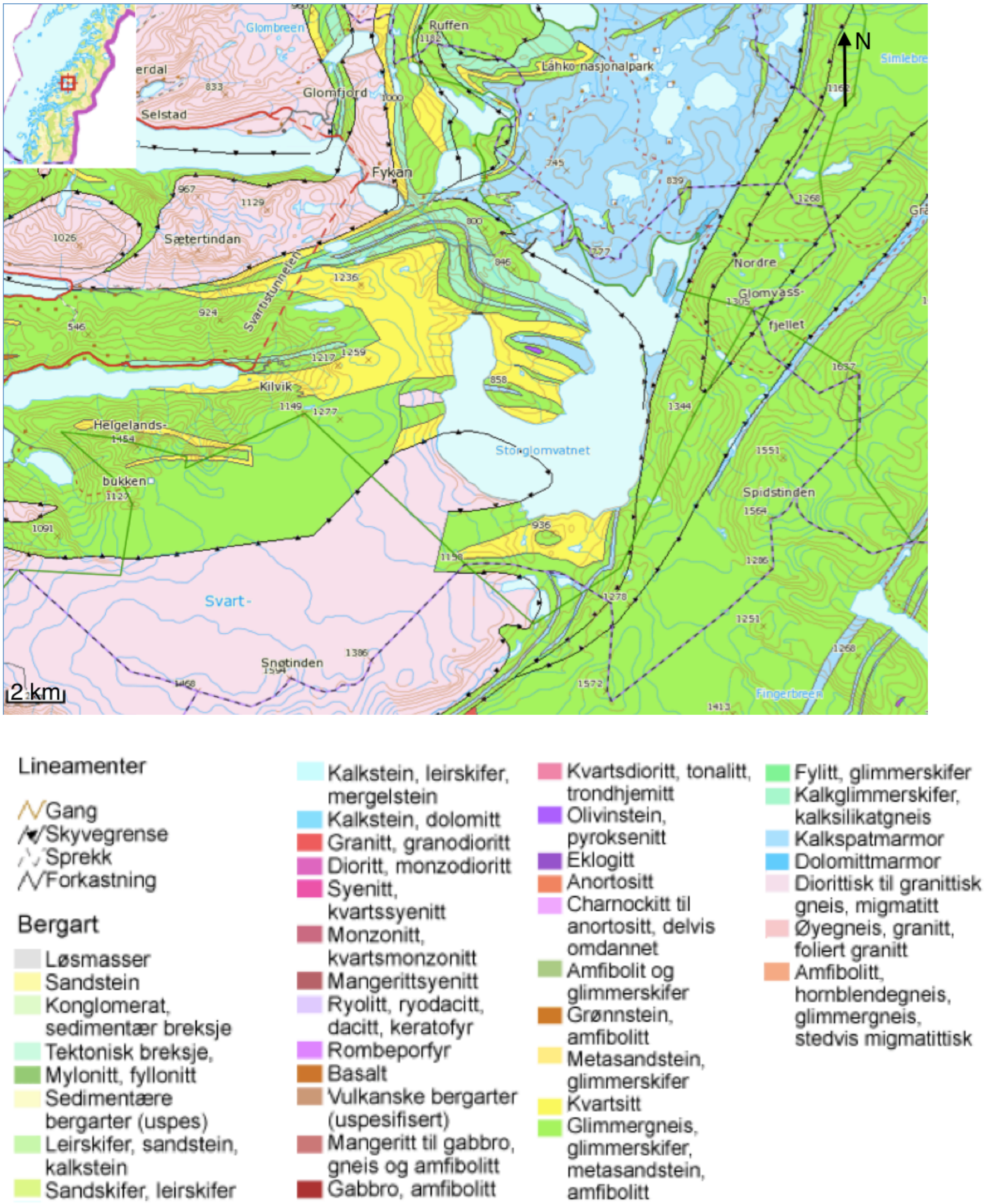
Figure 20. Map of the project area, showing the west and east tunnel systems in relation to Storglomvatn lake and the Svartisen glacier (Water Power & Dam Construction, 1992).

5.2.1 Geology and pre-investigation

Svartisen is situated in the Caledonides of Northern Norway, so the rocks there are all strongly folded and metamorphosed. The main strike direction is NE-SW, while the dip varies from horizontal to vertical. The three main types of rock in the tunnels of Trollberget are:

- Mica schist 80 %
- Limestone/marble 7 %
- Other types of rock 13 % (meta sandstone, gneiss, granite, granitic gneiss, diorite)

The limestone occurs in layers with thickness of centimeters to more than hundred meters. On the surface one can observe karstic features like caves and channels that disappears into the ground. (Drake & Johansen, n.d.).



Figur 1. Geological map of the rock types in the area of Svartisen hydropower plant. Trollberget is situated to the East of Storglomvatnet (NGU, n.d.b)).

5.2.2 Problems while excavating

From one access tunnel, approximately 41 km of tunnels were constructed at Trollberget using 4 TBMs. The construction of Staupåga diversion tunnel started in August 1990 and was finished in April 1992. An open TBM From the Robbins Company was used while boring the 8.2 km long tunnel with a diameter of 3.5 m (Water Power & Dam Construction, 1992). Probe drilling during excavation was not performed because it was assumed that it would be more time consuming to probe drill than performing measures if an incident occurred (Johansen, 2014)

In June 1990 a karst region was encountered and excessive leakage of 500 l/s occurred in the tunnel (Water Power & Dam Construction, 1992). The bedrock consisted of very folded marble with karst veins and clay zones (Drake & Johansen, n.d.), which were leading water from snowmelt on the surface into the tunnel. This resulted in very bad conditions for the operation of the TBM (Water Power & Dam Construction, 1992). A detailed mapping of the area was initiated to provide information of what could be expected further. At the same time the injection procedures where started to seal the tunnel (Haagensen & Johansen, 1992).

The water had to flow the whole tunnel length to drain and investigations suggested that the TBM might encounter more water. The tunnel boring was stopped and a 150 m long drainage tunnel with 5 m^2 in diameter was constructed 500 m behind the TBM. Handheld pneumatic drills were used to make the hole for blasting the tunnel (Water Power & Dam Construction, 1992). The measures initiated through the karstic region were: rock bolting, shotcrete, concrete and pre-injection. The bad conditions in the tunnel lasted for some hundred meters and caused delay in the project of 3-4 months, but the rest of the tunnel was bored in good conditions with advance rate of 200 m per week (Drake & Johansen, n.d.).

5.3 Discussion of experiences and key data from the tunnel projects in Nordland

The water leakage had a great influence on the progress in both of the tunnels. In Staupåga diversion tunnel the water leakage caused bad working conditions for the TBM. Injection and a separate tunnel for drainage had to be constructed. The project was delayed 3-4 months because of this. The water inflow in Upper Røssåga caused significant delays, damage on the tunnel and the different equipment and drainage of lakes nearby. The measures that were initiated were water pumps, redirection of water stream from Snelia and the construction of the tunnel continued with diamond boreholes to detect water in front of the face.

The leakage of 3000 l/s through the karst pipe at Upper Røssåga and 500 l/s in a karstic region in Staupåga diversion tunnel shows that the water ingress into Lower Røssåga can be severe if karst is encountered. It is therefore very important to take this into consideration and to plan efficient measures in case of similar situations. Key-data provided from the sources used in the literature study is shown in Table 2 and is taken into consideration when answering the issue of the master thesis. Unfortunately some of the key-data, which would have been useful for the prognosis of water inflow into the TBM-tunnel, was not possible to find and are therefore missing in the table.

Table 2. Key-data from tunnel projects in karstic ground in Nordland

Key data	Staupåga diversion tunnel	Tailrace tunnel at Upper Røssåga
Main rock types in tunnel	Limestone, marble and Mica schist	Amphibolite, Mica schist and marble
Total length of tunnel	8,2 km	4,5 km
Excavation method	Open TBM	Drill and blast
Cross section	9,6 m^2	65 m^2
Max. water inflow from karst feature	500 l/s in a karst region in marble	3000 l/s through a karst pipe, probably in marble
Waterpressure in karst feature	-	-
Overburden	200 m above karst region	-
Measures when encountering karst	Pre-injection, drainage-tunnel	Water pumps, redirection of water streams, probe drilling

6 Experiences from international TBM-projects in karst regions

To give an answer to the issue of this master thesis, it is useful to consider experiences from other tunnel projects in karst areas. Hence in this chapter, three different international TBM-projects in karst areas will be described with focus on the geology, the consequences of karst on the construction of the tunnel and how they dealt with the karst issue.The Kuhrang Transmission tunnel and Alborz service tunnel are both located in Iran. Both of them encountered karst in several places along the tunnel alignment, which caused water inflow and inundation of the tunnel. The SMART-project is situated in Kuala Lumpur in Malaysia. Kuala Lumpur is a known karst region and the ground in the project area is very sensitive to groundwater drawdown. Because of this the tunnel faced concerns regarding water inflow and had to be carefully planned and excavated to avoid incidents with karst and water inflow. Together these projects provide useful experiences for tunneling in karstic ground conditions, especially with regard to the water-inflow issue.

6.1 Kuhrang water transmission tunnel

The Kuhrang 3 transmission tunnel is located in the Zagros Mountains of Iran. The length of the tunnel is 23 km. It was constructed in the early 2000s to transmit 225 million m^3 of water per year, due to demand for water from an increasing population, from the Kuhrang River to the Zayanderud River. The Mountain of Zarab separates the Kuhrang River from the Zayanderud River (Movahednejad, 2005).

6.1.1 Geology

The transmission tunnel is located in the High Zagros geological zone as shown in Figure 21, which is a narrow, up to 80 km wide thrust belt, in the Zagros mountain range. The zone is characterized by extensively deformed overthrust anticlines mainly composed of Jurassic-Cretaceous outcrops (Sharifzadeh, Uromeihy, & Zarei, 2012). As seen in Figure 22 the tunnel alignment is composed of limestone and marlstone and several fault zones intersect the tunnel.

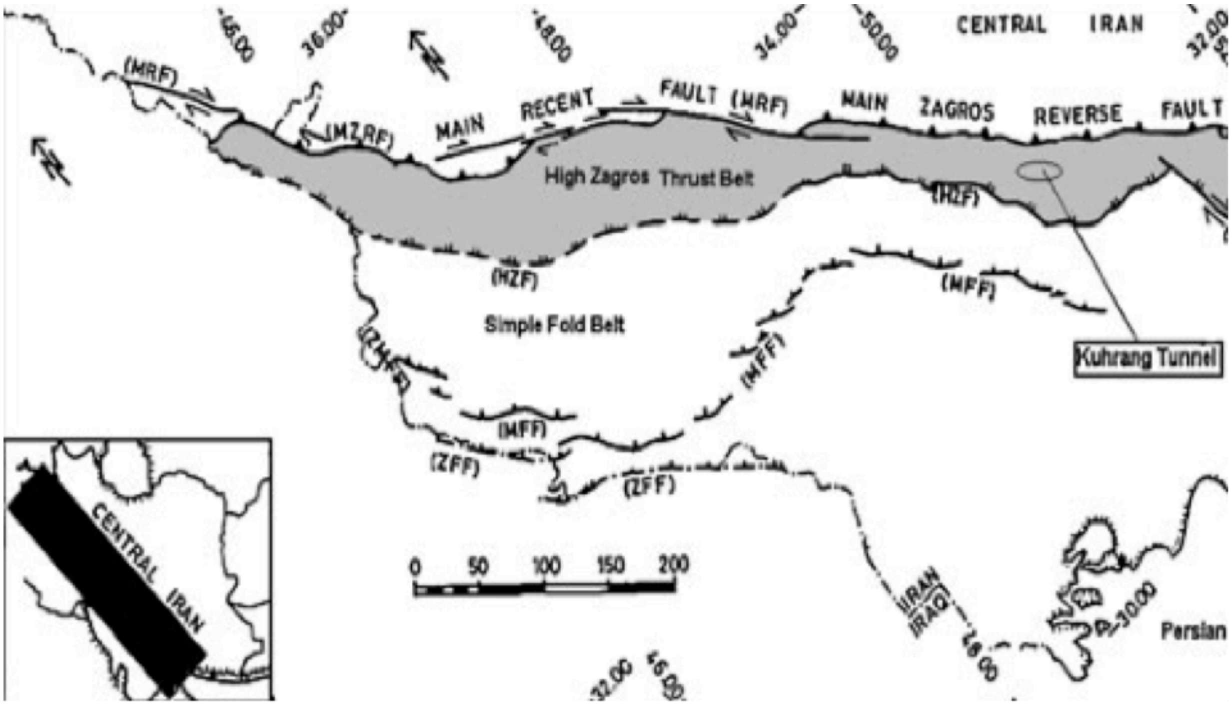


Figure 21. Major morphotectonic units of the Zagros Region of Iran, location of the High Zagros thrust belt and the project location of Kuhrang water transmission tunnel (Sharifzadeh et al., 2012). This is a segment from the original photo; the original photo is attached as Appendix 3.

The tunnel is located at an elevation of 2200 m.a.s.l. The highest elevation above the tunnel is 3500 m.a.s.l., which means that the tunnel will have a maximal overburden of 1300 m. Karst features observed on the surface is located along the tunnel alignment between:

- Km. 3+500 - 11+400
- Km. 16+000 - 17+000

Results from exploratory boreholes during pre-investigation indicated that the tunnel was placed under high groundwater level, ranging from 50 to 500 m (Sharifzadeh et al., 2012).

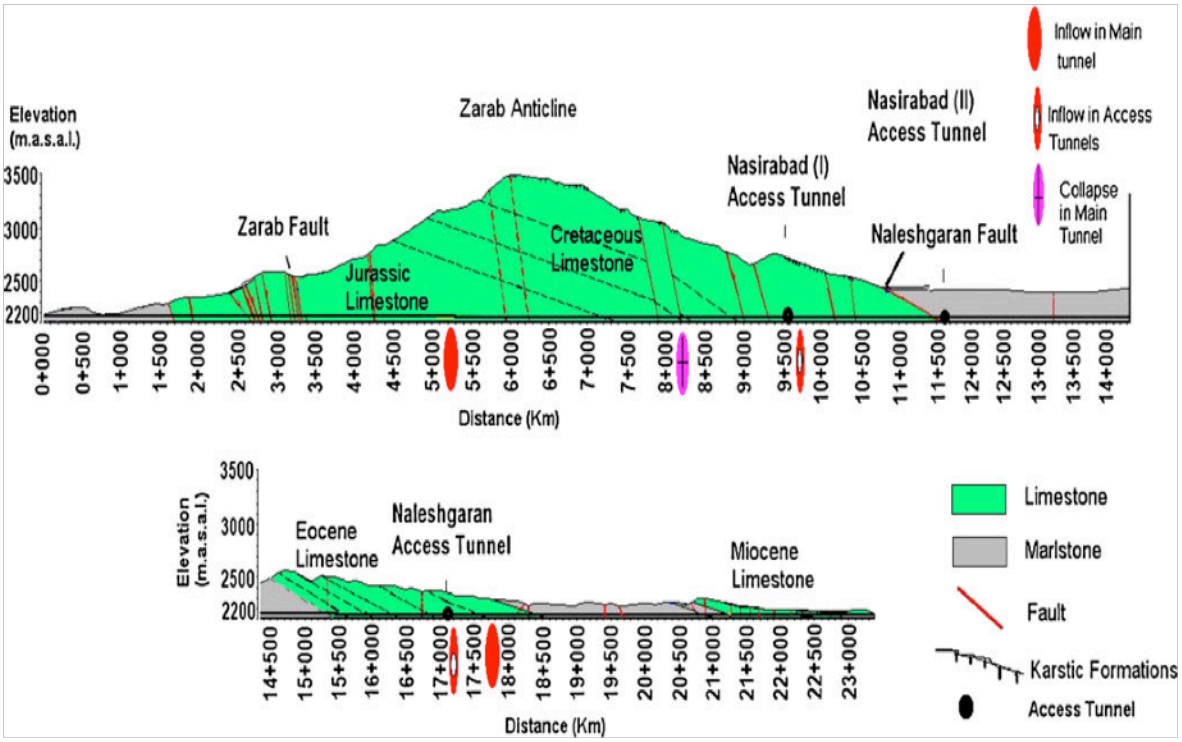


Figure 22. Geology of the Kuhrang transmission tunnel, as well as the incidents during construction of the tunnel (Sharifzadeh, Uromeihy, & Zarei, 2012).

6.1.2 Excavation

Conventionally tunneling was partially used during the excavation, but the largest part was excavated with open TBM (Movahednejad, 2005). The location for where incidents occurred during excavation is shown in Figure 22, and one can observe that several incidents with karst occurred during the tunnel construction:

- In km 5+250 a karstic cave was exposed in the tunnel floor as illustrated in Figure 23. This led to huge water inrush into the tunnel, which after a few hours exceeded 1200 l/s. The water level in the tunnel rose to a level of 2.5 m and flooded a section of 18 km. The tunnel was drained gravitationally before the water ingress halted and the excavation could continue. The groundwater pressure measured 30 bars in the karst channel (Sharifzadeh et al., 2012).

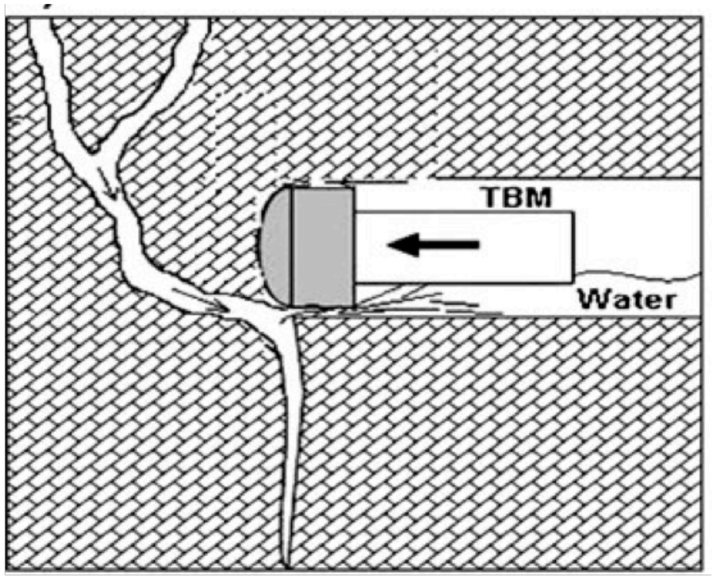


Figure 23. Illustration of the incident with the encountered karst in km 5+608 (Sharifzadeh et al., 2012).

