NTNU – Trondheim
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Engineering Geological Assessments of a Tunnel for the Proposed High Speed Railway Link Between Oslo and Bergen

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for
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ENGINEERING GEOLOGICAL ASSESSMENTS OF A TUNNEL FOR THE PROPOSED HIGH SPEED RAILWAY LINK BETWEEN OSLO AND BERGEN

Background

The Ministry of Transport and Communication gave a mandate to The Norwegian Rail Administration to assess the possibility of a high-speed railway connection between cities in Southern Norway. The assessment shall provide alignment studies for four different scenarios for a high speed rail link, based on different design speeds and alignment parameters. A potential station location assessment and environmental impact assessment are included. The Norwegian Rail Administration has assigned Sweco Norge AS as lead consultant for the so-called *West Corridor* (Oslo-Bergen-Stavanger). The work is almost to the final stage and the deadline for the completion of assessment is set for 1st February 2012.

In this respect, Sweco is interested to engage a student in carrying MSc study on this project. The main focus of the study is to concentrate on the evaluation of engineering geological conditions and possible rock engineering and tunnelling challenges that may be met while tunnelling.

MSc Project task

This MSc thesis work is related to the engineering geological assessment of the tunnels for proposed High Speed Railway Link between Oslo and Bergen, and shall include:

- Brief description of the high speed railway corridors under consideration for southern Norway, presentation of corridor connecting Oslo and Bergen, and selection of the longest tunnel for further assessment.
- In-depth assessment on the geological conditions of the selected tunnel alignment.
- Analysis on the possible ground water ingress, challenges related to weakness and fault zones and stress induced instability using analytical, empirical and numerical methods, respectively.
- Analysis of cost and construction time required.
- Highlight on the required future geological investigations.
- Highlight also potential challenges related to long tunnels (>10-20 km), i.e. construction methods to be used.

Relevant computer software packages

Candidate shall use *roc-science package* and other relevant computer soft wear for the master study.

Background information for the study

- Relevant information about the project such as reports, maps, information and data will be provided by **Sweco, Oslo**.
- Collected data by the candidate from various sources.
- Scientific papers, reports and books in Norwegian tunnelling.
- Literatures in rock engineering, rock support principles and rock mechanics.

Cooperating partner

Sweco; Oslo is the cooperating partner of this thesis. Mr. Kenneth Haraldseth, Engineering Geologist from Sweco, will be the main contact and also co-supervisor for this MSc thesis.

The project work is to start on January 16, 2012 and to be completed by June 15, 2012.

The Norwegian University of Science and Technology (NTNU)
Department of Geology and Mineral Resources Engineering

January 10, 2012

Krishna Panthi

Dr. Krishna K. Panthi
Associate Professor of geological engineering, main supervisor

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This master thesis has been conducted at the Department of Geology and Mineral Resources Engineering, Norwegian University of Science and Technology (NTNU) in Trondheim, Norway. Sweco, Lysaker has been cooperating partner and the work has been conducted during the spring semester 2012.

Dr. Krishna K. Panthi, associate professor of geological engineering at NTNU, has been my main supervisor for this study. He have gladly and with enthusiasm shared his knowledge and experience in the field of engineering geology, and given me much appreciated support, guidance and advices during the work. I am very thankful for his commitment in my work.

I would like to thank the people at Sweco, Lysaker, who let me have my office at their place during the semester. I has been of great help and motivation to be able to take a part of their team of engineering geologists and benefit from their knowledge and experience. Sweco has provided me with very helpful data and every single person I have asked for help have gladly contributed. I would especially like to thank Kenneth Haraldseth, engineering geologist at Sweco and my co-supervisor for the thesis. His support and guidance has been most helpful for me and I am very grateful to him.

I would also like to thank by brother Espen for proofreading my thesis. His academic skills has provided me with good input and advices when completing the work.

Abstract

Today, the larger cities in Norway are linked with railways, but the rail transport system is in need of an upgrade in order to compete with other means of transport. Long journey times associated with the Norwegian railway is one of the main factors contributing to that most people prefer travelling by air when travelling longer distances. Thus, a great potential exists for high-speed railway lines between the major cities of Southern Norway, which will reduce journey times significantly.

On February 19, 2010, in a mandate from the Ministry of Transport and Communications, the Norwegian National Rail Administration was asked to study the possibilities of constructing high-speed railway lines in Southern Norway. One of the assessment's main conclusion is that development costs for the alternative routes are substantial and vary considerably depending on the proportion of tunnels on the respective lines. This is especially relevant for the link between Oslo and Bergen which will have to cross large mountainous areas. With strict requirements regarding inclination and stiffness of alignment, there will be numerous long (> 5 km) to very long (>10 km) and partly deep tunnels along this particular route.

In addition to large investment cost, challenges related to long and deep tunnels are considerable. Important aspects which needs to be considered are related to construction works, geology, environment and operation. Geology plays an important role since adverse and especially unforseen geological conditions may influence construction time and costs. In Southern Norway, the rock mass mainly consists of crystalline basement rocks of good quality. However, rock mass in faults and weakness zones found within the basement rocks have reduced quality. Cambro-Silurian sedimentary rocks also exist, which generally have lower strength and higher deformability than the basement rocks.

This thesis focuses on one long tunnel proposed for the high-speed railway link between Oslo and Bergen. The tunnel will be about 40 km long, has a maximum overburden of 900 m and crosses under the Hardangerjøkulen glacier. The thesis evaluates engineering geological conditions of the tunnel and the most crucial aspects of tunnel stability problems are covered. Theoretical aspects of main factors influencing on tunnel stability are evaluated, including water inflow, potential swelling, faults and weakness zones and stress induced problems. Water inflow, tunnel squeezing and spalling are analysed and predicted by analytical and empirical approaches. Numerical modelling is used for analysing brittle failure in the rock mass. Special challenges related to long and deep tunnels are also emphasized. Based on findings in the stability assessment, construction costs and construction time for the tunnel are estimated. This includes estimates for both conventional and TBM tunnelling methods.

It is concluded that the tunnel will face different geological challenges such as 1) spalling in massive brittle basement rocks, 2) tunnel squeezing in weak phyllite and 3) considerable water inflow under high pressure in fractured rock and weakness zones, which most likely will influence on the stability during tunnel excavation. Using conventional excavation methods, estimated construction time for one tunnel tube is 9.5 years. On the other hand, using TBM, estimated construction time is 4.9 years. Total costs for both main tunnel tubes are estimated to be 9.9 and 10.7 billion NOK for conventional and TBM excavation methods, respectively.

Sammendrag

I dag er de fleste større byer i Norge forbundet med jernbane, men jernbanesystemet har et behov for oppgradering for å kunne konkurrere med andre transportmidler. Lange reisetider som i dag er assosiert med den Norske jernbanen er en av hovedfaktorene som gjør at mange velger å reise med fly på lengre distanser. Det er derfor et stort potensial for høyhastighetsjernbane mellom de største byene i Sør-Norge, noe som vil redusere reisetiden betraktelig.

I februar 2010 ble Jernbaneverket gitt et mandat fra Samferdselsdepartementet for å utrede muligheten for bygging av høyhastighetsbaner i Sør-Norge. En av utredningens konklusjoner er at utbyggingskostnadene er betydelige for alle alternativer og varierer i stor grad med tunnelandelen på de forskjellige strekningene. Denne konklusjonen er spesielt relevant for den utredede linjen mellom Oslo og Bergen som nødvendigvis må krysse store fjellområder. Med strenge krav angående stigning og kurvatur til en høyhastighetsbane vil det bli et høyt antall lange (> 5 km) til veldig lange (> 10 km) og til dels dype tunneler på denne strekningen.

I tillegg til høye investeringskostnader er utfordringene knyttet til lange og dype tunneler betydelige. Konstruksjonsmessige aspekter samt geologi, miljø og funksjonskrav er viktige forhold som må evalueres nøye. Geologi spiller en viktig rolle siden ugunstige og spesielt uforutsette geologiske forhold i høy grad kan påvirke byggekostnader og drivetider. I sørlige deler av Norge består berggrunnen hovedsakelig av krystalline grunnfjellsbergarter av god kvalitet. Forkastninger og svakhetssoner reduserer imidlertid kvaliteten stedvis. I tillegg finnes det kambrosiluriske sedimentære bergarter som generelt har lavere styrke og høyere deformasjonsevne enn grunnfjellsbergartene.

Denne masteroppgaven fokuserer på en tunnel som er foreslått for høy- hastighetsbanen mellom Oslo og Bergen. Tunnelen vil bli ca. 40 km lang, har en overdekning på maksimum 900 m og krysser under Hardangerjøkulen. Oppgaven vurderer ingeniørgeologiske forhold langs tunnel traseén og de viktigste aspektene ved stabilitetsproblemer er identifisert. Teoretiske aspekter angående hovedfaktorer som påvirker stabiliteten i tunnelen er diskutert, inkludert vannlekkasjer, potensiell svelling i leire, forkastninger og svakhetssoner og spenningsrelaterte problemer. Innlekkasjer, "tunnel squeezing"og sprakeberg i tunnelen er analysiert ved bruk av analytiske og empiriske tilnærminger. Numerisk modellering er brukt for å analysere sprø deformasjon i bergmassen. Det er også lagt vekt på spesielle utfordringer knyttet til lange og dype tunneler. Basert på resultatet fra stabilitetsanalysen er byggekostnader og drivedtider estimert for tunnelen. Inkludert er estimater basert på konvensjonell drivemetode og driving med TBM.

Det er konkludert med at tunnelen vil møte ulike geologiske utfordringer hvor 1) sprak og bergslagsaktivitet i sprø og harde grunnfjellsbergarter, 2) "tunnel squeezing"i svak fylitt og 3) store vannlekkasjer under høyt trykk i oppsprukket berg og i svakhetssoner er vurdert til å være de mest sannsynlige forhold som kan påvirke stabiliteten under driving av tunnelen. Ved konvensjonell drivemetode er byggetid per hovedløp estimert til 9.5 år, mens med driving med TBM er estimert byggetid 4.9 år. Totale kostnader estimert for de to hovedløpene er 9.9 og 10.7 milliarder NOK med henholdsvis konvensjonell drivemetode og TBM.

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List of symbols and abbreviations

D & B	Drill and Blast
GSI	Geological Strength Index
HSR	High-speed railway
NTNU	Norwegian University of Science and Technology
TBM	Tunnel Boring Machine

D	Disturbance factor
D_i	Damage index
Δh	Hydrostatic head (m)
E	Young's modulus/modulus of elasticity (GPa)
K	Hydraulic conductivity (m/s)
K	Ratio of horizontal stress component to the vertical stress component
R_f	Depth of failure (m)
Q	Inflow/leakage rate (m^3/s)

σ_H	Maximum horizontal stress component (MPa)
σ_v	Vertical stress component (MPa)
σ_1	Major principal stress (MPa)
σ_3	Minor principal stress (MPa)
$\sigma_{\theta max}$	Maximum tangential stress (MPa)
$\sigma_{\theta min}$	Minimum tangential stress (MPa)
σ_c	Uniaxial compressive strength of intact rock (MPa)
σ_{cm}	Rock mass strength (MPa)
ρ	Density (g/cm^3)
γ	Specific weight of rock mass (MN/m^3)
ν	Poisson's ratio
p_i	Support pressure (MPa)
ε	Tunnel strain (%)

1 Introduction

1.1 Background

On February 19, 2010, the Ministry of Transport and Communication (Samferdselsdepartementet) gave the Norwegian Rail Administration (Jernbaneverket) a mandate to assess the possibility of high-speed railway lines (HSR) between cities in Southern Norway. The assessment was finished in late 2011, including proposed alignments for a new HSR link between Oslo and Bergen. It was concluded with that it is feasible to construct and operate HSR lines in Norway. However, construction costs are significant and is highly dependent of the share of tunnels included in the different proposed railway alignments (Jernbaneverket 2011). One of many challenges for a new HSR link were found to be the crossing of the mountainous areas in the middle of Norway. Only one solution exist with the stringent requirements with respect to curvature of a high speed rail, and that is putting a large share of the railway line in tunnel. The tunnels will be many and some of them very long and deep.

Challenges related to long tunnels are many and several aspects must be thoroughly evaluated before an eventual construction process may begin. Aspects of major importance and that is decisive for the feasibility of a tunnel project is geological conditions, construction time and costs. The work must be completed in a safe way resulting in minimal disturbance to the environment, as well as a secure tunnel where operational requirements are fulfilled.

Adverse geological conditions commonly encountered in long and deep tunnels are often difficult to predict due to the limited access to the tunnel level before excavation starts. Examples of such conditions are unfavorable stress conditions leading to spalling and rock burst in hard rocks, tunnel squeezing and plastic deformation in soft rocks subjected to high stresses, high water pressure and high water inflows, weakness zones often associated with poor rock mass strength and swelling clay. If such geological conditions are encountered and not secured sufficiently by means of rock support, it might lead to severe instability causing an insecure tunnel environment both during the construction phase and operational phase.

In addition to bring a certain risk to tunnel projects, geological conditions are highly influencing construction time and costs. Basically, constructing a long tunnel is a time-consuming and very expensive process. Adverse tunnelling conditions will increase the costs and construction time due to increased need for rock support. Thus, reliable estimates of geological conditions at the tunnel level is crucial for reliable estimates of construction costs and time which in its turn is determining for the political will for investing in such a large project.

1.2 Introduction to the high speed railway project

1.2.1 Project background

The HSR assessment has been carried out in three predefined phases with the following objectives (Jernbaneverket 2011, Sweco 2011) :

- **Phase 1.** A total overview and presentation of the knowledge base that exists in Norway were assessed. This also included the final report on high-speed rail in Sweden.
- **Phase 2.** Common premises for HSR concepts that might be relevant for Norwegian conditions were studied and identified. This included a market analysis, evaluation of different conceptual solutions related to the use of high-speed tracks, multipurpose tracks, stop-pattern and station design, different speed standards and possibilities of incremental development of the existing railway network.
- **Phase 3.** Based on findings in Phase 2, specific individual corridor analysis of different scenarios was performed. This included detailed investigations in order to confirm whether a HSR link is feasible or not and recommendations for long-term development strategies.

Today, the larger cities in southern Norway are linked with railways, but the rail transport system is in need of an upgrade in order to compete with other means of transport. Due to long travel times, most Norwegian people prefer to travel by air rather than existing railways when traveling longer distances. A high frequency of flights between the larger cities, like Oslo, Bergen and Trondheim does also contribute to a higher amount of air passengers.

In Phase 2 it was found that the Norwegian people are very positive to high-speed trains. 7 out of 10 would choose HSR rather than traveling by air if the expenses were the same. Saving of time, having possibility to work on board as well as comfort makes people choose HSR (Jernbaneverket 2011). As a result, Jernbaneverket decided to let four different consulting groups assess four separate corridors:

1. Corridor West (Oslo - Bergen with link to Stavanger)
2. Corridor East (Oslo - Gothenburg and Stockholm)
3. Corridor South (Oslo - Kristiansand/Stavanger)
4. Corridor North (Oslo - Trondheim)

For each corridor, the consulting groups were selected and asked to find representative, feasible alignments for the HRS network. The alignment studies aimed at deciding number of stations, design speeds, estimation of construction costs and environmental impacts .The study has tested four different scenarios for a high speed rail link, based on different design speeds and alignment parameter agreed upon for all corridors.

The consultancy firm Sweco was assigned corridor West, and a few numbers of sub consultants were also engaged in this particular project.

1.2.2 Corridor West: scenarios and proposed tunnel alignments

Corridor West consist of routes connecting Oslo with Bergen and Stavanger. With the present infrastructure system in Norway, the train between Oslo and Bergen have a traveling time of approximately 6,5 hours, while Stavanger is reached in about 8 hours from Oslo. There are no direct railway connection between Stavanger and Bergen, and travelers going by train have to travel via Oslo.

The HSR assessment, carried out by consulting group Atkins, found that a HSR connection between Oslo, Bergen and Stavanger will reduce travel times significantly (table 1). With a speed profile of 330 km/h, it is possible to travel from Oslo to Bergen in less than 2 hours.

Table 1: Fastest attainable travel times on each route with different scenarios (Sweco 2011).

Route	Alignment scenario		
	D1 330 km/h mixed traffic	D2 330 km/h passenger trains only	2* 250 km/h mixed traffic
1 Hallingdal	01:54		02:14
2 Haukeli	02:07		
3 Numedal	01:59		02:20
4 Coastal	01:24	01:19	

In corridor West the four following routes have been assessed (figure 1):

- **Route 1** Hallingdal
- **Route 2** Haukeli
- **Route 3** Numedal
- **Route 4** Coastal

Route 1 is planned as a new line from Bergen via Voss-Geilo-Gol-Hønefoss to Sandvika where it links up to the existing railway line to Oslo. The existing railway line between Bergen and Oslo is also traveling via these stations. For a new HSR line with mixed traffic a significant number of long tunnels and bridges have to be constructed. Particulary problematic are long tunnels through mountainous areas, amongst them the 40 km long tunnel under the glacier Hardangerjøkulen (Sweco 2011).

Route 2 is planned as a new line from Bergen via Odda-Røldal-Haukeligrend-Bø-Notodden-Kongsberg to Drammen where it links up to the existing railway line to Oslo. Several large tunnels have to be constructed, amongst them a long tunnel under the glacier Folgefonna. In Røldal the line is separated, and one line will go to Stavanger via Sauda and Haugesund. This line to Stavanger will have to cross the Boknafjord with a very long subsea tunnel reaching 370 meters below sea level (Sweco 2011).

Route 3 is planned as a new line between Geilo and Kongsberg. At Geilo the route connects with the Hallingdal route Geilo - Bergen, while in Kongsberg it connects with route 2, Kongsberg - Drammen. With this link, a new line will be established between Bergen-Geilo-Kongsberg-Drammen. The number of tunnels and bridges in route 3 are significantly lower than in route 1.

Route 4 will connect Bergen and Stavanger via Haugesund. The main challenge will be the crossing of the Boknafjord in a very deep and long subsea tunnel.

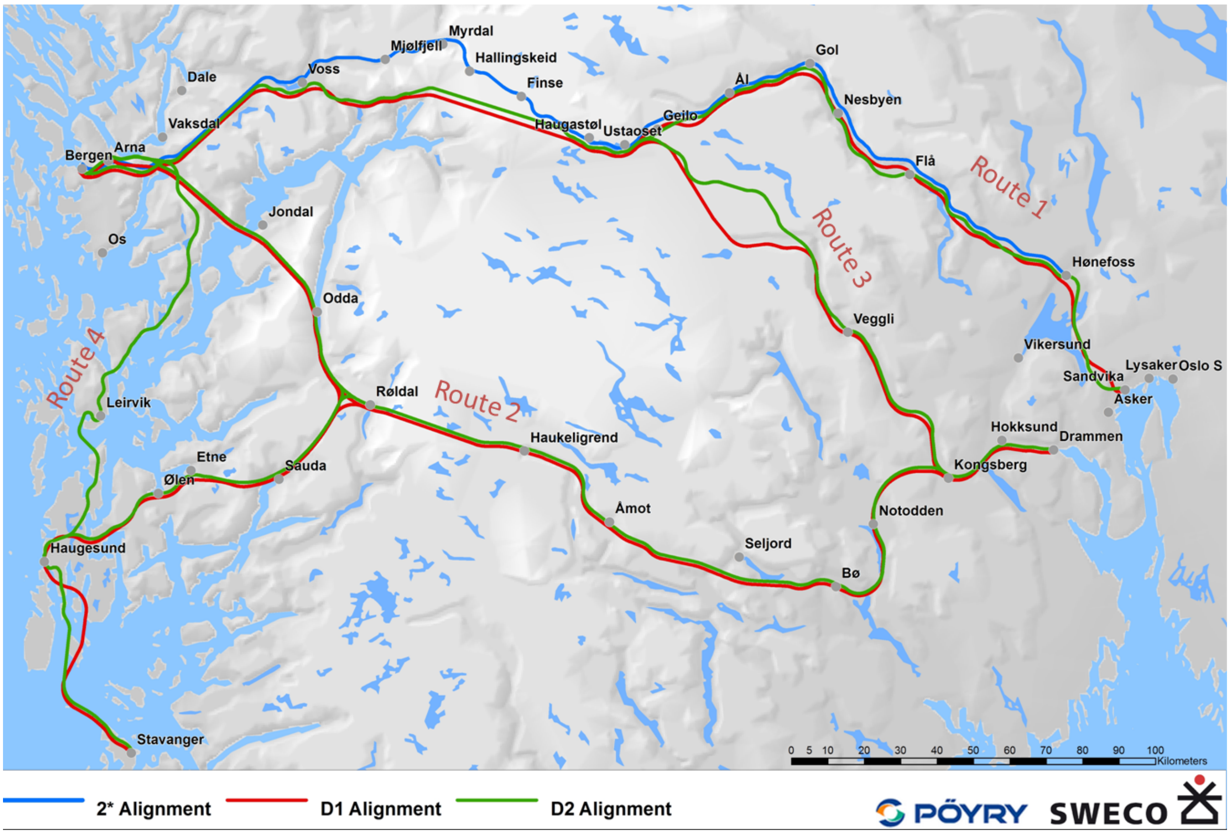


Figure 1: Overview of corridor West showing proposed routes and railway alignments for scenario 2*, D1 and D2 (modified from Sweco 2011).

Based on the mandate, the study to be carried out had to consider four different scenarios.

Scenarios tested are based on different design speeds and alignment parameters agreed upon for all corridors, this to assess which alternatives that are best suited to meet the goals of the transport policies in the different corridors. The following scenarios were considered:

- Scenario B: Measures for the existing track, where the travel time shall be cut by 20 %.
- Scenario 2*: An upgrade of existing railway lines for mixed traffic and a speed profile of 250 km/h.
- Scenario D1: New high-speed lines for mixed traffic and a speed profile of 330 km/h.
- Scenario D2: New high-speed lines for passenger trains only and a speed profile of 330 km/h.

Proposed alignments for scenarios 2*, D1 and D2 are shown in figure 1. Railway lines for mixed traffic must be designed for freight trains in addition to high-speed trains, requiring a stiffer line with larger horizontal and vertical curves and smaller gradient. Due to freight traffic, technical design parameters for scenario D1 are limited to a maximum gradient of 1,25 % and a minimum horizontal curve radius of 6300 m. For scenario D2 the gradient may be increased to 3,5 % and the horizontal curve radius decreased to minimum 3000 m. The alignments have been tried located close to existing stations and communities.

1.2.3 Selection of alignment for MSc Thesis

For an in-depth assessment of the HSR-project, route 1 Hallingdal, scenario D1 (figure 2) is chosen for further study in this MSc thesis with a particular focus on one tunnel. Engineering geological aspects of this route are very interesting with respect to the crossing of mountainous areas in the middle of Norway. Many challenges will be met when constructing long and partly deep tunnels, both with respect to geology and construction works.

Every assessed route and alignment has several tunnels with different characteristics. What seems to be one of the common challenges for most of the alignments is that the total share of tunnels is very high, and that long tunnels (>10 km) can not be avoided. There are several challenges related to construction and maintenance of long tunnels; technical issues, environmental and safety issues etc. Strict requirements regarding the rigidity of a HSR line and the diverse Norwegian landscape with valleys and mountains makes it impossible to avoid long tunnels along the route. For route 1, scenario D1, 52 tunnels with an overall length of approximately 217 km have to be constructed. 8 of these have an length exceeding 10 km (Sweco 2011).

The tunnel chosen for this MSc study is located between Osa and Bergsmultfjorden in the area known as Hardangervidda. It is approximately 40 km long and is proposed

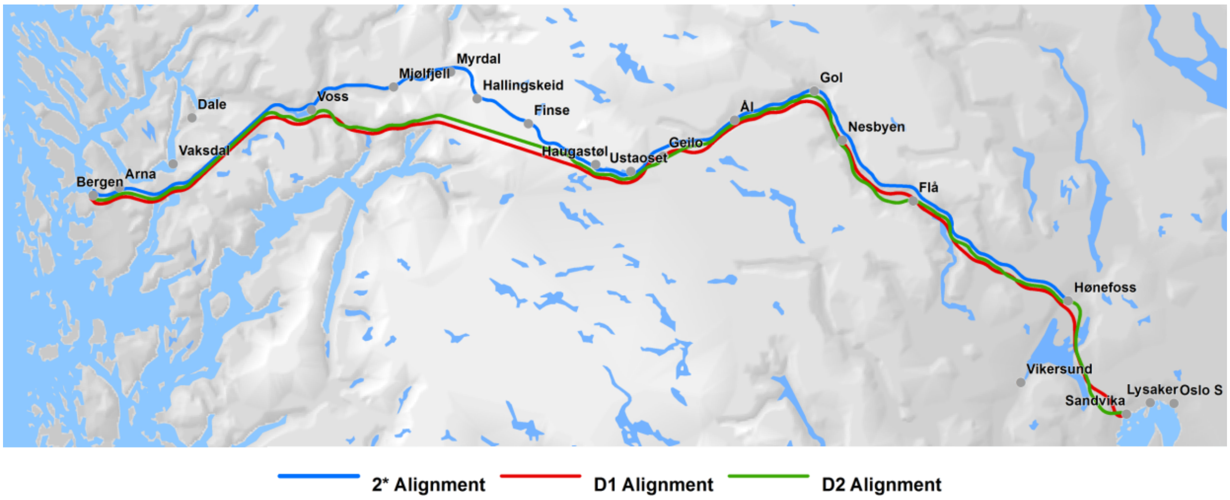


Figure 2: Route 1 Hallingdal with proposed alignments for scenario 2*, D1 and D2 (Sweco 2011).

to cross under the glacier Hardangerjøkulen, the sixt largest glacier in Norway (figure 3). Hardangerjøkulen constitutes a part of the preserved landscape area known as "Skaupsjøen /Hardangerjøkulen landskapsvernområde", which borders towards Hardangervidda national park. This MSc thesis will focus on this particular tunnel project as it is found especially interesting due to it's length, depth and location and because it is found to be a challenging project regarding engineering geology and construction aspects. In the thesis, this tunnel will henceforth be named the Hardangerjøkulen tunnel.

Figure 3: The glacier Hardangerjøkulen seen from Hårteigen where one of the proposed HSR tunnel will cross under. The tunnel will be approximately 40 km long. Source: Wikimedia Commons.

1.3 Purpose and scope of the work

The main goal of this MSc thesis is to assess engineering geological conditions of the Hardangerjøkulen tunnel together with analysis of construction time and costs. Overall, the thesis includes:

- Study of geological conditions along the tunnel alignment
- Literature review of long and deep tunnels
- Brief review and theory of potential geological challenges
- Analysis of water ingress, tunnel squeezing and stress induced instability by analytical and empirical approaches
- Numerical analysis of stress induced instability
- Analysis of required construction time and costs
- Recommendation of future geological investigations

The material used for the study is mainly:

- Relevant information about the HSR project such as reports and maps provided by Sweco, Lysaker.
- Literature in rock engineering, rock mechanics and rock support
- Literature in Norwegian tunnelling
- Literature in long and deep tunnels
- Geological maps

The review of geology along the tunnel route is mainly based on the study of geological maps existing for this area. Engineering geological conditions and potential challenges discussed in this thesis are based on available literature in rock engineering, and the conditions are especially discussed with respect to Norwegian tunnelling practise. Norwegian literature is also used for gaining experience from other tunnelling projects in similar conditions as the Hardangerjøkulen tunnel. In addition to the study of available literature and maps, computer software from Rocscience is used for numerical analysis. This includes the software programs Phase^2 and RocLab. The analysis of construction time and costs are based on experiences from Norwegian tunnel projects, and relevant data are provided by Sweco if no else is stated.

2 Geological conditions of alignment

2.1 Geological overview of southern Norway

Norway mainly consists of crystalline basement rocks formed 1750-900 million years ago. The basement rocks in southern Norway may be divided into two complexes separated by a major fault zone known as the Mandal-Ustaoset fault zone (figure 4). As indicated by the name, it stretches from Ustaoset right in the middle of Norway down to Mandal at the South coast of Norway. East of the fault zone, the basement rocks are mainly composed of metamorphosed sandstones and rocks of volcanic origin. These are partly intersected by plutonic rocks. Plutonic rocks are formed by glowing melt deep down in the earth's crust that later has cooled down and hardened. West of the fault zone a variety of such plutonic rocks like granites, metamorphosed granite and gneiss are found.