- In km 17+814 another incident with karst occurred with resulting water inrush and tunnel flooding. The inflow reached 70 l/s with a water pressure of 16 bars. The water inundated the tunnel and prevented the operation of the TBM. The excavation was therefore continued from the other side of the zone from the Naleshgaran access tunnel by blasting method. The schematic plan of this is illustrated in Figure 24, a and b. In km 17+705 another water inrush occurred of about 250 l/s which completely flooded the tunnel within 4 days. The excavation through the karstic zone continued with TBM. Pre-grouting was performed before excavation, which took 9 months and 3027 tons of cement (Sharifzadeh et al., 2012).

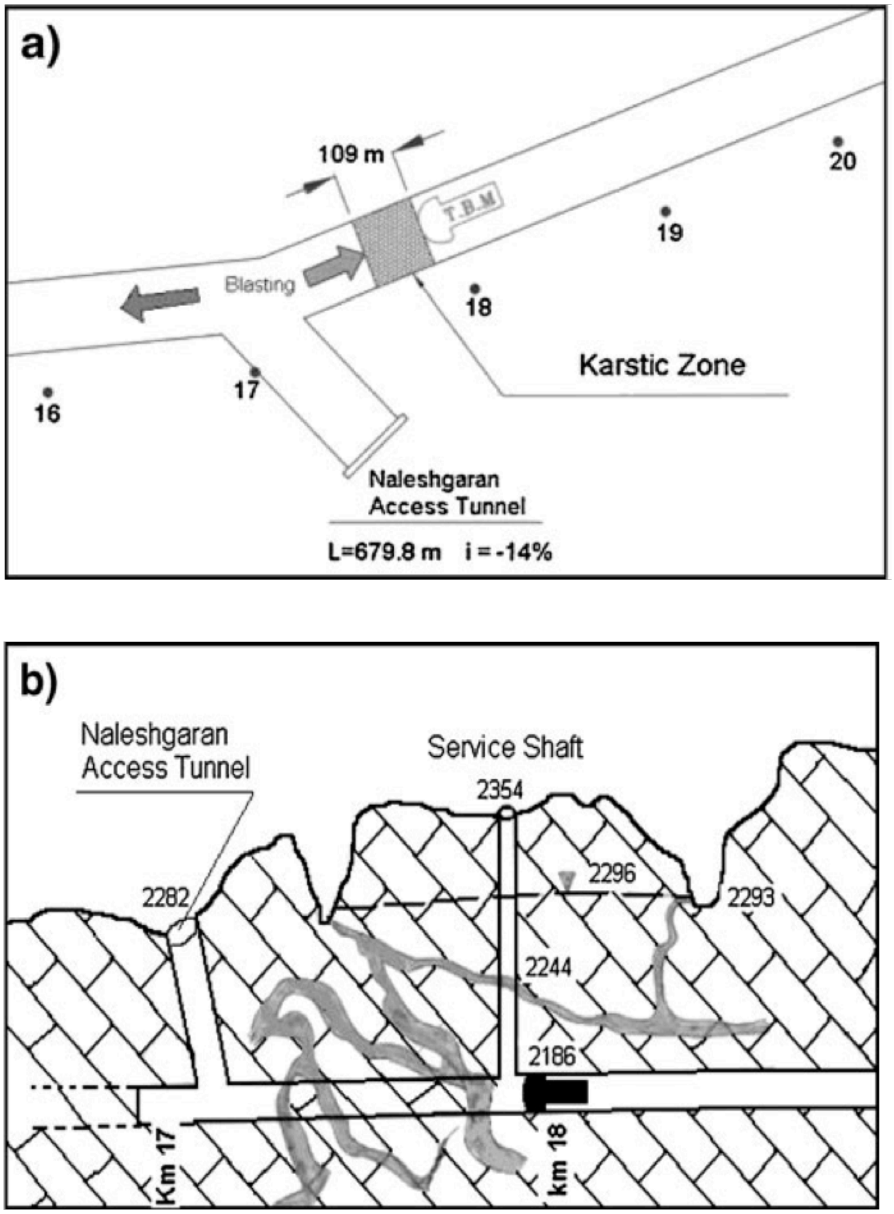


Figure 24. a) Schematic plan of the karstic zone in km 17+705 -17+814, b) Profile of the Kuhrang Tunnel in karstic zone in km 17+705-17+814 (Sharifzadeh et al., 2012).

- Water inrush was also experienced during the excavation of the Nasirabad access tunnel. At the portal the inflow into the tunnel was measured as 150 l/s the first week and after 7 months it was reduced to 50 l/s. The reason for this incident was karst features that had developed in relation to the cretaceous carbonate rocks along the Naleshgaran fault, which crosses the tunnel where the water inrush occurred. The geological condition in this section is shown in Figure 25. Pre-grouting and tunnel lining was not successful to prevent water inflow, so a drainage pipe was installed around the tunnel lining to connect the lower and upper parts of the karst channel (Sharifzadeh et al., 2012).

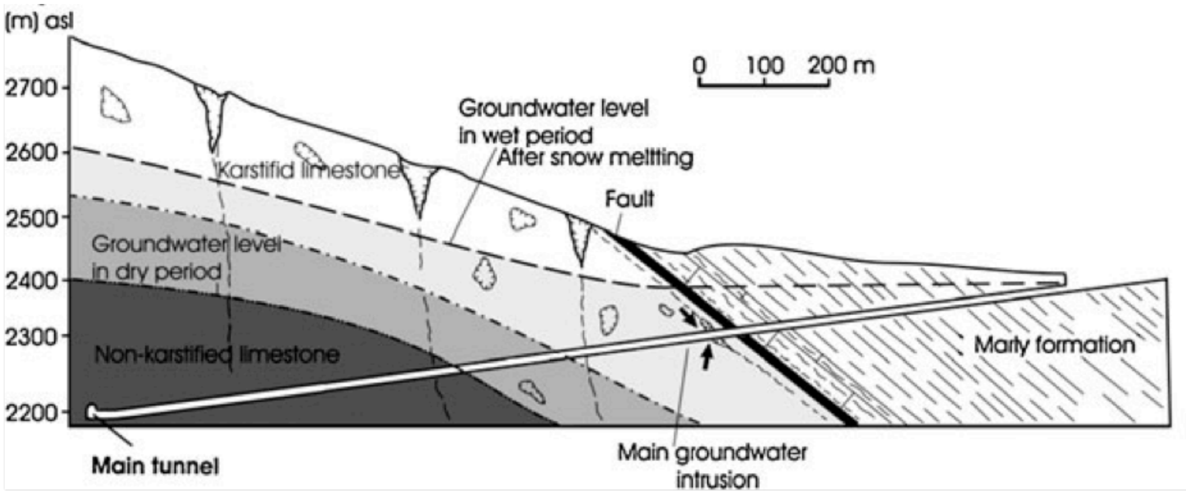


Figure 25. Geological conditions for the section with groundwater inrush through karstic fault zone in the Nasirabad access tunnel (Sharifzadeh et al., 2012).

- In km 8+063 a TBM was trapped under the filling material in a karstic cave as illustrated in Figure 26. The length of the cave was 20 m along the tunnel axis and it took 50 days to excavate through it (Sharifzadeh et al., 2012) with 75 cm stroke steps with steel ribs, shotcrete and lagging plate protection (Movahednejad, 2005).

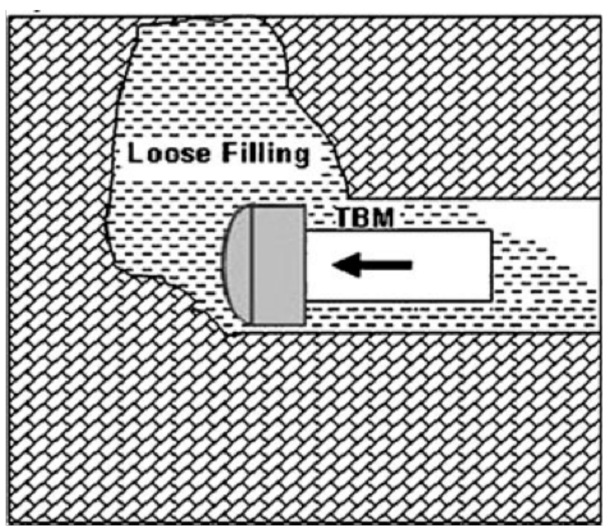


Figure 26. An illustration of the incident in km 8+063 where a cave with filling material was encountered (Sharifzadeh et al., 2012).

6.2 Stormwater Management and Road Traffic – The SMART project in Kuala Lumpur, Malaysia

The Klang River has been flooding the city of Kuala Lumpur regularly during monsoon season. Therefore some kind of flood control was strongly needed. The idea of a drain linkage between the Klang and the Kerayong Rivers to bypass the city centre was proposed as a solution for this. The major drawback for a large storm drain is that it would mostly be empty. So a dual use solution for the tunnel was implemented with a motorway section comprising two decks and a permanent storm waterculvert beneath the deck, as shown in Figure 27.

The tunnel was constructed in the period between 2003 and 2007. There is a holding/regulating reservoir at each tunnel end. In a flood situation, diversion facilities in the Klang River will direct excess flow into the upstream regulating basin. As this fills, controlled outflow into the tunnel will begin before discharge via gravity into the downstream regulating basin. The stormwater tunnel between the two regulating reservoir is 9.7 km long. It is only for the mid section of 3 km that the double deck highway will be incorporated (T&T, 2004).

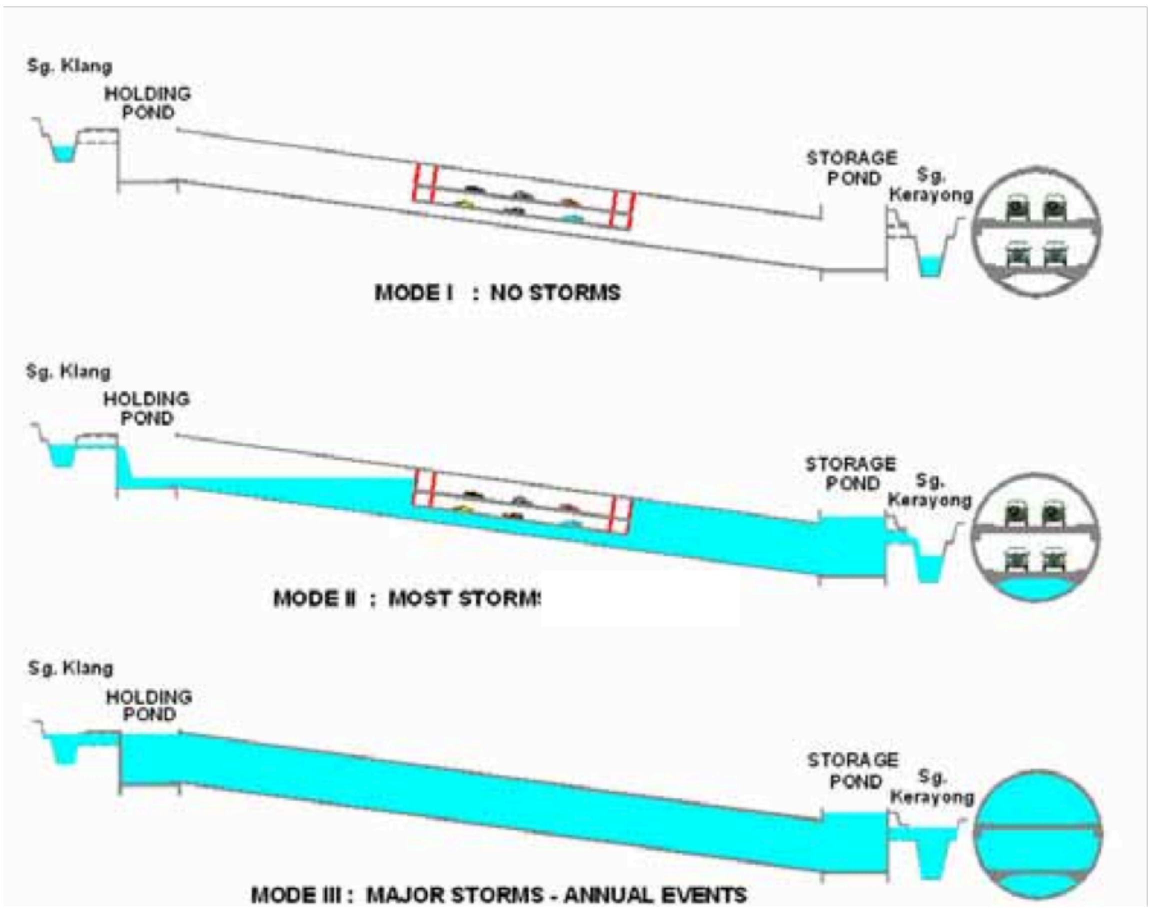


Figure 27. The three-mode operation of the SMART-project (Darby & Wilson, 2005).

6.2.1 Geology and pre-investigations

The tunnel route goes through the Kuala Lumpur limestone formation at shallow depth. To determine the profile of the rock face and to understand better the highly variable karstic limestone the following site investigations were carried out:

- 334 boreholes over a 12 km project length.
- Microgravity Survey conducted over sections where there was identified significant drop in rock head and presence of cavities.
- With respect to the above data, further tests such as resistivity measurements and seismic measurements were carried out. These tests cover nearly 50 % of the entire alignment.

The karst formation is characterized as mature and is covered with silty sand, peat and mine tailings. The thickness of the deposit is generally 4-5 m thick, but the depth to bedrock may vary due to karstic features. The groundwater table is 1.5 to 2 m below the surface. The limestone has generally low permeability, but groundwater can move through soft alluvial overlay and karstic features and cause major leakage while tunnelling.

The geological profile of the tunnel alignment as it was expected from the pre-investigations is shown in Figure 28. The tunnel alignment runs from the highest elevation through alluvium and mine tailings for about 2 km, and mixed conditions for the next 2.5 km. The mid section of about 5 km goes through karstic limestone. The last 1,5 km is in karstic limestone with shallow rock head cover and some residual soils (Klados & Parks, 2005).

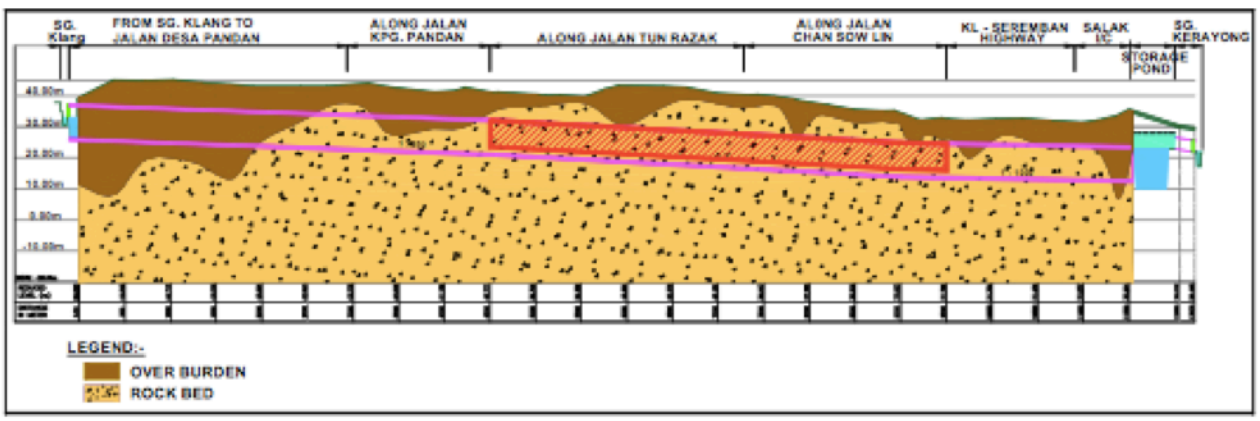


Figure 28. Geology of the SMART tunnel (Klados & Parks, 2005). A picture in a larger scale can be seen in Appendix 4.

6.2.2 Excavation

The area is very sensitive to groundwater drawdown. Groundwater drawdown was known to cause sinkhole incidents in the karstic areas and differential settlements in the alluvial cover. With the majority of the tunnel in karstic rock conditions and the need to control groundwater, it was considered appropriate to select slurry Mixshield TBM with an air bubble control system (T&T, 2004). The support pressure in the excavation chamber is precisely managed using an automatically controlled air cushion, which means that heterogeneous geologies and high water pressures of more than 15 bars can be controlled safely. Even small pressure and volume fluctuations in heterogeneous geologies can be controlled exactly. This allows good control of heave and settlement of the ground, which is important to avoid especially with small overburdens (Herrenknecht, n.d.) These machines are also able to work in the mixed face conditions at the site, with sudden drop in rock head and filled karstic caverns. Two Mixshields TBM´s with diameter of 13,21 m were purchased from Herrenknect in Germany (Klados & Parks, 2005).

To face the risk of running into karst cavities the machines were equipped with probe drilling facilities to probe ahead into the crown of the tunnel and into the invert. Seismic probing was also included in the TBMs to identify as far as possible the size and location of karst cavities and other obstacles in and ahead of the tunnel face (T&T, 2004). The machines were equipped with two probe-drilling rigs. However, probe drilling from the TBM was only used occasionally due to, among other things, the limited value provided over such a large tunnel-face. The seismic and resistivity geophysical survey methods, which were used to investigate the ground conditions from the surface, proved to be useful additional information to replace the original intention to supplement the site investigation with probing ahead of the TBM. It was performed monitoring of settlements in sensitive areas. Grouting was also performed from the surface as well as from the TBM in these sensitive areas if it was needed (Klados & Parks, 2005).

The karstic conditions of Kuala Lumpurs bedrock, and the unpredictable and extensive nature of the void system, is well known by all who have excavated tunnels in the city in the past. The challenging ground conditions were therefore recognized before the project was embarked on (T&T, 2004) and the necessary measures could therefore be implemented to avoid incidents with karst and water inflow. Choosing a TBM suitable to the ground conditions, thoroughly site investigations, ground monitoring of settlements and drilling and grouting from the surface has been useful measures to be able to tunnel through these ground conditions. The project has been challenging, but it has shown that tunnelling in this type of ground is possible (Klados, Kok, Parks, & Tavender, 2007).

6.3 The Alborz Service Tunnel

The Alborz Service Tunnel is excavated in advance of the Alborz twin tunnels for the purpose of site investigation, drainage of the rock mass, providing access for main tunnel excavation and for service, ventilation and drainage during operation of the complete tunnel system. These tunnels are a part of the Tehran freeway project in Iran, which is a new freeway between the capital Tehran and the city of Chalus. The total length of this project is 121 km. The Alborz Service Tunnel is the longest tunnel on the route with a length of 6.4 km (Wenner & Wannenmacher, 2009). It was constructed between 2006 and 2008. The location of the project is shown in Figure 29.

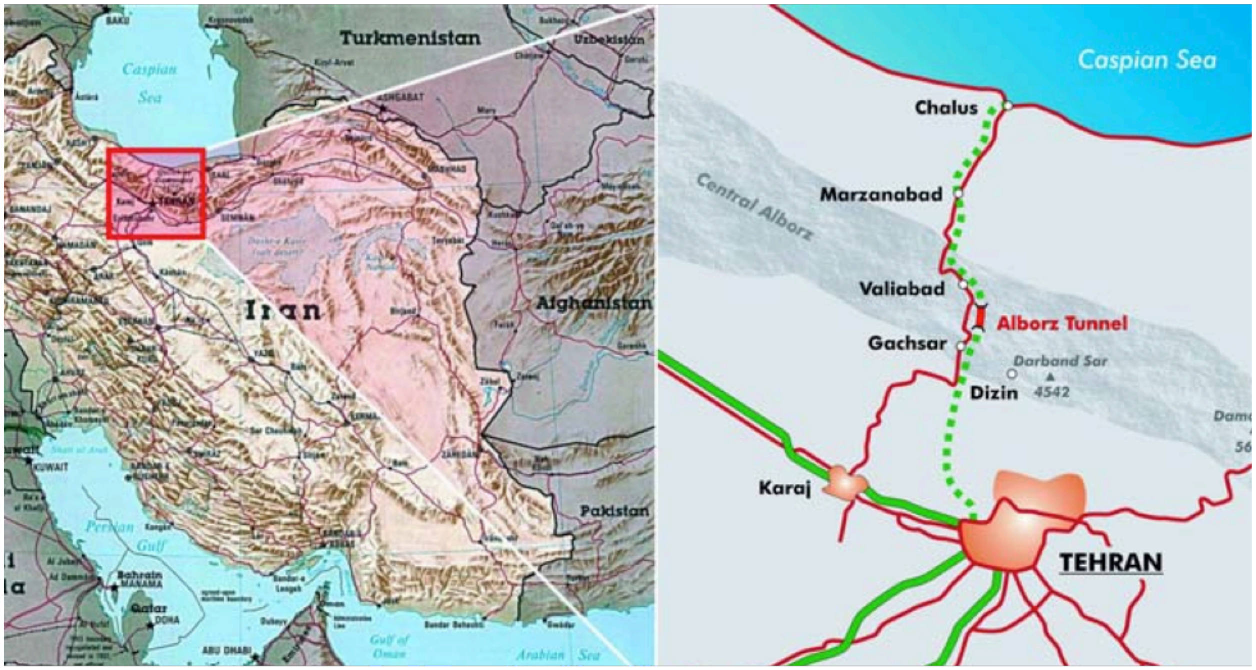


Figure 29. Project location of Alborz service tunnel (Wenner & Wannenmacher, 2009).

6.3.1 Geology of the project

The predicted geological conditions were complex and overall heterogeneous. From the north, Triassic and Jurassic argillites with some sandstones and thin coal layers were expected, followed by sandstone and then limestone formation. A thick fault zone was predicted approximately halfway along the tunnel alignment. Further south Oligocene clastic sediments were predicted; including massive gypsum/anhydrite bodies with a length of up to 300 m on tunnel level. At the surface the gypsum shows massive karstic features with unknown extend below surface. The rest of the tunnel was situated in Eocene tuffs, shales and other layered rocks. The overburden of the tunnel is up to 850 m. The rock types as they occurred along the tunnel alignment are given in Table 3.

Table 3. Geological units along the tunnel alignment of Alborz service tunnel (Hajali et al., 2013).

Rock description	Location along tunnel alignment (from north) (m)
Argillite and sandstone sequences with some coal lenses and dacitic dykes	0-2550
Sandstone, limestone and dacitic dyke	2550-3025
Tuff, anhydrite and andesite	3025-3425
Anhydrite with lenses of black tuff	3425-3950
Gray tuff with interbedding of black tuff	3950-4900
Anhydrite with interbedding of black tuff	4900-5390
Tuff, andesite, sandstone and limestone	5390-6374

6.3.2 Excavation

The excavation of the Alborz service tunnel was done mainly with an open gripper hard rock TBM from Wirth with diameter of 5,2 m, but also 314 meter of the tunnel was excavated with drill and blast method. The designed rock support consisted of a variety of predefined rock support types, ranging from only wire mesh in the crown for head protection against small stones to Swellex rock bolts in the crown every 75 cm plus wire mesh and 15 cm of shotcrete all around.

Several challenges occurred during excavation of the tunnel related to geological condition that had to be faced. This includes presence of methane gas, hydrogen sulphide and carbon monoxide, squeezing conditions that resulted in blockage of the TBM, high water ingress and inrush of filling material related to karst. The problems with water inflow during excavation of the tunnel and the measures taken to overcome the problems will be further discussed.

During TBM excavation, detailed geological mapping was performed. It was performed 145 percussive probe drillings ahead of the cutter head with a drill unit mounted to the TBM. Ten core drillings up to 105,7 m in length were performed ahead of the cutter head and above the shield. Three tunnel seismic prediction tests were also performed to investigate structures ahead of the TBM.

The northern section had only low water inflow. At TM 2582, first significant water ingress was encountered in the order of 125 l/s. At TM 3015 the tunnel encountered a fault zone with karstic void fillings when entering into the first anhydrite section. Water into the tunnel in the range of 110 l/s occurred together with approximately inrush of 100 m^3 mud and stone material. The other end of this anhydrite was unproblematic with respect to water ingress and did not cause any tunneling problems.

The main water ingress occurred at TM 4524 in a fault zone, with estimated water inflow of 800 l/s. This fault zone is not reported as karstic. To give an impression of the quantity of leakage through this zone, the water ingress in the TBM-backup is shown in Figure 30. After a period the water inflow decreased and it was possible to excavate through the zone. At the end of the southern anhydrite zone another karstic fault zone was encountered with ingress of 220 l/s. At that time the total water inflow into the tunnel was in the order of 600 l/s, which continued to reduce with time (Wenner & Wannenmacher, 2009).

Figure 30. Water inflow with estimated leakage of 800 l/s in the TBM backup (Wenner & Wannenmacher, 2009).

The main hazard of these water ingresses and inrushes into the tunnel was related to:

- Inrush of material and subsequently collapse of washed out voids eventually leading to the blockage of the cutter head.
- Harsh working conditions for workers with cold water inrush, especially during winter.
- Bad effect of water on various electrical installations and high voltage cables being submerged under water.

Most of the water ingress could be anticipated due to probe drillings, but the pre-injection of the zones to reduce leakage was never successful. High water pressures and flow rates, together with poor availability of equipment and experienced personnel for these works, was the reason for the bad success with pre-injection. It was decided to drain the water through the tunnel. Foam developing resin was used to fill up the voids and stabilize the collapsed ground. During the excavation of the Alborz service tunnel the challenges with karst had influence on the progress rate and on costs. However, the challenges could be passed without any major accidents (Wenner & Wannenmacher, 2009).

6.4 Discussion of experiences and key data from the international tunnel projects

In this chapter the experiences from the international projects that are useful for the topic of this master thesis will be discussed.

Kuhrang

In the Kuhrang water transmission tunnel several incidents occurred with karst, which serves as examples of how karst can affect the excavation of a tunnel and which mitigation measures that can be taken. The greatest water leakage that was experienced in this tunnel was related to a karst cave with water pressure of up to 30 bar and water inrush into the tunnel of 1200 l/s. After a short time the water had inundated the tunnel in its whole length. A karstic fault zone caused water inflow of 150 l/s into the tunnel. One of the encountered karst caves in this tunnel had a filling material that caused TBM-jamming. The measures that where taken against karst and water inflow were draining the water gravitationally through the tunnel, pre-injection in a karst section and installing drainage pipe outside of the lining to connect lower and upper part of a karst channel.

SMART:

The possibility of encountering karst was widely known from former construction projects in the limestone formation of Kuala Lumpur and the ground was known to be very sensitive to groundwater drawdown. Because of this the necessary measures was implemented to avoid problems with karst and water inflow while excavating the tunnel. . It is important to be prepared and to implement a system, which can cope with all of the anticipated situations in this type of ground. Thoroughly site investigation was performed to have a good overview of the ground conditions before excavation, but monitoring during excavation was also necessary. It is important to choose the correct TBM for the ground conditions, and in this case the Mixshield TBM´s performed well to control the water inflow and the challenging ground conditions. Grouting and pre-injection was available and could be performed from the TBM and from the surface. The experience from this project shows that if the necessary measures are taken during the planning and excavation of the tunnel, then tunneling in this type of ground can be performed without any major incidents.

Alborz service tunnel:

The experiences from Alborz service tunnel shows some consequences that karst can have for the tunnel construction; Inrush of material and collapse of washed out voids hampered the progress of the TBM and water ingress caused bad working conditions and posed a risk for the electrical installations in the tunnel. Most of the water ingress into the tunnel could be predicted with probing, but pre-injection was not successful partly due to lack of experienced personnel and available equipment. The mitigation measure was therefore to drain the water through the tunnel. Experiences with failed performance of pre-injection shows how important it is to have the necessary equipment available if karst is encountered and personnel who has experience in coping with such situations. Even though major challenges were met due to karst, which had a large influence on the progress rate and cost for the project, these challenges could be passed without any major accidents.

Key-data from the projects:

Key-data provided from the sources in the literature study are summarized in Table 4. The data is taken into account when answering the issue of this master thesis on which consequences karst can have for the excavation of the TBM-tunnel and which measures can be taken if karst is encountered. The amount of inflow into the tunnels is useful for the prognosis of water leakage in the TBM-tunnel at Lower Røssåga.

Table 4. Key-data from the literature study of the international TBM-projects.

	Kuhrang	SMART	Alborz
Main rock types	Limestone and marble	Limestone and different soils	Argillite, sandstone, limestone, gypsum and anhydrite.
Total tunnel length	23 km	9,7 km	6,4 km
Tunnel diameter	4,5 m	13,21 m	5,2 m
Highest water inflow from karst feature	1200 l/s from karst cave,	-	220 l/s from karstic fault zone
Highest waterpressure	30 bar in karst cave	-	-
Higest overburden over karst feature	1100 m	Ca. 10 m.	Ca. 800 m
Groundwater level above tunnel	50-500 m	Ca. 0,5-8 m	-
Type of TBM	Open TBM	Slurry Mixshield TBM	Open TBM
Measures to avoid incidents with karst during excavation	-	Probe drilling, seismic probing, monitoring of settlements.	Percussive probe drillings, core drillings, seismic probe drillings
Mitigation measures when encountered karst	Drainage through tunnel, pre-grouting, drainage pipe	Grouting and pre-injection from the surface and from the TBM in the sensitive areas	Pre-grouting, drainage through tunnel, foam developing resin

7 Analysis of potential problems related to water inflow for TBM-tunneling at Røssåga

In this chapter a prognosis for the expected problems with water leakage in the headrace tunnel will be presented on the basis of the engineering geological conditions and experiences from the relevant tunnel projects. A particular focus in the prognosis will be on karst since it is possible that this will cause significant water inflow if it is encountered. A review of the possibility of encountering karst during excavation has therefore been performed and taken into consideration when preparing the prognosis. Possible consequences of encountering karst for the construction of the TBM-tunnel and different suggestions for measures against karst will be discussed. Theory of groundwater in rock will first be presented to give an understanding of which characteristics influences the amount of water inflow into a tunnel.

7.1 Groundwater in hard rock

Tunnels are normally located beneath the groundwater table and the permeability of the rock masses is therefore crucial for the size of the water leakage (Løset, 2006). Permeability and hydraulic conductivity, usually given in m/s, are measures of the ability of a formation to transmit water. Permeability is a more rational concept than hydraulic conductivity as it is independent of fluid properties and depends only on the properties of the medium (Singhal & Gupta, 2010).

The hard rock types who make up the majority of the bedrock in Norway have generally low porosity and permeability when not jointed, so most of the water in the rocks moves through joints. Therefore will the permeability of the rock mass be dependent on the joint characteristics:

- Intensity of joints
- The length of the joints
- Joint fillings
- Roughness of the joints
- Aperture of the joints
- The orientation of the joints

It is therefore crucial to consider the jointing of the rock mass when predicting water leakage into tunnels. The chances of high permeability increase with higher degree of jointing. The length of the joints and the number of joint sets will influence the communication between the joints. Good communication between joints will give better permeability. Some joints have fillings and others can be open, without any fillings. The permeability of the joint will normally be small if the joint is completely filled. Hard minerals like quartz, feldspar and epidote normally gives very tight joints. This is also often the case for clay and calcite, but circulating water may wash out clay over time and calcite may be dissolved by water creating channels. The variations in the roughness of the joints can be great and influence the permeability of the joints (Løset, 2006). High roughness creates many channels where the water can flow (Holmøy, 2008). Joints parallel with the tunnel must be avoided because this poses very difficult conditions for water sealing (Klüver, 2000).

The difference in mechanical properties and different tectonic history will make a difference in the jointing and, as a consequence, for the permeability of the rock. The degree of jointing will be dependent on the rock type. During deformation a hard and stiff rock type will be more fractured than a softer rock type (Løset, 2006)

The rock stresses are important for the rocks permeability. The stresses, both the vertical and the horizontal will generally increase with higher overburden and compress the joints, so the chances are higher for open joints near the surface. This is why the conductivity often decreases with depth (Løset, 2006). Stresses can also occur that are caused by geological or topographical conditions, and this can lead to anisotropic stress conditions. Open water-filled joints on a deep of 200 meters bears witness of low stresses normal on the fracture plane (Klüver, 2000). It is often the steep joints that are oriented perpendicular to the minor horizontal stresses that can cause water leakage into the tunnel (Løset, 2006).

Løset (2006) defines a weakness zone as a zone in the earth´s crust where the rock mass quality is worse than its surroundings. Weakness zones normally causes special hydrogeological conditions when compared to the surrounding rock. The rock mass composition, structure and tectonic influence mostly determine the hydrogeological properties of the weakness zones. For example in the faults of the bedrock in Norway the side-rock of the weakness zone is often very conductive while the zone itself often contains a lot of clay that seals for water. Faults in other geological regions in Norway, as for example in the Oslo-fields eruptive rocks, can contain a larger amount of crushed rock and less clay then the fault zones of the bedrock (Klüver, 2000). Holmøy (2008) found support for a relationship between the major principal stresses and water-bearing discontinuities; water-bearing discontinuities are often sub-parallel with the major principal stress in the area. Holmøy (2008) also found a relationship between the weakness zones and the rock mass quality; it is encountered most water inflow in the marginal of the weakness zone, with Q-values between 0.6 and 15.

To summarize this chapter: Water leakage into tunnels is dependent on the permeability of the rock mass. The permeability of the rock mass is dependent on the type of rock, the characteristics of the joints and degree of jointing, the stress condition and the presence of weakness zones. By doing a detailed investigation of these characteristics it is possible to say a great deal about the probability of water leakage into a tunnel (Løset, 2006). As described in Chapter 3.2, the occurrence of karst can have a big influence on the permeability of rock mass, so this is also important to take into consideration.

7.2 A review of the possibility of encountering karst in the TBM-tunnel

Nordland with its high abundance of marble is one of the most important karst regions in Norway and it is known that karst has caused problems with water inflow into tunnels in the past. The likelihood of encountering karst in this area can therefore be considered to be great. It is therefore important to consider the possibility of encountering karst when planning tunnels in this area.

The dissolution of rocks can occur in almost all rock types that are calcareous. Having this in mind it is important to consider the rock types that will occur in the tunnel. At the site of Lower Røssåga the rock types that can be prone to dissolution and development of karst are primarily limestone/marble, skarn and also mica schist/mica gneiss if it is calcareous. The experiences with karst in marble from the nearby tunnel projects in the same geological area indicate the possibility of encountering karst in marble in the TBM-tunnel at Lower Røssåga. Pure and coarse-grained carbonates are more prone to karst than the rocks with silicates and fine-grained minerals. The risk of encountering karst could therefore be considered to be higher in marble and limestone than in the other rock types in the area. This was also experienced when karst was encountered at Fallfors. The karstified rock type at Fallfors was marble. There was nothing that indicated karstification of the calcareous mica schist, which was also observed near the intake. The calcareous mica schist at Fallfors has a completely different character than the marble; it has a lot of dark minerals and it is homogenous without much fracturing. However, the risk of encountering karst in mica schist must not be underestimated.