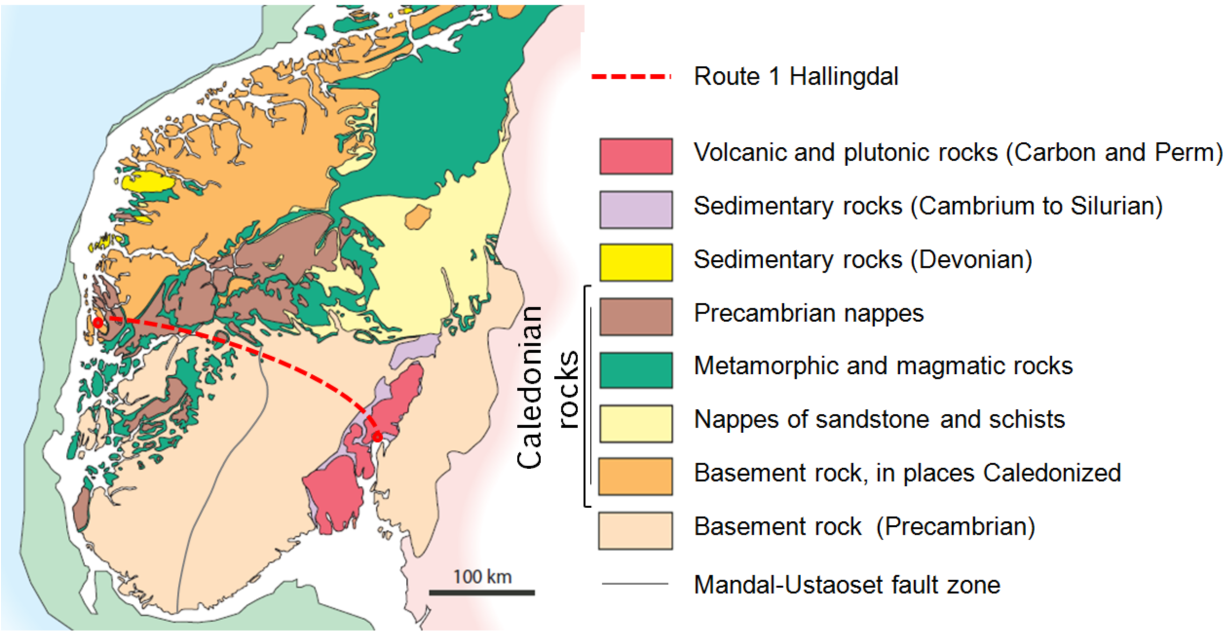


Figure 4: Geological overview of southern Norway (modified from NGU 2012).

The basement rocks were weathered during millions of years, giving rise to what today is known as the sub-Cambrian peneplain. When the ocean forced its way over the peneplain, gravel, sand, carboniferous and calciferous clay and limestone were deposited at the seabed above the basement rocks. The rocks formed from these deposits are today known as the Cambro-Silurian sedimentary rocks.

During the Cambrian age, an ocean existed between Norway, Greenland and the American continent. The two continents collided about 400 million years ago resulting in the Caledonian orogenesis. Big sheets of Precambrian bedrock like gneiss and granite, and supracrustal rocks deposited at the seabed, unfastened and were pushed towards south-east. The Cambro-Silurian sediments became heavily folded and metamorphosed during this process. The clay became shale, phyllite and mica schist, the sand became meta-sandstone and quartzite and the limestone became marble.

The final result from these geological processes is a stratified geological structure with the oldest layers at the bottom and top and the Cambro-Silurian sedimentary rocks in the middle. Depending of the degree of weathering and erosion, these geological structures form the basis of southern Norway except from the Oslo region (Ramberg, Bryhni & Nøttvedt 2007 and Sigmond 2011).

From Hønefoss to Sandvika and Oslo, the geology differs from what is found in rest of Norway. This region is marked as pink and lilac in figure 4. The geology is complex with different rock types that may be categorized into the following main-units:

- Cambro-Silurian sedimentary rocks
- Intrusive plutonic rocks
- Volcanic rocks

During the Silurian age sediments with different characteristics were deposited. Later, the weak sedimentary rocks were deformed and folded due to the Caledonian orogenesis. During the Carboniferous and Permian age volcanic activity became high in the Oslo region. What is known as the Oslo- rift, a huge graben-structure, was formed at this time. The region has many rare minerals and rock types formed by crystallization of melts, amongst them the quite known rhombus porphyry. Due to erosion and weathering, plutonic rocks in addition to extrusive igneous rocks are today exposed to the surface in larger areas. Intrusive igneous rocks are found throughout the Oslo region as dykes with different size and geometry (Ramberg et al. 2007).

2.2 Geological units along Route 1

Kilometer 0 - 28, The Bergen region: This region is formed like topographic arcs around the city of Bergen and is therefore known as "Bergensbuene", i.e. the arcs of Bergen (figure 5). The arcs is in fact composed of different nappes transported and deformed during the Caledonian orogenesis. The railway line will start in Lille Bergensbue, then go through the Blåmann nappe, then the Lindås nappe and then Store Bergensbue (table 2).

Table 2: Geological units in the Bergen region

Tectonic unit	Rock types
Lille and Store Bergensbue	Ophitic rocks and attached supracrustal rocks, mainly quartz-diorite, mica schist, marble and metabasalt
Blåmann nappe	A nappe formed of metamorphosed basement rocks and younger sedimentary rocks like quartzite, gneiss and migmatite
Lindås nappe	Overthrusted basement rocks, anorthosittic and charnockittic rocks

Kilometer 29 - 53, The Bergsdalen tectonic unit: The Bergsdalen tectonic unit is composed of rocks of Precambrian age and intrusive rocks that are influenced by the Caledonian orogenesis. Main rock types are tonalittic and grannitic gneiss.

Kilometer 54 - 110, Phyllite: This area is composed of autochthonous phyllite which originates from Cambro-Silurian sedimentary rocks. The sediments have been overthrusted by Caledonian nappes and thus it has been metamorphosed into phyllite and mica schist.

Kilometer 78 - 99, The Slettafjell nappe and Skorafjell nappe: Within the phyllite, the Slettafjell nappe is found. This is supracrustal, metamorphosed rocks like quartziferous shale and meta-sandstone. Above the Slettafjell nappe lies the Skorafjell nappe, a unit with several sheets of Pre-Cambrian age, deformed and metamorphosed. It mainly consists of granitic to granodiorittic gneiss and augen gneiss.

Kilometer 111 - 165, Precambrian basement rocks west of the Mandal-Ustaoset fault zone: Before entering the basement rocks the railway line passes through a relatively small area with autochthonous and overthrusted rock of sedimentary origin lying above the basement rocks. Here the longest tunnel of the line is planned, the 40 km long tunnel crossing under the glacier Hardangerjøkulen. The autochthonous basement rock is mainly composed of granite, augen gneiss, granitic and granodioritic gneiss. As it get closer to the Kalhovd fault, which is the western boundary of the Mandal-Ustaoset fault zone, the rock mass gets more folded.

Kilometer 165 - 258, Precambrian basement rocks east of the Mandal-Ustaoset fault zone: Approximately at km 165 the line enters into the Mandal-Ustaoset fault zone, one of the major fault zones in Norway. It is 3-4 km wide and confined by the Kalhovd fault to the west. It was formed at great depth and thus the rocks are deformed. However, the newest fault, the Kalhovd fault, was created at lower depth and brecciated rocks are found in a smaller area (Sigmond 2002).

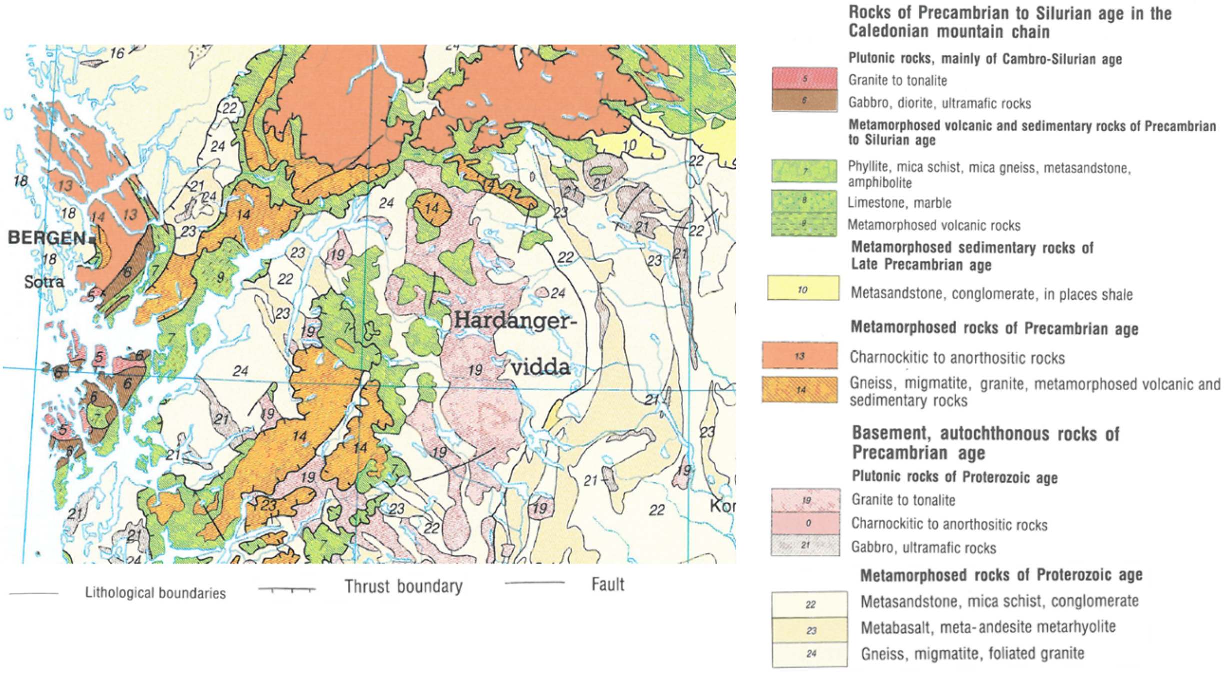


Figure 5: Geological overview of western parts of route 1 (modified from Sigmond 1985).

East of the Mandal-Ustaoset fault zone, the area is mainly composed of plutonic rocks like gabbro, granite, augengneiss and migmatite. The line also passes through an area composed of metatuff; metamorphosed volcanic metasediments.

Kilometer 259 - 317, The Kongsberg Complex: The Kongsberg complex, situated between Noresund and Hønefoss, is composed of autochthonous, metamorphosed rocks of Precambrian age. The line will mainly cross through banded gneiss with a varying amount of amphibolite, hornblende gneiss, biotite gneiss and dykes of granite.

Kilometer 317 - 355, The Oslo region. Between Hønefoss and Sandvika, the line passes into what is known as the Oslo region. First, the line passes through metamorphosed rocks of Proterozoic and Cambro-Silurian age. These are sedimentary rocks like sandstone, limestone and schists. Then autochthonous, extrusive igneous rocks like gabbro, rhombus porphyry and basalt are found. Finally, the line once again passes through Cambro-Silurian sedimentary rocks like sandstone, limestone, clay shale.

2.3 Geological setting - Hardangerjøkulen tunnel

The geology in the area of the Hardangerjøkulen tunnel is mainly composed of 3 geological units separated from each other by major faults. Geological map is found in figure 6 and geological profile in appendix A. Identification and description of the geological units are based on the study and interpretation of geological maps produced by the Geological Survey of Norway (NGU)(Sigmond 1998, Askvik 2008 and Henriksen 2000)

Undre Jotun nappe: Nappe transported during the Caledonian orogenesis composed of metamorphosed Precambrian rocks. Charnockittic and anothosittic rocks together with granitic to monzonittic gneisses constitutes most of the nappe. These are originally plutonic rocks that later has been metamorphosed. Several phases of folding and different grades of metamorphism are characterizing rock found here. A major thrust fault is confining the nappe which borders to the underlying unit, Cambro-Silurian sedimentary rocks.

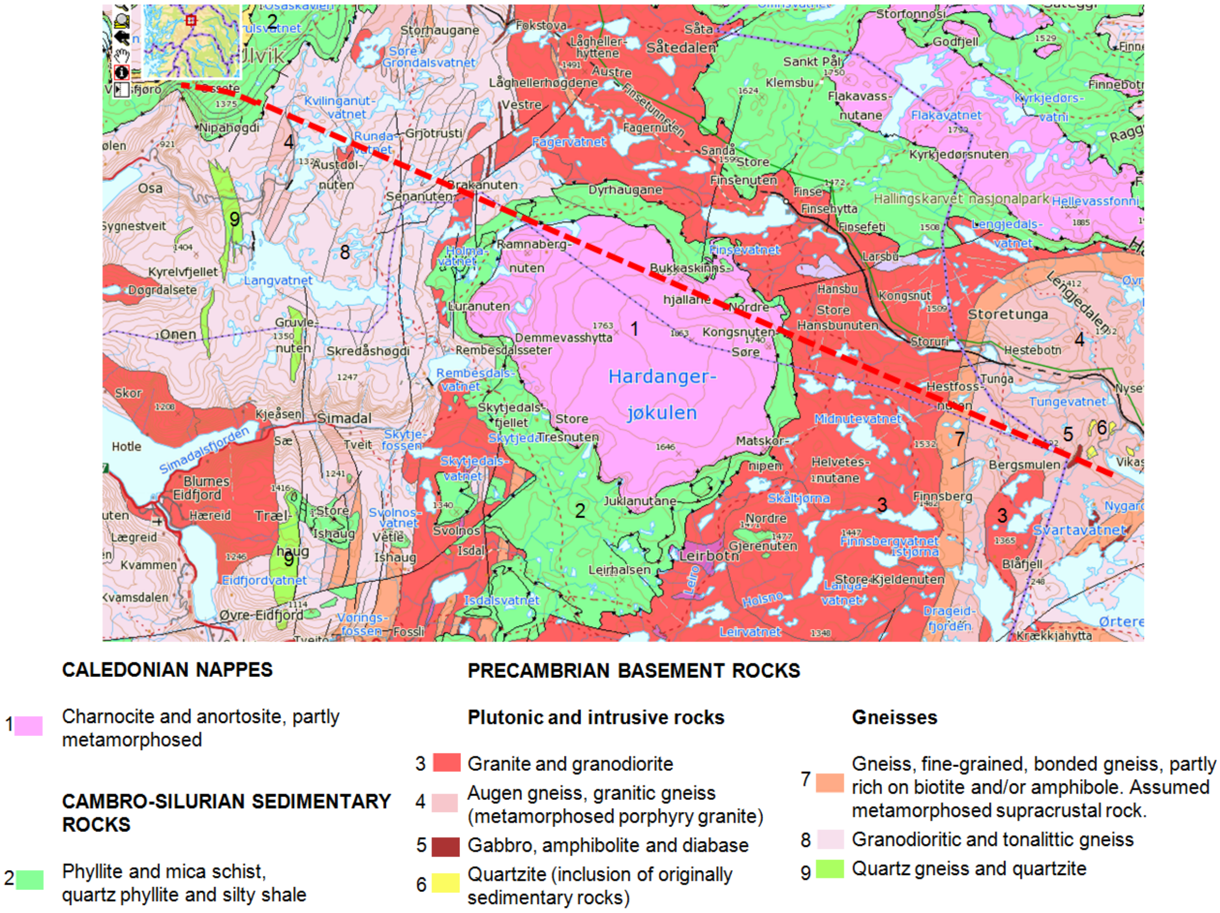


Figure 6: Geological map of the Hardangerjøkulen tunnel. The tunnel alignment is shown by the red dotted line (modified from NGU 2012).

Cambro-Silurian sedimentary rocks: Autochthonous and overthrusted rocks of Cambro-Silurian age. When the ocean flooded the Subcambrian peneplain, clay shale, conglomerate and sandstone were deposited. These were later deformed during the Caledonian orogenesis, resulting in rocks like phyllite and mica schist. Different types of phyllite exists in the area. Some are carbonaceous and graphite bearing, while some are rich on quartz. Generally, these rocks have a fissility oriented E-W or ENE-WSW and dips towards south or SSE.

Pre-cambrian basement rocks: Rocks belonging to the previously described basement rocks west of Mandal-Ustaoset fault zone. They form the geological basis in the area, underlying the Caledonian nappes and Cambro-Silurian sedimentary rocks. The basement rocks may be divided into two units; plutonic and intrusive rocks (number 3-6 on geological map) and different types of gneisses (number 7-9 on geological map). The area which the tunnel pierce is mainly composed of different types of gneiss and granite:

- Granite and granodiorite
- Granitic gneiss
- Augen gneiss
- Banded gneiss
- Tonalittic/granodioritic gneiss
- Gabbro/Amphibolite

2.4 Rock types and their mechanical properties

The Hardangerjøkulen tunnel will pierce through the different rock types listed in table 3. In addition to geological maps, geological information is also obtained by experiences from nearby tunnel projects located in the same geological formations. Values of uniaxial compressive strengths σ_c is based on rock samples tested in the rock mechanical laboratory at NTNU/SINTEF (table 4). The measurements listed are from the nearby locations Sima, Sysenvatnet and Aurland. A more general classification suggested by Hoek (2009) is also found in table 4.

Undre Jotun nappe and the Cambro-Silurian sedimentary rocks situated directly under the glacier are found to be near flatlying (figure 6, No. 1 and 2). The geological maps suggests a maximum thickness of approximately 500 m of these units. Thus, with rock overburdens varying between 600 and 900 m, the tunnel is not likely to go through these geological formations and they will not be further discussed and analyzed here.

Table 3: Rock types along the Hardangerjøkulen tunnel

Chainage kilometer	Rock type	Rock overburden (m)
0 - 0.650	Calcareous phyllite/ quartz phyllite	0 - 270
0.650 - 1.850	Phyllite	270 - 690
1.800 - 3.800	Migmatitic gneiss	560 - 790
3.8 - 4.4	Augen gneiss	490 - 560
4.4 - 5.8	Granitic gneiss	430 - 490
5.8 - 6.4	Migmatitic gneiss	420 - 430
6.4 - 7.3	Granite	420 - 430
7.3 - 7.8	Granitic gneiss	410 - 420
7.8 - 9.0	Tonalittic/granodioritic gneiss	420 - 450
9.0 - 9.8	Granitic gneiss	450 - 490
9.8 - 9.9	Tonalittic/granodioritic gneiss	490 - 500
9.9 - 11.3	Granite	500 - 520
11.3 - 12.1	Tonalittic/granodioritic gneiss	520 - 550
12.1 - 12.5	Granite	540 - 550
12.5 - 13.8	Bonded gneiss	540 - 620
13.8 - 32.5	Granite	420 - 900
32.5 -34.2	Bonded gneiss	490 - 530
34.2 - 35.5	Granitic gneiss	420 - 480
35.5 - 35.6	Augen gneiss	410 - 420
35.6 - 38.0	Granitic gneiss	390 - 480
38.0 - 38.4	Gabbro/amphibolite	450 - 480
38.4 - 40.2	Granitic gneiss	0 - 450

Cambro-Silurian sedimentary rocks

Between chainage kilometer 0 and 1,85 the tunnel is situated in different types of phyllite:

- Phyllite, dark and graphite-bearing
- Calcareous phyllite
- Phyllite, greyish green with quartz lenses. Since this sub-unit is thrusted into the other phyllites, it is not known whether it will come in contact with the tunnel or not.

Phyllite is metamorphosed slate which has developed almost visible crystals of muscovite. Parallelism of fine mica platelets creates shimmering foliation surfaces and the rock is typically highly anisotropic (Goodman 1993). Phyllite may represent a stability problem in underground openings and may cause roof falls and instability of the working face. A particular problem with such foliated rocks arises when tunneling is attempted at a small angle with the foliation.

Table 4: Uniaxial compressive strength of rock types found in the Hardangerjøkulen tunnel (Myrvang 2001).

Rock type	σ_c (MPa) (Myrvang 2001)	σ_c (MPa) (Hoek 2009)	Term (ISRM 1978 1978)
Granite	142	> 250	Extremely strong
Gneiss	136 - 172	100 - 250	Very strong
Granittic gneiss	110 - 141	-	-
Phyllite	30 - 65	50 - 100	Strong
Augen gneiss	77 - 276	-	-
Amphibolite and gabbro	-	100 - 250	Very strong

Phyllite samples from Aurland, a few kilometers away from the Hardangerjøkulen tunnel, are found to have a uniaxial compressive strength varying between 30 and 65 MPa. According to Hoek (2009) most phyllites have a uniaxial compressive strength ranging between 50 and 100 MPa. It is a significant weaker rock than gneiss and granite, but this is dependent of the degree of weathering and quartz content. At the surface this rock may show poor qualities with high degree of weathering and disintegration. On the other hand, below ground surface where not weathered, the rock mass properties might be quite good. When exposed to atmospheric conditions when excavated it might change character to the worse. If the content of quartz is high, this will give a stronger rock less prone to weathering. Due to its often ductile character it might also be prone to squeezing under unfavorable stress conditions. From a hydro power tunnel situated very close to the Hardangerjøkulen tunnel it is known that the phyllite might contain graphite (Løset 2006).

Pre-cambrian basement rocks

Granite: Granites are often light colored rocks, whose feldspar is chiefly orhoclase and that contain abundant quartz. Granodiorites might also be present, a rock that has a higher amount of plagioclase than granite. The granite in the area is massive, homogeneous and fine- to middle grained. Granite is generally a very solid and competent rock, excellent for underground excavations. Granites and granodiorites shows generally very good stability except from in weakness zones. Some granites tend to be quite brittle. Spalling and rock burst might then occur if in-situ stresses are high. Granite samples from Sysenvatnet, just south of the Hardangerjøkulen tunnel, are found to have an average uniaxial strength of 142 MPa which may be classified as very strong. Granodiorite is generally also a very strong rock, typically with UCS varying between 100 and 250 MPa (Hoek 2009).

Gneiss: Gneiss is a banded rock with fairly continuous segregations of different minerals. Gneiss originate from metamorphism of granitic rocks, which they resemble except from the banded, foliated structure. Gneiss has a good reputation in engineering construction, often being a competent and strong rock (Goodman 1993). Massive, unfoliated gneiss create excellent conditions for underground openings. The granitic gneiss in the area is found to be homogenous and fine- to middle grained with a light red-gray color. Gneiss in general will typically have a UCS between 100 to 250 MPa. Gneiss from Sima and Sysenvatnet are found to have a UCS varying between 136 to 172, while granitic gneiss from Sysenvatnet are found to have a UCS between 110 and 141 MPa. At the Sima power plant the mechanical properties of the gneiss was found to correspond well to the average Norwegian Pre-Cambrian gneisses.

When the segregation of different minerals in distinct bands has developed in the gneiss, it is known as banded gneiss. Being more anisotropic than normal gneiss it might represent a stability problem especially if the tunnel is aligned close or parallel to the foliation. Migmatitic gneiss is a banded gneiss with highly curved or wavy bands, lenses and pods generated by partial melting or by mixing from fingerlike intrusions (Goodman 1993).

Growth of new minerals that force the host rock apart generates lenticular or eye-shaped porphyroblasts, termed augen. Augen gneiss, like gneiss, shows generally good quality and is a strong rock.

Gabbro/amphibolite: Gabbro is a dark, plutonic crystalline silicate rock without quartz, while amphibolite is metamorphosed volcanic or intrusive rocks. Gabbro and amphibolite are generally strong rocks suitable for underground excavation works (Løset 2006). Amphibolites in Norway are found to have UCS varying between 80 and 150 MPa , while Gabbro often shows strength exceeding 200 MPa. Rocks containing amphibole tends to be ductile. The mineral will give strong rocks, but might have reduced strength if the amphibole is transformed into mica or chlorite.

2.5 Joints

Nearby tunnelling projects have experienced a few open joints filled with gouge material, often calcite or swelling clay. The gouge material is often washed out, and this has caused water leakages.

Phyllite: Metamorphic rocks like phyllite may contain four or more sets of regularly spaced, extensive joints. Typically, there is one set of joints parallel to the original bedding direction of the rock mass, one set parallel to the foliation and two or more sets of fracture surfaces in other directions (Goodman 1993). The phyllite in the area is found to be strongly foliated and often schistose (Løset 2006).

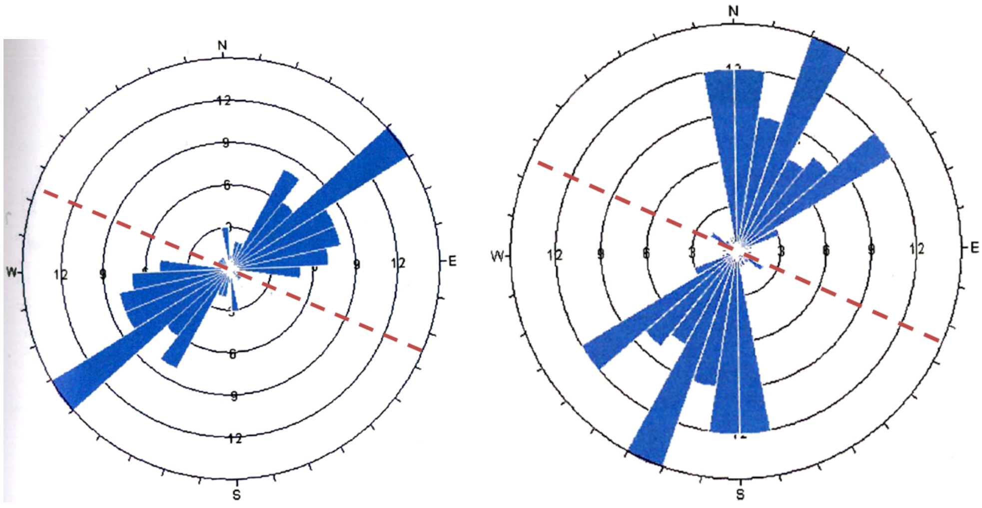
Granite: Granitic rocks often contain fairly regular planes of jointing, often in two or three directions through the rock mass. In hard, unweathered granites, joints tend to be rough surfaces with considerable friction, but they do cut the rock into blocks. Jointing is especially well developed in the margins of some igneous intrusions where unequal rates of cooling has set up locally high strains.

Frequently in granitic rocks there has been chemical alteration by hot fluids and thus the jointing is abundant. The spaces between the joint walls may contain clay infillings. Such granitic rock masses are difficult to excavate, especially if there are swelling clay like montmorillonite (Goodman 1993). At the nearby Eidsfjord hydropower plant it was experienced a few joints with infillings of clay, chlorite and graphite (Løset 2006). This seems to be quite typical for granites and gneisses in the area.

Gneiss: Gneiss will typically contain one set of joints parallel to the bedding direction of the rock mass in addition to 3 or more sets of regularly spaced extensive joints. At Aurland power plant, constructed in gneiss of same geological formation, the rock mass was found to be jointed with several and varying joint sets (Gunleiksrud & By 1987).

2.6 Faults and weakness zones

Weakness zones in the area of the Hardangerjøkulen tunnel are often characterized by crushed rock and clay (Løset 2006). As shown in figure 7 most of the fault zones are oriented approximately NE-SW.



(a) Hardangervidda-Setesdalsheiene (b) Jotun nappes

Figure 7: Joint rosettes based on NGU's geological maps showing orientation of the largest fault zones in the area of a) Hardangervidda-Setesdalsheiene and b) the Jotun nappes (Løset 2006). The tunnel alignment is indicated by the red dotted line.

From geological maps (Askvik 2008, Henriksen 2000), a total of 14 weakness zones are identified along the tunnel alignment. These are either faults or major fracture zones oriented N-S to NE-SW. The orientation is favorable with respect to the tunnel axis as they cut the tunnel with a large angle. Minor faults and weakness zones in addition to the ones identified from geological maps should also be expected.

Experiences from nearby tunnelling projects and study of geological maps indicate that there are a few weakness zones, and that these are likely to be filled with clay:

- Sima power plant: Few, but well distinguished fault zones were encountered. Some of the weakness zones contained swelling clay (Myrset & Lien 1985).
- Aurland power plant: Weakness zones with a width of approximately 1 m was encountered, some of them filled with clay (Henriksen 2000). During construction of a tunnel between Skrulsvatn and Floskefonn, close to the eastern portal of the Hardangerjøkulen tunnel, a thrust fault was hit. Located in phyllite, it was 1.5 m wide with a central zone containing graphite and swelling clay.
- The Finse tunnel: No major weakness zones were found, and only one zone required concrete lining (Dokken 1992).

Weakness zones found in the basement rocks are most likely formed by tectonic activity. Massive and brittle rocks, like granite, are then typically crushed into a more coarsely grained material. The surrounding rocks will often be affected showing a higher jointing density. Weak and ductile rocks like phyllite tends to be deformed in smaller zones into a fine-grained material (Nilsen & Broch 2009).