On the geological map from NGU (NGU, n.d. a)), the southern part of the tunnel that consists of mica schist and mica gneiss is reported as calcareous. None of the other sections, where these rock types occur, is reported as calcareous. Mica gneiss is reported to contain calcite in the area, but not along the tunnel alignment. However, the rock types vary more frequently than what is given on the map and it cannot be excluded that calcareous mica schist/mica gneiss can occur in the rest of the tunnel alignment as well. It is therefore possible that karst can occur in mica schist and mica gneiss in all of the sections along the tunnel alignment. Skarn is also a calcareous rock type, but since it is likely less pure than the marble and limestone it is likely to be less prone to karstification. But the risk of encountering karst also in skarn must not be underestimated.

There are some indications of karst development that can be observed on the surface at Lower Røssåga. Experiences from the site visit suggest that there are surface karst features nearby Tullavbekken, probably in relation with marble. According to the pre-investigations it has been observed karst features on the air-photos and in the field above the headrace tunnel, but the locations of these observations are not specified. The karst features that are observed are depressions filled with water, without any drainage (Statkraft, n.d.). This can be interpreted to be sinkholes or irregularities in the rock due to uneven karstification of the ground. From the air-photos of the area (finn.no AS, n.d.), several small lakes and smaller ponds (>10 m) can be observed. Most of these features are observed in the area above the southern part of the tunnel alignment. Smaller ponds could also exist, but they are too small to be observed on the air-photos. It is possible that some of these are formed in relation with karst, but to be able to determine this it is necessary to do observations and investigations in field.

Since karstic caverns are clustered in zones of close jointing or fracturing and the permeability of the rock is one of the most important factors for karst development, it is assumed that the probability of karst in the tunnel is higher in the weakness zones than the surrounding rock. There are several weakness zones in the area, where the risk of encountering karst during tunneling must be recognized.

The rocks that are beneath unconsolidated deposits will be more prone to karst. The deposits in the area can be seen on Figure 5. Most of the tunnel alignment has thin layers of humus and peat above, which probably will have a positive effect on the content of carbon dioxide in the water. The water will thus be more effective in dissolution of the rock beneath.

The karstification is dependent on depth; it decreases with depth especially below the lowest level of the groundwater table. Therefore will the chances of running into karst in the tunnel be dependent on the overburden. The overburden over the headrace tunnel is mostly 100-200 meters. Karst occurred down to a level of 200 m in marble at Staupåga diversion tunnel. The experiences from this tunnel show that it is possible to also encounter karst at this depth in the headrace tunnel at Røssåga. The sections with the lowest overburden will be the locations with the highest chances of encountering karst.

It is not possible to provide a concrete answer for whether karst will occur or not in the TBM-tunnel. However, what can be said for sure is that karst has been encountered in tunnels before in Nordland and the geological conditions of these tunnels are similar to the TBM-tunnel at Lower Røssåga (same geological area and similar rock types). Observations of karst terrain on the surface indicate the possibility of meeting karst in the tunnel. The question is whether the process of karstification has been extensive enough for the karst features to intersect the tunnel. To give a better answer to this it is necessary to perform more investigations in the area.

7.3 Remarks on prediction of water inflow

Experiences have shown that it is hard to predict the exact amount of water into a tunnel on the basis of the pre-investigations. It is however possible to do certain calculations, for example in the form of numerical modeling, but this method has not been widely used (Løset, 2006). According to Holmøy (2008), there are basically three main approaches that are used to predict groundwater inflow in hard rock tunnels:

- Analytical approaches: Formulas that are obtained through theory are used to calculate the water inflow into tunnels. In this type of approach it is necessary to do simplifications to be able to derive the formulas that are being used in the estimations.
- Semi-empirical approaches: Combining the experiences from previous tunnel projects to find correlations useful for estimating possible water inflow.
- Empirical approaches: The experiences from previous projects in the same geological setting are being used for estimating the water inflow. The results are presented in an expected range of water leakage.

Empirical approximations are most commonly used in Norway. The problem with this method is that the geological settings are never identical and the method can therefore only give rough assumptions. The relations between groundwater inflow and the geological conditions that are reviewed in the Chapter 7.1 are commonly used, but the result will depend on the people using them. Another problem is that a lot of experience gathered over time is not reported, and is therefore not easily available. The results will hardly be more than a qualified guess for water inflow, but the expected water leakage described as for example small, moderate, large and extremely large is possible to prepare (Holmøy, 2008).

7.4 Evaluation of the engineering geological conditions at Røssåga with respect to water inflow

To give a prognosis for water leakage into the TBM-tunnel it is necessary to evaluate the conditions at Røssåga with respect to water inflow. The degree of jointing, the weakness zones and the different rock types with their respective characteristics will be reviewed.

7.4.1 Degree of jointing

There are in general three joint sets in the area. Two of the joint sets (direction N-S) are oriented at a small angle to the tunnel. This is not favorable because joints that are parallel to the tunnel often give very difficult conditions for water sealing. The third joint-set has strike direction N110°Ø, which is a more favorable direction to the tunnel for avoiding water inflow. Two of the joint sets in the area have a steep fall and earlier experiences have shown that it is often the steepest joints that are most water-bearing (Klüver, 2000).

The intensity of the jointing is characterized as low to moderate. It is favorable that the jointing is not high because the intensity of the jointing has a great influence on the permeability of the rock and the leakage into a tunnel. Two water-bearing joints often give double as much water leakage as one joint (Klüver, 2000).

The overburden is mostly between 100-200 m, which means that the stresses at this depth probably have made a contribution to the closure of the joints. Some parts of the tunnel have lower overburden and it is therefore possible that these sections will experience more open joints. Open, water-bearing joints at 200 m depth can appear if the stresses are anisotropic and low stresses acts on fracture surfaces. The largest regional principal horizontal stress is assumed to be in the direction of NV-SE, so it is possible that the steep joints oriented in this direction will be the most open and water-bearing.

7.4.2 Weakness zones

There are several assumed weakness zones in the area (See Chapter 2.5) that crosses the tunnel alignment. They are characterized as small to moderate in size. The weakness zones in the calcareous rock types can be particular water-bearing if karst occurs. Also, the weakness zones that are oriented in the same direction as the largest horizontal stress (NV-SE) can be more open and water-bearing than the rest.

7.4.3 Evaluation of the different rock types with respect to water inflow

Mica schist: This rock type has in general low conductivity in the range of 10^{-12} to 10^{-13} m/s (NBG, 1985b). The joints in mica schists are often closed with high clay content. This can cause the formation of small to larger channels, which can lead to a large number of small leakages into the tunnel especially in relation to defects in the rock (Statens vegvesen, 2004). The mica schist often exhibits schistosity (jointing) along the foliation, which are normally formed by the occurrence of muscovite or biotite in the fractures (Goodman, 1993). Foliation in metamorphic rock has a big influence on groundwater movement and is the most significant discontinuity in this type of rock (Singhal & Gupta, 2010). As discussed in Chapter 7.2, the mica schist can be prone to karst and significant water inflow can occur if this is encountered.

Mica gneiss: The mica gneiss also exhibits jointing along the foliation, referred to as gneissosity, but the foliation is not as intense as in mica schist, which results in that the mica gneiss is stronger (Goodman, 1993). Mica gneiss is, as the mica schist, generally an impervious and soft rock type and usually has small joints that are easily squeezed together (Brattli, 2012). It is possible that the mica gneiss in the area can be calcareous, so it can also be prone to karst and water inflow can occur in relation with karst in this rock type.

Marble/limestone: The conductivity of limestone is generally classified as moderate in the range of 10^{-10} to 10^{-7} m/s. The conductivity of *karstified* limestone is classified as low and in the range of 10^{-7} to 10^{-4} (Singhal & Gupta, 2010) so the conductivity of this rock type is highly dependent on the dissolution of the rock. From the core logging it is observed that the marble is mostly pierced by two joint sets and that the joint surfaces are mostly unaltered or slightly altered with smooth/rough and undulating texture (Statkraft, n.d.). The observation of karst at Fallfors showed dissolution along two joint sets along the foliation and perpendicular to the foliation. Two joint set is favorable for the permeability, because it increases the interconnection between the joints. These rock types are considered to be the rock types with highest probability of developing karst and water inflow can occur in this rock type dependent on the dissolution of the rock.

Granite and granodiorite: The conductivity of granite and granodiorite is generally in the range of 10^{-10} to 10^{-11} m/s (NBG, 1985a), but the weathering of this type of rock has a high influence on the conductivity. The problem with weathering is generally concentrated in the upper 60 m below the ground surface, but in Scandinavia the glaciers have often eroded away the weathering profile. The groundwater is therefore primarily expected to be concentrated in fractured rock adjacent to faults and along extensive open joints (Goodman, 1993). Granite and granodiorite is normally not very jointed, but the rock types have a very high stiffness so it will fracture easily during deformation (Løset, 2006). In hard and unweathered granites the joints tend to have rough surfaces with considerable friction (Singhal & Gupta, 2010). The rock type is competent, meaning that the joints in this rock type tend to be more open with depth than other more incompetent rock types (Brattli, 2012).

Quartzite: The conductivity of quartzite is in the range of 10^{-11} to 10^{-13} m/s (Ingeniørgeologi berg, handbook). Quartzite is also a competent rock type, where the joints tend to be open down to a great depth (Brattli, 2012). Quartzite has often high conductivity in areas that have been exposed to tectonic, but there are also examples of weakness zones in this type of rock that is sealed because the rock mass is so crushed down and finely grounded (Klüver, 2000).

Calcareous skarn: Significant water inflow can occur in this rock type if karst is encountered.

To conclude, the rock types in the area that is likely to cause the highest water inflow are primarily the calcareous rock types like the limestone, marble and in minor extent calcareous skarn, mica schist and mica gneiss. The crystalline rock types, like the granite and granodiorite, are competent rock types, which is likely to have open joints down to a great depth. However, inflow can occur in minor extent in all of the different rock types in relation to the weakness zones and through open joints

7.5 Prognosis for expected problems with water inflow

The tunnel profile is divided into segments with respect to possibility of encountering water leakage during construction; the profile is shown in Figure 31. Each of the segments and the possibility of water leakage will be reviewed in this chapter. The possibility for water inflow is given on a scale of Very low, Low, Moderate, High and Extremely high. The amount of water inflow that can be expected into the TBM-tunnel will be evaluated in the end.

Section 1:

Rock type: Mica schist, mica schist with grenade and mica gneiss.

The rock types in this section are generally impervious and the joints tend to be closed. The southern weakness zones are directed parallel with the assumed largest principal stress in the area (NV-SØ) and can therefore be more water bearing when compared to the other weakness zones. Because of low overburden, which increases the possibility of karst and open joints near the surface, and several weakness zones the possibility of water inflow is given as Moderate.

Possibility for water inflow: Moderate

Section 2:

Rock type: Marble/limestone

The permeability of marble and limestone is very dependent on the dissolution of the rock. The overburden is moderate, but there are several weakness zones, which increases the permeability and thus the possibility of karst. The southern weakness zones are directed parallel with the assumed largest principal stress in the area (NV-SØ) and can therefore be more water bearing when compared to the other weakness zones and are therefore also likely to be more prone to karst. Marble and limestone are considered to be the rock types in the area that is likely to be most prone to karst. The possibility of water inflow is given as high because of the risk of encountering karst.

Possibility for water inflow: High

Section 3:

Rock type: Mica gneiss, granite/granodiorite, marble/limestone and possibly quartzite.

The overburden is moderate and there are few weakness zones in this section. Mica gneiss is generally considered to be impervious with joints that tend to be closed with depth. The joints in granite/granodiorite and quartzite tend to be more open. It is smaller section of calcareous rock types like marble/limestone and skarn, it is also a possibility that mica gneiss can be calcareous. Moderate overburden and few weakness zones decrease the possibility of karst when compared to other sections. Smaller water inflow can be expected to occur through open joints in granite and quartzite. The possibility of water inflow is therefore given as Low-Moderate.

Possibility for water inflow: Low-Moderate

Section 4:

Rock type: Mica gneiss and calcareous skarn.

Mica gneiss is expected to be generally impervious with closed joints. The overburden is Moderate and no weakness zones are expected. The overburden has probably contributed to the closure of joints. Experiences from site visit suggest that this section will be very tight and dry. It is possible to encounter karst in both mica gneiss and calcareous skarn, but the absence of weakness zones reduces the possibility of encountering karst. The possibility for water inflow in this section is therefore given as Low.

Possibility for water inflow: Low

Amount of water inflow:

As previously described, the amount of water inflow that can be expected during tunneling can be very difficult to evaluate, but the expected water leakage described as small, moderate, large and extremely large is possible to prepare. The water inflow, which was encountered in the tunnel projects from the literature study, shows a various amount of inflow from 220 l/s in a karstic fault zone in Alborz service tunnel, to 3000 l/s in a karst pipe in the tailrace tunnel at Upper Røssåga. The amount of inflow is dependent on a variety of parameters like the size and quantity of the karst features and groundwater level above the tunnel. The water inflow into the tunnel will be dependent on the geological structure that is encountered. In the TBM-tunnel *small* to *moderate* water leakages can be possible from open joints and especially in relation with weakness zones. The largest amount of water leakage is expected in large open karst conduits or in network of fractures widened substantially by dissolution. It is not possible to say if karst will occur or not, but if it does it can cause *extremely large* water inflow in the same range as what was experienced in the literature study.

Experiences from the tailrace tunnel at Upper Røssåga shows that the karst-system that was encountered during construction was connected with the small lakes and waters in the area, which were providing the channels in the tunnel with water. The river of Røssåga and Stormyrbassenget at Lower Røssåga is possible water-sources, which can provide the karst-system with large amount of water that can be drained into the TBM-tunnel. The karst-system that caused the inflow into the tunnel in both Staupåga and Upper Røssåga was also highly influenced by the seasonal variations, directing water from snowmelt and reacting quickly on variation in precipitation. The amount of water inflow from karst features into the TBM-tunnel can therefore be expected to be highly dependent on seasonal variations in precipitation and other factors like snowmelt.

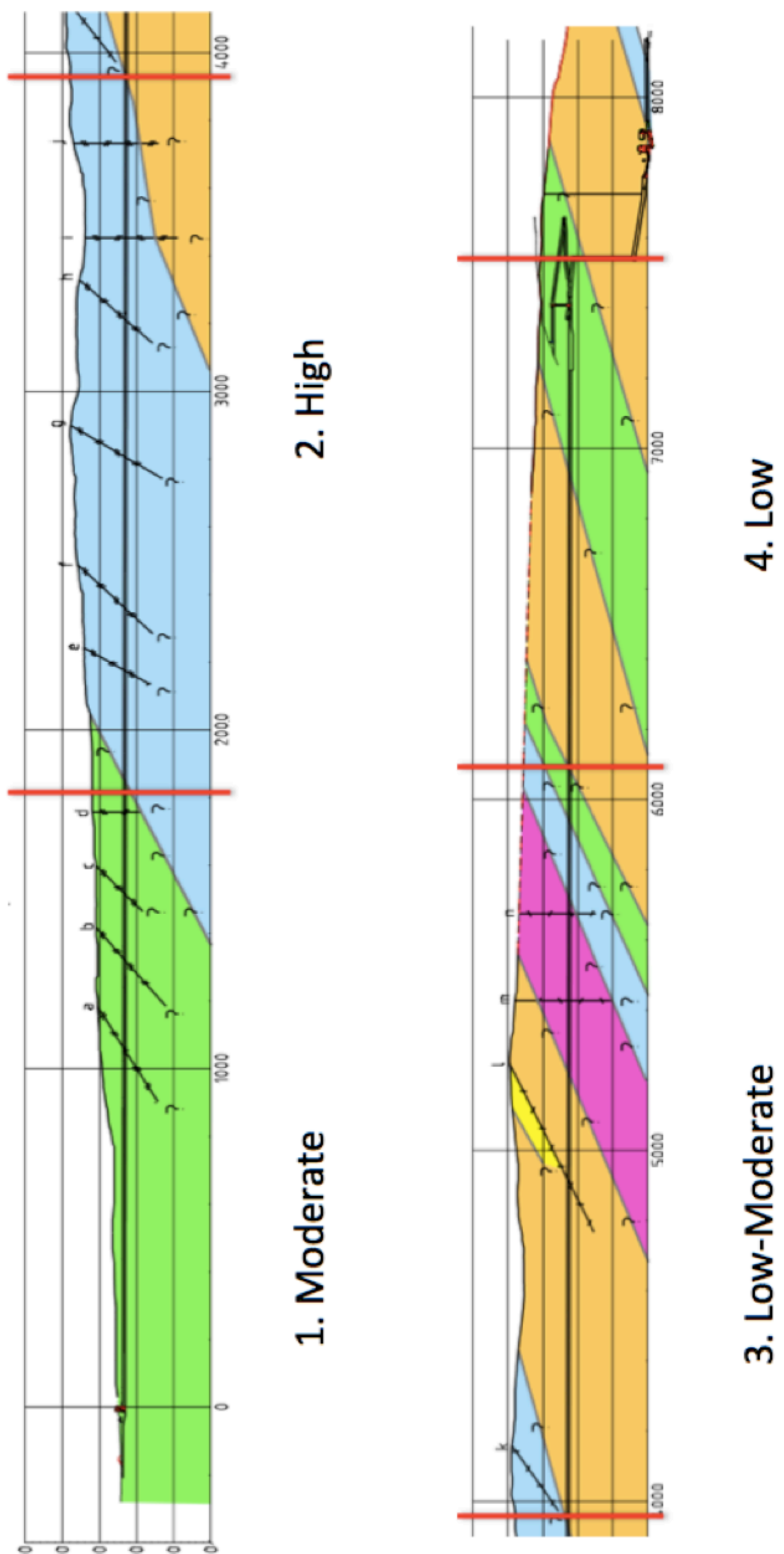


Figure 31. Prognosis for the possibility of water inflow in the TBM-tunnel. The geological profile is provided from the map in Appendix 5 and the map legend is also given there.