2.7 Rock Overburden

The rock overburden of the tunnel alignment will vary as shown in the geological profile, appendix A and table 3. Except from at the portal areas, the rock overburden is exceeding 400 m with a maximum of 900 m. As opposed to shallow underground excavations, the interlocking forces in rock mass will be very good due to the high overburden. But high overburdens may also cause high in-situ stresses. The stresses might induce new cracks and fissures in the rock mass, and will in worst case lead to rock burst (Nilsen & Broch 2009).

3 Review of long and deep tunnels

Recent projects of very long railway tunnels (from 20 to 50 km or more) at great depth (thousand meters and more) have only emphasized what has been recognized for many years with existing long railway and road tunnels (from 5 to 15 km or more): the work is a complex civil engineering system, to be designed under certain conditions with stringent requirements for safety during construction and operation (ITA 2010).

The need and request of long tunnels is increasing with today's high-speed and high-capacity transport projects. Several benefits are associated with long tunnels such as shortening of route milage and travel time, raised train speeds, reduction or mitigation of environmental impacts and improvement of safety and quality of travel compared to old difficult routes involving many curves.

Construction of long tunnels already started in the late 19th century with the great Alpine crossings. In Japan, the construction of long tunnels started about 1920 with the Shimizu tunnel (9.7 km). Long tunnels were required, and still is, in order to cross mountainous areas. During the last 25 years several long railway tunnels have been constructed, the longest being the Seikan tunnel in Japan (53.9 km) and the Channel tunnel between France and England (50.5 km, figure 8). In addition to long traffic tunnels, several long tunnels are constructed around the world for hydropower and water supply etc. Long tunnel projects are also at the time being under construction and at the planning stage. Amongst them is the Gotthard base tunnel through the Swiss Alps which will be the world's longest tunnel (57 km) when it opens for traffic in 2016 (ITA 2010).

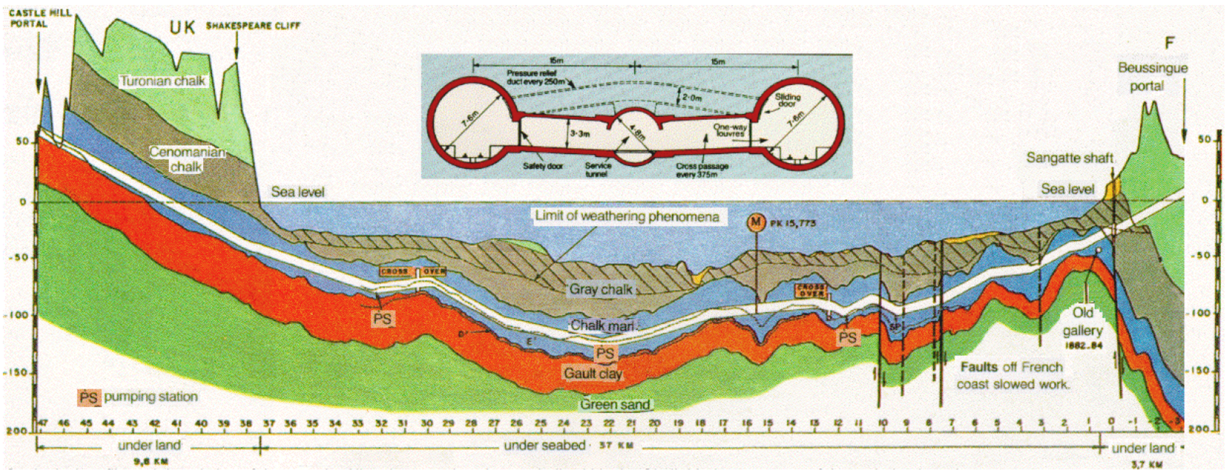


Figure 8: Geological profile and sketch of cross section of the Channel tunnel connecting England and France (Pompée 2012).

Extreme conditions for risk assessment and risk management is characterizing long and deep tunnels during the project's lifetime. Both non-technical and technical aspects is of major concern. Many factors must be taken into consideration during planning and construction, like geology and hydrogeology, tunnel design, construction method, costs and funding, operation and maintenance, operation safety, work hygiene, work safety and environment.

3.1 Major concerns and constraints for long and deep tunnels

Most long tunnels, and especially those at great depths, are located in mountainous areas with limited infrastructure. Thus, geological, hydrogeological and geotechnical information is often limited. The deeper the tunnel, the larger are the uncertainties and the higher is the probability of encountering adverse and unforseen conditions. Great effort and cost for site investigation are essential in order to reduce the uncertainties.

Common adverse tunnelling conditions often encountered are many (ITA 2010):

- High in-situ stresses which are difficult to estimate and not easy to measure
- Strong squeezing and high plastic deformation in rocks
- Spalling and rock burst in hard rocks associated with high in-situ stresses
- Swelling and creep in argillaceous rocks
- High water pressure and/or large water inflows
- Faults and shear zones, often associated with poor strength and high water pressures
- High ground temperatures, which may require ventilation and even cooling during excavation
- Heavily tectonized and fractured zones with poor mechanical and anisotropic conditions
- Environmental impact on underground water or large aquifers

Unforeseen geological conditions may lead to erroneous choices in the construction development with dramatic consequences for the workers and the work. Uncertainties in construction parameters may also influence construction time and cost. Thus, risk-minimized engineering solutions should be developed. This includes site investigations like the use of systematic probe-drilling, cost-benefit analysis and integrated design of the investigations, the tunnel options and the construction methods.

Not only geological conditions, but also logistics and operation are of major importance in long tunnels. The need for equipments for ventilation, cooling system, de-gassing, dewatering and mucking are strongly related to site conditions as well as length and depth of a tunnel. The maximum drive length from a portal is limited by logistic constraints and extension of time, thus often requiring deep access shafts and/or construction access tunnels, which sometimes also are required to be long and deep. Long access roads are often to be built, even for the site investigations.

When operating a long tunnel, ventilation and cooling requirements are of major concern. This is especially an issue if the tunnel is deep as well and the temperatures high. All technical, safety, operative and environmental aspects must be considered. Especially important is the fire aspect. The chimney effect resulting from large pressure differences between shafts, intermediate adits and the tubes has to be prevented with screens, lock chambers and over pressurization in rescue rooms, or similar devices.

Not only technical aspects are of importance when a long tunnel is to be built, but also non-technical:

- There must be political will, often at the highest level of responsibility, the government.
- There are great investments involved, and the project must be economical feasible.
- The sources of unforeseeable factors are many and they may influence cost and time required for construction.
- A multiplicity of techniques are required (traffic forecast, running, maintenance, layout, signalling, traction, aerodynamics, civil work, safety..), involving several specialists which requires a high degree of co-ordination.
- During the long implementation time of a project, changes are quite common in the political, economical and social conditions, the material and labor conditions, and staff involved in the project.

3.2 Site investigations

Before construction of a tunnel can start, the project undergoes several project phases. This includes strategic planning, technical feasibility study, preliminary design, final design, tendering and construction design. During the preliminary study and design phases sufficient information about the ground conditions must be provided. This should be obtained from the following (ITA 2010):

- Aerial photographs, satellite images and geological maps
- Terrain system mapping - a scientific classification of a large area based on topography, soils, rocks and vegetation correlated with geology, geomorphology, hydrogeology and climate.
- Field mapping and study of the identified corridor.
- Data on existing properties and utilities in the project corridor.
- Drilling and geophysical explorations supplemented by in-situ laboratory tests.

- 3-dimensional ground models
- Investigations for the final design

Study of maps, field mapping, geophysical exploration, drilling and laboratory tests must be executed to understand the ground structure and its engineering behavior at the tunnel level. An example is The results should be combined into a 3-dimensional model of the ground along the project corridor, which will be of great help in testing the sensitivity of alignment variations. Engineering geological and geomechanical characterization of each potential alignment for the tunnel option must be found. The collected information are used for technical feasibility study, preliminary design analysis and environmental impact assessment of the project.

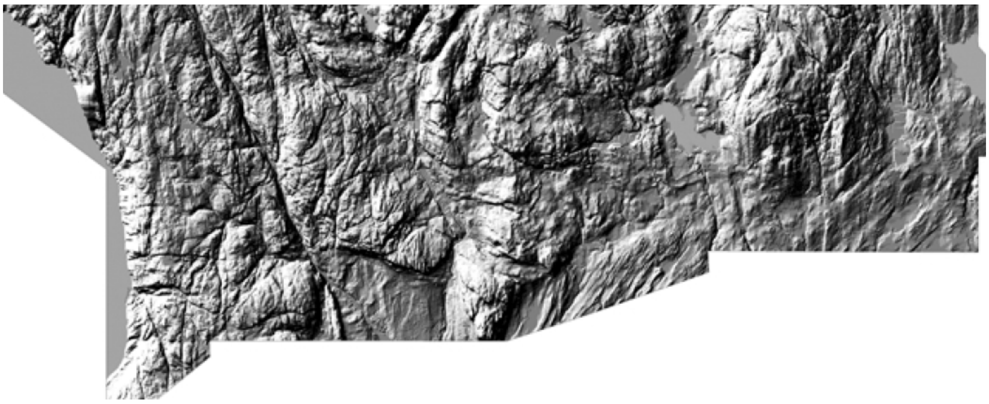


Figure 9: Digital elevation model of the Oslo region used for identifying weakness zones (Kveldsvik, Erikstad, Holm & Enander 2007).

As a final stage of the investigation program, and if found necessary, it can be of great benefit to execute investigations at the level and in the alignment of the tunnel. This may be especially important where geological conditions are not easily predicted from above ground like in deep tunnels, and may include horizontal adits or directional drilling. Horizontal adits may be used for in-situ testing and convergence measurements on different types of support. Such adits may be long and expensive, but their cost can be paid back by integration into the final project. It can for example have the function of a safety tunnel, access tunnel to critical zones for the construction of main tubes or drainage tunnel at a later stage. Directional drilling is also a widely used option, especially where the access of an adit is not existing or not feasible, like for subsea tunnels (ITA 2010).

3.3 Tunnel system design

Possible tunnel systems for long tunnels may be presented by 4 basic concepts (ITA 2010):

1. Single tube double track tunnel with intermediate rescue station
2. Single tube double track tunnel with a parallel safety-service tunnel
3. Two parallel single track tunnels, with intermediate rescue station
4. Two parallel single track tunnels, with a third service tunnel

The choice of tunnel system should consider required transport capacity, the construction aspects, equipment for operation and safety like ventilation, lighting, signalling, communications and cableway. Each option has advantages and drawbacks and the worlds longest railway tunnels have different system options. On the other hand, safety requirements for railway tunnels becomes more and more stringent and thus there has been a tendency towards two parallel single tracks for tunnels with lengths exceeding 10 km during the last 10 - 15 years (Kalager 2009).

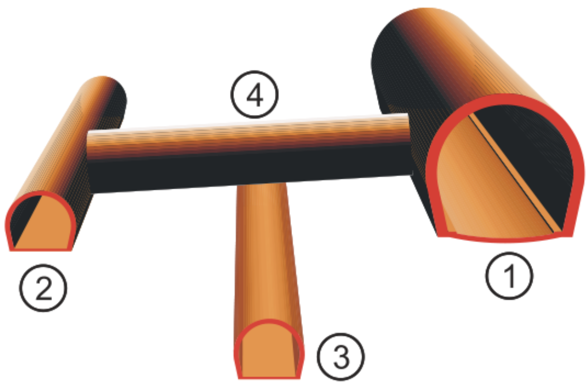


Figure 10: Example option 1: Seikan subsea tunnel; (1) main tunnel, (2) service tunnel, (3) pilot tunnel, and (4) connecting gallery

Common arguments are that one should have two separate tubes if the tunnel is long and are to be dimensioned for high speed trains, a high number of passing trains and mixed traffic (both freight and passenger trains). Traffic safety is of very high importance and is thus often justifying the extra costs associated with two tubes instead of one. For safety reasons, cross passages between two parallel tubes or between one tube and a service tunnel have to be constructed at an interval not greater than 330 to 500 m for railway tunnels.

3.4 Choice of construction method

Construction of long tunnels will be either by conventional drill and blast (D & B) or by tunnel boring machine (TBM). Several advantages and disadvantages are associated with both methods, and they have been competing for more than 30 years. While D & B is more flexible, the TBM will normally have better progress if the equipment is specially designed for the job. However, time and design for mobilization is longer for the TBM and TBM is more vulnerable to unexpected geological conditions and requires a higher effort for ground investigations during the planning stage (Sweco 2011). Several conditions have to be established and evaluated before a decision can be taken on the excavation method, but often as in most other manufacturing the most cost effective is the preferred one.

3.4.1 Conventional tunnelling

The conventional tunnelling concept is based on the type of excavation equipment, and the term is often used for any tunnel that is not excavated by a TBM. In the context of this work, conventional tunnelling means the construction of underground openings of any shape with a cyclic construction process of:

1. Excavation by using the drill and blast method
2. Mucking
3. Placement of relevant rock support elements such as rock bolts, shotcrete or concrete lining

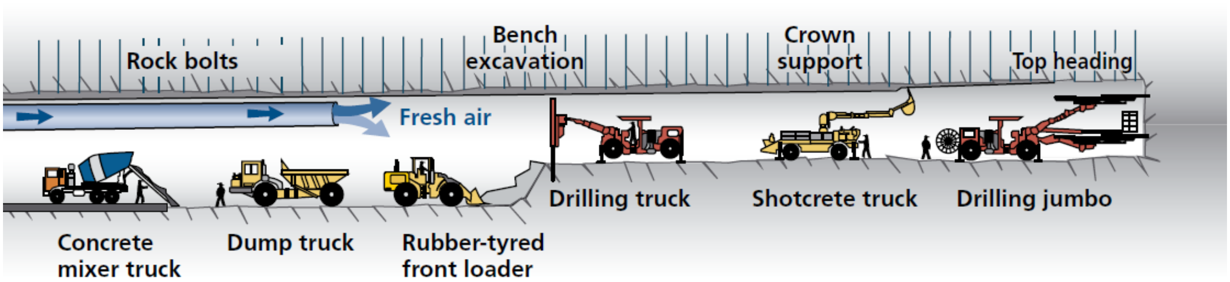


Figure 11: Sketch of drill and blast method (Unterschütz 2004).

The method is mainly using standard equipment including a drilling jumbo to drill holes for blasting, rock bolting, grouting etc., lifting platform allowing workers to reach each part of the tunnel crown or tunnel face, lifting equipment for steel sets, loader or excavator for loading excavated material onto dump trucks, dump trucks for hauling excavated material and shotcrete manipulators for application of wet or dry shotcrete (figure 11). The use of such equipment allows access to the tunnel excavation face at almost any time, and makes the method very flexible (figure 12). Changes can easily be applied during construction if ground conditions change or if monitoring results require action (ITA 2009). This includes:

- Increase or decrease of support
- Variation of ring closure time - the time between the excavation of a section of the tunnel and the application of support (partial or full support)
- Variation of explosives charge per blasting round and variation of detonator sequences.
- Increased or decreased length of excavation round
- Partial excavation by splitting the excavation face into segments

If special ground conditions are encountered, the conventional tunnelling method can react and utilize a variety of auxiliary construction technologies like different types of grouting and technologies to stabilize and improve the ground ahead of the actual tunnel face like forepoling and ground freezing. The possibility to adapt to the ground conditions as an excavation advances, and the easily changed shape of the excavated area, makes often conventional tunnelling method the most advantageous in many projects.

(a) (b)

Figure 12: Example of conventional tunnelling processes; a) blasting and b) face support with shotcrete (ITA 2009).

Main advantages compared to the TBM method may be summarized as follows (ITA 2009, Kalager 2009, Nord 2006):

- Flexibility when rock conditions are varying, and different methods exist to deal with potential unforseen problems.
- Enables a greater variability of the shape of the tunnel.
- Better knowledge of the ground by using systematic exploratory drilling at the tunnel level ahead of the face.

- Easier optimization of the primary support using the observational method in special cases.
- With respect to advance rate, D & B is less depending on strength characteristics of the rock than TBM.
- Equipment being used in D & B tunnels can be adapted to fit the actual tunnel size, and thus the relative decrease in advance rate with increasing tunnel area is less than for TBM tunnels.
- Faster advance rate when heavy support is required.
- Equipment for D & B can normally be transported as it is, and costs for transport and erection are a very small part of the total costs.

One of the main disadvantages are the danger associated with handling and using explosives. Men and equipment have to be moved out of the way before blasting, and fumes have to be exhausted. In long tunnels, an additional problem arises with long headings and the potential for prolonged stoppages resulting from the need to halt other traffic while explosives are being moved to the working face. The logistics of transporting excavated material and construction material also become even more complex . If the topography is suitable, several adit tunnels might be used to decrease the length of headings and thus also reducing the construction time (ITA 2010, Sweco 2011). The main disadvantages may be summarized by the following:

- The method requires often several adits in order to achieve an acceptable construction time, especially in long tunnels.
- Excavation rates are highly dependent upon watertightness criteria and the requirement for grouting.
- The method often requires more rock support than with TBM.
- Danger is associated with handling and using explosives.

3.4.2 Mechanized tunnelling - TBM

Mechanized tunneling make use of hard rock excavating machines with circular rotating head equipped with disc cutters (figure 13 and 14). The cutters are rolling along a circular path on the tunnel face. The load on the cutters makes the rock fail and the loosened pieces fall by gravity down to the tunnel invert where they are picked up by buckets mounted along the periphery of the cutter head of the TBM.

In sound rock TBMs make normally more rapid progress than conventional tunnelling, and this is one of the main advantages with the method. Options are available which permit the erection of final lining as the excavation progresses, so that faster completion is assured.

TBM is also safer when excavating overstressed brittle rock since popping rock is confined by the face of the shield and by the lining erected under the cover of the tail shield.

The many advantages may be summarized as follows (Hansen 2012):

- The circular profile will often give a better stability of excavation eliminating all sharp corners.
- Minimal disturbance to the rock mass confining the tunnel and to the surroundings like noise and vibration.
- Reduced extent of rock support, and also more easily predictable.
- The cross-sectional area of water tunnels may be reduced by a factor of approximately 2/3 of blasted cross section. This reduces the amount of excavated material which needs to be deposited or reused.
- Longer tunnels may be excavated without the need of extra adits because of reduced need of ventilation and (often) higher excavation rate. Fewer adits and working sites will reduce road construction and thus reduce environmental disturbances to nature.
- The use of continuous conveyor system for transportation of excavated rock masses out of tunnel reduces the need of diesel-powered vehicles and whit that less exhaust fumes.
- The use of linings for support and watertightness gives more predictable cost and excavation rates than by conventional tunnelling with grouting and heavy rock support.
- In principle, the tunnel may be finished parallel with the excavation.
- The cost during the operation of the tunnel is reduced for a TBM tunnel compared to a blasted tunnel.

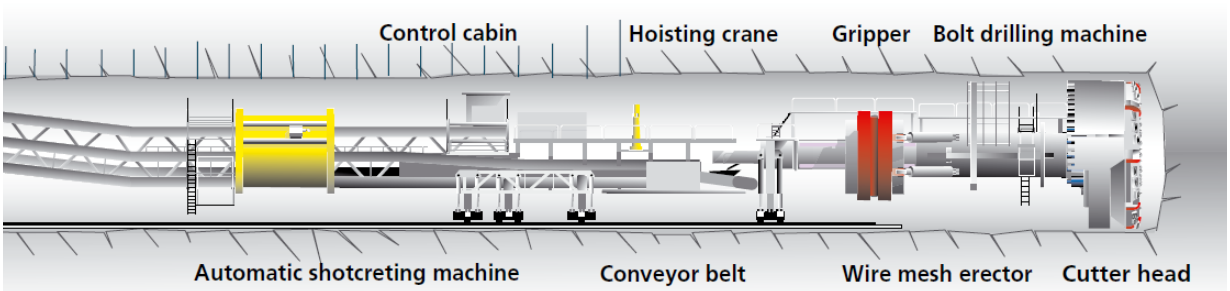


Figure 13: Longitudinal section of a TBM (Unterschütz 2004).

Considering safety and working environment, one of the main advantages of TBM compared to D & B is the eliminated use of explosives. With TBM, utilization of renewable energy as power supply for the project will reduce the emission of environmentally unfriendly gases and thus improve safety and working environment inside the tunnel. In addition, there are no need of storage and handling of blasting agents.

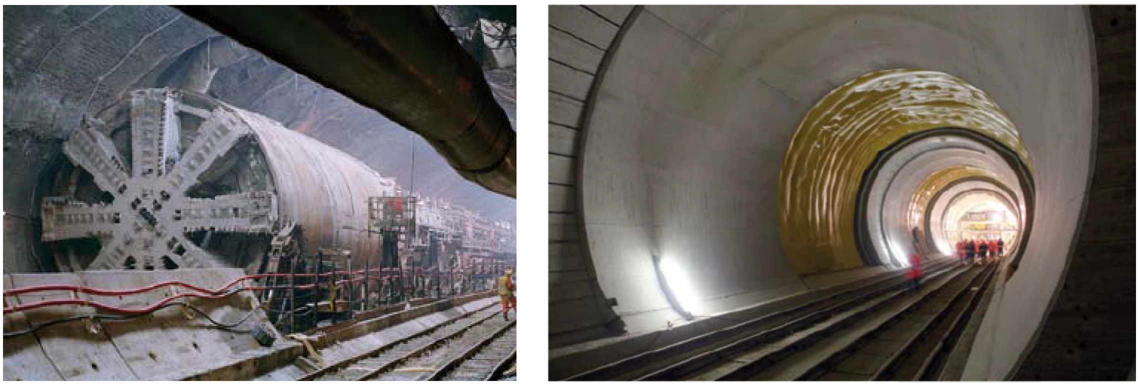


Figure 14: Shielded TBM machine and an almost completed TBM tunnel with concrete lining (ITA 2010).

To go with a TBM option is associated with large investments, and the tunnel must have a reasonable length to motivate for the large costs of buying the machine. This is also linked to the market for used TBMs of the actual size required for the project. Another solution is to rent the machine if that option is available. Additional costs are also associated with the infrastructure needed at site. A TBM site requires normally a more demanding infrastructure with respect to roads power supply, handling of the excavated muck and land at the portal areas for workshops storages and transport arrangements. This normally means higher cost and longer time for TBM mobilization. Transportation of the equipment to site comes in addition. This requires good roads with high load bearing capacities.

Compared to D & B TBM tunnelling is less flexible. Adjusting and adapting the alignment and the cross section's shape and size becomes more difficult when the tunnel is under construction. Rock conditions are also having a higher influence of excavation rates and costs than with conventional tunneling and thus more effort must be put into the preliminary ground investigations in order to get reliable construction costs and time estimates.

Main disadvantages of the TBM method compared to D & B may be summarized by the following (Hansen 2012, ITA 2010, Nord 2006):

- Investment requirements are high.
- Longer delivery time of new equipment (10-12 months for TBM versus 5-6 months for a drilling rig).
- Change of tunnel diameter is limited when using a TBM and requires a reconstruction of the machine.

- For road and railway purposes, a circular cross section is less convenient. The excess area at the bottom of the profile may be used for ventilation and cable ducts.
- A more powerful electricity supply is required for TBM tunnelling compared to conventional tunnelling.
- Blasting of niches, branch tunnels and breaks is more difficult and troublesome when using a TBM, especially with small cross-sectional areas.
- To be able to choose and prepare the appropriate TBM, more comprehensive and detailed geological mapping are necessary.
- Larger tunnel diameters results in decreased advance rates.
- A TBM excavation will suffer from down town caused by changes of cutters, regripping, daily maintenance, break downs etc.
- Difficulties with installations of support right behind and ahead of the cutter head.
- Requires more demanding infrastructure at site and mobilization time than with D & B. Total costs for transport, TBM rigging and erection is significant.

3.5 Safety and environment

3.5.1 Designing a safe tunnel

Safety is one of the key elements for the design of long and deep tunnels. Through all the project phases from the design phase to construction and operation, an optimal level of safety must be aimed at each stage. This has an impact on the design of the equipment, ventilation, cross passages, emergency exits and so on. Implementation of a safety organization, safety philosophy and objectives and safety measures into the project are of major importance.

Safety issues during construction are related to the construction methods. If using D & B a special problem is related to health and safety and working conditions with ventilation of gases from explosions and transport vehicles during construction. Extra ventilation shafts might be required at certain intervals during construction. Drainage might be a problem in horizontal or slightly inclined tunnels, and drainage and pump systems must be designed to take care of water leakages and process water during construction.

Because of the confined environment in a tunnel, accidents and fires in tunnels may have dramatic consequences. Arrangements must be found to allow for evacuation of people, retaining of dangerous substances, and for an efficient engagement of rescue crew. Technical innovations should be followed and adapted in order to reduce risks for not only users and workers, but also the environment (ITA 2010).

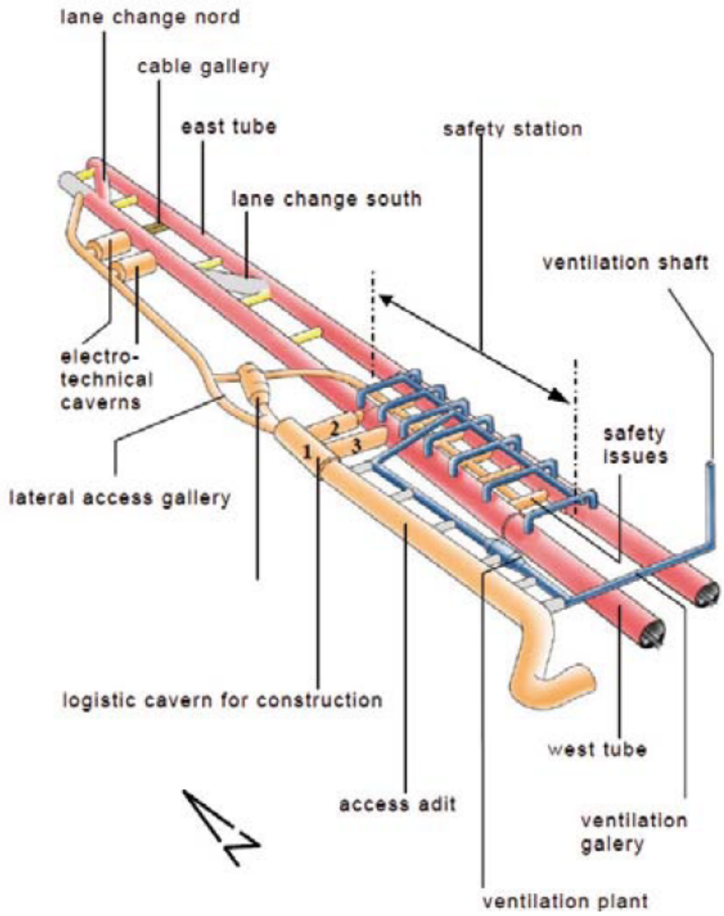


Figure 15: Ferden rescue station in Lötchberg base tunnel, Switzerland (ITA 2010).

The safety design of a railway tunnel includes the signal system, electric traction, lighting, sound system, ventilation and smoke evacuation. Requirements exists regarding indications of derailment, access routes to the tunnel opening, water supply, fire extinguishers, escape walkways, emergency lighting, signs, communications, power outlet and fire protection of construction. Facilities must also be planned for rescue services and for evacuation and protection of passengers. This includes access galleries, bypasses, refuges and first-aid rooms. The safety of a railway tunnel requires minimum (Jernbaneverket 2012):

- For single-tube systems, the maximum distance between successive smoke-free exists or safe areas shall be in the order of 1000 m (lateral or vertical emergency exit to the surface).
- For double tube system the cross-passages to the other tube shall be provided at least every 500 m.
- Alternative system able to provide same level of safety.

Often will an adit from the construction of the tunnel be equipped as emergency exits. Special shafts should also be considered. The cross-passages between two tubes shall have a minimum width of 1.5 m and height of 2.25 m. Emergency exits shall also act as an access route to the railway for the rescue crew, and shall have a minimum width and height of 2.25 m.

Based on length and daily number of trains, a railway tunnel may be classified into a risk category. A long tunnel (> 15 km) will most likely be classified into the highest risk category, unless the distance between adits acting as emergency exists are small (< 15 km). The following supplementary safety measures are highly relevant for long tunnels (Jernbaneverket 2012):

- Ventilation
- Extension of cross section
- Guide vanes
- Railed transport for evacuation
- Summit

At the time being, safety requirements for railway tunnels are leading towards a design with two single track tubes. The safety scenario in long tunnels also lead to design tracks for passing trains (loop lines), and crossovers to allow trains to change track. A few new long railway tunnels have included safety stations into the tunnel design where evacuated passengers may escape to the surface via ventilated access tunnels or shafts.