7.6 Influence of karst on the construction of the TBM-tunnel

Limestone and other carbonate rocks exhibit in general good geotechnical behavior and favorable tunneling conditions. However, if the rocks are karstic it may occur several problems when tunneling (Marinos, 2001). Experiences from the literature study show that in addition to water leakage into the tunnel under high pressure, instability of voids and inrush and instability of filling material can also occur if karst is encountered. Mixed face conditions can also occur in karst conditions, which is not favorable for the TBM (T&T, 2003). The focus will however be on the consequences of water leakage for the construction of the tunnel, since this is the issue of the master thesis.

7.6.1 Consequences of water leakage

The consequences for water leakage into tunnels can in general be divided in three (NFF, 2010):

- <u>Consequences for the environment:</u> Water leakage into tunnels can among other things cause draining and settlements of the surrounding soil, causing damage on buildings in populated areas and can dry out lakes and cause damage on the vegetation. Sometimes damage can occur that one in advance does not have any overview of. In towns and densely built-up areas, a single tunnel construction cannot be viewed in isolation, but the influence of existing and possibly future underground structures needs to be taken into consideration.
- <u>Consequences for the construction of the tunnel:</u> Leakage during the construction of a tunnel can among other things cause unfavorable working environment, stability problems in the tunnel and difficulty with performing work in a proper way such as charging and grouting.
- <u>Consequences for the permanent tunnel operation:</u> This depends on the use of the tunnel. The main problem with water leakage in water tunnels is the lost production because of water leaking *out* of the tunnel. Tunnels that normally should be filled with water might stand empty at times, so the environmental consequences of inflow of water must therefore be considered. However in road- and railway–tunnels water can cause damage during tunnel operation, with for example the electrical installations (NFF, 2010).

The functional requirements with respect to inflow and outflow of water are set with regard to these consequences. The following up of the functional requirements are done by performing water loss measurements before and after grouting, inflow/outflow measurements in the structure, pore pressure measurements, groundwater level measurements and measurements of settlements in the area above the tunnel. The water inflow criteria for inflow into a tunnel takes into account the consequences the water leakage will have. The inflow criteria are normally given as maximal allowed liter/minute/100 meter of tunnel (NFF, 2010). Experiences suggest that the requirements are possible to fulfill if pre-injection is performed correct (Statens vegvesen, 2004).

The inflow criterion that is set for a tunnel will have influence on the construction of the tunnel. This is because it is necessary to implement measures during construction to ensure that the tunnel will be adequate waterproof. This applies whether the inflow criterion is set due to consequences for the environment, for the construction operation or for the permanent tunnel operation.

For the TBM-tunnel at Lower Røssåga there are not set any specific water inflow criteria that need to be fulfilled. There are no buildings or settlement in the area above the tunnel and it is not considered a huge problem if drainage will influence the vegetation above, so there are no functional requirements set for the tunnel regarding the environment. It is not set any specific functional requirements for the permanent tunnel operation either, but leakage *out* of the tunnel during operation could be a problem (Johansen, 2014). If karst features are present it is important to seal the tunnel so the water does not connect to the karst system during operation of the tunnel. Since karst ground is highly permeable, it is possible with large water outflow from the tunnel. An acceptable volume for the water-outflow in an unlined/shotcrete-lined tunnel may be defined as 1-1,5 l/min/m tunnel (Panthi & Nilsen, 2010).

The main problem with water inflow in the TBM-tunnel will be for the construction of the tunnel, but there are not set any specific functional requirement for this either (Johansen, 2014). Water-inflow can cause adverse internal environment in the tunnel, leading the TBM to its functioning limit when the flow is higher than what can be drained or pumped out of the tunnel. Limiting situation for a TBM is when and where a machine doesn't work for what it was designed and manufactured for and the advance is significantly slowed down or even obstructed. The most limiting situations for a TBM are among others the occurrence of karstic caves and the inflow of groundwater (Peila & Pelizza, 2009).

An open TBM will be used at Røssåga (Nilsen & Log, 2013). The main principle for an open TBM is shown in Figure 32. The front of the TBM is a rotating cutter head, which holds disc cutters. While the cutter head is rotating the disc cutters are pushed into the rock. This creates fractures in the rock and causes chips to break away from the tunnel face. The TBM can be steered while pressing the gripper shoes to the sidewalls. This ensures the advancement forward (Robbins Company, n.d.). An open TBM has only a front shield, but elsewise it is free access to the rock mass (Nilsen & Log, 2013). Open TBMs are particularly exposed to problems with water inflow, instability and inrush of filling material. While in shielded TBMs the treatment of the rock can be performed inside of the shield in safe conditions. If an inadequate machine encounters this type of conditions, it is possible that the situation becomes critical and could cause severe delays, increases in costs and sometimes risks for the workers (Peila & Pelizza, 2009).

According to Barton (2000) it could also be difficult to install support when the conditions are wet and unstable.

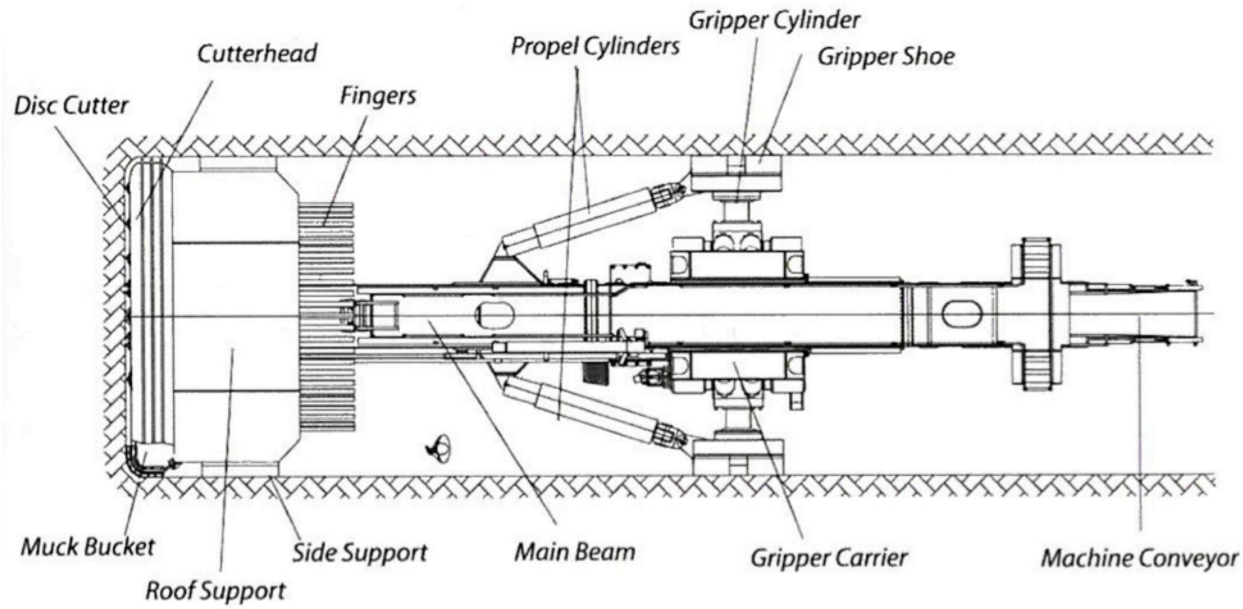


Figure 32. The main principle of an open TBM (Nilsen & Log, 2013).

According to the operation manager of the TBM at Røssåga, the machine can tolerate a lot of water as long as the tunnel is not inundated and it is also available a pump that will be able to pump away a certain amount of water. According to Johansen (2014) will the most likely problem with water inflow for the tunnel operation be the muck and difficulty with transporting this. The muck is transported to the conveyor belt by muck buckets (see Figure 32), which are scoop shaped part in the cutterhead. The muck bucket picks up muck and drops it into conveyor belt. From there it is transported to the back of the machine for removal out of the tunnel (Robbins Company, n.d.). It will be difficult to pick up and transport the muck if it is too wet. A lot of water will make the muck spread out in the working area. Water and mud will generally have an adversely impact on the working conditions. It is an advantage that the headrace tunnel is slightly inclined in the direction of construction, because this gives the water opportunity to drain away from the TBM through the tunnel (Johansen, 2014).

7.7 Possible measures if karst is encountered

The different measures against karst will be reviewed in this chapter. It is considered an advantage for a TBM to be equipped to probe ahead of the face, to detect highly permeable or weak zones when boring through areas with karst (Fischer et al., 2009), in this way pretreatment of the ground can be performed when needed. For TBM the technique has only been partly utilized in a few former TBM-projects and it seems to be the a general opinion that the technique is very time consuming and should be avoided as far as it goes in TBM-tunneling. However, according to Log (2011) detection and pre-injection of the ground is the most effective technique for handling zones with bad rock conditions and/or leakage, also for TBM-tunnels. The procedure for probe drilling and pre-injection performed from the TBM will therefore be reviewed as a measure against water leakage if karst is encountered. Some other measures against water inflow and instability related to karst in the TBM-tunnel will also be reviewed briefly.

7.7.1 Probing ahead of face and pre-injection

The main goal by performing injection is to satisfy the requirements for watertightness that is set for the tunnel, but it does also improve the rock mass quality. Injection can be done as pre-injection and post-injection. Pre-injection means injection ahead of face. Post-injection means injection behind face, but this is ineffective and not recommended (NFF, 2011).

7.7.1.1 Boring the pre-injection screen

Boring of probe holes or pre-injection holes are for TBM performed with special drilling hammers that are installed as close to the TBM´s head as possible, as shown in Figure 33. The standard is 1-2 drilling hammers per TBM. A drilling hammer could potentially do a systematic probe drilling in one hole without this interfering with the production, but if it is necessary with a more extensive pattern of probing or if boring of injection holes is necessary, then this would be very ineffective and time consuming. It is therefore recommended installation of more bore hammers if the diameter of the machine is large enough (Log, 2011).

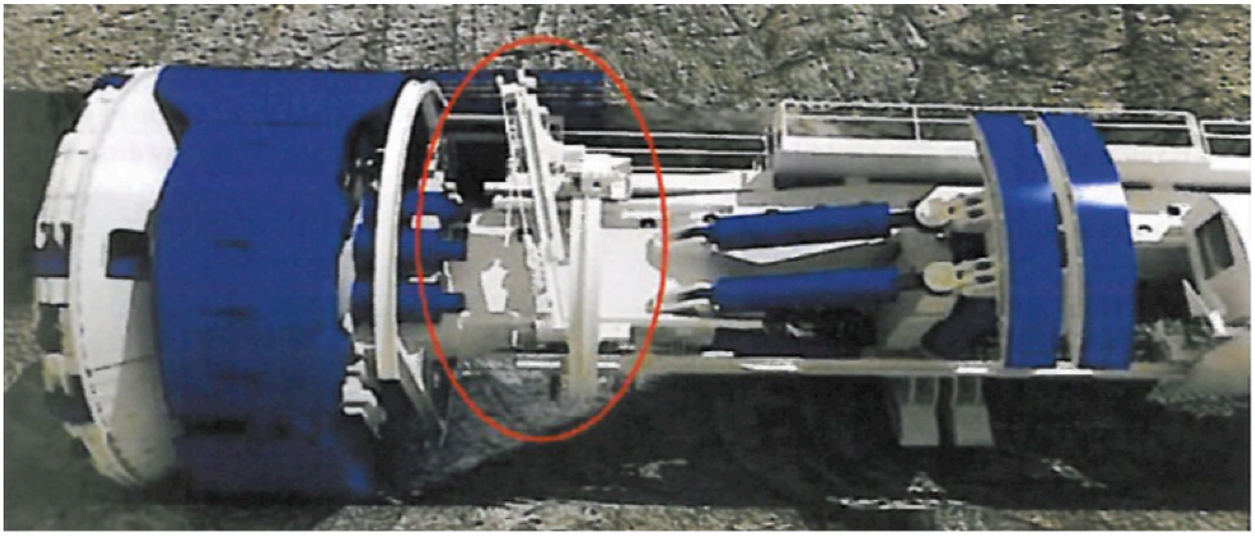


Figure 33. Placement of the drill hammers in open TBM (Log, 2011)

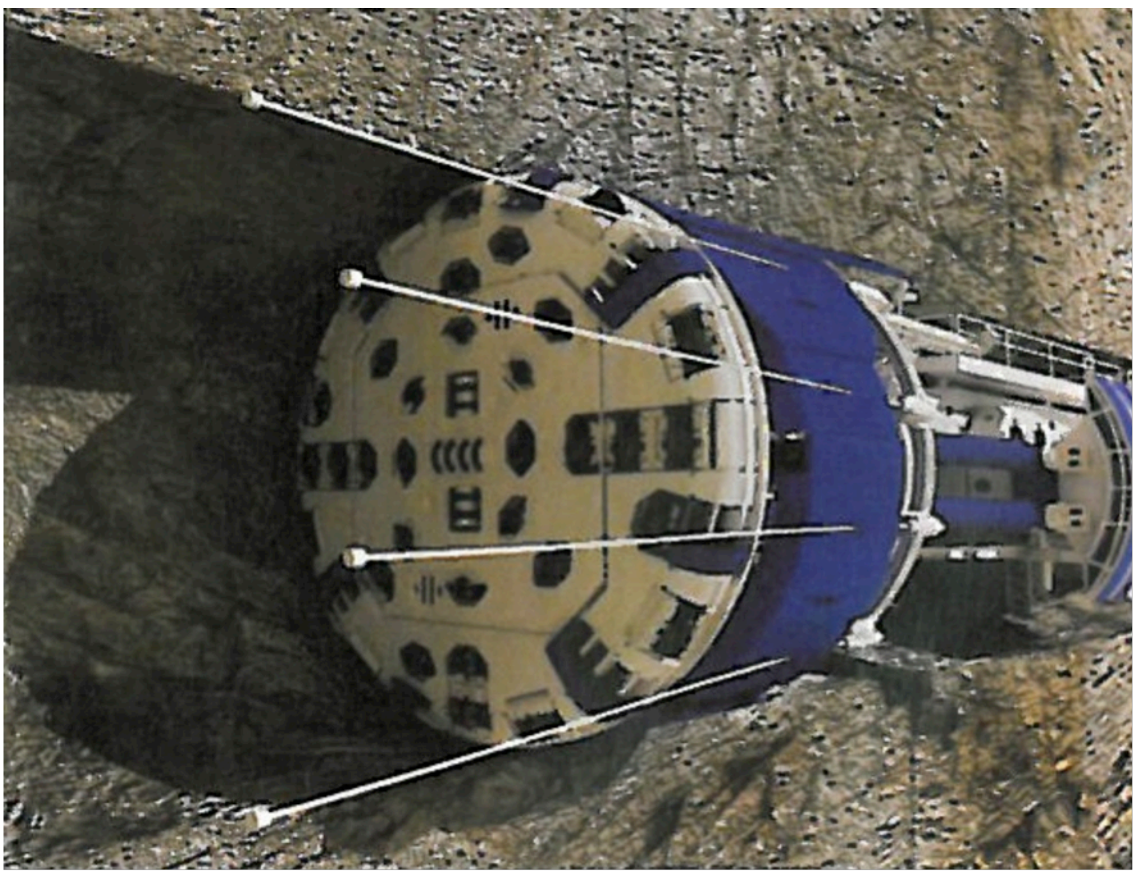


Figure 34. Drilling of holes for grout-curtains in open TBM (Log, 2011).

The probe drilling holes can be bored with a length of up to 30-60 meters. The holes should have an overlap of 5-10 m if using systematic probing. The design of the grout-curtains is based on the results from the probe drilling holes and a geological evaluation.

The injection holes are commonly bored with a length of 20-30 m with an overlap between the screens of 4-8 m (Log, 2011). Horizontal boreholes are normally performed through TBM-cutting head; inclined boreholes are normally possible from behind the cutter head in an open TBM (Peila & Pelizza, 2009). The drilling of pre-injection holes from an open TBM is shown in Figure 34.

The logging of the boreholes can be performed with MWD (Measuring while drilling), which is a term for the gathering and interpretation of the drilling data from the drill rig. This gives the opportunity to adjust the grout curtain, adaptation of the rods and to decide the limits for the grouting pressure and suitable injection material. The leakage from the boreholes can be measured by using a measuring rod. This is a short injection rod with an open packer. Using a stopwatch and a bucket, the amount of water is measured over a given time interval (NFF, 2011).

7.7.1.2 Equipment and materials for injection

To make the injection as optimal as possible it is various factors to take into account when selecting equipment and materials.

The injection-pumps and –mixers:

They are positioned in suitable locations on the rear rig of the TBM to facilitate the supply of cement and other grouting (NFF, 2011). There are great demands for the injection capacity and the documentation of the performed injection work. It is preferred that the injection-pumps have a capacity of 100 l/min when the pressure is approximately 80 % of the maximum permitted pressure. Equipment should be available for automatically logging of parameters like injected volume of different recipes in each hole, pressure and time for start/stop for different mixtures (NFF, 2010).

Packers:

Packers are used to close the boreholes so that sufficient pressure can be achieved. There are different types of packers. The choice of the packers is done on the basis of the application area for the packers. The choice needs to be suited for the pressure of injection; there are packers for low pressure (<60 bar) and for high pressures (60-100 bar). There are onetime packers and reusable packers. Onetime packer is the most used packer-type. It has a valve in the front and is installed in the borehole with an injection rod. There are locking washers at the end, which secures the packer in the hole, allowing the rod to be unscrewed after completed injection (NFF, 2010). The procedure for the installation of the packer in the borehole is shown in Figure 35. The packers must be placed at sufficient distance (normally 2 to 3 meters) ahead of the drill head so that the grout does not escape into the tunnel. This means that the grouting rods used in TBM-tunnels will have to be longer than those employed in drill and blast-tunnel (NFF, 2011). The packers are mostly 15 cm long, but can also be double-packers with length of 30 cm.

Reusable packers can be used in special injection jobs and when measuring water loss. They can be released after use and removed from the hole (NFF, 2010).

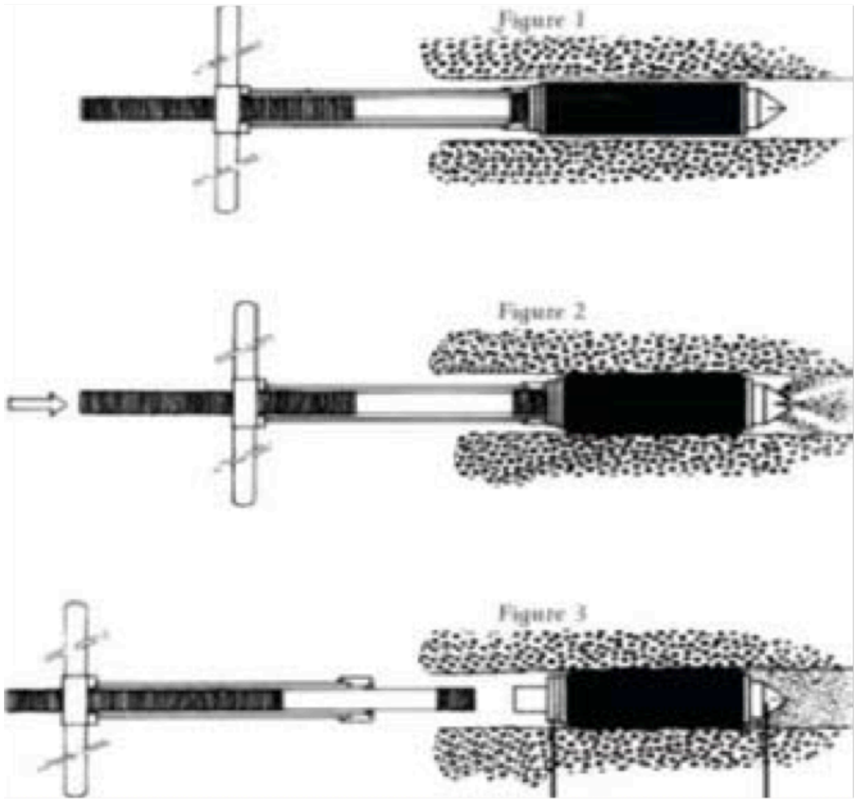


Figure 35. The principle of placement of the packers (NFF, 2010).

Standpipes and self-drilling injectable rods:

A standpipe can be used if so much water is encountered that there are problems with installing traditional packers. It has a vault that can be closed. Self-drilling injectable rods are used when the rock has very bad quality.

Injection-materials:

Cement-based products are the most common injection material in Norway. Injection-cement can be divided in two types: industrial cement and micro cement. Industrial cement has the coarsest material of those. Micro cement is normal cement that is extra finely ground. A general rule is that suspensions can penetrate fractures with an opening three times the suspensions maximum grain size (Statens vegvesen, 2004). This is why the size of the fractures is important to consider when choosing injection-material.

For the injection-material to work optimal it is necessary with additives. Stabilizing compounds is added to avoid bleeding, which means separation of water from the suspension. Silica slurry is the most used stabilizing compound in Norway. To prevent the masses from clump together superplasticisers are added to the suspension. Accelerating additives can be added if controlled curing is needed (NFF, 2010).

7.7.1.3 The performance of the injection

The injection is performed from stationary platforms near the drill hammers. When the procedure for the injection is to be determined it is important to consider the requirements for watertightness, the size and location of the tunnel and the rock mass that the tunnel will be located in. Injection can be performed both systematic and if deemed as necessary. Packers should normally be installed in all of the boreholes that are bored, before the injection starts. The holes in the sole should be injected first, because they are most demanding and should be injected before they are affected by other holes. Cement is in addition heavier than water and the gravity will then be utilized to squeeze the water forward and upward (NFF, 2010).

It is recommended that the injection is performed as a continuous process, with no interruption. Need for curing must be considered from the conditions. At normal water pressure it is normally no need for extra curing, but at high water pressure this might be necessary (NFF, 2010).

7.7.1.4 Special consideration for probe drilling and pre-injection in karst conditions

As previously described in Chapter 4.2 it can be difficult to detect water-bearing channels, like karst features, with probing. If a geological structure or condition, which might require grouting, is thought to exist, probe holes should be drilled at orientations designed to intersect and most effectively grout the suspected zone (Henn, 1996). For karstic conditions it is necessary to probe in several places at the face and the holes should be angled outwards to investigate outside of theoretical profile so that any karst phenomena that can affect the tunnel will be discovered (Dammyr, 2014). According to Nilsen & Log (2013) systematic probe drilling during excavation of the TBM-tunnel at Lower Røssåga will be performed and only *one* drilling hammer is installed on the TBM. If the water leakage through the probe drilling holes is considered large enough, pre-injection will be implemented. Only one probe hole will be drilled in each round, so the chances of detecting karst are not great. The probe holes will be logged with MWD (Johansen, 2014). Log (2011) recommends installation of more drill hammers so that the probe drilling process can be performed efficiently.

Klüver (2004) classifies 4 different rock mass types according to their rock mass properties and gives recommended injection strategies according to their classifications. One of the rock mass types includes rock masses with karst structures that have caused extreme fracturing and/or open caves/voids in the rock. Klüver (2004) does not recommend any particular injection material for this type of rock mass, but states that in these conditions it has previously been successful with the injection of concrete material with composition and grain size that is suitable for the opening of the fractures and for the rock conditions. Klüver (2004) also states that it often will be the correct choice to use cement-based materials with grain sizes dependent on the specified demand for water-tightness in the tunnel.

7.7.2 Different measures

If potential karst features are identified early, which represent large risk for the tunnel, then a realignment of the tunnel can be considered. This is dependent on early identification. In significant karst areas this may not be possible and planning and implementing mitigation strategies is the best way of dealing with such problems (T&T, 2003).

If instability occurs in the tunnel because of karst features and/or bad rock mass quality in combination with possible large water pressure it is important to have an effective system for installation of rock support. It is important to have a detailed scheme planned for all different situations that can be anticipated, and the necessary equipment within reach (Dammyr, 2014).

If pre-injection of the tunnel is not effective enough, an alternative solution can be steel or concrete lining to properly seal the tunnel (NFF, 2010). This would be to ensure that not too much water is leaking out during operation of the water tunnel, but installing this would be a worst-case scenario.

7.8 Discussion of potential problems related to water inflow for TBM-tunneling at Røssåga

Along the tunnel alignment for the new headrace tunnel at Lower Røssåga, there are several calcareous rock types, like marble/limestone, skarn and possibly mica schist and mica gneiss. Mica schist and mica gneiss is not given as calcareous rock types in the whole tunnel alignment, but the rock types in the area are more varying then what is given on the map. It must therefore be considered a possibility that calcareous mica schist and mica gneiss can occur even in those sections that is not given on the map. Since karst can occur in almost all calcareous rock types it is possible that karst will be encountered in all of the section where these rock types occur. It has been observed karst features in the area and above the tunnel alignment and experiences from former project indicates the possibility of encountering karst in the tunnel, but it is possible to conclude with whether karst will occur in the TBM-tunnel or not. To be able to predict karst at tunnel level it will be necessary to perform more investigations. More field mapping to identify critical sections and borehole investigations to predict karst at tunnel level is recommended. However, the possibility of karst in some sections of the tunnel is higher then in the rest. This applies to sections with marble and limestone, areas with lower overburden and areas with higher permeability, as for example in the weakness zones.

A prognosis has been prepared for the expected problems with water inflow into the tunnel. The main problem with water inflow is expected to occur in the sections where there is possibility of karst. However, karst is very unpredictable and so is the prediction of water inflow, so the prognosis possesses high uncertainty. The amount of water inflow if karst is encountered can possibly be extremely large. Minor water leakages, in the range of small to moderate can also occur in weakness zones and through open joints, particular in the crystalline rocks.

At Røssåga the water inflow into the tunnel must mainly be controlled with respect to the construction operation. This is because the consequences of water inflow for the environment and for the permanent tunnel operation are not considered to be great. A lot of water can lead the TBM to its functioning limit and will hamper the TBM-progress, but the TBM, which are operating at Røssåga, can withstand a lot of water as long as the tunnel is not inundated with water. The most likely influence of water inflow will be difficulties with the muck transport in the TBM and poor working environment with water and muck spreading out in the working area. Water can also cause unstable conditions and difficulty with installing rock support. An open TBM is particular exposed to these problems, since it does not have a shield and it is open to the rock mass in the working area. It is therefore very important to keep the water inflow to an acceptable amount. It is also important to have planned a system for all expected events that can occur in relation to karst, which might pose a risk for the tunneling, and to have available equipment and experienced crew for handling these situations. In brief, water inflow will hamper the progress of TBM because of poor working conditions, difficulty with installing support, unstable conditions and the time it takes for implementing measures to control the water inflow.

Probe drilling and pre-injection is also available with TBM. It is considered the best method for controlling water inflow and for treating difficult rock conditions, also for TBM. Karst can be predicted in advance and the treatment of the rock can be performed while the TBM is still in safe conditions. The problem with this, especially in relation to karst, is that the chances of encountering water-bearing channels is low. Many probe drilling holes is necessary to be able to reliably detect these features. Too much investigation during construction will hamper the progress of the TBM, while insufficient investigation can cause incidents, which will also hamper the progress of the TBM. At Røssåga only one probe drilling hole will be drilled at the time and the chances of detecting such karst conditions before its too late are therefore considered to be small. It would be advisable to perform more investigations from the surface in areas that are considered to have high risk. Surface mapping of karst features may be performed and borehole investigations from surface in sections, which are identified from the mapping. In this way, critical sections of the tunnel can be identified and the amount of probe drilling holes could be increased through these sections. It is also possible to install more drill hammers, which can operate at the same time to increase the efficiency of the probe drilling. In this way, an appropriate balance between the investigation and the production will be obtained. If the risk of karst and water inflow from investigations is considered to be too great, a realignment of the tunnel can also be discussed.

8 Conclusion

The main conclusions in this thesis are:

- Karst ground is very complex and unpredictable, and the focus of karst detection before excavation should not be on detecting all of the karst features along the tunnel alignment, but enough information should be obtained to be able to plan the tunneling sufficiently. It is considered appropriate to perform mapping of surface features, which might foretell the degree of dissolution and perform borehole investigations in critical section. In this way karst at tunnel level may be predicted.
- The possibility of encountering karst in the tunnel is present in all of the calcareous rock types such as marble/limestone, calcareous skarn, mica schist and mica gneiss. Experiences and observations indicate a possibility for meeting karst in the tunnel, but it is not possible to conclude with whether karst will occur during tunneling or not. However, the possibility of encountering karst is present and some sections in the tunnel have higher possibility of karst then the rest of the tunnel.
- A prognosis has been prepared for the possibility of water inflow into the tunnel and one section with marble is given High possibility for water inflow. The main problem with water inflow into the tunnel is expected to be in those sections with possibility of encountering karst. Extremely large water inflow can occur if karst is encountered.
- Since detection of karst before excavation is very uncertain, it is important to perform investigations during the construction. Measures against karst and water inflow can be probe drilling and implementing pre-injection if karst is detected. This is considered the best method for controlling water inflow, also in TBM-tunneling.
- The TBM at Røssåga can withstand a lot of water as long as it is not completely inundated. For the TBM-tunnel the most likely consequences for water inflow is considered to be poor working conditions because of water and muck in the working area. Difficulty of installing support in wet and unstable conditions can also occur. An open TBM is particular exposed to these problems. It is also time consuming to implement measures for controlling water inflow. In short, water inflow into the tunnel will hamper the progress of the TBM.
- It is recommended more investigations from the surface in areas where the possibility of encountering karst is considered to be great. In this way it is possible to identify high-risk sections for water inflow. More probe drilling should be performed in these sections to be able to reliably detect karst features in front of the tunnel face.

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10 Appendix

Appendix 1. The Q-method (NGU, 2013)

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m³)	0-25
B	Poor	(20-27 joints per m³)	25-50
C	Fair	(13-19 joints per m³)	50-75
D	Good	(8-12 joints per m³)	75-90
E	Excellent	(0-7 joints per m³)	90-100

Note: i) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value

ii) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate

2	Joint set number	J_n
A	Massive, no or few joints	0.5-1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random heavily jointed "sugar cube", etc	15
J	Crushed rock, earth like	20

Note: i) For tunnel intersections, use 3 x J_n

ii) For portals, use 2 x J_n

3	Joint Roughness Number	J_r
	a) Rock-wall contact, and b) Rock-wall contact before 10 cm of shear movement	
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5

Note: i) Description refers to small scale features and intermediate scale features, in that order

	c) No rock-wall contact when sheared	
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1

Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening)

iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction

4	Joint Alteration Number	Φ_r approx.	J_a
	a) Rock-wall contact (no mineral fillings, only coatings)		
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.		0.75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening).	20-25°	3
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc gypsum, graphite, etc., and small quantities of swelling clays.	8-16°	4
	b) Rock-wall contact before 10 cm shear (thin mineral fillings)		
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but <5mm thickness).	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness).	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5mm thickness). Value of J_a depends on percent of swelling clay-size particles.	6-12°	8-12
	c) No rock-wall contact when sheared (thick mineral fillings)		
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated.	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	8-12
N	Thick continuous zones or bands of clay. Strongly over-consolidated.	16-24°	10
O	Thick, continuous zones or bands of clay. Medium to low over-consolidation.	12-16°	13
P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

5	Joint Water Reduction Factor	J_w
A	Dry excavations or minor inflow (humid or a few drips)	1.0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0.66
C	Jet inflow or high pressure in competent rock with unfilled joints	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	0.33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0.1-0.05

Note: i) Factors C to F are crude estimates. Increase J_w if the rock is drained or grouting is carried out

ii) Special problems caused by ice formation are not considered

6	Stress Reduction Factor	SRF
	a) Weak zones intersecting the underground opening, which may cause loosening of rock mass	
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see 6L and 6M	10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)	7.5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)	5
D	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)	2.5

Note: i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening

	b) Competent, mainly massive rock, stress problems	σ_c/σ_1	σ_θ/σ_c	SRF
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0.3-0.4	0.5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
K	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	<2	>1	200-400

Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to 0.75 σ_c. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to 0.5 σ_c, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory)

iii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)

	c) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure	σ_θ/σ_c	SRF
M	Mild squeezing rock pressure	1-5	5-10
N	Heavy squeezing rock pressure	>5	10-20

Note: iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)

	d) Swelling rock: chemical swelling activity depending on the presence of water	SRF
O	Mild swelling rock pressure	5-10
P	Heavy swelling rock pressure	10-15

Note:
The values for J_r and J_a should be chosen based on the orientation and shear strength, τ, (where $\tau \approx \sigma_n \tan^{-1}(J_r / J_a)$) of the joint or discontinuity that gives the most unfavourable stability for the rock mass, and along which failure most likely will occur.

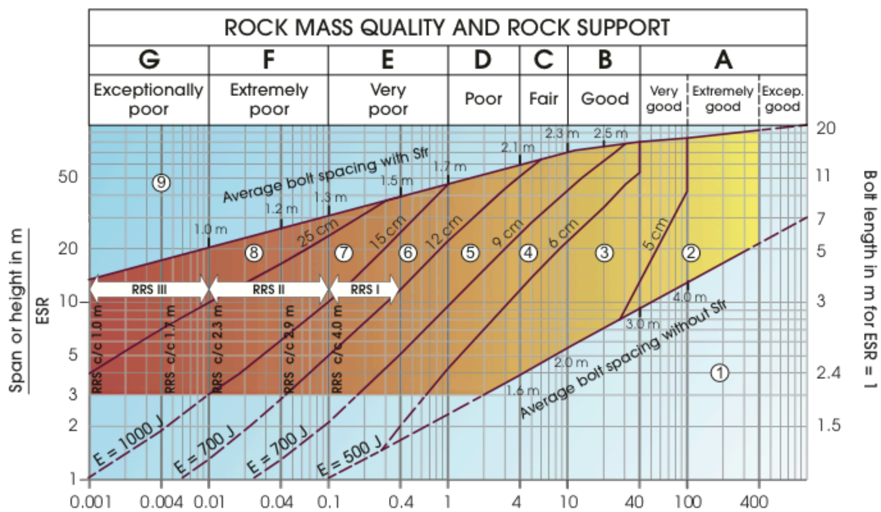
Table 7 ESR - values		
	Type of Excavation	**ESR**
A	Temporary mine openings, etc.	ca. 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	ca. 2.5 ca. 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks) water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0.8
G	Very important caverns and tunnels with a long lifetime, ≈ 100 years, or without access for maintenance.	0.5

For the types of excavation B, C and D, it is recommended to use ESR = 1.0 when Q ≤0.1. The reason for that is that the stability problems may be severe with such low Q-values, perhaps with risk for cave-in. ESR together with the span (or wall height) gives the Equivalent dimension in the following way:

$$\frac{\text{Span or height in m}}{\text{ESR}} = \text{Equivalent dimension}$$

Table 8 Conversion from actual Q-values to adjusted Q-values for design of wall support		
In rock masses of good quality	Q > 10	The actual Q-value is multiplied by 5
For rock masses of intermediate quality	0.1 < Q < 10	The actual Q-value is multiplied by 2.5 (In cases of high stresses the actual Q-value is used)
For rock masses of poor quality	Q < 0.1	The actual Q-value is used

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$



Rock mass quality $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$

Support categories

① Unsupported or spot bolting
② Spot bolting, **SB**
③ Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B+Sfr**
④ Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500)+B**
⑤ Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700)+B**
⑥ Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E700)+RRS I +B**
⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000)+RRS II+B**
⑧ Cast concrete lining, **CCA** or **Sfr (E1000)+RRS III+B**
⑨ Special evaluation