3.5.2 Environmental aspects

Long tunnels have several benefits to the surrounding environment. However, tunnel activities related to the surface environment will be concentrated around tunnel openings, adits, transport activities, logistics and disposal of tunnel spoil.

From the early design stages to the operation of the tunnel, a sustainable development is a key issue for the environmental management of the project. During construction potential environmental impacts are many (ITA 2010):

- Settlements, slope failure and surface impacts
- Acoustic emissions
- Dusts and air pollution
- Vibrations caused by blasting or mechanical excavation
- Site water evacuation and treatment
- Transports (noise, dust)

- Drying up of water springs and ponds or loss of flow rates of hydro-thermal waters

Figure 16: Handling of excavated material at Amsteg, Gotthard base tunnel, close to existing buildings (AlpTransit Gotthard Ltd 2012).

Already at the early design stage, portals, shafts and adit locations should be assessed with respect to areas sensitive to nature and landscape. For long tunnels, the amount of spoil is significant and handling and transport of tunnel muck will be a great challenge. Tunnel spoil and drain water from tunnels are basically contaminated and measures for cleaning drain water from tunnel openings and deposits for excavated material should be prepared (Sweco 2011).

Environmental loads must be kept to an acceptable level with proposed and implemented risk mitigation. After the construction of the tunnel, during the operational phase, disposal of non degradable pollutants into air, water and soil must be kept at or below a defined and acceptable value. Care must be made in order to dispose the excavated material in a justifiable way, and it should be aimed at recycling or reusing the tunnel spoil. Final deposit of material must take national regulations into account.

3.6 A case study: Gotthard base tunnel

The Gotthard base tunnel is a part of the AlpTransit project linking countries north and south of the Alps closer together. Construction works began in 2003 and the tunnel is scheduled to become operational at the end of 2016. It is situated in the Swiss Alps in the cantons of Uri and Ticino. With a length of 57 km and a total of 151.84 km of tunnels, shafts and passages, it is the world's longest railway tunnel. Maximum rock overburden is 2300 m, making it also the world's deepest railway tunnel ever constructed. The motivation for building the tunnel was to increase transport capacity across the Alps, especially for freight traffic, and to shift freight volumes from road to rail. The journey time for national and international passenger trains will also be cut significantly. The tunnel is designed for a maximum speed of 250 km/h. AlpTransit Gotthard Ltd is the constructor of the tunnel and it is a wholly owned subsidiary of the Swiss Federal Railways (SFF). The project employs approximately 2600 persons and is estimated to have a price tag of about 7 billion CHF (approximately 44 billion NOK).

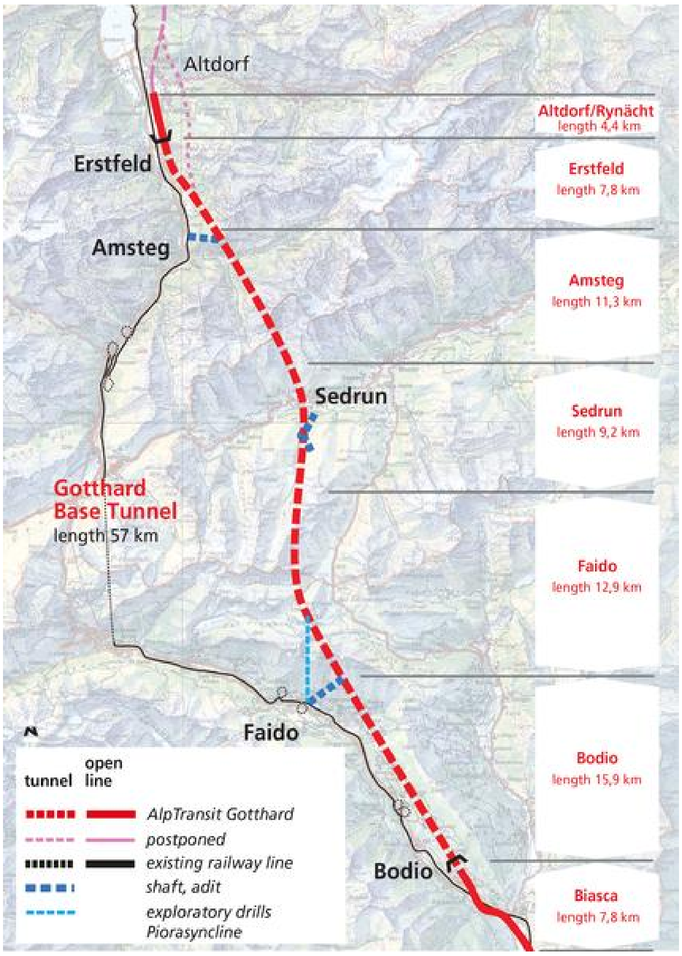


Figure 17: Overview of Gotthard base tunnel (AlpTransit Gotthard Ltd 2012)

Tunnel design and safety: For construction purposes, the Gotthard Base Tunnel was subdivided into 5 sections where construction work proceeded simultaneously (figure 17 and 18). The tunnel consists of two single-track tubes which are connected together every approximately 312 meters by cross passages. Several access adits provided access to the

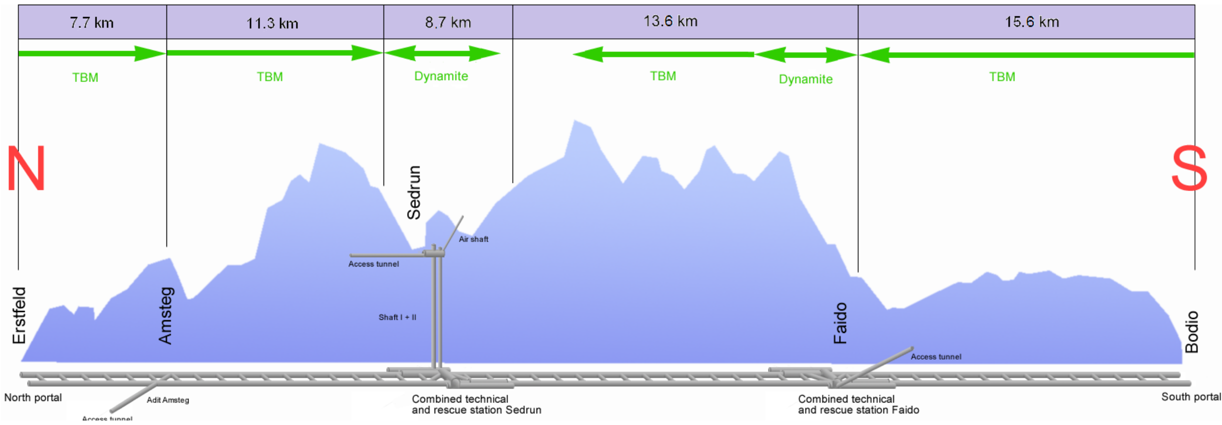


Figure 18: Scheme of Gotthard Base Tunnel

underground working sites. Two multifunction stations were constructed at Faido and Sedrun (figure 19). Each contain emergency stop stations and two track crossovers. In case of an incident in the tunnel, the affected train travels out of the tunnel into open air, otherwise the driver stops the train at the emergency stop. The emergency stop stations provide a place to stop and from where passengers can escape and evacuate. If smoke occurs, it will be sucked out of the affected tunnel and fresh air will be blown into the emergency stop station through side tunnels and connecting galleries. An overpressure will prevent smoke entering the escape route. An evacuation train will transport passengers out of the tunnel from the emergency stop station, and if a train stops before it reaches an emergency stop station, passengers may use the connecting galleries to escape to the other railway tunnel (Unterschütz 2004).

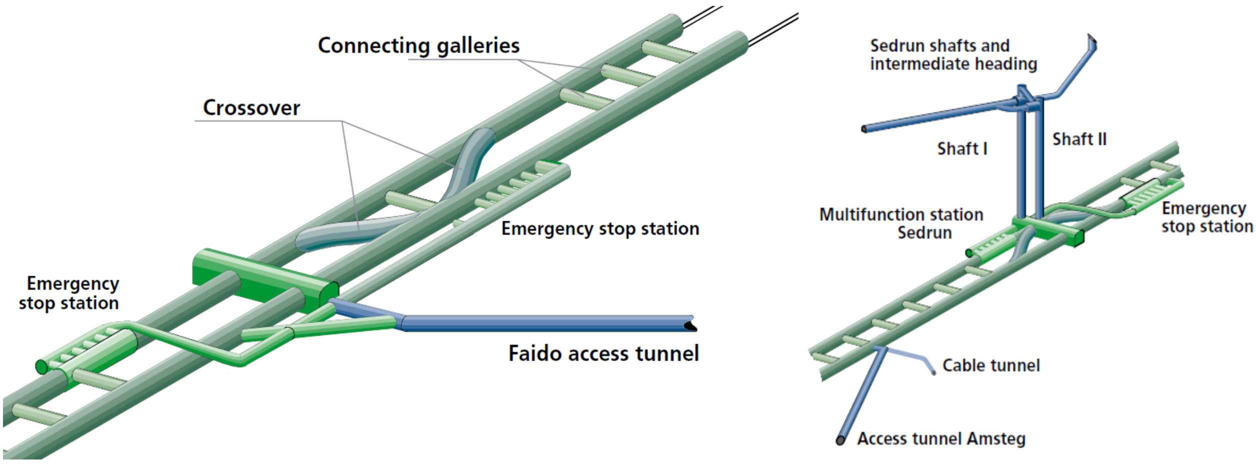


Figure 19: Simplified sketches of the multifunction stations at Faido (left) and Sedrun (AlpTransit Gotthard Ltd 2012).

Construction concept and excavation methods: To optimize time and costs, the tunnel was driven at five separate sections of different length simultaneously (figure 18). Both TMBs and D % B were used. Before excavating the two main tubes, the multifunction stations, all necessary adits and shafts were constructed.

The northerly section, Erstfeld, were excavated with TBM except from the first part of the tunnel where an open trench were established which will be covered after completion. The second section from north, Amsteg, were reached from a 2.2 km long adit driven by D & B to the position of the railway tunnels where assembly caverns were excavated. From these caverns, two TBMs started cutting south towards the Sedrun section. At the Sedrun section access from the surface was through a 1 km long horizontal adit and two 800 m vertical shafts. From there, the main tubes were excavated by D & B (figure 20). Due to the geological conditions TBMs were not used here. The section of Faido is reached from the surface through a 2.7 km long adit with a gradient up to 13 %. When the two TBMs excavating from the southern portal, Bodio, arrived here, they were repaired and modified before they continued north towards the Sedrun section. The first part of the southernly section, Bodio, was constructed above ground, and then then through loose rock before entering solid rock where TBMs were used (Unterschütz 2004).

In total, 66.311 km (43.7%) were excavated using conventional excavation method with D & B. This includes the Sedrun section (figure 20), cross passages and access adits. The excavation diameter varied between 8.8 to 13.08 m. For the main tubes TBMs were primarily used, excavating a total of 85.529 km (56.3%). Four TBMs were in operation, two south bound and two north bound. These had a total length of up to 450 m including backup train. The machines were in service 320 days per year with a daily operation cycle of 2 × 9 hour driving and 6 hour maintenance. Diameters used were 8.8 m and 9.5 m. The average daily performance with TBM varied between 9.82 to 14.77 m at the different sections, while the maximum performance achieved was 56 m a day.

Figure 20: Excavation at the Sedrun section using drill and blast. Here, the main tubes were supported by a tight pattern of steel arches (AlpTransit Gotthard Ltd 2012).

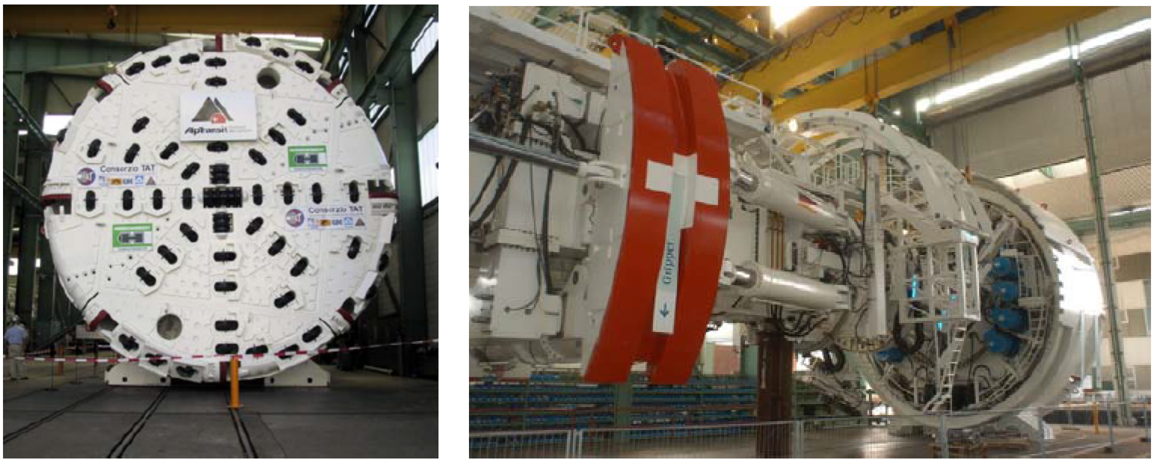


Figure 21: Open hard-rock gripper TBM used in Gotthard Base Tunnel (AlpTransit Gotthard Ltd 2012) .

Rock supports and linings: When excavating faults and other difficult rock conditions injection grouting were frequently used in order to reduce water ingress and to increase rock stability. Initial supports to prevent rock fall included anchors, shotcrete and steel arches in variable combinations depending of the geological condition. The high speed of the trains required a smooth inner lining of concrete. This lining has a minimum thickness of 30 cm and also acts as a permanent support. Where the lining was subjected to heavy stresses it was reinforced with steel. An impermeable sealing foil were placed behind the concrete lining and drainage pipes are preventing direct water ingress into the tunnel. This also prevents build-up of groundwater pressure (Unterschütz 2004).

Geological challenges: Geology was found to be the central element of risk in the project. In the case of Gotthard base tunnel, the major risks were mainly unknown geological and hydrogeological conditions at depths of up to 2500 m below the ground surface. The project demonstrated that even with extensive explorations, the geological risk cannot be ruled out entirely. For example, several times experiences was made that in one of the tubes no driving difficulties occurred, while in the other tube only 40 meters away, rockfalls occurred which caused months-long interruptions. On the other hand, conditions more favorable than predicted were also encountered during the construction.

Situated in the complex geology of the Alps, predicting geological conditions at tunnel level is no easy task. When the European and African tectonic plates collided, rocks formed from marine sediments were pushed together and lifted. In a process lasting tens of millions of years, different layers of rocks were transported, squeezed and twisted together resulting in different geological formations. Gotthard base tunnel is mainly situated in the Aare and Gotthard massifs, formations consisting of gneisses and granites. Wedged in between are younger sedimentary rocks, some of very poor quality.

Based on geological investigations during the preliminary phase, two zones were identified as very threatening for the project; a zone at the Faido section and in the area of the Sedrun intermediate heading. Around 100 million CHF (approximately 600 million NOK) were spent to investigate the so-called Piora syncline at the Faido section where "running ground" was feared, a granular dolomite mixed with water. The investigation program included several very long exploratory bore holes which revealed water and sugary dolomite above the tunnel level (figure 22). At the tunnel level, dolomite anhydrite was present, but no water. Based on the findings, special measures were planned for the approach to the Piora syncline and it was later traversed without problem by the TBM (Ehrbar & Lieb 2011).

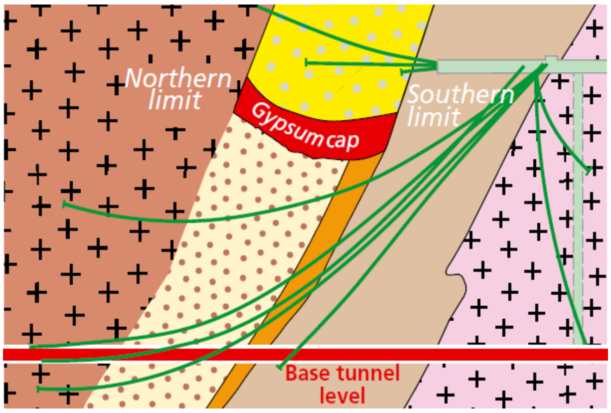


Figure 22: Geology, exploration tunnel (green) and test-bore system of the Piora syncline (Unterschütz 2004). The yellow dotted area above the gypsum cap were found to be sugary dolomite, while the brown dotted area below are dolomitic marble (approximately 130 m wide zone at tunnel level).

At the multifunction station at Sedrun two constructionally very unfavorable rock formations were anticipated. This was mainly due to highly ductile rocks under high pressure and thus severe squeezing was expected. Based on the findings from exploratory bore holes and laboratory testing of the rock, a new construction concept was developed to deal with the unfavorable conditions; steel inserts in the form of deformable steel rings in conjunction with full face excavation with systematic heavy rock support of the face. Trials of this method were executed in a side tunnel close to the actual point of application. The expected length of the squeezing zone were more than 500 m and the advance rate was calculated to be 1.1 m pr day. However, it turned out that the actual length was 305 m and a advance rate of 1.9 m per were achieved per working day . A great advance on the construction schedule was thus achieved.

Construction works at the southern sections were not progressing as expected. Unexpected pressure effects brought the TBM to a standstill and made reprofiling and repair work necessary which lasted for more than a year. The geological and constructional problems that arose at the multifunction station Faido resulted in repositioning of the station.

Not only repositioning of the multifunction station Faido, but also other adjustments had to be done during construction due to unforseen geological conditions. Examples are construction of the second shaft at Sedrun, modifications to the air exhaust system, changes to section boundaries and changes in distances between the connecting passages between the main tubes. The large number of additional or modified excavations resulted in an increase of excavated rock of about 900 000 tonnes, and additional storage possibilities had to be found close to the construction site. Other unforseen conditions included a zone with squeezing rock conditions and a 150 m long fault zone.

Even though several important unexpected events with effects on the construction time occurred, the overall time schedule had a favorable development and in 2009 AlpTransit Gotthard Ltd. postponed the opening of the tunnel with scheduled train services forward by one year from 2017 to 2016 (Ehrbar & Lieb 2011).

Spoil management When finished, about 28 million tons of excavated rock had been transported out of the tunnel. This is equivalent to the volume of more than 5 Cheops pyramids, or to the length of a freight train stretching from Zurich to Chicago (7160 km). As for all long tunnels, management of these large quantities of excavated rock presents a major challenge and the following requirements had to be met (Lieb 2011):

- Spoil management must not be a performance-determining factor.
- Recycling/reuse of the excavated rock must be maximized.
- The environmental burden must be minimized.
- The spoil management concepts must be economically sound.

A high grade of reuse of excavated material was striven for. Experiments and research programmes were established in order to prove that the fine-grained rock ships from the TBMs were suitable as concrete aggregate. Normally, such rock ships are not accepted for the manufacture of high-quality concrete. By recycling the excavated rock, this also allowed for a high degree of self-sufficiency of the construction sites. Construction of the tunnel required more than 9 million tons of aggregate for the production of concrete and shotcrete and the conversion were done locally on the construction sites. Self-sufficiency made also the logistics easier since it then were no need for third-party supplies of concrete aggregate.

9.4 million tonnes, or 33 % of the excavated material were found to be suitable as aggregate for concrete production. 0.2 million tonnes of slurry from the drives were used for reactor landfill. The rest 18.6 million tonnes which were found unsuitable for concrete production were mainly used for railroad embankments, landfill, renaturing and ballast.

Such large quantities also represent major logistical challenges, both management of the excavated material at the time it is produced and to ensure supplies of the required aggregates. Removal of excavated rock, as well as the supply of aggregate for concrete and

Figure 23: Spoil management at Erstfeld (Lieb 2011)

shotcrete production, had to be assured 24 hours a day, 7 days a week, summer and winter and also in mountainous conditions. Transport of spoil underground were done either by belt conveyor or by spoil-removal train. Approximately 70 km of belt conveyors was installed at the construction sites. This allowed transportation either directly to the spoil-processing plants or to the permanent landfills or temporary storage sites. Longer overground transportation took place by rail, and road transport was only exceptionally used.

Even though generous bandwidths were assumed from the start of the project, the actual development of the project created a need for extensive and cost-intensive adjustments to the concepts and systems for managing the large volumes of excavated rock. Recognition of the need for adjustments prevented the spoil management from becoming performance-determining and was therefore considered as successful.

4 Potential geological challenges

4.1 Experiences from nearby projects

During the seventies and the eighties, the construction of several large hydropower plants took place in Norway. Two of them, Sima and Aurland power plant, are located in the same geological formation as the Hardangerjøkulen tunnel. Experiences gained from these projects and the railway tunnel at Finse has shown that the Pre-Cambrian basement rocks in this area are of generally good quality. However, two major issues seems to be common; stress-driven instabilities and weakness zones containing swelling clay.

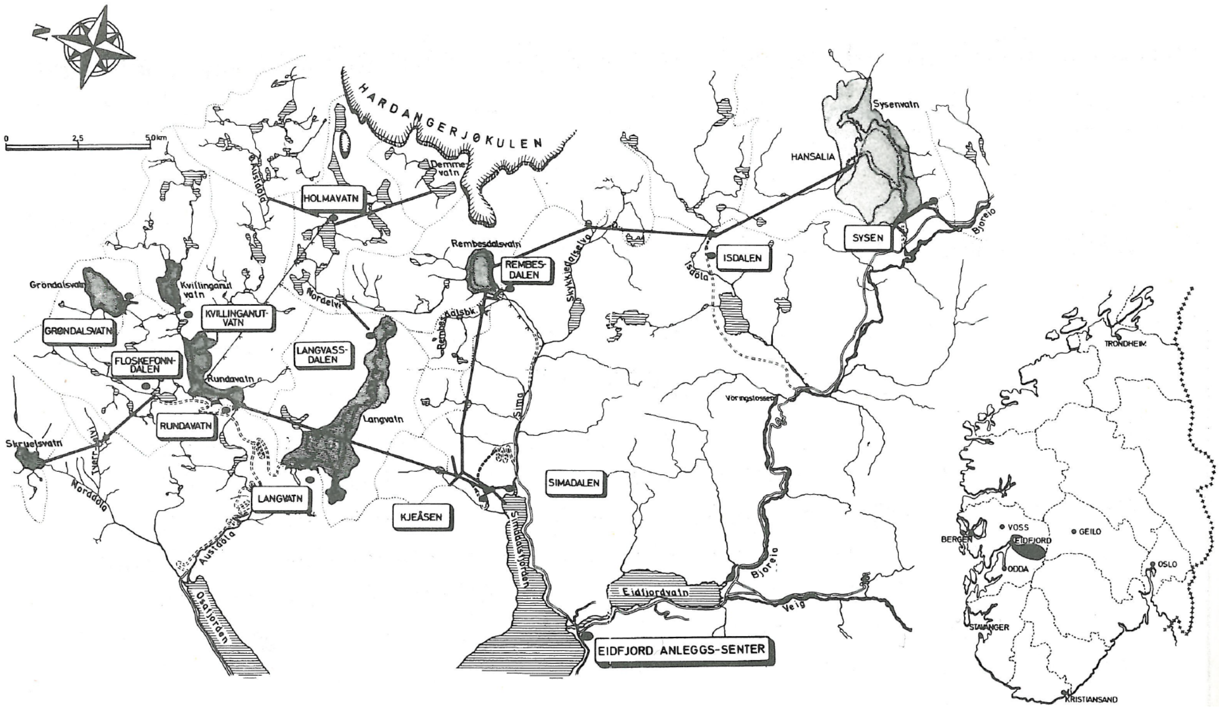


Figure 24: Overview of the hydropower plants in Eidfjord (Johansen 1976).

Sima hydropower plant

Sima hydropower plant, which once were Norwegian largest, is located west and south west of Hardangerjøkulen (figure 24). The geology of the area consists mostly of granite and granitic gneiss belonging to the same geological formation as found in the Hardangerjøkulen tunnel. The rocks were found to be quite homogeneous. The power plant, built in 1977-1979, is situated 700 m below surface level. During and also after finishing the construction works, extensive spalling and rock burst was experienced due to high stress levels. High tensional forces in the rock under extreme pressure caused rock burst as the pressure was relieved by the tunneling. To deal with the spalling and rock burst a high amount of rock bolts were installed (3-8 m long) together with steel mesh and a 120 mm thick layer of shotcrete. Even this was not sufficient to prevent spalling after finishing the construction (Martin 1990).

Besides rock burst and spalling, the second worst problem was found to be weakness zones containing swelling clay (Myrset & Lien 1985). Few, but well distinguished fault zones were discovered. Some minor weakness zones were found to be water bearing, but with concrete work and injection at low pressures (< 30 bar), the water leakages was brought down to a level below 1 l/s. In general the Pre-Cambrian gneiss was found to be favorable as a construction material showing very little water leakages. This was confirmed by permeability test which showed very good results.

Stress measurements were conducted during construction close to the power plant. The horizontal stress was found to be 5 times larger than the theoretical prediction. Mechanical properties of the gneiss were found to vary little and they corresponded well to average values of Norwegain Precambrian gneisses (Johansen 1976).

Aurland hydropower plant

During construction of Aurland hydropower plant, similar conditions were experienced as at Sima power plant. The geology here consist of foliated granitic gneiss. High anisotropic in-situ stresses were encountered, and thus spalling and rock burst were found to be the major issues. Several and varying joint sets were encountered in the rock mass. A few weakness zones were encountered with a width of approximately 1 m, some of them with clay filling (Gunleiksrud & By 1987).

The Finse tunnel

In 1992 a 10.3 km long railway tunnel was constructed at Finse, north of the glacier Hardangerjøkulen. It is located 1200 - 1300 m above sea level and the overburden is ranging between 50 and 250 m.

The rock mass at the tunnel level, mainly Pre-Cambrian granite, was found to be very competent. The rock here is described as coarsely grained with a high content of quartz and no particular foliation. The degree of jointing was found to be low to moderate except from in the weakness zones. The stability was found to be good. No major weakness zones were encountered, and only one zone required full concrete lining. Small tendencies of spalling was registered.

A zone with leaking water required extra attention and was post-grouted. Only random water leakages was encountered apart from this special zone (Dokken 1992).

4.2 Brief review on relevant theory

4.2.1 Water inflow

Even though the rock itself is more or less impermeable, there will always be joints and fissures that might lead water into a tunnel. Several issues are related to such water ingress (Nilsen & Broch 2009):

- Leakages or major water inflows may prevent or stop further construction.
- Aggressive water may cause damage to equipment and technical installations in the tunnel.
- Wash-out of gouge material and crushed rock material resulting in reduced stability.

Leakages might slow down and complicate the construction works. High water pressure and mud break-ins might cause boring, charging and blasting more difficult. Water may also reduce the friction of joints, especially if there are clorite and smectite present, resulting in reduced stability of the excavation. Other problems associated with water leakages are disintegration, swelling, rock falls and corrosion (Nilsen & Broch 2009).

It is mainly the ground water found in joints and openings that will affect operation and use of a tunnel. However, if rock overburdens are high, small joints are likely to be closed or filled with material like carbonate, epidote, quartz or clay minerals. Major water inrushes are seldom experienced in rocks like the Norwegian granite and gneiss, and leakages will most likely be restrained to faults, crushed zones and a limited amount of open joints while the rock itself is generally impervious. In soft rocks like phyllite, open joints leading water are often few. Such easily deformed rocks are also often found to be less water leading even in crushed zones because they are less prone to develop open joints (Nilsen & Broch 2009).

When considering water leakages into a tunnel, faults and crushed zones are of major importance. Ground water is attracted to a fault zone because of the greater conductivity of the fractured and loosened rock to be found here. Since the unaffected wall rock outside the zone of faulting can be expected to be significantly less permeable, the water can be expected to move much more freely parallel to the fault than across it. Often, the water will carry rock blocks and fault zone detritus with it and the excavation runs the risk of caving. Sudden loss of stability in the face of a tunnel caused by water is a serious and frequently dangerous accident in tunneling (Goodman 1993).