Bolts spacing is mainly based on Ø20 mm

E = Energy absorbtion in fibre reinforced sprayed concrete

ESR = Excavation Support Ratio

Areas with dashed lines have no empirical data

RRS - spacing related to Q-value

I: **Si30/6 Ø16 - Ø20 (span 10m)**
D40/6+2 Ø16-20 (span 20m)

II: Si35/6 Ø16-20 (span 5m)
D45/6+2 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

III: D40/6+4 Ø16-20 (span 5 m)
D55/6+4 Ø20 (span 10 m)
D70/6+6 Ø20 (span 20 m)

Si30/6 = Single layer of 6 rebars, 30 cm thickness of sprayed concrete

D = Double layer of rebars

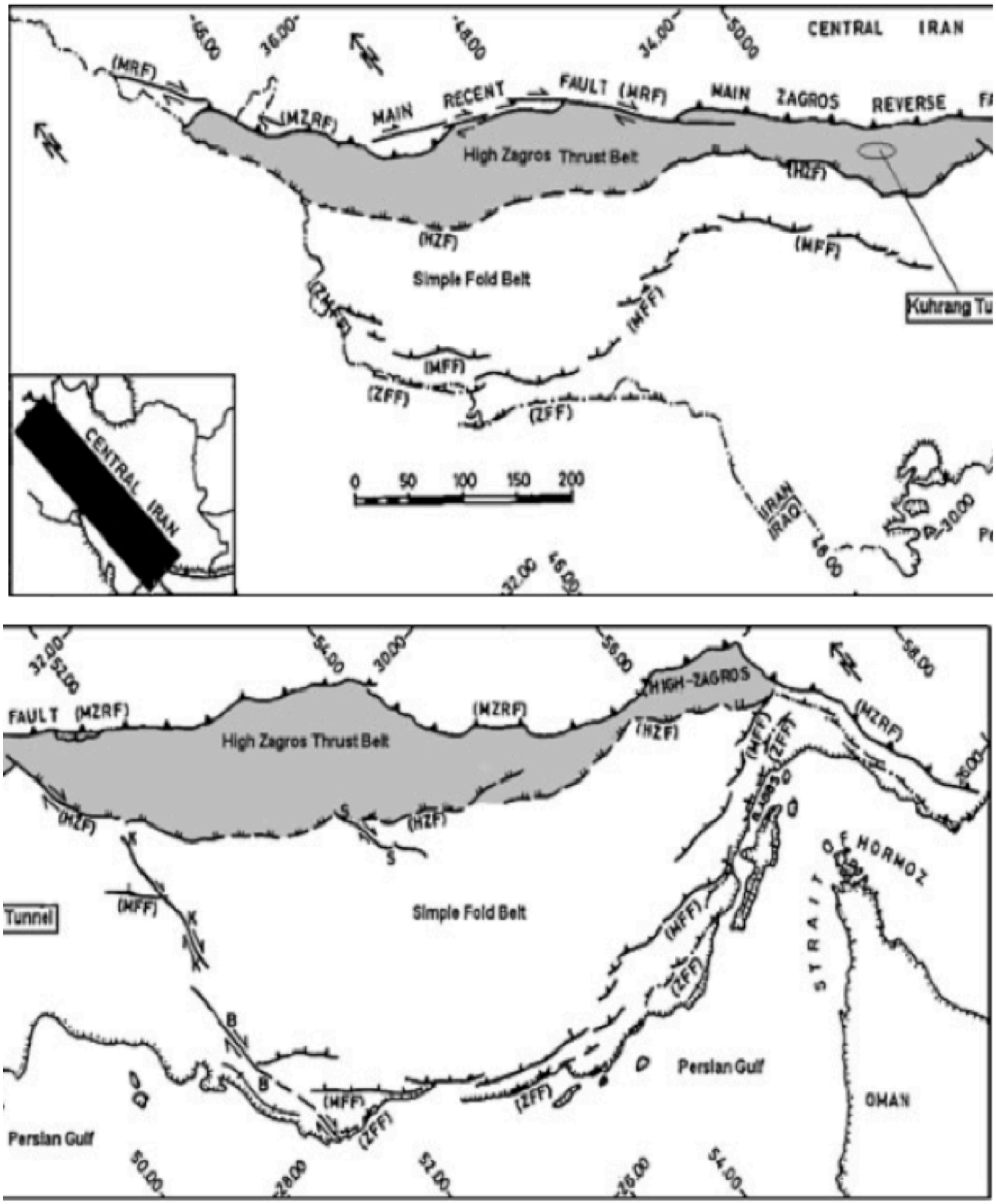
Ø16 = Rebar diameter is 16 mm

c/c = RSS spacing, centre - centre

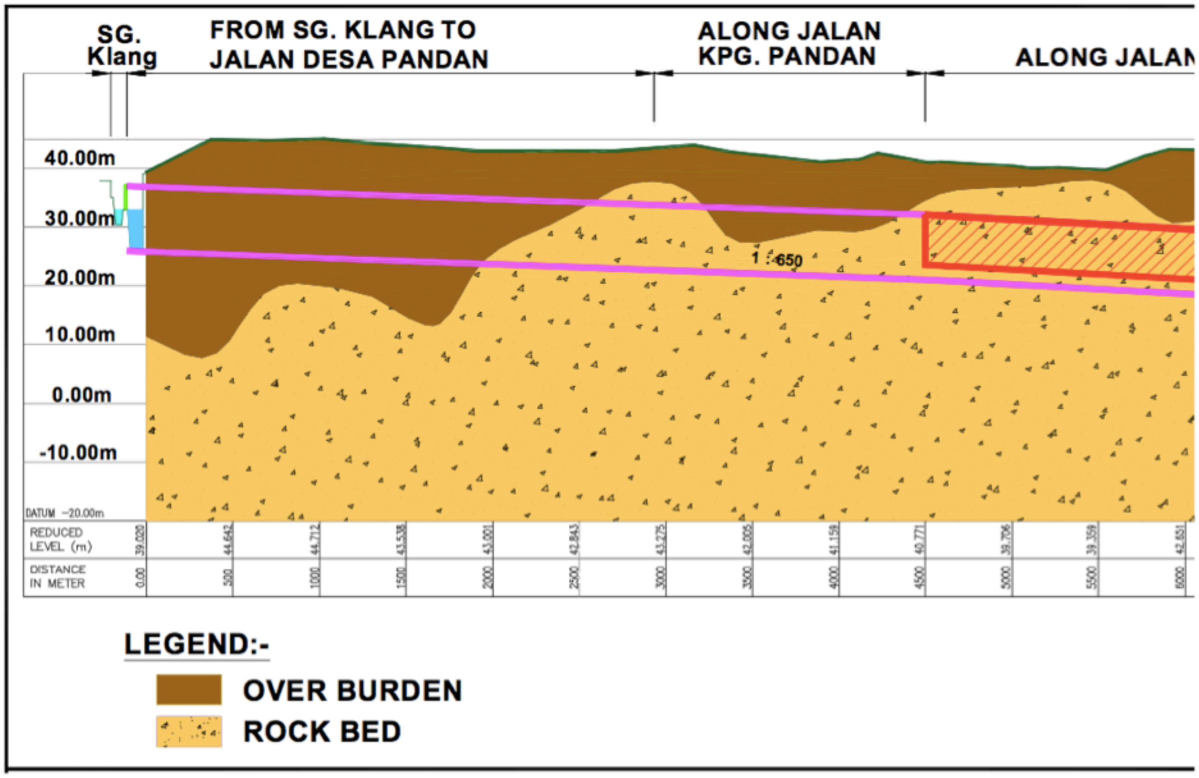
Appendix 2. Strike and dip measurements from the site visit.

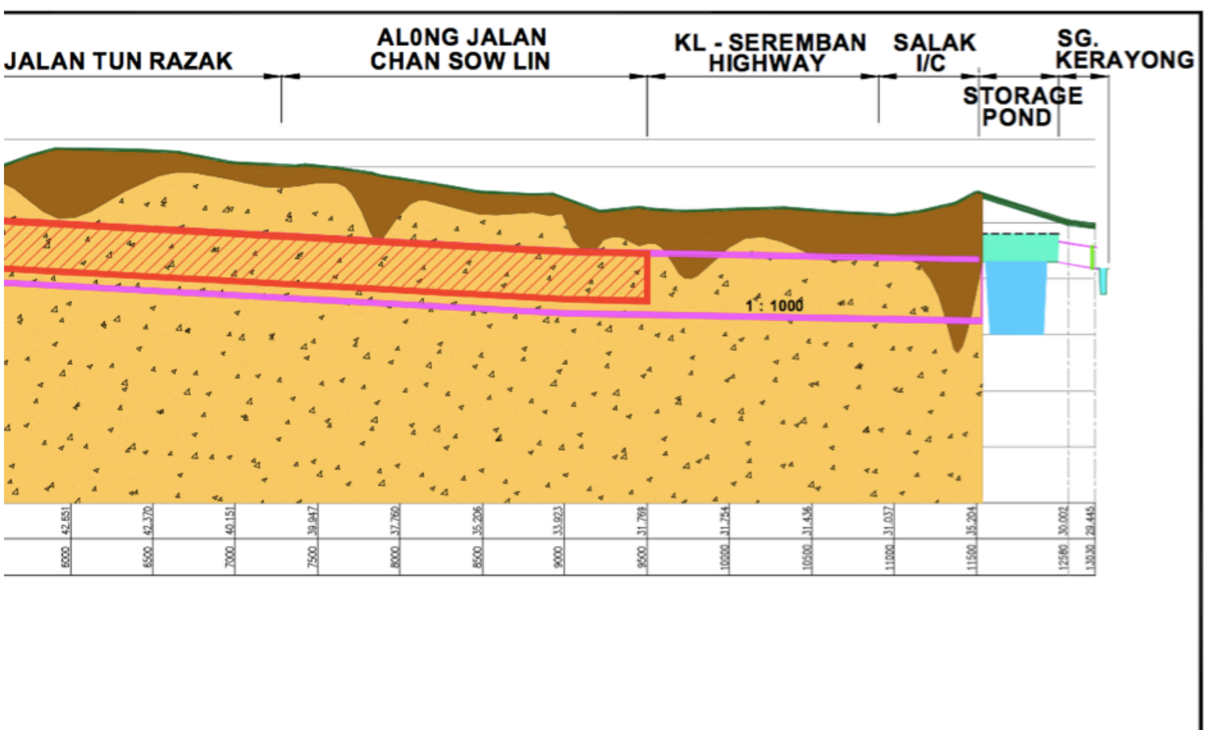
Dip (Degrees)	Dip-direction (degrees from North)	Comments
28	271	Foliation
48	280	Foliation
30	260	Foliation
18	285	Foliation
70	320	Joint
63	305	Joint
76	310	Joint
44	250	Joint
19	270	Foliation
19	263	Foliation
24	246	Foliation

Appendix 3. Major morphotectonic units of the Zagros Region of Iran and location of the High Zagros thrust Belt and the project location of Kuhrang water transmission tunnel (Sharifzadeh, Uromeihy, & Zarei, 2012).

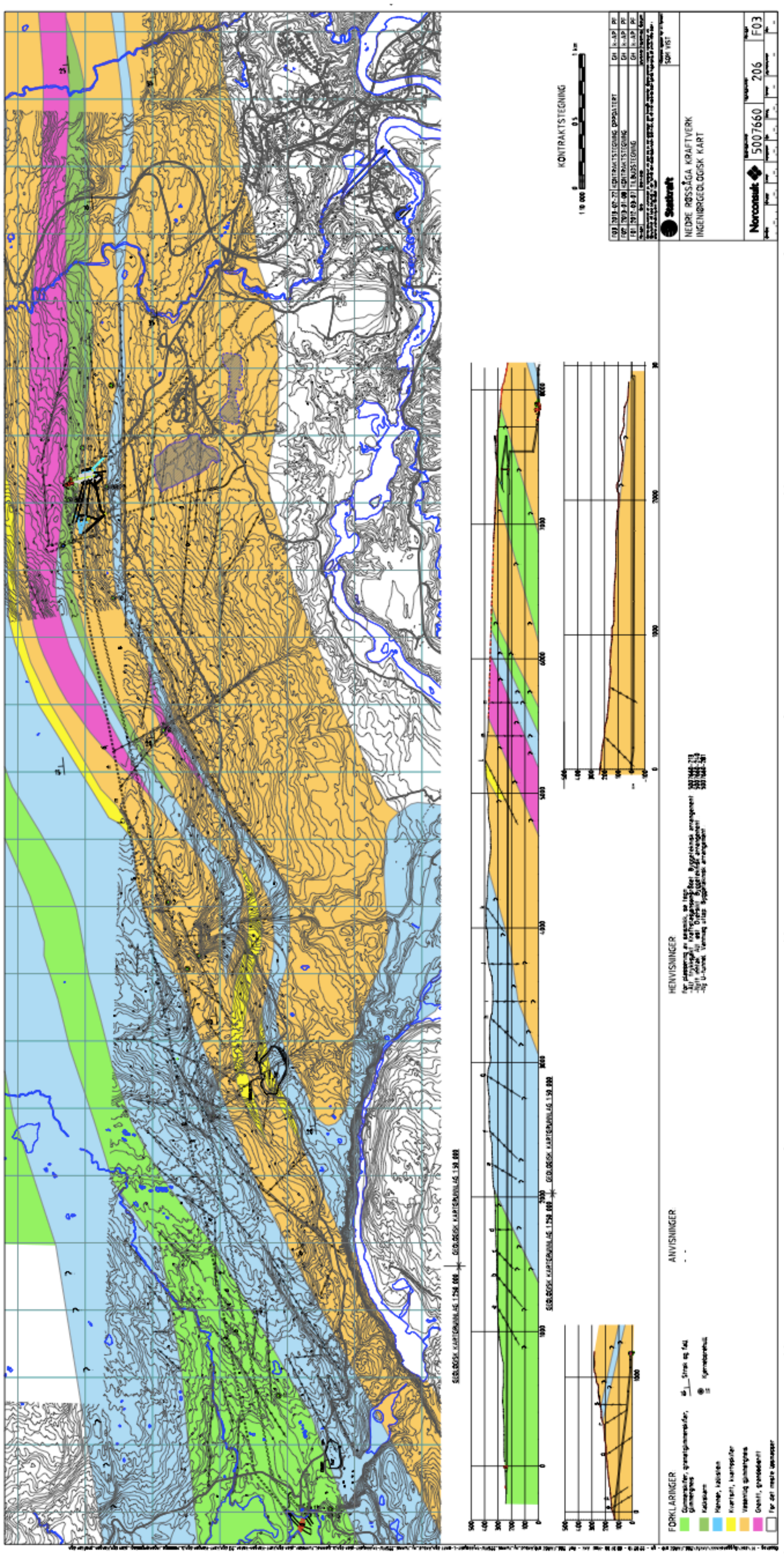


Appendix 4. Geological profile of the SMART-tunnel in Kuala Lumpur, Malaysia (Klados & Parks, 2005).

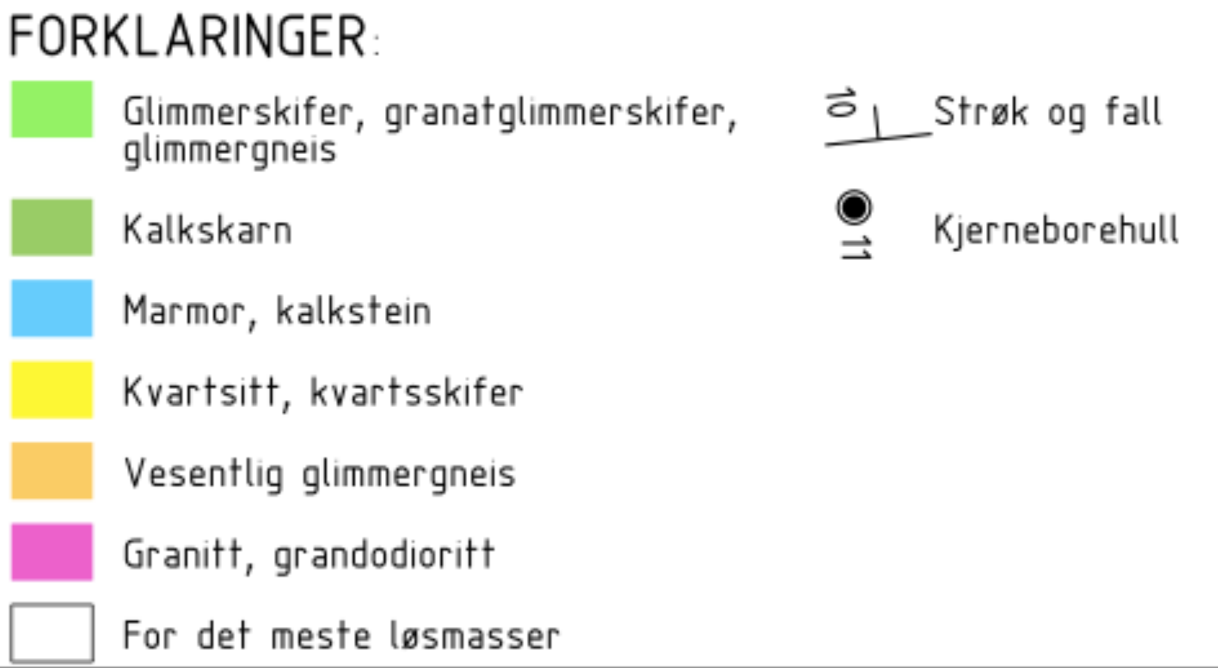




Appendix 5. Geological map of the area and geological profiles of the tunnels (Statkraft, n.d.)