Predicting possible inflow and leakage into a planned underground excavation is a difficult task. This is mainly due to the fact that the permeability distribution of fractured rocks is very heterogenous. However, simple mathematical models have been published that can explain or predict the behavior of groundwater inflows into tunnels in rocks with homogenous hydraulic conductivity and Darcian flow. For a deep tunnel (tunnel radius $r <<$ depth of tunnel below water table) in a homogenious rock mass with a linear constant pressure boundary at the surface, the steady state inflow rate into a tunnel segment of length L can be described by the analytical function reported by Goodman, Moye, Schalkwyk & Javandel 1965:

$$Q = \frac{2\pi K L \Delta h}{2.3\log(2\Delta h/r)} \tag{1}$$

Where; Q is the inflow or leakage rate in m^3/s, K is the hydraulic conductivity in the (vertical) plane of flow in m/s, L is the tunnel length in m under consideration, Δh is the hydrostatic head (depth below groundwater level) in m and r equals the radius of the tunnel in m. As seen from the equation, calculations of groundwater inflow to a tunnel will mainly depend on the area of the tunnel, the depth and the original permeability of rock.

The function do not take into consideration stratification of the geology or any cross-flow gradient. However, the analytical function will work well for conditions that can be regarded as homogenious and where the cross-flow gradient is small (Johansen 2007).

Hydraulic conductivity of the rock mass may be found from permeability tests like the Lugeon test. Approximate hydraulic conductivities are suggested for some common rock types in figure 25. However, they should be used with care since the conditions and size of joints may vary considerably and since hydraulic conductivity also is a function of depth below ground surface.

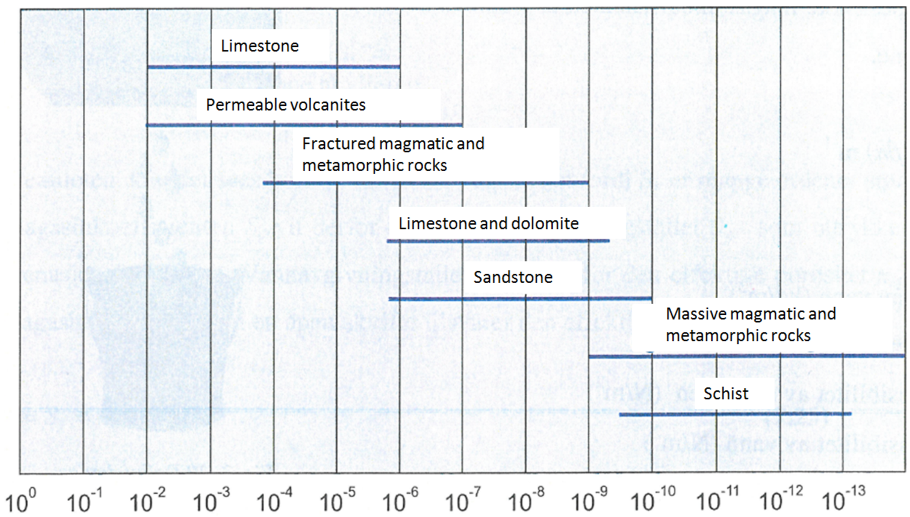


Figure 25: Hydraulic conductivity of some common rock types (modified from Brattli 2009.)

4.2.2 Potential swelling

Occasionally, difficulties caused by swelling clays in faults are experienced when tunnelling underground. The definition of the phenomena swelling is given by the International Society of Rock Mechanics by the following (Stefanussen 1999):

Swelling of rock is the time dependent volume increase involving physical-chemical reaction with water. The physical-chemical reaction with water is the major contributor to swelling but it can only take place simultaneously with or following stress relief.

Faults often act as conduits for flow of water, and this explains why rocks adjacent to them often are hydrothermally altered. Replacement of original minerals by clay minerals, as well as precipitation of these minerals in void spaces, often changes the character of the rock in and near a fault zone (Goodman 1993).

Crushed zones with clay infilling exists in all Norwegian geological formations. Smectite and the swelling mineral montmorillonite are common minerals that swells in contact with water. The precambrian gneisses will often have high amounts of feldspar close to fault zones, and the decomposition of feldspar will result in smectite and illite. Montmorillonite is formed by hydrolysis of plagioclase feldspar.

The pressure caused by the swelling clay represent a stability issue, and rock falls due to low consolidation and wash-out might occur. Clays tend to be very slippery and thus the friction angle of joints becomes very low. The swelling pressure is highly related to the deformations of rock, and swelling is a time-dependent phenomena. Often will swelling start immediately after excavation, only a few minutes if water is accessible (Stefanussen 1999). Depending on the swelling pressure, appropriate rock support must be adapted to the zones with stability problems. If there are swelling, the support should allow some deformation before the support system is loaded. Concrete elements is therefore more suitable than shotcrete as the shotcrete will stick close to the tunnel walls and thus very little deformation is allowed (Nilsen & Broch 2009).

In-situ determination of swelling clay are often difficult and laboratory analysis of the gouge material is therefore the most appropriate method for identification of swelling clay. Different methods for analyzing samples exists where the most important are mineralogic analysis for determination of specific minerals, determination of plastic properties and direct measurements of swelling and swelling pressure (Nilsen & Broch 2009).

4.2.3 Faults and weakness zones

In Norway, serious instability in underground excavations has rarely been experienced, and when such serious stability problems have been experienced most of them have been related to faults and weakness zones. Possible stability problems associated with weakness zones are (Nilsen & Dahlø 1994):

- Cave-in at the working face during tunneling.
- Instability at the working face.
- Water inflows into the tunnel.
- Swelling of smectite, especially when smectite occurs in combination with other problem- minerals like calcite (solvable) and chlorite (low friction).
- Cave-in after completion

Figure 26: Image from rock fall in the Hanekleiv tunnel. The rock fall was caused by a weakness zone containing swelling clay (Picture in courtesy of Statens Vegvesen).

Face instability is often encountered in weakness zones due to heavily fractured and low-strength properties of rocks. Such instabilities may develop very quickly. If not sufficiently supported the result may be cave-in at the working face. In worst case a cave-ins may propagate several tens of meters above tunnel level. Cave-in has also been found to occur after completion of tunnel projects, often several years later. If not sufficiently supported, a weakness zone may cause serious instability and fall-out as shown in figure 26.

Water is very often associated with faults and weakness zones as discussed in the previous chapter. Especially in brittle fault zones with open joints, major water inflows into the tunnel may cause serious problems for the excavation works. Swelling clay is also common in weakness zones.

Faults may be classified according to type of rocks they are formed in:

- Faults in hard and brittle rocks like granite and gneiss.
- Faults in weak and more ductile rocks like phyllite.

Rocks will respond different to applied stresses depending on the rock's strength and elasticity. Faults in rock types that are hard and brittle, like granite, will often result in a wide zone with crushed and coarse material (figure 27). Faults in ductile rocks like phyllite will often be characterized as a thinner zone containing fine grained material (Nilsen & Dahlø 1994).

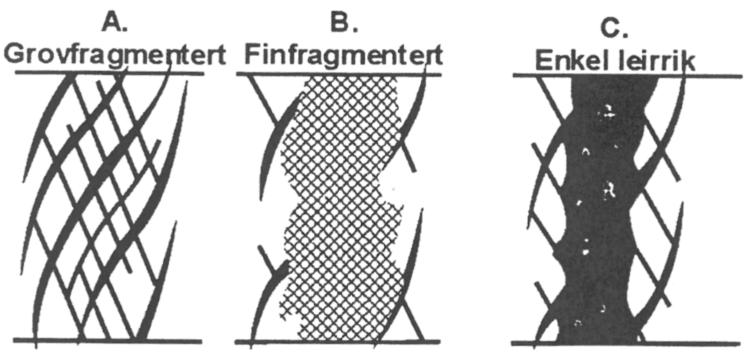


Figure 27: Classification of crushed zones; (A) coarsely fragmented, (B) finely fragmented and (C) containing clay (Nilsen & Broch 2009).

Faults and weakness zones will most likely create a need for additional rock support. The amount of extra support is dependent of the characteristics of the fault and the surrounding rock, like thickness of the fractured zone, the amount of jointing and crushing,types of minerals present and cohesion. The high water inflows often associated with weakness zones must be sealed and tightened with for example injection grouting.

4.2.4 Tunnel squeezing

The International Society of Rock Mechanics defines the phenomena squeezing by the following (Stefanussen 1999):

Squeezing of rocks is the time dependent large deformation, which occurs around the tunnel, and is essentially associated with creep caused by exceeding a limiting shear stress. Deformations may terminate during construction or continue over a long time period.

Squeezing behavior is associated with poor rock mass deformability and strength properties loaded under high in situ stresses. Weak rocks such as phyllite, shale, slates and weakness/fracture zones are incapable of sustaining high tangential stress. When the rock mass strength becomes less than induced tangential stresses, a time dependent deformation take place along the periphery of the tunnel. This time-dependent inward movement (plastic strain or creep) of rock material is defined as tunnel squeezing. Most of this inward movement occurs less than approximately two tunnel diameters behind the face . The result might be stability problems as well as an unfavorable contour (figure 28). In order to minimize and prevent instabilities it is important to establish stabilizing measures and optimize the support well in advance of the excavation works. Hence, prediction of the extent of squeezing is essential (Panthi & Nilsen 2007).

Figure 28: Failure of a tunnel section due to very severe squeezing (background) and re-mined and re-supported tunnel (foreground) (Hoek & Marinos 2009).

Several methods have been developed for predicting tunnel squeezing. One of the most commonly used is the semi-empirical approach suggested by Hoek & Marinos (2000) which estimates tunnel strain ε and tunnel support pressure p_i. In this approach, the ratio of rock mass strength σ_{cm} to the overburden stress σ_v and support pressure p_i is related to the percentage tunnel strain ε (equation 2). With no rock support in the tunnel, i.e. support pressure p_i equals zero, equation 2 may be rewritten to equation 3. Rock mass strength of phyllite may be estimated by equation 4 proposed by Panthi (2006), where σ_{ci} is the uniaxial compressive strength of intact rock.

$$\varepsilon = 100 \times (0.002 - 0.0025\frac{p_i}{\sigma_v}) \times (\frac{\sigma_{cm}}{\sigma_v})^{2.4\times\frac{p_i}{\sigma_v}-2} \tag{2}$$

$$\varepsilon = 0.2 \times (\frac{\sigma_{cm}}{\sigma_v})^{-2} \tag{3}$$

$$\sigma_{cm} = \frac{\sigma_{ci}^{-1.5}}{60} \tag{4}$$

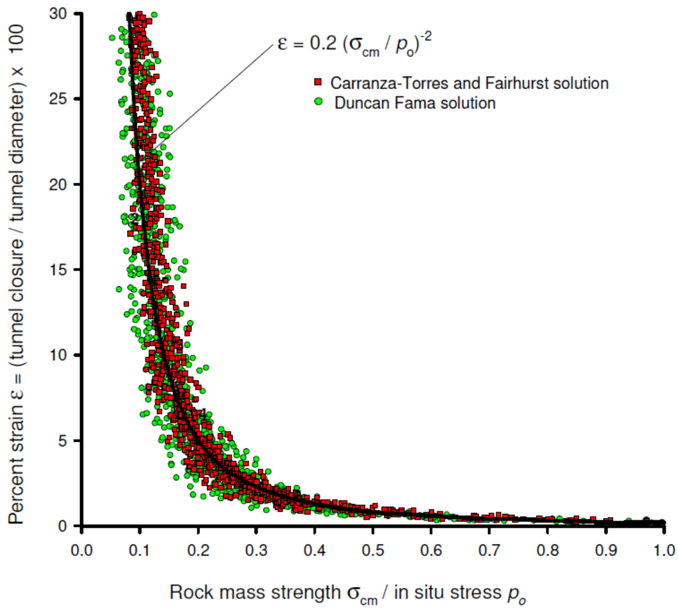


Figure 29: Plot of tunnel convergence against the ratio of rock mass strength and in-situ stress (Hoek & Marinos 2000).

Equation 2 is a result of a study based on closed form analytical solutions for circular tunnels in a hydrostatic stress field. Total tunnel strain ε is defined as 100 x the ratio of tunnel closure to tunnel diameter. The relation between total tunnel strain and the ratio between rock mass strength and in-situ overburden pressure are illustrated in figure 29 which is based on Monte Carlo simulations in a wide range of rock mass conditions. The behavior of all the analyzed tunnels follows a clearly defined pattern, which is well predicted by equation 3 (Hoek & Marinos 2000). Approximate guidelines to characterize the degree of difficulty that can be encountered at different levels of tunnel strains are shown in figure 30.

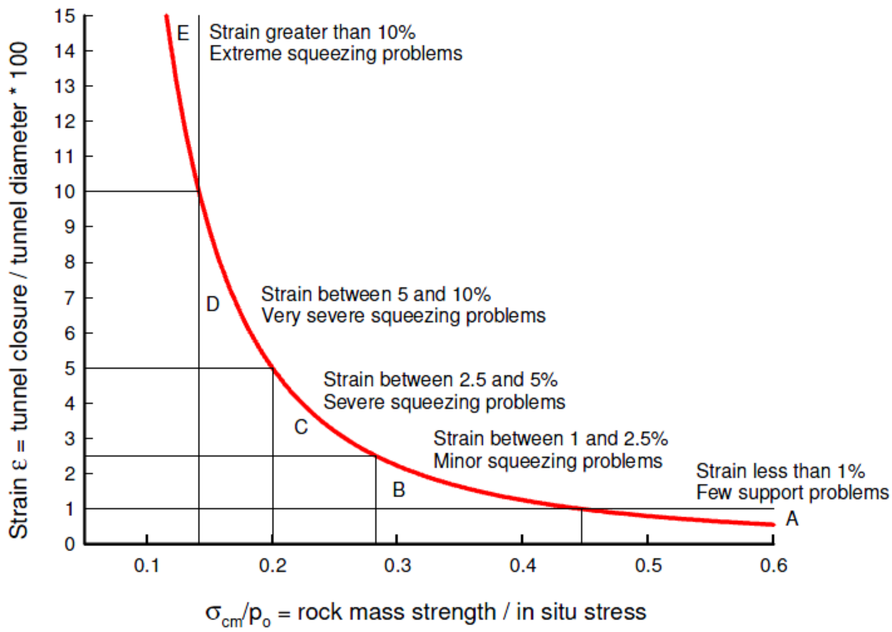


Figure 30: Approximate relationship between strain and the degree of difficulty associated with tunnel squeezing. The curve is for tunnels with no support (Hoek & Marinos 2000).

4.2.5 Spalling and rock burst

One of the major stability issues commonly found in hard rocks are stress-driven instabilities. This is often referred to as brittle failure, spalling or rock burst. In massive, brittle rocks, such as granite, failure around the tunnel occurs in the form of spalling and rock burst when the rock mass is subjected to high stresses. Such failure leads to formation of slabs and cracks behind the tunnel walls and may represent a safety issue (figure 32a and 31). In extreme cases, slabs buckle and flies off the wall violently, a condition known as rock burst or popping rock. In blocky or schistose rock, spalling and rock burst seldom occur (Goodman 1993).

Spalling is a result of stress relief. It does not necessarily make the excavation works considerably more difficult except that rock bolting of loose slabs may be required. Rock burst, on the other hand, may stop the progress of the work until the possibility of explosive detachment of rock is eliminated by application of rock support. In Norway, spalling and rock burst are a common phenomena when tunnelling underground and has caused many accidents. Thus, rock burst requires heavy rock support and reinforced shotcrete, rock bolts and wire mesh over the interior surface are commonly used for this purpose (Goodman 1993, Myrvang 2001).

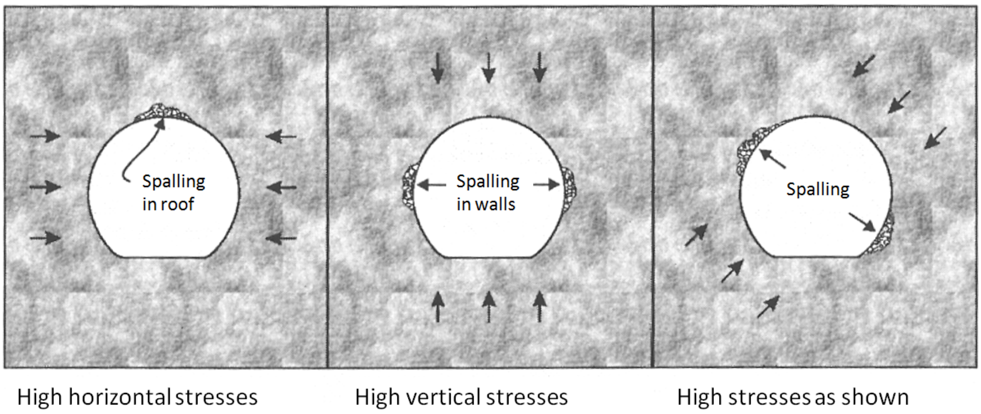
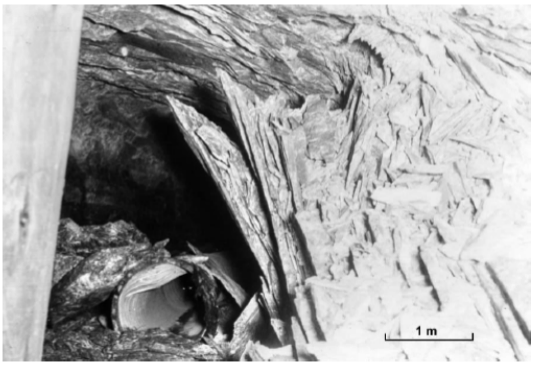


Figure 31: Spalling at different orientation of the major principal stress (modified from Myrvang (2001)).

(a) Brittle failure in the walls of a bored vertical shaft in a hard rock deep mine



(b) Result of a rockburst in a deep level gold mine in South Africa

Figure 32: Brittle failure in hard rock (Hoek & Marinos 2009).

When brittle failure occurs around an underground opening, this is a function of the geometry of the opening, the far-field stresses and the strength of the rock mass. A notch starts to propagate from the point of maximum tangential stress until it reaches the deepest point of damage in the direction of the minor principal stress (figure 31). In a homogenous and isotropic elastic rock mass, maximum and minimum tangential stresses at the boundary of a circular opening can be estimated by the well-known Kirsch equations:

$$\sigma_{\theta max} = 3\sigma_1 - \sigma_3 \tag{5}$$

$$\sigma_{\theta min} = 3\sigma_3 - \sigma_1 \tag{6}$$

If the maximum tangential stress $\sigma_{\theta max}$ exceeds the rocks compressive strength, failure will occur in the form of spalling or rock burst (Myrvang 2001). The failure zone that forms

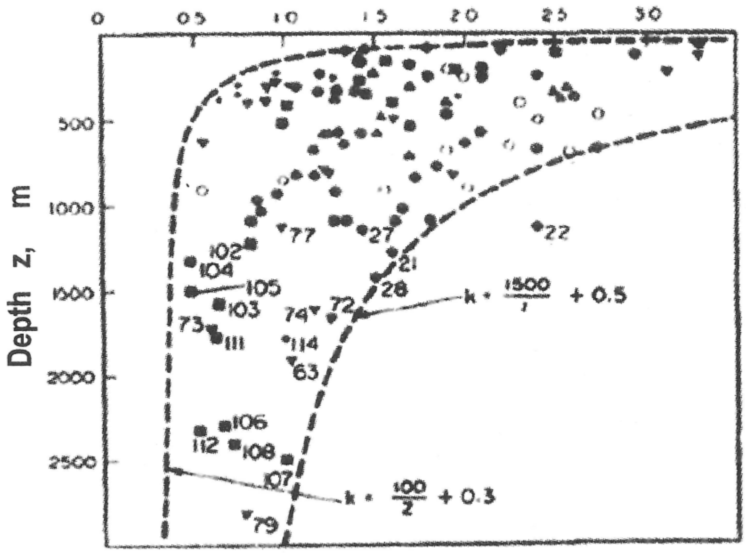


Figure 33: Measured horizontal stresses as a function of depth (Hoek & Brown 1980)

around the underground opening due to spalling and rock burst may take different shapes. In addition to geometry of the excavation and rock strength, this is also controlled by the factor K. K is the ratio of the maximum stress to the minimum stress (σ_1/σ_3) in the plane of the tunnel cross section. As K increases, so will the extent of the slabbing process (Martin, Kaiser & McCreath 1999). Estimating the in situ stress ratio K is a difficult task. While the vertical stress component may be estimated by the overlying rock masses ($\sigma_v = \rho g h$), the horizontal stress component (assuming no lateral strains) is given by:

$$\sigma_h = \frac{\nu}{1-\nu}\sigma_v = K\sigma_v \tag{7}$$

Where; ν is the Poisson's ratio. However, due to tectonic stresses, erosion of overlying rocks etc. it often turns out that σ_h is larger than the vertical stress. Figure 33 shows the plot of measured average in situ stress ratio K with depth. Hoek & Brown (1980) have suggested two envelopes for the compiled data:

$$\frac{100}{Z} + 0.3 < K < \frac{1500}{Z} + 0.5 \tag{8}$$

Where Z is the depth in meters and K is the ratio of average horizontal stress to vertical stress. With this equation it is possible, within broad limits, to estimate the variation of horizontal stress with depth (Ramamurthy et al. 2010). In Norway, and especially in the area of the Pre-Cambrian basement rocks found in the western parts of the country, large horizontal stresses exists, even at great depths. One example is the Lærdal tunnel where the rock overburden exceeds 1000 m and severe spalling was experienced in the tunnel roof indicating a horizontal stress component larger than the vertical stresses (Myrvang 2001).

Depth of failure R_f can be approximated by a linear relationship given by equation 9, where a is the tunnel radius, $\sigma_{\theta max}$ is the maximum tangential stress and σ_{ci} is the uniaxial compressive strength of intact rock (Martin et al. 1999).

$$\frac{R_f}{a} = 0.49(\pm 0.1) + 1.25\frac{\sigma_{\theta max}}{\sigma_c} \tag{9}$$

Failure will then initiate when $\sigma_{\theta max}/\sigma_c \approx 0.4 \pm 0.1$. This relation which is based on empirical data is illustrated in figure 34. Where the tunnel is D-shaped, an effective tunnel radius is used (figure 34, right).

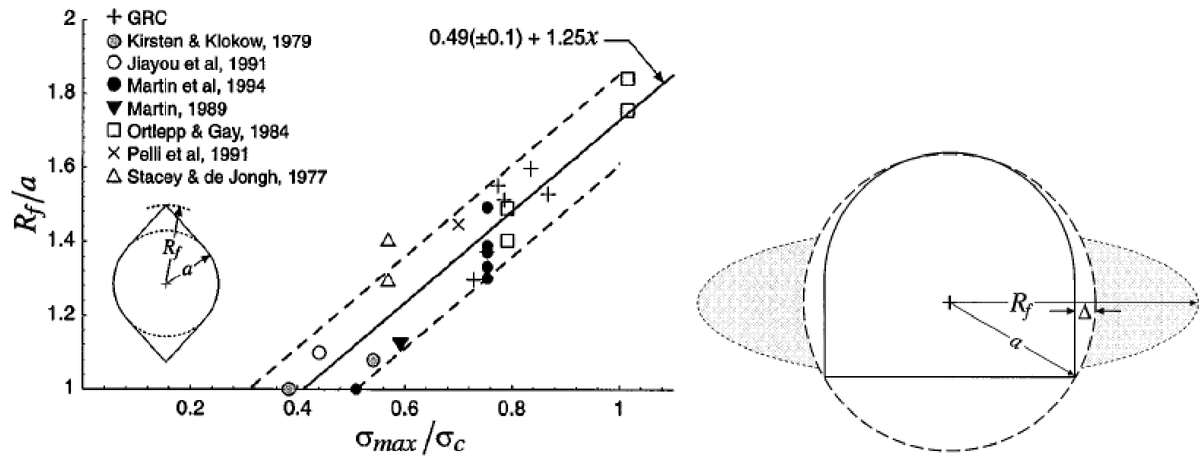


Figure 34: Relationship between depth of failure and maximum tangential stress at the boundary of the opening (Martin et al. 1999).

In figure 35, an empirical stability classification suggested by Martin et al. (1999) is found. The classification is a conversion of the Hoek-Brown stability classification developed for square tunnels in South Africa with low horizontal stresses. Based on the damage index D_i, the ratio of $\sigma_{\theta max}$ to the laboratory unconfined compressive strength σ_c, the extent of spalling and required rock support might be estimated (table 5).

The stability classification in figure 35 and table 5 indicates that for $D_i < 0.4$, the rock mass is basically elastic and no visible damage is recorded. This implies a maximum rock-mass strength near the opening of approximately $0.4\sigma_c$. This notion that the field strength of massive or moderately jointed rock is approximately one half of the laboratory strength has been reported by several researchers for a wide range of rock types (Martin et al. 1999).

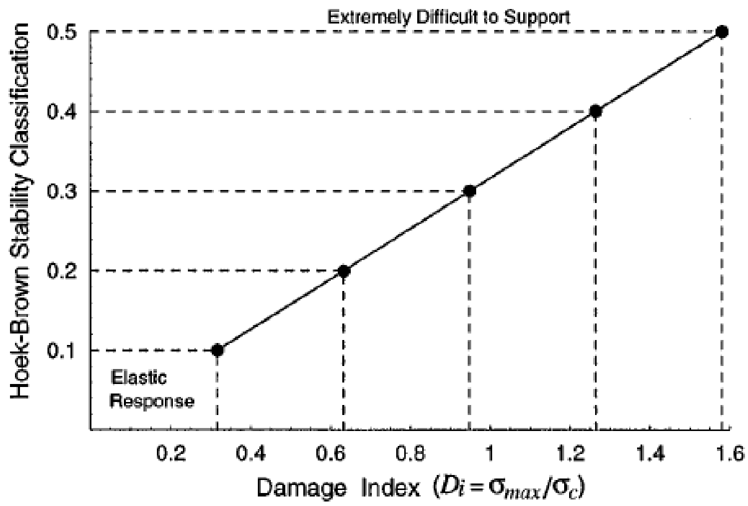


Figure 35: Stability classification according to the damage index D_i expressed as a function of the ratio of $\sigma_{\theta max}$ to σ_c (Martin et al. 1999).

Table 5: Stability classification according to damage index D_i (Martin et al. 1999).

Damage Index D_i	Classification
≤ 0.4	A stable opening with no need of support
0.6	Minor spalling can be observed, requiring moderate support
1.0	Severe spalling, requiring moderate support
1.3	Heavy support is required to stabilize the opening
1.6	Stability of the opening may be very difficult to achieve, extreme support required

5 Analysis of stability problems

5.1 Geometrical design parameters

During the HSR assessment, two parallel single track tubes were found to be most suitable for the tunnel system design for scenario D1 and D2. The shape of the tunnels will depend of the construction method. If the main tubes are to be excavated by conventional drill and blast, this will give typical D-shaped tunnels, while excavation with TBM will give perfectly circular shapes.

The size of the cross-sectional area is based on the assessment of several parameters like air resistance, energy consumption, comfort, pressure buildup, safety and costs. A free air volume of 65 m^2 was found to be necessary for a design speed of 330 km/h. This gives a total cross-sectional area of approximately 80 m^2 if drilled and blasted (figure 36). The distance between the two tunnels is recommended to be generally 15 m, but this is also a parameter dependent on engineering geological conditions (Sweco 2011).

Choice of construction method is dependent on economical circumstances, geology and length of tunnel. At the time being, conventional drill and blast is by far most the preferred tunnelling method in Norway. To simplify calculations, a circular tunnel with radius of 5 m is assumed in the analysis of ground water ingress, squeezing and spalling.

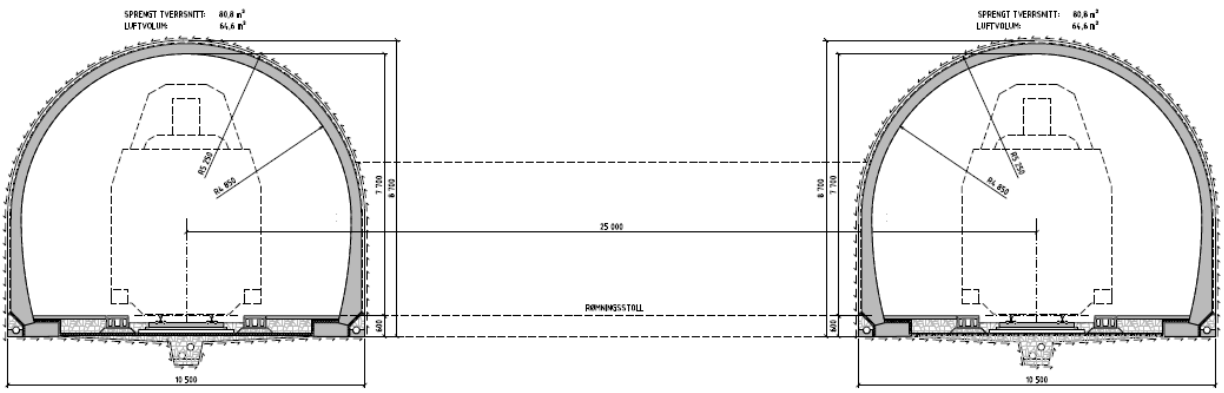


Figure 36: Proposed tunnel design for the HSR link for tunnels excavated by drill and blast method. Total cross-sectional area is 80.8 m^2 and distance from track to track 25 m (Sweco 2011).

5.2 Ground water ingress

In the analysis, equation 1 is used for predicting ground water leakage into the Hardangerjøkulen tunnel. Besides hydrostatic head h, the most important parameter in this analytical formulae is the hydraulic conductivity K of the rock mass. Leakage to a tunnel is proportional to the hydraulic conductivity. Hydraulic conductivity in m/s for some common rocks are found in figure 25.

Acceptable leakage rates into a tunnel is depending on type of tunnel (traffic tunnel, water supply tunnel etc.) and possible influence and damage to overlying buildings and environment. Kluver & Kveen (2004) suggests the following classification of inflow requirements for a traffic tunnel with a diameter of 8.5 m:

- Extremely strict requirements: 1-3 liter per minute per 100 m tunnel
- Strict requirements: 3-7 liter per minute per 100 m tunnel
- Medium strict requirements: 7-15 liter per minute per 100 m tunnel
- Moderate requirements: $>$ 15 liter per minute per 100 m tunnel

The range of hydraulic conductivities which are best found to represent the rock types found in the tunnel and thus used in the analysis are presented in table 6. As already discussed, hydraulic conductivity is highly dependent of degree of fracturing in the rock mass. Phyllite and massive basement rocks which are sparsely jointed are expected to be highly impermeable. Fractured rock mass on the other hand, like faults and weakness zones, are expected to have significantly higher conductivity.

Table 6: Range of hydraulic conductivities (K) for predicting water leakage to tunnel

Input variable	Maximum	Minimum
Hydraulic conductivity of phyllite (m/s)	10^{-10}	10^{-13}
Hydraulic conductivity of massive basement rocks (m/s)	10^{-9}	10^{-15}
Hydraulic conductivity of fractured basement rocks (m/s)	10^{-4}	10^{-9}

Figure 37 shows estimated leakage rates in liters per minute per 100 m tunnel along the profile with different hydraulic conductivities. The hydrostatic head, or tunnel depth below ground water surface, is assumed to equal the rock overburden.

For hydraulic conductivities $< 10^{-10}$, the inflow rate is insignificant ($<$ 1 l/min per 100 m) and the tunnel may be regarded as tight. A hydraulic conductivity of 10^{-9} gives inflows ranging between 3 to 6 l/min per 100 m, which in most cases would be regarded as low to extremely low leakage rates (Kluver & Kveen 2004). When the hydraulic conductivity is increased to 10^{-8}, the inflow rates are increased significantly exceeding 50 l/min per 100 m where hydrostatic heads are high. In a tunnel to be used for public transport, such leakage rates will require sealing by for example injection grouting.

Estimated leakage rates for phyllite, massive basement rocks and fractured rock are found in table 7, 8 and 9. The portal areas are not considered. The estimation aims at predicting a "worst case scenario", a "best case scenario" and a "most likely scenario" in the different geological conditions.

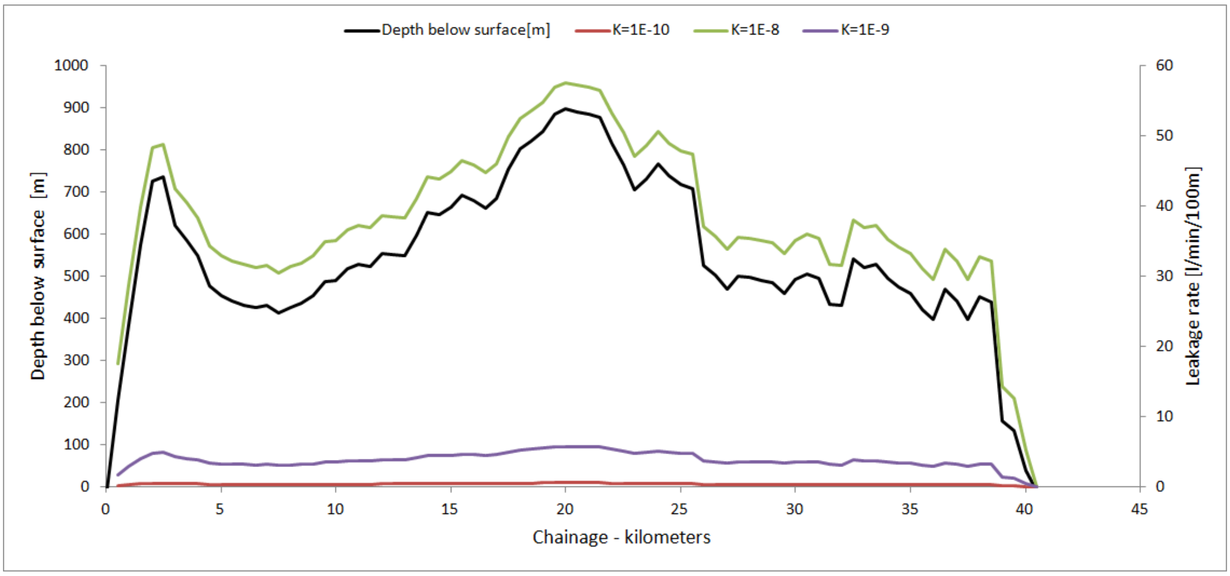


Figure 37: Leakage rate in l/min per 100m along the tunnel profile as a function of hydraulic conductivity K and depth below surface.

In phyllite the estimated leakage rates are very low (< 1 l/min per 100 m) for all three scenarios. In such conditions no extra sealing like injection grouting should be required. Estimated inflow rates in massive basement rocks shows similar result as for phyllite, and the expected leakage into the tunnel is less than 1 l/min per 100 m tunnel. Maximum leakage is estimated to approximately 6 l/min per 100 m tunnel. This can be where the tunnel is situated 900 m below ground water table and a few joints exists in the rock mass. Depending on inflow requirements, such conditions might require some sealing by injection grouting.

A different situation exists for faults and weakness zones. A worst case scenario is a weakness zone at great depth (900 m) with open joints leading water. With a hydraulic conductivity of 10^{-4} and hydrostatic head of 900 m, the estimated inflow rate is 5770 l/min in a 1 meter wide zone. Such extreme leakages requires a exceptionally effort for sealing and supporting in order to prevent construction difficulties, face instability and cave-ins. The "most likely scenario" predicts a inflow rate of 38 l/m per meter tunnel, a situation which also will require great effort in order to establish a tight tunnel.

The amount of leaking ground water in weakness zones and fractured rock is highly dependent on the width of the zone and characteristics of the crushed rock. If clay exists in the weakness zones, this might have a sealing effect and reduce the inflow rate. Further and thorough ground investigations is required in order to get reliable characteristics of such zones.

Table 7: Estimated leakage rates in phyllite

Input variable	Minimum	Maximum	Mean/expected
Hydraulic conductivity (K) of phyllite (m/s)	10^{-13}	10^{-10}	10^{-11}
Hydrostatic head (h) (m)	200	690	400
Leakage (Q) in l/min/100m tunnel	0.00	0.46	0.03

Table 8: Estimated leakage rates in massive basement rocks

Input variable	Minimum	Maximum	Mean/expected
Hydraulic conductivity (K) of massive basement rocks (m/s)	10^{-15}	10^{-9}	10^{-11}
Hydrostatic head (h) (m)	200	900	550
Leakage (Q) in l/min/100m tunnel	0.00	5.8	0.04

Table 9: Estimated leakage rates in faults, weakness zones and fractured basement rocks

Input variable	Minimum	Maximum	Mean/expected
Hydraulic conductivity (K) of fractured rocks/faults/weakness zones (m/s)	10^{-9}	10^{-4}	10^{-6}
Hydrostatic head (h) (m)	200	900	550
Leakage (Q) in l/min/m tunnel	0.02	5770	38

5.3 Tunnel squeezing in phyllite

A combination of expected weak phyllite and high overburden might lead to tunnel squeezing in the Hardangerjøkulen tunnel. Approximately the first 1.8 km of the tunnel will be located in phyllite (figure 38). Excluding the portal areas, the tunnel alignment in this section has rock overburdens ranging from 200 to 690 m.

Tunnel squeezing is estimated from equation 3 and 4. The estimation aims at predicting a "worst case scenario", "best case scenario" and "most likely scenario" by using maximum, minimum and mean input variables. Input variables that are assumed to be representative for the phyllitic rock mass in the Hardangerjøkulen tunnel are found in table 10. Intact rock strength σ_{ci} is obtained from measurements of rock samples from Aurland (Myrvang 2001).

Estimated minimum, maximum and mean strain ε as a function of tunnel depth is found in table 11. Recalling figure 30, Hoek & Marinos (2000) suggests the following relationship between tunnel strain and associated degree of difficulty:

- Class A (strain less than 1%); Few support problems

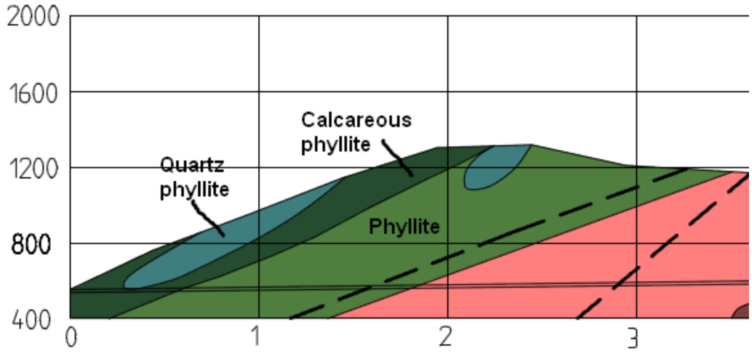


Figure 38: Profile of Hardangerjøkulen where phyllite exists.

Table 10: Estimated input variables for squeezing analysis.

Input variable	Unit	Minimum	Maximum	Mean
Overburden height (h)	m	300	700	500
Specific weight (γ)	MN/m^3	-	-	0.027
Overburden pressure ($\sigma_V = \gamma h$)	MPa	8.1	18.9	13.5
Intact rock strength (σ_{ci})	MPa	30	65	47.5

- Class B (strain between 1 and 2.5%); Minor squeezing problems
- Class C (strain between 2.5 and 5%); Severe squeezing problems
- Class D (strain between 5 and 10%); Very severe squeezing problems
- Class E (strain > 10%); Extreme squeezing problems

In the case of maximum rock strength, strains < 1% are estimated for rock overburdens ranging up to 700 m (ε_{min}). Strain < 1% is not associated with considerable stability problems and ordinary tunnel support methods may be used. Tunnel support recommendations based on rock mass classification will then provide an adequate basis for design.

For the "most likely scenario" (ε_{mean}), the maximum estimated tunnel strain is 2.4 %, and for rock overburdens exceeding 500 m minor squeezing problems is expected. Generally, rock bolts and shotcrete are used to deal with such conditions, sometimes supplemented by steel sets of lattice girders or armed shotcrete ribs (Hoek & Marinos 2000).

In the case of very weak rock masses and high overburden, estimated tunnel strain are close to 10% (ε_{max}) which indicate very severe squeezing problems and face stability problems. Forepoling and face reinforcement are usually applied in such conditions, and yielding support may be required in extreme cases (Hoek & Marinos 2000).

Table 11: Estimated strains as a function of rock overburden for the input variables given in table 10

		Rock overburden [m]		
		300	**500**	**700**
Tunnel strain ε in percent	ε_{min}	0.17	0.48	0.94
	ε_{mean}	0.44	1.22	2.4
	ε_{max}	1.75	4.85	9.53

5.4 Spalling and rock burst

High overburdens, high horizontal stresses and expected brittle rock masses might lead to spalling and even rock burst in the Hardangerjøkulen tunnel. Experiences from nearby projects do also confirm this presumption. In order to design adequate rock support and excavation method, a reliable estimate of the behavior of the rock mass is required.

Estimation of spalling in the Hardangerjøkulen tunnel is based on the following assumptions:

- Uniaxial compressive strength of 150 MPa
- A D-shaped tunnel with effective radius a equal 6 m.
- Principal stresses oriented horizontal/vertical
- The vertical stress is given by the weight of the overburden; i.e. $\sigma_v = \rho g h = \gamma h$, where $\gamma = 0.027$ MN/m^3.
- The maximum tangential stress at the tunnel wall is given by the Kirsch equation (Eq. 5)
- K, the ratio of the horizontal stresses to the vertical one is decreasing towards depth (Eq. 10, figure 39).

High horizontal stresses are characterizing the region. From measurements at several locations it is known that at 400 m depth, the maximum horizontal stress σ_H=25 MPa and K=2.3. Empirical, it is also known that the horizontal stress decreases very little with lower rock overburdens, and often high horizontal stresses are found close to the surface. Myrvang suggests that the horizontal stress in this particular region may be calculated towards depth by the following equation (Myrvang, personal communication, May, 2012):

$$\sigma_H = 22MPa + \frac{\nu}{1-\nu}\sigma_v \tag{10}$$

Where ν is Poisson's ratio and σ_v is the gravitational vertical stress component. With ν = 0.2, K will decrease towards depth as shown in figure 39.

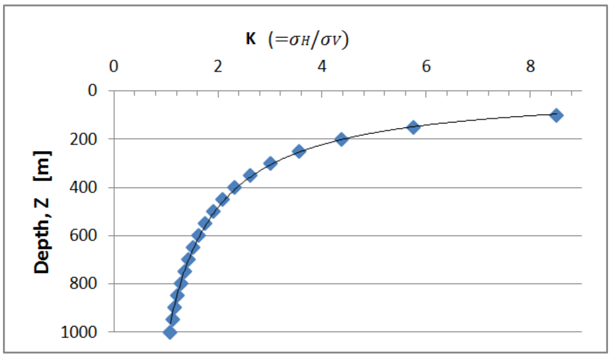


Figure 39: K, the ratio of horizontal stress to the vertical one as a function of depth Z.

Rock mass behavior with respect to brittle failure is assessed by the damage index D_i (figure 35 and table 5) and the ratio of uniaxial compressive strength σ_c to the major principal stress σ_1 (table 12). Since $K > 1$ for rock overburdens < 900 m, the major principal stress will in this case equal σ_H. Depth of failure R_f is estimated from equation 9 (figure 34). Where $R_f >$ tunnel radius, failure at the tunnel profile has initiated. The analysis considers rock overburdens (Z) from 200 to 900 m and calculated results are presented by the graphs in figure 40.

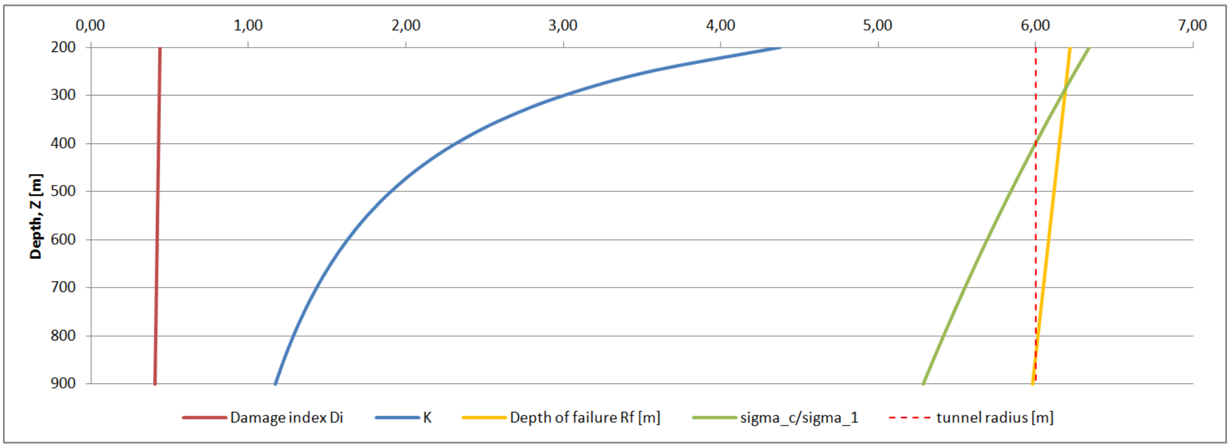


Figure 40: Estimated damage index D_i, depth of failure R_f and σ_c/σ_1 as a function of K and depth below surface Z.

The damage index is found to be fairly constant for the analyzed depths, ranging from 0.44 at 200 m to 0.41 at 900 m. According to Martin et al. (1999) and Grimstad (2006), minor spalling can then be observed, requiring moderate rock support. Even though stresses are high, they are most likely not threatening the stability of the excavation.

The ratio of σ_c to σ_1 is found to vary between 6.3 to 5.3, decreasing with depth. As for the estimated damage index, this only indicates minor spalling.

Considering R_f, maximum depth of failure is estimated at 200 m depth where $R_f = 6.2$

m. With a tunnel radius of 6 m, this means that the yielded zone will reach 20 cm outside the tunnel periphery. R_f is found to decrease with depth, and at 900 m depth no spalling is estimated.

Table 12: Relationship between σ_c/σ_1 and $\sigma_{\theta_{max}}/\sigma_c$ and associated stress condition (Grimstad 2006). According to the analytical results, Hardangerjøkulen tunnel categorizes into class C.

Class	Hard, competent rocks, stress induced problems	σ_c/σ_1	$\sigma_{\theta max}/\sigma_c$
A	Low in-situ stresses, open joints. Rock fall outs due to low stresses. Usually near surface.	> 200	< 0.01
B	Medium high in-situ stresses, favorable stress condition	200 - 10	0.01 - 0.3
C	High stresses. Usually favorable stress condition, but some spalling might occur	10 - 5	0.3 - 0.4
D	Moderate spalling after > hour in massive rocks	5 - 3	0.5 - 0.65
E	Spalling and rock burst after a few minutes in massive rocks	3 - 2	0.65 - 1
F	Intense spalling and immediate bursting in massive rocks	< 2	> 1

Due to the risk of erroneous assumptions in the analysis, the possibility of intensive spalling and rock burst can not be excluded in the tunnel, especially not when such conditions are known encountered at nearby projects. An adequate rock support strategy must be implemented, using experiences from other tunnel projects. In Norway, many experiences have been gained when tunnelling in unfavorable stress conditions. One example is the already mentioned Lærdalstunnel, where severe spalling and rock burst were encountered. At most, a depth of spalling of 3.8 m from the the tunnel periphery was observed. In Norway, rock bolts and shotcrete are usually applied to deal with such conditions. In the Lærdalstunnel, the following procedure were established to achieve sufficient stability (Palmstrøm, Nilsen, Pedersen & Grundt 2003):

- Low or moderate spalling: End anchored bolts after each blasting round followed by reinforced shotcrete applied once a day.
- Intensive spalling: Reinforced shotcrete and end anchored bolts for each blasting round.

When intensive spalling were encountered, shotcrete were usually applied in two layers; the first layer were applied before installing bolts. After next blasting round, a second layer of shotcrete was applied. 30-40 bolts were used per blasting round (Grimstad & Kvåle 1999, Grimstad 2006, Palmstrøm et al. 2003). A similar rock support concept might be relevant for the Hardangerjøkulen tunnel if stress conditions are found to be unfavorable.

6 Numerical modelling of brittle failure

6.1 Introduction

Numerical simulations may be utilized for modelling rock masses and excavations. This method of calculation is used for quantitative assessment, and may give a better understanding of mechanisms, a verification of "rule of thumbs" and an extension of measurement results. In rock engineering, the main application is stability analysis and rock support design.It is widely used to assess stability of underground caverns, tunnels, rock slopes and dam foundations. It may also be used to assess water seepage to tunnels and through dam foundations.

The finite element method (FEM) is a widely used numerical method. It is used in all engineering disciplines for solving problems represented by a set of discrete partial differential equations. In rock engineering it can be used in solving most types of problems, and is particulary applicable to rock mass with complicated material properties. In FEM, the rock mass is considered to be a continuous medium, modelled by a set of appropriate finite elements interconnected at point called nodes. Elements may have physical properties such as density, Yongs's modulus, shear modulus and Poisson's ratio. The elements make up a mesh found inside the domain to be modelled which is limited by a defined external boundary. A complex set of equations express the forces and displacements of the nodes inside the domain, influenced by the excavation (Myrvang 2001).

A number of commercial codes exists, amongst them Phase^2 which was used for this work. Phase^2 is a 2D finite element analysis program for excavations and slopes, developed by RocScience Inc. It is an elasto-plastic stress analysis tool, covering most commonly used material models for rock masses, such as the Mohr-Coulomb and Hoek-Brown failure criterions. The program is commonly used for the calculation of stresses and displacements around underground and surface excavations in rock. A wide variety of rock support types can also be modeled, including rock bolts, shotcrete and concrete lining (Rocscience 2012).

6.2 Simulating brittle failure in hard rock

In this work numerical modelling has been used for predicting the extent of spalling, which is expected to be one of the main concerns regarding stability of the Hardangerjøkulen tunnel. Characteristics of brittle failure and spalling are discussed in chapter 4.2.5. Unlike ductile materials in which shear slip surfaces can form while continuity of material is maintained, brittle failure deals with materials for which continuity must first be disrupted before kinematically feasible failure mechanisms can form.

Different approaches may be used for simulating brittle failure in hard rock masses. In Phase^2 the rock mass may be treated as plastic or elastic. If treated as plastic, the material can yield and exhibit non linear stress-strain behavior. Then, yielded elements, volumetric strains and maximum shear strains may be used as indicators of failure. In an elastic analysis, the stability of an excavation may be evaluated through a strength factor (SF). SF is calculated by dividing the rock strength (based on the failure criterion) by the induced stress at every point in the mesh. Since an elastic model cannot "fail", an SF value less than 1 is then considered to represent failure within the material. Both elastic and plastic models may be used to estimate depth, shape and extent of fallout caused by brittle failure (Edelbro 2009).

Hoek et al. (1995) suggests that an elastic-brittle-plastic material model should best represent massive brittle rock. In a plastic material model, the failed rock is then assigned very low residual strength values in order to simulate the elastic-brittle-plastic failure process which results in the rock spalling and falling away from the roof of the excavation (figure 41).

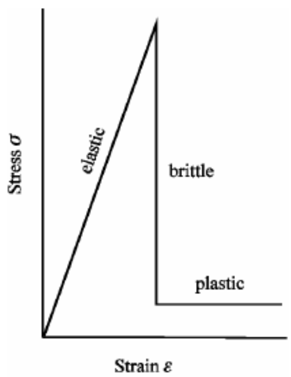


Figure 41: Elastic-brittle-plastic behavior assumed for massive brittle rock (Hoek, Kaiser & Bawden 1995).

Martin et al. (1999) showed that the application of certain strength parameters in the Hoek-Brown criterion could be used to estimate the location and depth of brittle failure around a tunnel in massive to moderately fractured rock when using a value of m_b close or equal to zero and $s = 0.112$ in an elastic analysis. This represents brittle failure initiation at $0.3\sigma_{ci}$. m_b and s are material constants used in the Hoek-Brown criterion dependent upon rock mass characteristics. Shape and extent of the brittle failure is more difficult to predict with such an elastic analysis.

6.3 Input parameters

Input parameters used for the analysis are found in table 13. The rock mass has been approximated as a homogeneous, isotropic continuum, since the rock mass is assumed to be sparsely jointed and the fallout is not structurally controlled. The stress conditions can be represented by a plane strain assumption, since the length of the tunnels are much longer than the cross-sectional dimensions. A gravitational vertical stress component σ_v is assumed. K, the ratio of the horizontal stress σ_H to σ_v is found from equation 10.

Table 13: Input parameters for numerical analysis.

Input data	Symbol	Unit	Value
Unit weigth	γ	MN/m^3	0.027
Geological strength index	GSI		80
Material constant	m_i		32
Disturbance factor	D		0
Intact rock strength	σ_{ci}	MPa	150
Young's modulus	E_i	Gpa	40
Poisson's ratio	ν		0.2
Dilation angle	$^\circ$	degree	0

In the analysis, the Hoek-Brown failure criterion is used for determination of failure at the tunnel periphery. This is an empirical failure criterion widely used in rock engineering, and was originally introduced in order to provide input data for analysis required for the design of underground excavations in hard rock. The generalized Hoek-Brown criterion to jointed rock masses is expressed as (Hoek, Carranza-Torres & Corkum 2002):

$$\sigma_1 = \sigma_3 + \sigma_{ci}(m_b \frac{\sigma_3}{\sigma_{ci}} + s)^a \tag{11}$$

Where; σ_1 and σ_3 are the major and and minor effective principal stresses at failure, σ_{ci} is the uniaxial compressive strength of the intact rock material and m_b, s and a are constants for the rock mass. m_b, a reduced value of the material constant m_i, and s are constants dependent upon the geological strength index (GSI) and the disturbance factor D, while a depends upon D only. D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. The rock mass is assumed to undergo minimal disturbance during construction and $D = 0$ is therefore assumed. GSI is based on the description of two factors; rock structure and block surface conditions. Due to expected good rock conditions with massive rock and few joints, GSI = 80 is assumed.

In elastic analysis in Phase^2, a dilation parameter is included as one of the strength factors. Dilatancy is a measure of the volume increase in a material during shearing. A parametric study done by Edelbro (2009) concluded with that the choice of dilation angle is not significantly affecting the zone of yielded elements, and that shear bands becomes more distinct with low dilation angles. It is reasonable to assume that for hard rock masses with tight interlocks at large depths, the possibility for dilation is small. A dilation angle of 0 is therefore assumed.

The rock mechanical parameters σ_{ci}, Young's modulus E and Poisson's ratio ν are based on measurements of Norwegian granites (Myrvang 2001). Peak strength parameters m_b and s, are derived by using the Hoek-Brown failure criteria in the RocScience program RocLab. The material constant m_i is set to be 32, which is the suggested value for granite (Hoek 2009).

6.4 Material models

Two different approaches are used for analyzing brittle failure in the Hardangerjøkulen tunnel. This includes one plastic and one elastic material model. Different indicators were used for determination of brittle failure in the two material models.

Elastic-brittle-plastic model: Hoek et al. (1995) suggested that elastic-brittle-plastic analysis are adequate for most practical purposes. This model is supposed to take into account for the stress redistribution as failure progresses (Martin et al. 1999). In order to simulate the elastic-brittle-plastic failure process typical for overstressed granite, the Hoek-Brown parameters are assigned very low residual values; $m_{br} = 1$ and $s_r = 0.01$ (unit less). In this way the elastic-brittle-plastic failure process is simulated which results in the rock spalling and falling away from the roof of the excavation. s is an empirical rock-mass cohesive strength constant while m_b is a frictional strength constant, both estimated by RocLab. If these peak strengths are exceeded, residual strength values (m_{br} and s_r) are applied. Yielded elements and maximum shear strain are used as indicators of failure as suggested by Edelbro (2009).

Elastic model with Hoek-Brown brittle parameters: The concept of using Hoek-Brown brittle parameters to define the damaged region around an underground opening was originally developed for massive unfractured granite. Later it has been found that the Hoek-Brown parameters are applicable to a wide range of rock mass types (Martin et al. 1999). In this approach an elastic model with the Hoek-Brown brittle parameters m_b=0 and s=0.112 is used for estimating depth of failure. Since elastic material can not "fail", an SF value less than 1 is considered to represent failure within the material. Martin et al. (1999) found this approach to predict the maximum depth of failure more accurate than the elastic-brittle-plastic analysis. They also found this approach to give a good estimate of the extent of failure.

6.5 Results from numerical analysis

For assessing stability of the tunnel the presented material models and input parameters are used for modelling stresses, strength factors, strains and yielded elements in Phase^2. The modelled excavation have dimensions as discussed in chapter 5.1, and both main tubes are included (figure 42). Tunnel depths assessed are 300, 500, 700 and 900 m. The results is evaluated for estimating extent, depth and location of failed zones, using different indicators as discussed.