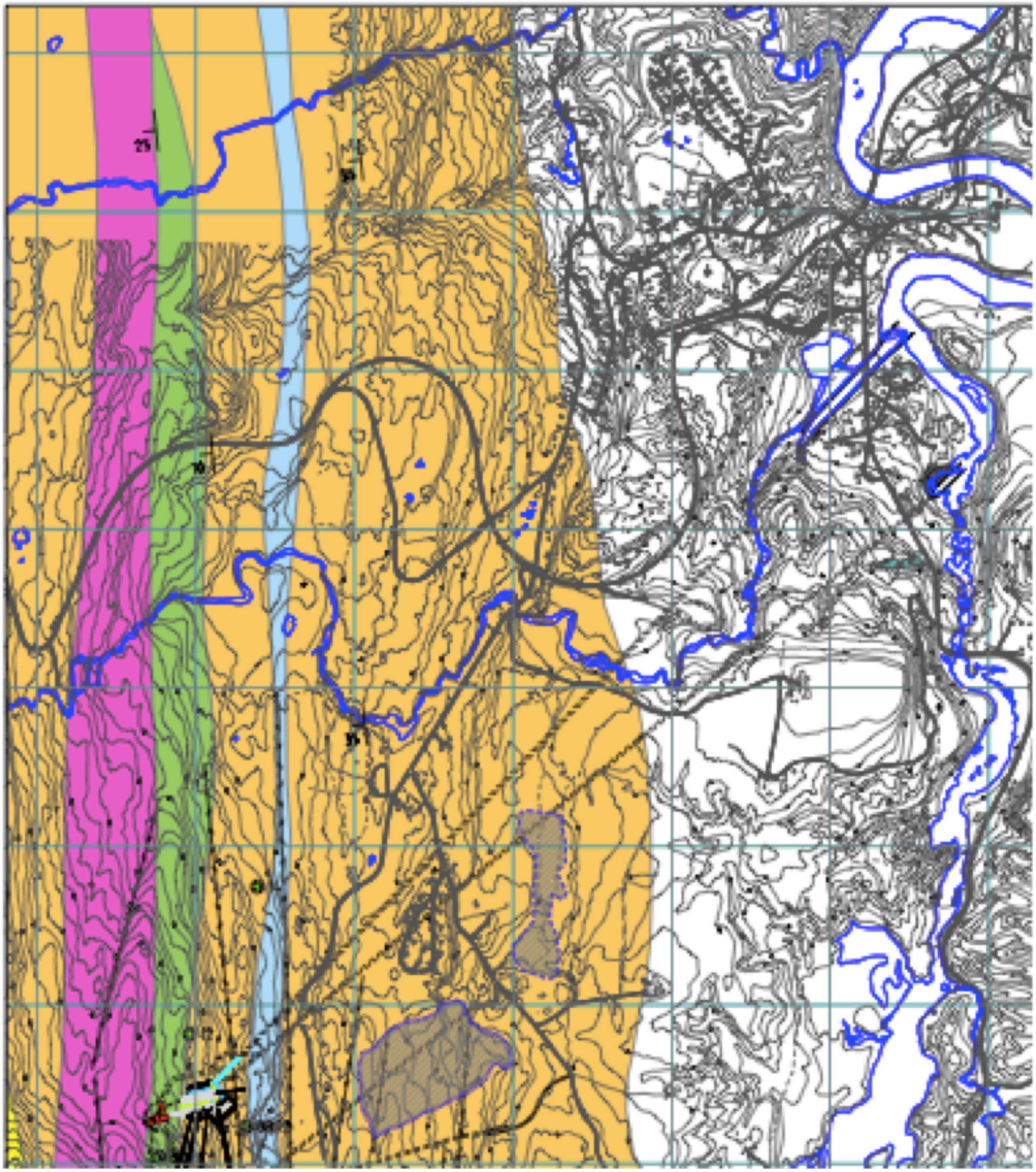


Appendix 5.1. Map Legend

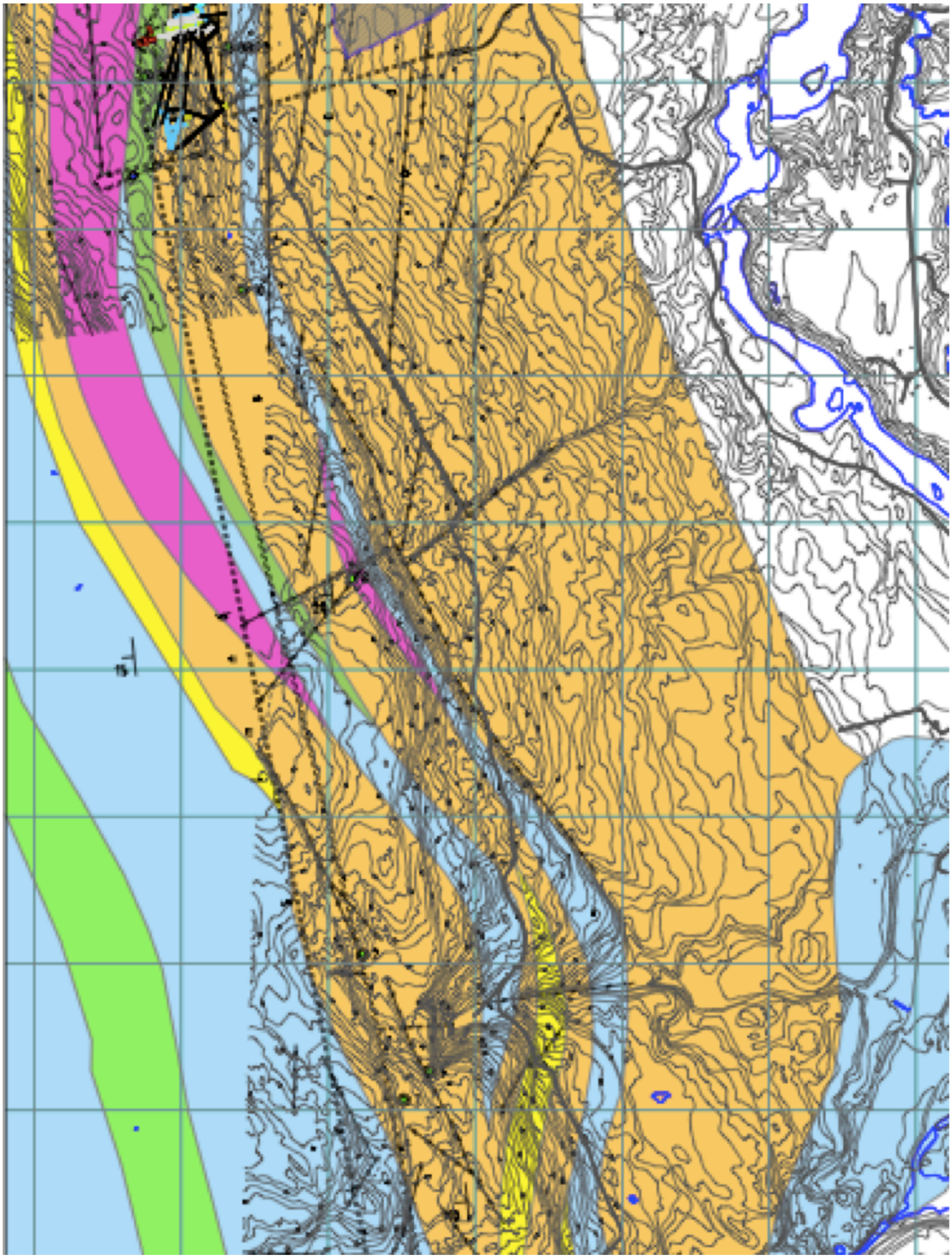


The geological map and the tunnel alignment of Lower Røssåga is in the following pages each divided into three parts, starting with part 1 from North.

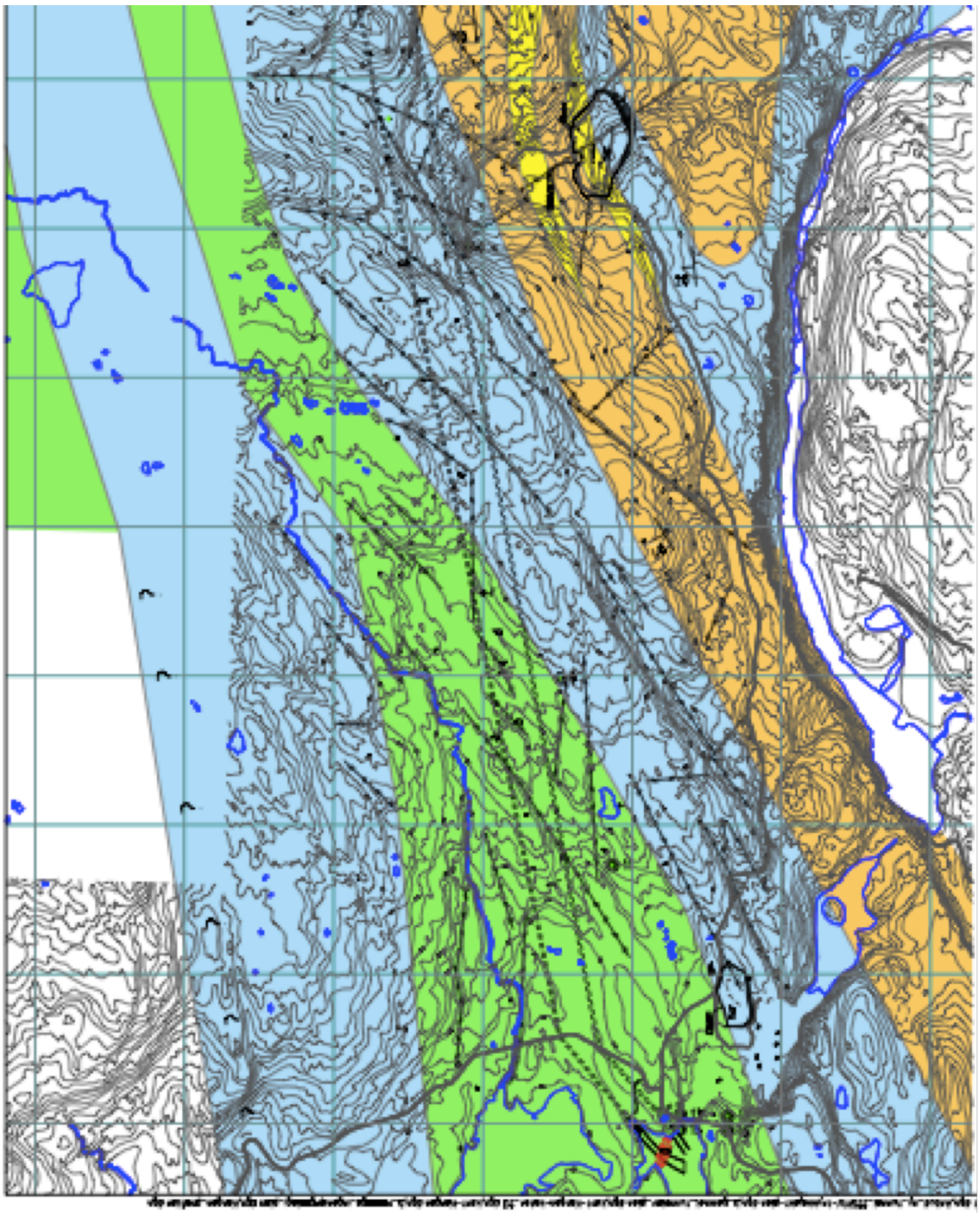
Appendix 5.2. Geological map, part 1.



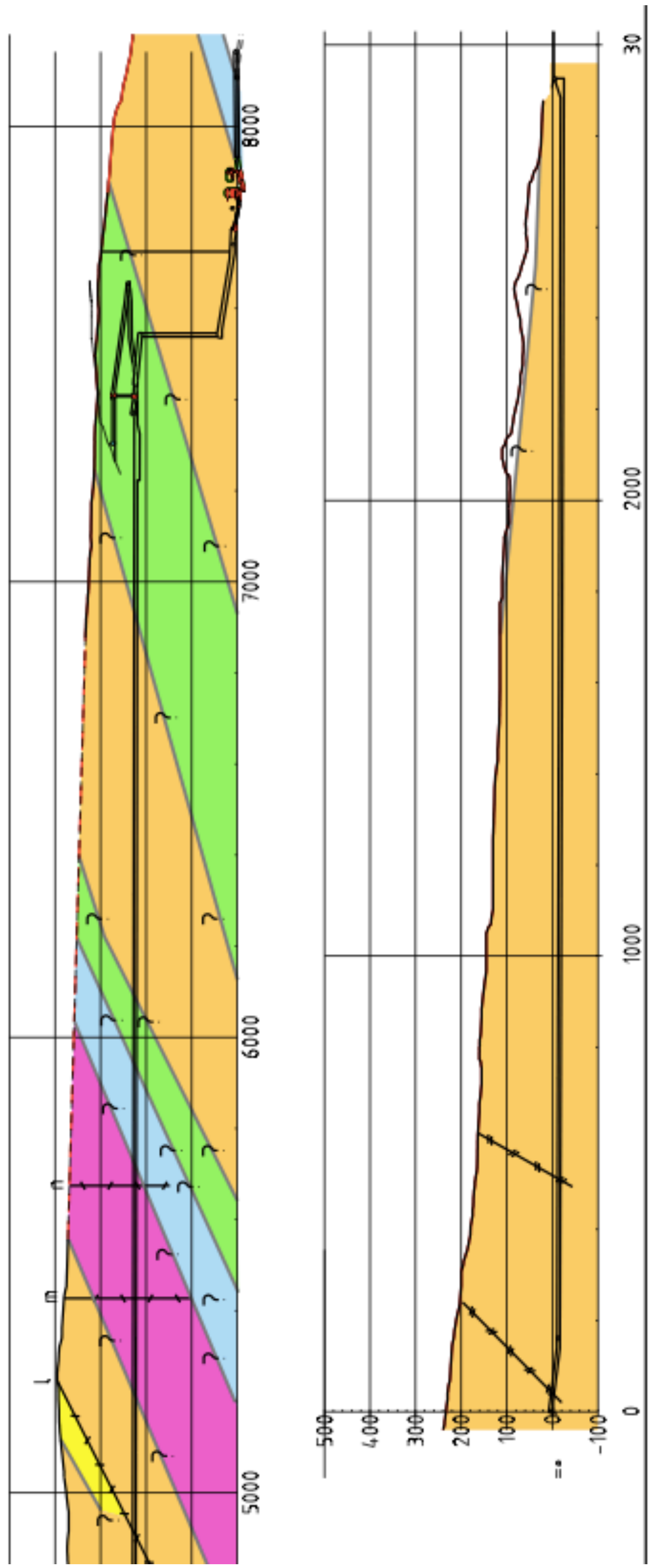
Appendix 5.3. Geological map part 2.



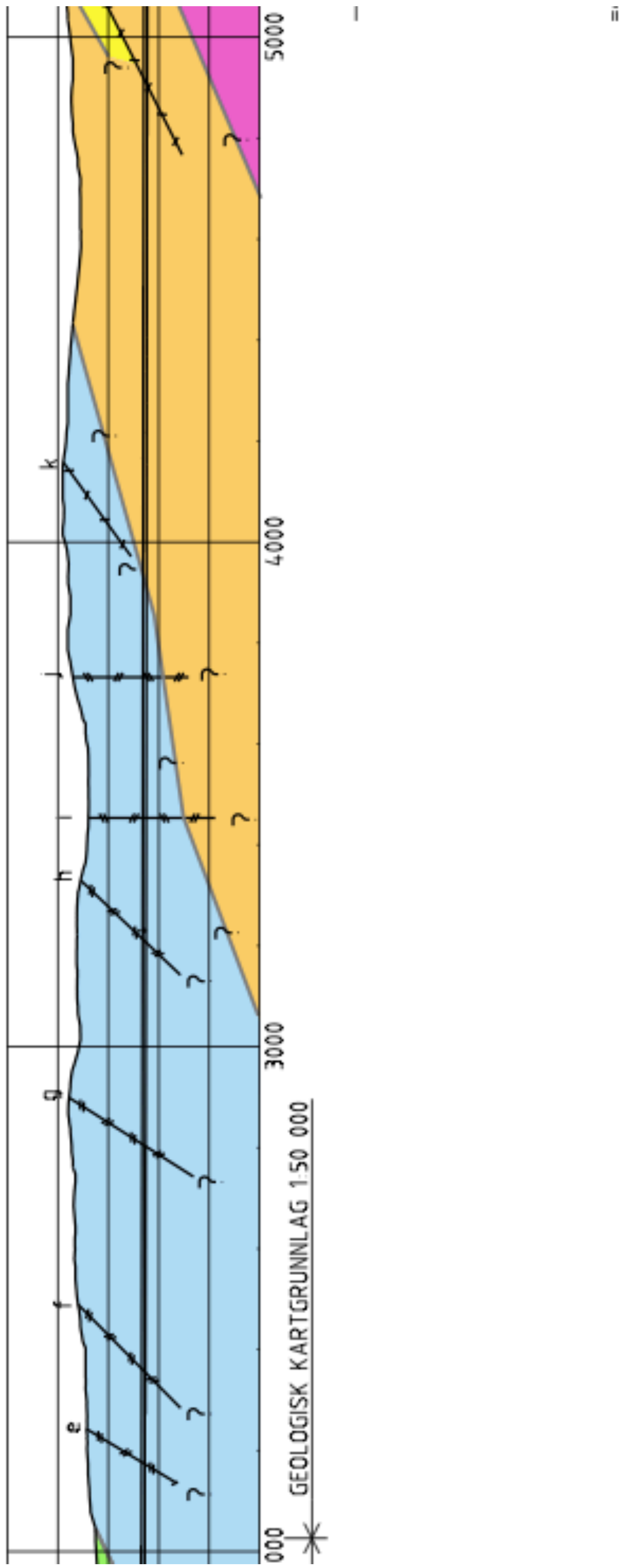
Appendix 5.4. Geological map part 3



Appendix 5.5. Geological profile part 1.



Appendix 5.6. Geological profile part 2



Appendix 5.7. Geological profile part 3.