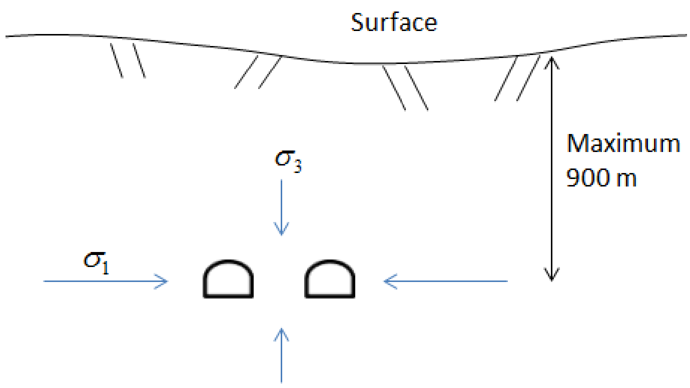


Figure 42: Schematic cross-section of the analyzed main tubes

The extent of the modelled domain is defined by an expansion factor, which determines how far the generated external boundary will be projected relative to the excavation dimensions. The extent of the modelled domain was in the analysis defined by an expansion factor of 5 times the tunnel diameter in order to eliminate any boundary effects. The domain is discretized with a finite element mesh of six-noded, triangular elements. A mesh gradation factor of 0.1 is used.

When modelling in Phase^2, the result is a post-processing module used for data visualization and interpretation. Data contours can be viewed (e.g. stress, strain, strength factor) and results can be displayed on the model. An example is shown in figure 43 where the tunnel is simulated with 900 m rock cover. Colored data contours represents the major principal stress, and stress trajectories are displayed showing the orientation of the principal stress at each point. What is seen is that the 2 excavations causes an alternation to the original stress field. Those stresses held by the removed material are transferred to the remaining rock mass, and stress concentrations appear around the tunnel periphery. Besides the magnitude and orientation of principal stresses, stress concentrations are dependent of the shape and geometry of the excavation; the smaller the radius of curvature, the larger the stress concentration. At corners, stress concentrations might get very high.

Figure 43 shows that the highest stresses are not found at the excavation boundary, but a bit outside the tunnel peripheries. This is probably due to overloading of the rock mass close and at the excavation boundary, causing the rock mass to fracture and yield. Thus the

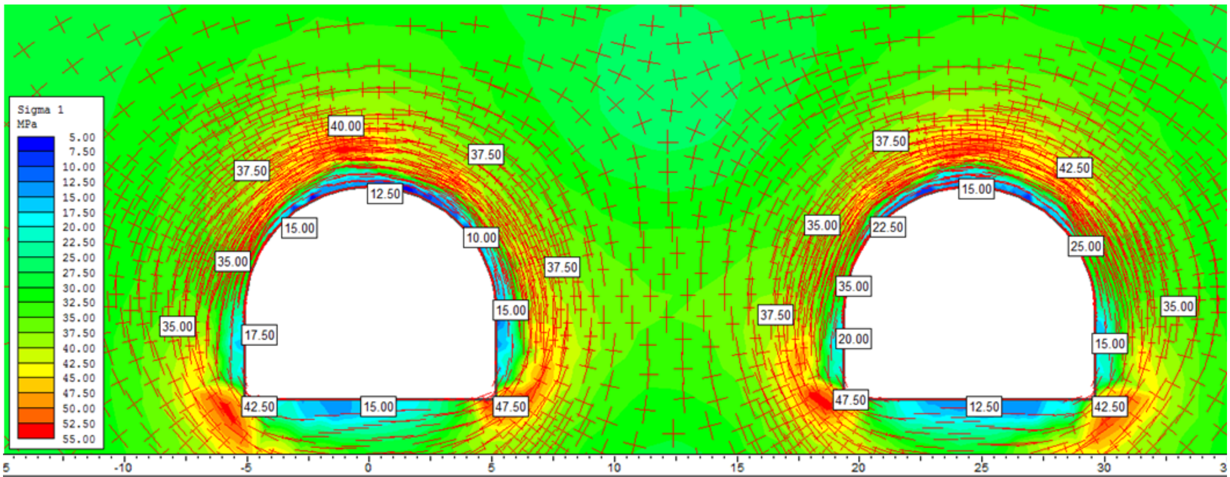


Figure 43: Stress trajectories and mean stress distribution at a chainage with 900 m rock cover.

rock mass here is not capable of retaining those high stresses, and the stress concentrations are moved from the excavation boundary and outwards.

6.5.1 Elastic-brittle-plastic analysis

Figure 44 shows results from the numerical analysis using an elastic-brittle-plastic material model with the low residual values $m_{br} = 1$ and $s_r = 0.01$. Failure of the rock mass is indicated by elements yielded in shear and tension.

What may be classified as minor spalling is found at all analyzed tunnel depths. At rock overburdens between 300 and 700 m, yielded elements are mainly found concentrated in the tunnel roof and floor. This is probably due to the high horizontal stresses found at these depths. At 900 m, where $K = 1.17$, a yielded zone has developed around almost the entire tunnel periphery. The yielded zones are mainly found as elongated areas following the tunnel boundary, and not as the typical V-notch shaped zones so often associated with spalling and rock burst.

In the process of spalling, parallel fractures will first initiate, and fallout can occur once fractures connect to the excavation boundary. Often, slabs fail at he outer ends through shear propagation or in the middle through tension (buckling). Thus, shear failure is likely to occur at the final process of the formation of a fallout. When one slab has fallen out, new slabs are formed. Since shear occurs in the outer ends of the slabs, the final shape of the fallout may be captured by shear bands or yielded elements failed in shear (Edelbro 2009). Maximum shear strain are displayed for the analyzed tunnel depths in figure 45. Well developed shear bands can be seen in the tunnel roofs and at 900 m depth, shear bands have developed around the entire tunnel periphery. Fallout caused by shear is assumed

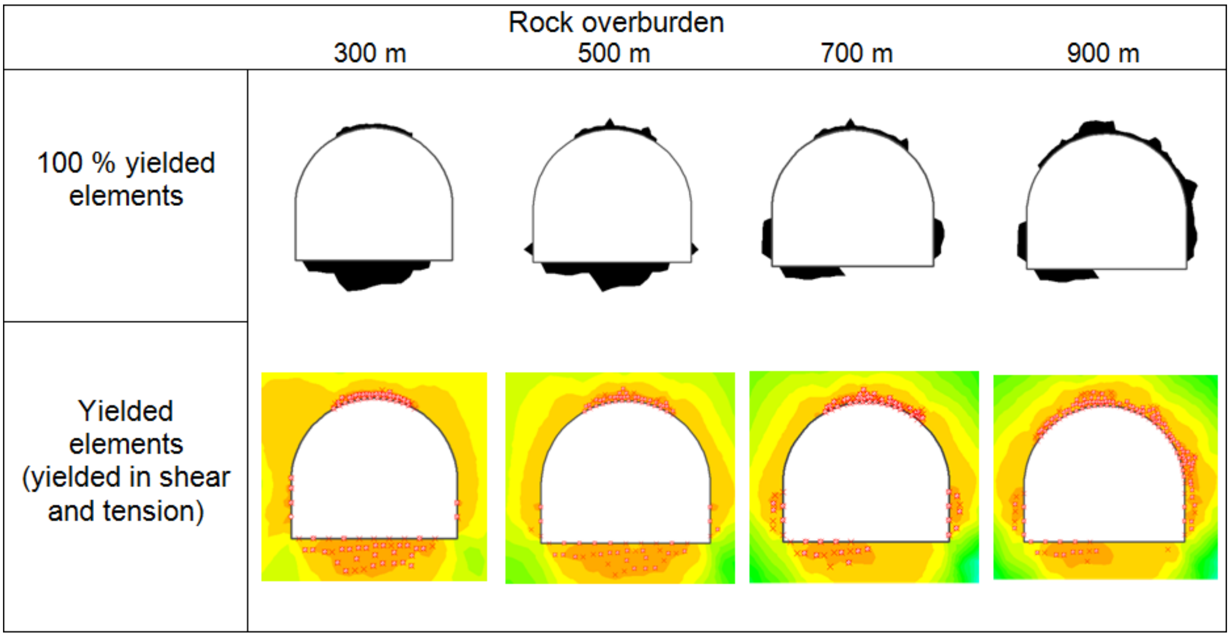


Figure 44: Yielded elements (red dots) at different tunnel depths.

to occur when two shear bands cross or form a coherent arch (Edelbro 2009). The shear bands in figure 45 forms V-notched arches, which is very typical for brittle failure. The area within the arches are likely to fall out causing instability to the excavations. Especially at 700 m depth, depth of failure seems to be quite severe. At 900 m, rock slabs which might fall out are also found at the tunnel walls.

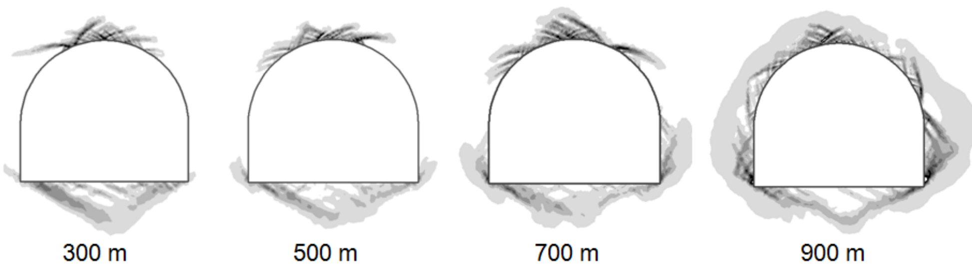


Figure 45: Maximum shear strain (0-0.004 %) at different tunnel depths.

6.5.2 Elastic analysis with Hoek-Brown brittle parameters

For estimating depth of spallin, an elastic analysis with Hoek-Brown brittle parameters is carried out using strength factor (SF) as indicator of failure. Figure 46 shows an example of how depth of failure may be estimated from the SF distribution; approximate depth of failure is the distance from the excavation boundary to where SF exceeds 1.

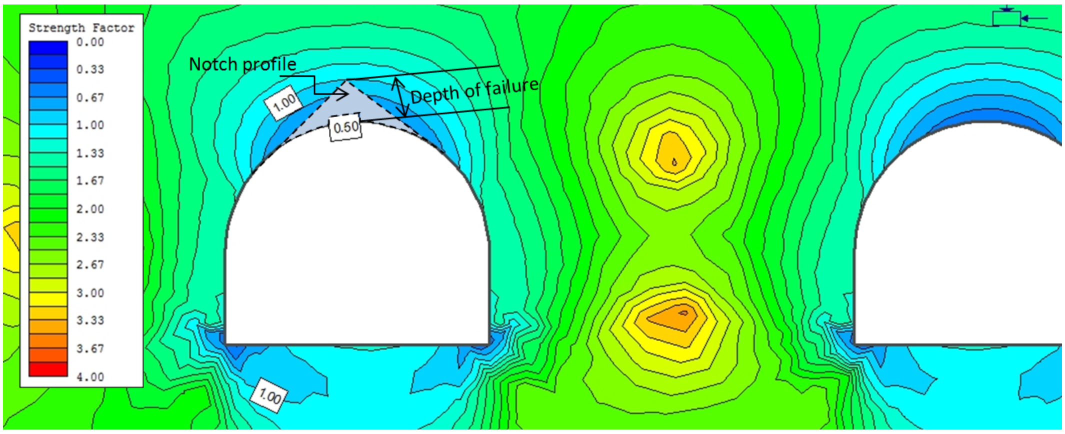


Figure 46: The depth of brittle failure may be interpreted from the analysis of strength factor distribution

The results from the elastic analysis are presented in figure 47. A small zone where SF < 1 can be seen in the tunnel roofs and corners. With rock covers of 300, 500 and 700 m, the estimated depth of failure in the tunnel roof is approximately 20 cm outside the excavation boundary. At 900 m tunnel depth, zones with SF < 1 are seen as spikes in the tunnel roof. Maximum depth of the spikes are found to be approximately 60 cm.

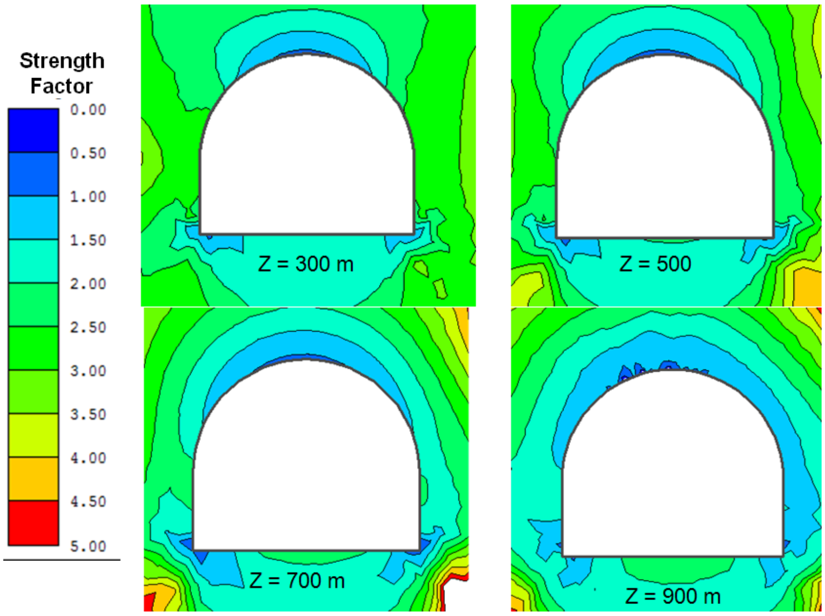


Figure 47: Calculated values of strength factor in Phase^2 when using an elastic model with Hoek-Brown brittle parameters.

6.5.3 Circular tunnels

Since the stress concentrations initiated around an underground excavation also are a function of size and geometry of the excavated area, circular tunnels are also analyzed. As opposed to a D & B tunnelling, TBM driven tunnels will have a perfectly circular shape. The numerical modelling is performed with two circular tunnels with 5 m radius and a 25 m distance from track to track as discussed in chapter 5.1. Tunnel depth analyzed is 900 m.

Results from the analysis of circular tunnels are presented in figure 48 and 49. A circular shape of the tunnels are found to eliminate the stress concentrations at any sharp corners, but are not likely to eliminate brittle failure. With the elastic-brittle-plastic material model, the analysis shows that yielded elements are found concentrated around the tunnel periphery very similar to the analyzed D-shaped tunnels (figure 49). Well developed shearbands are forming zones prone to fall out around the entire excavation boundary (maximum shear strain, figure 48). Depth of failure is estimated to be approximately 25 cm, using the elastic model with Hoek-Brown brittle parameters (strength factor, figure 48).

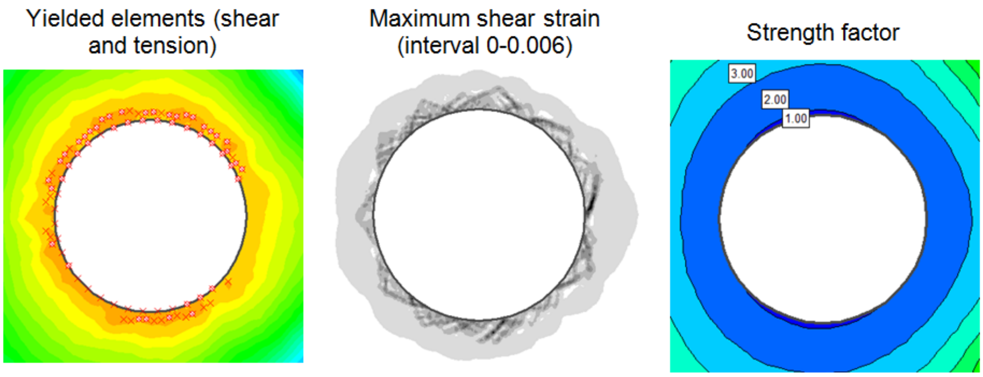


Figure 48: Yielded elements, maximum shear and strength factor of circular tunnels at 900 m tunnel depth.

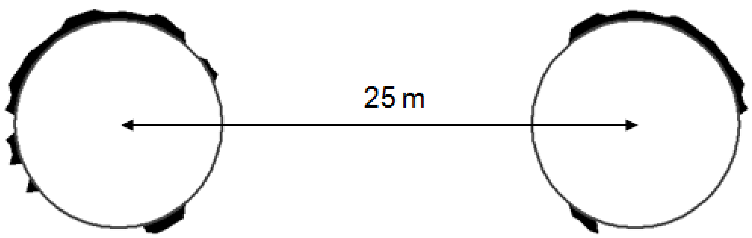


Figure 49: 100 % yielded elements at 900 m tunnel depth

6.6 Effect of rock support

To see whether conventional rock support is able to handle the stress-situation and induced rock mass failure in the Hardangerjøkulen tunnel, rock support is included in the analysis. When dealing with spalling rock, common practice in Norway is to use a combination of rock bolts and shotcrete (Nilsen & Broch 2009). In conditions with high and/or anisotropic in-situ stresses, end-anchored bolts with large triangular face plates supplemented with reinforced shotcrete is recommended (Sam 1994). The bolts should not be preloaded as they will be loaded when the rock mass deforms due to spalling.

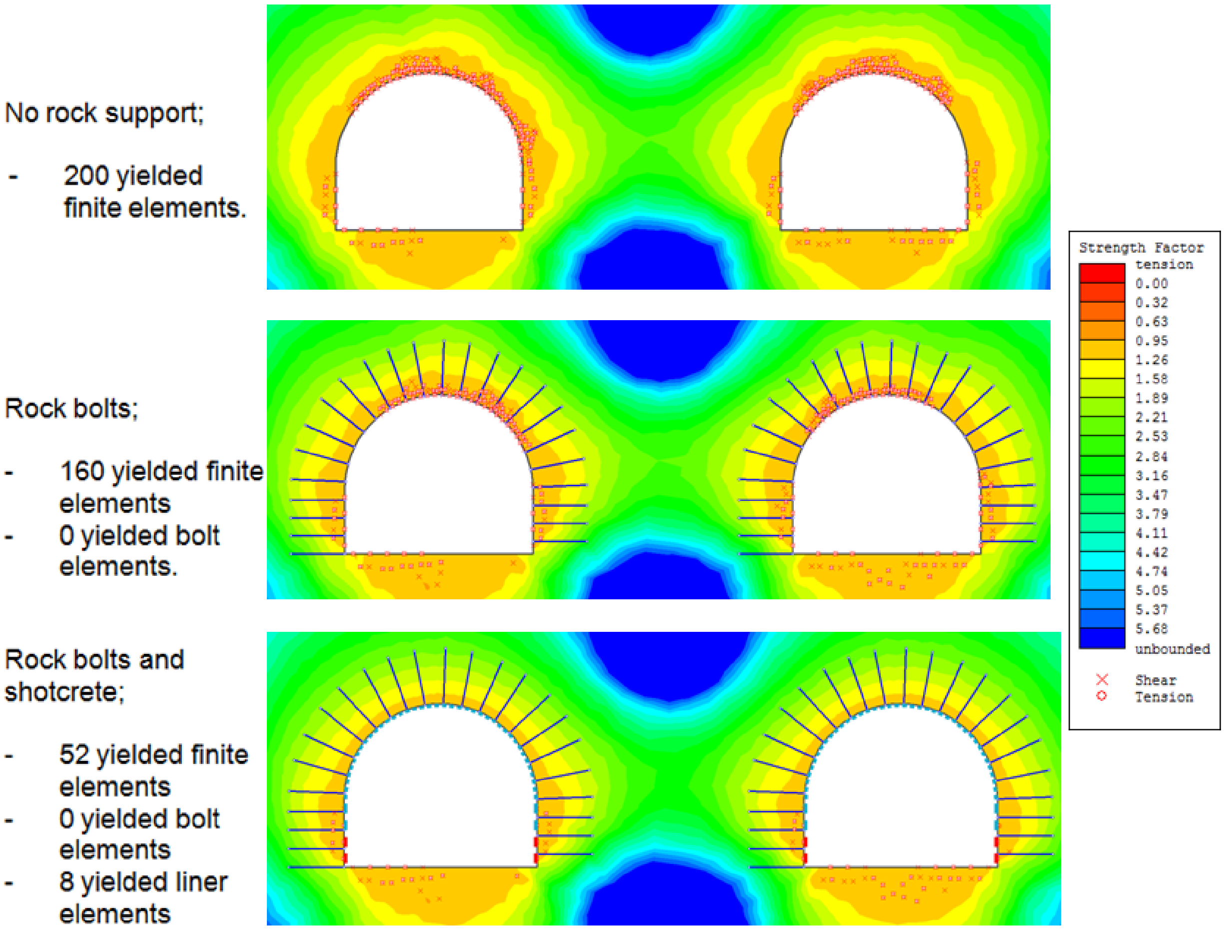


Figure 50: Calculated yielded elements with different amount of rock support at 900 m depth.

In the rock support analysis the elastic-brittle plastic material model is used with the material parameters found in table 13. A rock overburden of 900 m is used for this purpose. The simulation is done with 1) no rock support, 2) with bolts and 3) with bolts and shotcrete. Support elements are assumed to be installed directly after excavation.

Bolts used is end anchored with a bolt diameter of 20 mm and maximum load capacity of

10 tonnes. The pre-tensional force is set to be 0. The bolts are installed as shown in figure 50 with 1 m spacing and length of 3 m. This length assures that the bolts reaches well behind the failed zone. Applied shotcrete has a thickness of 250 mm, compressive strength of 35 MPa and tensile strength 5 MPa.

The result from the numerical analysis including rock support is presented in figure 50. When unsupported, total number of yielded elements is significant (200), and especially the hanging wall seems to be very unstable. Installation of rock bolts reduces the amount of yielded elements to 160 and seems to have a positive effect on the stability of the hanging wall. However, the amount of yielding is still significant. Shotcrete is found to be essential for supporting the rock mass, as once the shotcrete layer is applied yielded elements are reduced 52. After shotcrete is added, no yielded elements are found in the hanging wall, only at corners and floor. Thus, the excavation might be considered stable since no rock fallouts are likely to occur. It can also be seen that once the shotcrete is applied the strength factor around the excavation is redistributed towards the positive . Some additional support might be required at the corners since the liner have yielded here. There are no yielded bolts.

7 Discussion on the stability assessment

Evaluating and predicting possible geological challenges in the Hardangerjøkulen tunnel have been the main scope of this work. This includes analytical and numerical analysis of spalling and rock burst, tunnel squeezing and water ingress. Estimating potential geological challenges is of major importance since such issues will influence safety and stability of the excavation as well as construction time and costs.

7.1 Summary of main findings

The most important findings from the analysis based on analytical approaches and numerical modelling are:

- In phyllite and in massive and sparsely jointed basement rocks, estimated leakage rates are < 6 l/min/100 m tunnel. Average expected leakage rate is estimated to be 0.03 - 0.04 l/min/100 m tunnel.
- Unless mitigating measures are carried out, water leakages in fractured rocks, faults and weakness zones are estimated to vary between 0.02 to 5770 l/min/m. Average expected leakage rate is estimated to be 38 l/min/m tunnel.
- In phyllite, estimated tunnel squeezing is estimated to vary between 0.17 to 9.53 % depending on rock mass strength and rock overburden. With low overburden pressure and relatively high rock mass strength of phyllite, the estimated tunnel strain $\varepsilon =$ 0.17 %, while estimated tunnel strain with high overburden pressure and low rock mass strength is 9.53 %. Average tunnel strain is estimated to vary between 0.44 to 2.4 % depending on overburden pressure. This corresponds to a tunnel closure of 0.044 and 0.24 m respectively.
- By analytical approaches minor spalling is estimated at tunnel depths between 200 and 900 m. The extent of spalling is found to decrease with depth. At 200 m tunnel depth, estimated depth of failure is approximately 20 cm outside excavation boundary, while at 900 m tunnel depth it is found to be zero.
- By numerical modelling in Phase^2, brittle failure is found to occur at all tunnel depths between 300 and 900 m. Yielded elements are found concentrated in the hanging wall at tunnel depths between 300 and 700 m, and estimated depth of failure for these depths are approximately 20 cm outside the excavation boundary. At 900 m depth, tunnel walls are also found to yield and approximate depth of failure is found to be 60 cm.

Estimated tunnel leakages are found to vary considerably depending on the hydraulic conductivity. However, based on the analytical results and experiences from tunnel projects in similar rock conditions, it is expected that where the rock mass is massive and sparsely jointed, leakages to the tunnel will be more or less insignificant. This also applies for the phyllite, in which the leakage rates are estimated to be very low. On the other hand, where the basement rocks are fractured and jointed the leakage rate is expected to be considerably higher and injection grouting for sealing the tunnel might be necessary.

A worst case scenario with respect to water inflow will probably exist where water carrying faults and weakness zones are encountered. Estimated leakages in such zones with highly fractured rock and thus high hydraulic conductivities are very significant and might be threatening to the stability of the cavern as discussed in chapter 4.2.1. Such conditions will require exceptionally effort for sealing and stabilizing the rock mass. At the same time, weakness zones might have clay infillings which can have a sealing effect and reduce the amount of leaking water.

Significant water leakages in the tunnel may result in a drawdown of the water table which may cause drainage of overlying ponds and lakes. Especially important is it so preserve the existing nature in this protected landscape consisting of large, unspoiled areas with minor impact from human activities. Special attention should also be paid to the Hardangerjøkulen glacier. The impact the tunnel excavation might have on the environment, its sensitivity of a drawdown of the groundwater table and risk of damage should be carefully evaluated.

From the analysis, it is found that tunnel squeezing might occur where overburden pressure is high and if the rock mass is weak. In a "worst case scenario" severe squeezing might be encountered requiring heavy rock support. On the other hand, in a "most likely scenario" it is expected that only minor tunnel squeezing will occur, a situation which most likely can be dealt with by rock bolts, shotcrete and armed shotcrete ribs. Since squeezing is highly dependent on the rock mass strength, future geological investigations should aim at estimating this parameter more accurately. This requires not only field mapping, but also in-situ measurements and laboratory testing.

Spalling is found to be a serious issue that might influence the stability of the tunnel. Spalling is estimated at all tunnel depths by the use of analytical approaches and it is found to decrease with depth. While intense spalling and even rock burst are experienced in many nearby tunnel projects, only minor spalling is estimated by the analytical approaches in this work. This might be due to the model's sensitivity to K, the ratio of σ_H to σ_v. With increased K, the extent of spalling will also increase. With the values of K used in this analysis, the stress situation around the excavation seems to be quite favorable, and rock bolts and shotcrete should be able to deal with such conditions. However, if the horizontal stresses are higher than estimated by equation 10, moderate to severe spalling is likely to be encountered in the tunnel.

Occurrence of spalling is also confirmed by the numerical analysis. The estimation of brittle failure by analytical formulaes compared to the result from the numerical modelling is found to be quite consistent, but there are also some differences. While the analytical formula estimates depth of failure to decrease towards depth, the results from the numerical analysis indicates that spalling will increase with increasing rock overburdens. While the analytical equations only considers one rock mechanical property (σ_c) in addition to in-situ stresses, the numerical model also considers rock mass elasticity parameters, geological strength index (GSI) and other material constants. The rock mass strength is also calculated in a more sophisticated way by using the Hoek-Brown failure criterion and the software program RocLab.

The estimated failed zones in the numerical analysis caused by brittle failure in the rock mass, is found to vary a bit with the material model and "failure indicator" used. Brittle failure is estimated by using two different material models in Phase^2; one elastic and one plastic model. Yielded elements and shear bands were used as indicators of failure in the plastic model and strength factor in the elastic model (figure 51). Considering yielded elements, these are concentrated along the tunnel periphery forming a large elongated failed zone. The shear bands more or less forms V-notched arches which is very typical for spalling, while the region formed by SF $<$ 1 exhibits different shapes at different tunnel depths. It is therefore difficult to predict exact shape and depth of the failed region from the numerical analysis. On the other hand, independent on material model and failure indicator, it is a mutual trend that a failed zone is developing in the hanging wall and that this zone is becoming larger and deeper when rock overburden is increasing.

Figure 51: Result from numerical analysis at 900 m rock cover; left: yielded elements, middle: maximum shear strain, right: strength factor (dark blue).

Excavating a circular tunnel instead of a D-shaped is found to only have minor effects with respect to stability of the hanging walls. In the numerical analysis, the amount of yielded elements and development of shear bands were found to be very much the same comparing the two tunnel geometries. The positive effect by having a circular tunnel is the elimination of stress concentrations at the tunnel corners. If the rock mass is weak, failures initiating at these corners may in some special cases lead to floor heave.

Even minor spalling requires rock support for preventing fall outs and damage to the tunnel. The numerical analysis performed with rock support shows that with the amount of estimated spalling, conventional rock support methods should be able to give sufficient stability to the tunnel. Common practise in Norway is using rock bolts and shotcrete for this purpose, and the numerical analysis has shown that such rock support will give a stable tunnel preventing fallout from roof and walls.

7.2 Limitation and uncertainties of the stability analysis

It needs to be emphasized that the stability analysis is based on a limited amount of data. Results should be used with care and considered as rough estimates.

The accuracy of the analysis of water ingress, tunnel squeezing and spalling in chapter 5 and the numerical analysis in chapter 6 depends upon reliable estimations of input variables and reliable formulaes. Selection of representative variables are a difficult task since the geological conditions at tunnel depth are to a high degree unknown. Accurate estimation of variables requires field mapping at tunnel depth. Since that is beyond the scope of this thesis, the input variables used are based on measurements carried out at nearby locations, or when such measurements does not exist, average common values assessed to best represent the expected rock conditions are used.

Tunnel squeezing

The Hoek & Marinos (2000) approach for predicting squeezing requires reliable estimates of the overburden pressure σ_v and rock mass strength σ_{cm}. Estimating overburden stress is not a very difficult task since it can be calculated from the overburden height which in this case is known. Specific weight of the rock mass may be based on literature review, and the variation is relatively small and has only a little effect on overburden stress. σ_{cm}, on the other hand, is very difficult to estimate. Accurate estimates requires field mapping at tunnel depth, and it is also a parameter likely to vary along the tunnel corridor requiring several measurements.

Spalling and rock burst

As for analyzing tunnel squeezing, predicting spalling and rock burst requires reliable estimates of rock mass strength and in-situ stresses. The approach for predicting depth of failure suggested by Martin et al. (1999) implies a rock mass strength $\sigma_{cm} = 0.4\sigma_c$. In reality, the rock mass strength are most likely to vary along the tunnel corridor, and the real σ_{cm} can not be known exactly without further investigations.

The prediction of spalling and rock burst is also very sensitive to the in-situ stresses, and K, the ratio of the horizontal stress to the vertical one is especially of major concern. All experience gained from nearby projects indicates $K > 1$, even at great depths. However, there exists no empirical formulaes which can predict this parameter exactly as it is found to vary considerably from location to location, and with depth.

Water ingress

Predicting ground water inflow to a tunnel by means of a simple analytical assessment will only give a rough estimation. The formulae used in this work depends on h, the distance from tunnel to ground water table, the radius of the tunnel and K, hydraulic conductivity. Assuming a groundwater table close to the ground surface, estimating h is not a difficult task, and the tunnel radius is of course known. K is the major factor of uncertainty, a parameter difficult to estimate without in-depth investigations, and also a parameter highly influencing the inflow results. Also a limitation of the method is the assumption of homogenous conditions. Considering the fact that water mostly is limited to joints, and those joints are likely to vary considerably throughout the rock mass with respect to frequency and size, predicting water inflow is very difficult.

Numerical modelling

The major element of uncertainty for the numerical modelling is the limited knowledge of the mechanical rock mass properties. Uncertainties are associated with the choice of input parameters and those especially difficult to quantify are the rock mass elasticity parameters (E and ν) and the initial in-situ stresses. The result is highly dependent of the strength parameters used. It needs to be emphasized that the uncertainties of the input parameters affect the quality of the results of the numerical analysis, and the results should only be used for guidance and be carefully assessed through experience and practical judgement.

8 Analysis of construction time and costs

8.1 Engineering geological classification

Costs and construction time for a tunnel mainly depends on rock mass conditions and the risk of negative environmental influence. In order to obtain acceptable stability and security against deformations in poor rock conditions, more effort is put in rock support such as bolts and concrete linings. For the Hardangerjøkulen tunnel, groundwater issues is also relevant as there is a risk of draining lakes and drying out soil and vegetation of great value. A tight tunnel is obtained by the use of pre-injection grouting, a well established method in Norway, but also expensive.

Based on expected degree of rock support, the tunnel is divided into sections of three different tunnel classes; good rock quality, moderate to poor rock quality and very poor rock quality. The classification is based on the Q-system (Løset 1997) and Statens Vegvesen's rock support classes for road tunnels (Statens Vegvesen 2010).

Table 14: Tunnel classes and associated rock support concept.

HSR tunnel class	A	B	C
Rock condition	Good rock quality, sparsely jointed.	Moderate to poor rock quality, densely jointed or schistose rock mass.	Very poor rock quality
Q-value	4 - 100	0.1 - 4	< 0.1
SVV rock support class	I + II	III + IV	V + VI
Rock support concept	Systematic bolting (c/c 2 m), end anchored, tensioned and grouted. Shotcrete B35 E700, 80 mm thickness	Systematic bolting (c/c 1.5 m), end anchored, tensioned and grouted. Shotcrete B35 E1000, 150 mm thickness Armed shotcrete ribs if $Q < 0.2$ Spiling bolts if $Q < 0.2$	Systematic bolting (c/c 1.0 - 1.5 m), end anchored, tensioned and grouted. Shotcrete B35 E1000, 150 - 250 mm thickness Armed shotcrete ribs; double layered (D60/6+4, Ø20mm, c/c 1.5-2m) Spiling bolts; fully grouted; c/c 200 - 300 mm, ø32mm
Length	18.3 km (46 %)	17.1 km (42 %)	4.8 km m (12 %)

Tunnel class A refers to tunnelling using ordinary rock support such as bolts and shotcrete. Rocks of expected good quality such as massive, crystalline and metamorphic rocks with few joints usually belongs to class A. Class B implies increased use of bolts and shotcrete. Sporadic use of spiling and armed shotcrete ribs might also be relevant. The rock mass in class B is more jointed and/or there are unfavorable stress conditions. Phyllite and spalling rock will typically be represented by this class. Class C represents rock mass which requires heavy rock support such as full concrete lining, armed shotcrete ribs and pre-grouting. Weakness zones with surrounding rock are found in class C together with massive rock subjected to very unfavorable stress conditions.

Geological conditions along the tunnel alignment are discussed in chapter 2, 4 and 5. In table 14 the tunnel classes are represented with associated rock support concept and total share of tunnel length. Distribution of tunnel classes are estimated to be 46 % of class A, 42 % of class B and 12 % of class C.

8.2 Access tunnels and working faces

When planning a tunnel, different solutions exists with respect to the design of the project. One important aspect is the location and number of access tunnels. Adits must be placed such that the project may be accomplished in an efficient way with respect to construction time and costs. Due to the location of the tunnel under the Hardangerjøkulen glacier, long tunnel segments are difficult to avoid in the Hardangerjøkulen tunnel. As for the Gotthard base tunnel, it is recommended to construct at least two access adits for the two main tunnels. This will shorten an otherwise very long construction time.

The two adits should be located east and west of the glacier (figure 52). The tunnel will then be divided into 3 sections of approximately equal length (13.4 km) and a total of 6 working faces where the tunnelling may progress simultaneously (figure 53). The length of the adits will depend of the inclination of the tunnels, but a length of 2-3 km should be expected.

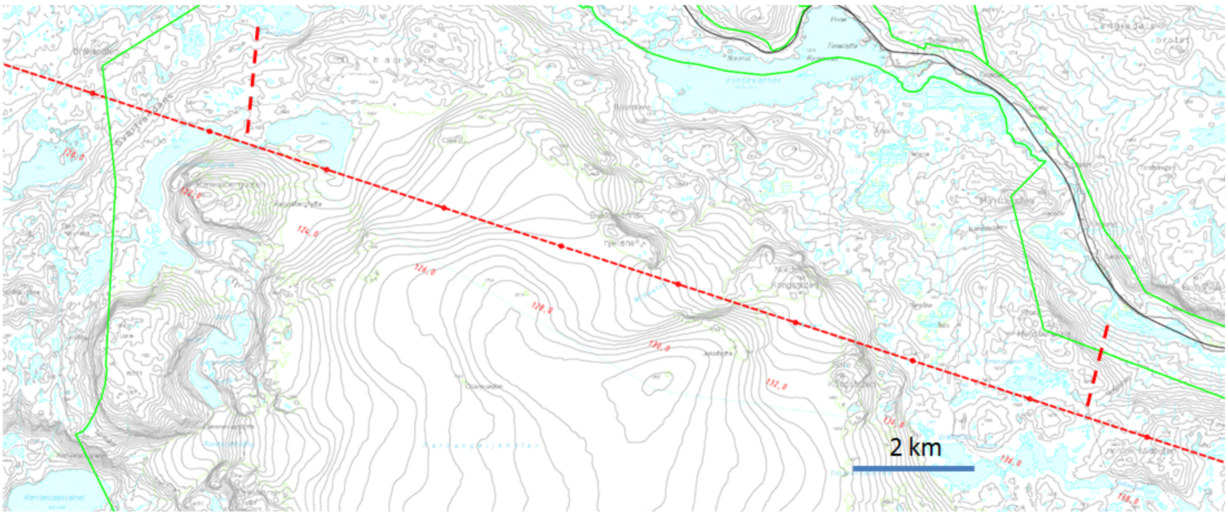


Figure 52: Suggested location of adits (map modified from Sweco (2011).)

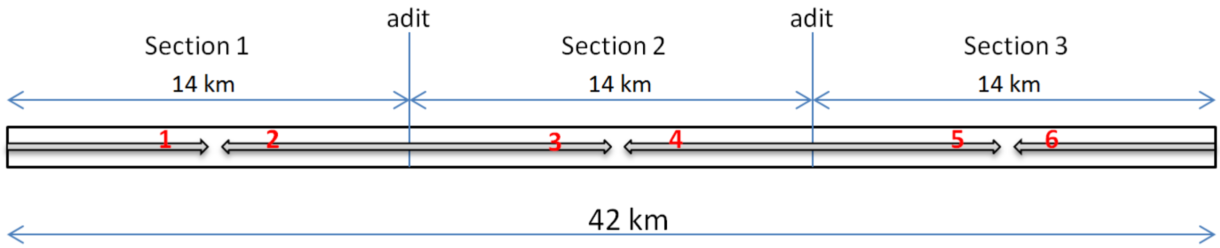


Figure 53: Schematic sketch of Hardangerjøkulen tunnel with 2 adits dividing the tunnel into 3 sections of equal length. This design will give 6 working faces.

8.3 Conventional tunnelling

8.3.1 Construction time

For estimating construction time, capacities obtained from the Norwegian Standard 3420; "Specification texts for building, construction and installations Part F: Earthworks - Part 1", and Project Report "2B-95 Tunneldrift - Prognoser konvensjonell drift" are used. The capacities are for a cross-sectional area of 80 m^2.

The prognosis of weekly advance rate excluding rock support works is based on the following assumptions (NTH-Anleggsdrift 1995):

- 45 mm drillhole diameter
- Hydraulic drilling rig AC COP 1838. Large hole diameter 102 mm.
- Round length 5.0 m
- Adequate transport capacity which is fully utilized
- 101 shift hours per week, 46 effective weeks per year

This includes all activities during a blasting round; drilling, charging and blasting, ventilation, loading, hauling and scaling. Capacities for rock support and injection grouting are based on capacities found in *Norsk Standard NS3420* (2008). This includes capacities for all processes normally performed at the tunnel face including spiling bolts, drilling of probe holes, injection works, rock bolts, shotcrete, armed shotcrete ribs and concrete lining.

The estimation of construction time is only regarding excavation and rock support capacities for the main tubes and does not include:

- Time for excavation and supporting of niches and connecting galleries
- Surface works like construction of tunnel portals etc.
- Technical installations

Scope of rock support for each tunnel class is based on table 14. See also table B-1, B-2 and B-3, appendix B.

Tunnel class A is supported by systematic bolting with a bolting density of 2 x 2m in the hanging wall supplemented by a 80mm thick shotcrete layer. Tunnel class B have a denser bolting pattern, 1.5 x 1.5m, and 150mm shotcrete. Spiling bolts in the hanging wall and armed shotcrete ribs in half of the length of class B are assumed. The armed shotcrete ribs are placed at intervals of 3 m. The bolting density in class C are 1 x 1m, the shotcrete layer 200mm thick and spiling bolts are assumed in each round, both in roof and walls. Armed shotcrete ribs at intervals of 2 m are assumed in class C.

Drilling of probe holes are included in the entire tunnel. 4 24 m long holes per cross section are included in class A, and 6 equivalent holes in class B and C. Probe drilling is assumed to be executed each 4. round which gives and overlap of approximately 4m. Concrete lining are assumed to be necessary in weakness zones only, in total 200 m. Armed shotcrete ribs are expected to give sufficient support in fractured rock, unless the ground conditions are exceptionally poor.

Capacities for pre-injection grouting works are included in class B and C (table B-4, appendix B). In class B each grout curtain will have 23 holes with 24 m long injection holes. On average, the distance between each curtain is assumed to be 30m. In class C systematic pre-grouting is assumed in the entire section, each grout curtain with 38 24 m long injection holes. For both tunnel classes a grout consumption of 1000 kg per injection hole is expected.

The results from the time analysis are found in table 15. For detailed calculations see table B–5, appendix B. Total working time at face per main tube including excavation, rock support, probe drilling and injection grouting is estimated to be 2628 weeks. With a tunnel length of 40.2 km this gives an average weekly advance rate of 15.3 m/week. Assuming that construction works can progress simultaneously and at same excavation rate at all 6 working faces, the total construction time of one main tube will be 458 weeks, or approximately 9 years and 6 months.

Table 15: Estimated weekly advance rates and total construction time.

Total working time at working face per main tube (weeks)	2628.2
Average weekly advance rate per working face (m/week)	15.3
Total construction time per main tube (weeks)	458
Total construction time per main tube (years)	9.5

8.3.2 Construction costs

Based on tunnel classes and associated rock support concepts found in appendix B, construction costs of the two main tunnels are estimated. The cost accounting is done in accordance with "*Håndbok 025 Prosesskode 1 Standard beskrivelstekster for vegkontrakter; Hovedprosess 3 Tunneler*", a Norwegian process code used for preparation of tendering documents in civil engineering works (Vegdirektoratet 2007).

Estimation of basic costs are based on a prediction model and simulation tool for costs of drill and blast tunnelling, developed at the Department of Civil and Transport Engineering, NTNU (Zare 2006). Basic costs includes costs for drilling, charging and blasting, scaling, loading and hauling and ventilation. This prediction model are based on the following assumptions:

- 48 mm drillhole diameter
- ANFO/emulsion with 5 % dynamite
- Medium drillability and blastability
- 10% extra for unforeseen costs

The prediction model does only consider tunnel lengths < 10 km, but a linear relationship is found between basic costs and tunnel length (Zare (2006)) figure 54). Costs are based on a cross-sectional area of 80 m^3 and it considers costs that vary with tunnel length such as hauling, ventilation, electrical installation and pumping of water.

Construction of two adits will give three tunnel sections with length of 13.4 km (figure 52 and 53). According to the extrapolated graph basic costs will then be approximately 300 NOK/m^3.

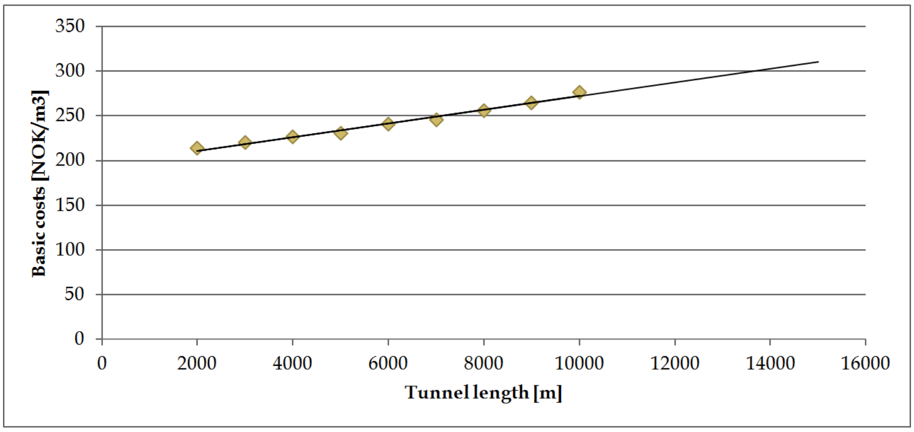


Figure 54: Basic costs as a function of tunnel length and cross-sectional area of $80m^2$. Price level is of August 2009.

Estimation of rock support costs are based on the tunnel classes and associated rock support concept in table 14. See also table B-1 - B-4, appendix B. Unit prices for rock support and injection grouting are gained from Norwegian tunnel projects. The estimation includes costs for:

- Rock support; rock bolts, spiling bolts, shotcrete, armed shotcrete ribs and concrete lining
- Probe drilling and pre-injection grouting
- Water and frost control

The estimation does not include costs for:

- Niches and connecting galleries
- Administration, planning and preliminary investigations
- Work outside the tunnel like portaling, removal of vegetation and blasting at surface
- Roads, telecommunication and power transmission lines
- Worksite upkeep such as roads, power transmission lines and housing
- Technical installations in the tunnel

The results from the cost analysis is presented in table 16. For detailed calculations see table B–6, appendix B. Total costs per meter tunnel including basic costs, costs for rock support, injection grouting, water and frost control are estimated to be approximately 123 000 NOK. 15 % is added for rigging and operation, and additional 10% is added for unforeseen costs. In total, including both main tubes, the estimated costs are approximately 10 billion NOK.

Table 16: Estimated costs for Hardangerjøkulen tunnel. All prices are given in NOK.

Process	Sum costs
Rigging and operation (15% of total costs)	571 431 705
Probe drilling and injection grouting	207 448 200
Excavation elements	964 800 000
Tunnel rock support	1 790 684 500
Water and frost control	846 612 000
Unforeseen costs (15 % of total costs)	571 431 705
Total costs incl. rigging, operation and unforeseen costs (NOK/m)	123 197
Total costs per km for main tunnels, rounded (NOK/km)	246 388 000
Total costs for main tunnels, rounded (NOK)	9 905 000 000

8.4 Mechanized tunnelling

Today, TBMs are used for most tunnelling purposes and in most rock conditions. Different types exists, like Open Hard Rock TBM, Single Shield, Double Shield and Mix Shield. These may be adapted to boring in different conditions. For this analysis a double shielded TBM are chosen due to strict functional requirements of railway tunnels in Norway. With this alternative, watertight concrete elements are installed continuously approximately 13 m behind the face. These concrete elements are expensive, but will replace huge amounts of probe drilling, injection grouting, rock support and water and frost control and will give more favorable conditions and lower costs for future maintenance. Since installation of concrete elements are done behind the working face and simultaneously as the TBM is boring, this process does not influence the advance rate (Jernbaneverket 2008).

For estimating time and costs for the tunnel excavated by TBM, the software program FullProf is used. The program is developed at the Department of Building and Construction Engineering at NTNU and is based on knowledge and experience data gathered by the department. Time consumption and excavation cost are estimated as described in the NTNU model for hard rock tunnel boring (Bruland 1998, 2002).

The rate of excavation is dependent on both geological parameters and machine parameters (table 17). Jointing frequency are of major importance as increased jointing will increase the advance rate of the TBM. The rock mass' drillability are expressed by a drilling rate index (DRI) and cutter life index (CLI). Quartz content is also influencing the drillability as a high content of quartz will increase the rock strength and reduce drillability. Registered rock parameters for some common rock types are found in figure C–1, C–2 and C–3, appendix C.

Table 17: Rock mass condition- and machine parameters influencing TBM excavation rate (Bruland 2002).

Rock mass conditions	Machine parameters
Jointing; frequency and orientation	Gross average cutter thrust
Drilling Rate Index (DRI)	Average cutter spacing
Porosity	Cutter diameter
Quartz content	

Input to the software FullProf is machine data such as TBM diameter, cutter diameter and geological parameters for an unlimited number of zones (table 18). Geology is described by drillability (DRI), cutter wear (CLI), quartz content and degree of jointing (fracture class and orientation of joints).

Table 18: Input parameters to software program FullProf

Parameter	Class A	Class B	Class C
Length (m)	18 300	17100	4800
Diameter of TBM (m)	10	10	10
Cutter diameter (mm)	483	483	483
DRI	51	45	50
CLI	7	9	31
Quartz content (%)	30	25	20
Fracture class	I	I-II	III
Angle	20	20	20

The cost model in FullProf is based on detailed estimations of all costs included in the excavation works. By excavation costs one means only the costs directly related to excavation of the tunnel, excluding rock support. This model is based on and linked to models for penetration rate, cutter life and advance rate. Advance rate is based on averaged data over the complete tunnel length. Geological and machine parameters are indirectly included in the model through net penetration rate and cutter life.

Costs for the concrete lining, water and frost control, rock support and injection grouting are based on experiences from the Norwegian railway project "Follobanen" (Jernbaneverket 2008). Currently under planning, this project has many similarities to the Hardangerjøkulen tunnel; the tunnel is very long (close to 20 km), the rock type is mainly Pre-Cambrian gneiss, and the tunnel is designed with two tubes with single track with same dimensions as for the Hardangerjøkulen tunnel (10 m diameter). In addition to concrete lining and water and frost control, costs for some rock support and injection grouting is included since otherwise there will be an unsupported length behind the TBM face before the concrete lining is installed.

In addition to the input parameters in table 18, the prognosis of construction time and costs are based on the following assumptions:

- 2 adits and 3 tunnel sections with lengths 13.4 km.
- 3 double shielded TBMs progressing simultaneously an at same advance rate in each tunnel section.
- Conveyor belt for transporting TBM cuttings out of the tunnel.
- 46 effective working weeks per year and 101 working hours per week.

The cost analysis does not include costs for:

- Adits and connecting galleries
- Technical installations

- Works outside the tunnel like portals

Estimated cost for the two main tubes are presented in table 19. For more detailed calculation see table C–1, appendix C. The estimation includes costs for rigging, operation, installation and dismantling of TBM, investment cost for TBM (100 MNOK), boring costs, transportation of excavated material out of tunnel, watertight lining and rock support. 10 % is added for unforeseen costs, erroneous assumptions etc. (Bruland 2002).

Total costs per meter tunnel is estimated to approximately 133 000 NOK/m. Including both main tubes, the total price will be approximately 10.7 billion NOK.

Table 19: Estimated costs for TBM tunnels

Process	Sum costs
Rigging and operation (15%)	639 541 800
Tunnel boring	1 210 050 000
Watertight concrete lining	3 053 100 000
Rock support and injection grouting	18 492 000
Unforeseen (10%)	426 361 200
Total costs (NOK/m)	132 575
Total costs incl both main tubes (NOK/km)	265 150 000
Total costs incl both main tubes (NOK)	10 659 030 000

Calculated by using of FullProf, estimated construction time is presented in table 20. With section lengths of 13.4 km and 3 TBMs at work, the achieved average weekly advance rate per working face is 59 m/week. This will give a total construction time per main tube of 227 weeks or 4.9 years.

Table 20: Estimated construction time per main tube

Working hours per week (h)	101
Effective weeks per year (weeks)	46
Average weekly advance rate (m/week)	59
Total construction time per main tube (weeks)	227.1
Total construction time per main tube (years)	4.9

8.5 Discussion of construction method

As discussed in chapter 3, both TBM and the drill and blast method have advantages and disadvantages, but when choosing construction method it basically comes down to time and costs. The analysis has shown that excavating the two main tubes with TBM is a competitive method compared to D & B, especially considering excavation rates.

Regarding costs, the TBM alternative is found to be approximately 8 % more expensive than the D & B alternative (10.7 versus 9.9 billion NOK). Considering this is only a rough estimate associated with large uncertainties of geological parameters, this is a relatively small difference, and the costs may change considerably if unforeseen geological conditions are encountered. Reducing investment costs, installation and dismantling costs for the TBM option is possible, having two TBMs operating from each portal. However, additional wages and costs for transportation, belt conveyor and ventilation will increase so much due to the lengthened tunnel sections, that the total costs will be higher than when using 3 TBMs.

Construction time, on the other hand, is found to vary considerably for the two methods. Predicted construction time using TBM is expected to be approximately half of the D & B construction time. A construction time of 4.9 years versus 10.5 is significant and can not be neglected. Again, the excavation rates are highly dependent of the geological conditions, especially for the TBM method, but still TBM is the method most likely to give the shortest construction time. One method for reducing the construction time using D & B is construction of more adits, but this might be a difficult task under the glacier Hardangerjøkulen.

Another very important aspect is the suitability of TBM in the expected hard and brittle rock subjected to high stresses. Rock bust and spalling will influence the TBM excavation in several ways (Gong, Wu, Zhao & Ting 2012):

- Non-violent spalling might be beneficial for the TBM excavation rate because additional joints are formed ahead of the excavation surface. On the other hand, it brings trouble to support system and muck removal.
- If violent rock burst is encountered, it is required to modify the TBM operation parameters to reduce the disturbance of the tunnel face and avoid cutterhead damage induced by rock burst and slabbing.
- Rock burst might influence support system, gripper moving, gripper force and cutter ring damage. In worst case, the TBM might get jammed.

Considering this, TBM might not even be an option if spalling and rock burst in the tunnel are found to be severe. Future thorough and in-depth ground investigations, stability analysis and laboratory analysis of rock mechanical properties must reveal wether the geological conditions are suitable for TBM or not. Such investigations and assessments must also be carried out in order to get more accurate and reliable estimations of times and costs. Acceptable costs, advance rates, a safe working environment and a safe tunnel must be aimed at, and maybe even a combination of TBM and D & B will be the most beneficial.

9 Suggested future geological investigations

For such a large project as the Hardangerjøkulen tunnel, geological investigations are of major importance both during the planning and construction phase. Preliminary investigations are determining for the design of the project and will be used not only by engineering geologist but will also be a part of the tender documents. Any errors or mistakes may have technical and economical consequences.

Investigations for the tunnel should be based on maps, aerial photos, geological maps and existing reports, as well as investigations performed in field and in laboratory. The results from the investigations should include an overview and characteristics of the following:

- Depth of soil and rock overburden
- Degree of weathering
- Rock types and their distribution
- Degree of jointing
- Weakness zones
- In-situ stresses
- Results from laboratory tests

The results are determining for several factors concerning the project and will be used by the entrepreneur(s) and proprietor to plan the blasting scheme (both design and working plan), rock support, construction time and quantity lists. The most important assessments based on the investigation results are:

- Positioning, orientation, design and dimensions of the tunnel
- Construction method
- Drillability and blastability
- Stability and water leakages
- Degree of difficulty and extent of rock support
- Utilization of excavated material
- Construction costs and time

9.1 Scope of investigations

The extent of investigations should be based on several factors; local ground conditions, type of project, phase of project planning, character of contract to be used and choice of construction method. Local ground conditions (geology, accessibility to the site, topography, rock overburden) and type of project and the demands it are to satisfy are of most importance when deciding the scope of investigations.

Local ground conditions may be classified with respect to difficulty (table 21), and the type of project may be classified with respect to requirements and challenges the construction of the project demands (table 22). From this, the project can be categorized into a "investigation class" which is determining for the scope of investigations (Palmstrøm et al. 2003).

The ground conditions in the Hardangerjøkulen tunnel are assessed to have a high degree of difficulty (table 21). Geology is expected to be surveyable as the geological structures in the area are assumed to be quite clear and mainly dominated by granitic and gneissic basement rocks. On the other hand, the glacier, depth of the tunnel and the remote location make surveying the area complicated. Thus, accessibility to terrain is assumed to be difficult and rock overburden significant. Weathering at surface is not known and is therefore assumed to be moderate. Summarized this will give a high degree of difficulty of the project.

Table 21: Classification of ground conditions

Ground condition	Grade	Classification
Geology	1.5	surveyable
Weathering at surface	1	moderate
Rock overburden	5	significant
Accessibility to terrain	3	difficult
Degree of difficulty	10.5	high

Construction requirements for the tunnel are assessed to be moderate (table 22) based on requirements with respect to stability/safety during construction and operation and the possibility of influence on external surroundings. According to Norwegian standards, functional requirements should always be set to be high for railway tunnels. Risk during construction is expected to be moderate mostly due to the spalling rock. Situated in a protected area drainage of ground water may impact the environment and this post is therefore set to be moderate. No influence on existing constructions or buildings is expected as this is a very remote area with no population.

Table 22: Classification of construction requirement

Circumstance of excavation	Grade	Classification
Functional requirements	5	high
Risk during construction	2	moderate
Environmental impact	1	moderate
Influence on other constructions	0.5	none
Construction requirements during building and operation	8.5	moderate

A high degree of difficulty and moderate construction requirements places the project into investigation class C (Palmstrøm et al. 2003). As seen in figure 55, a tunnel with a length of 40 km in investigation class C has a recommended scope of preliminary investigations of approximately 0.9 % of the total blasting costs. Costs for geological investigations during construction comes in addition.

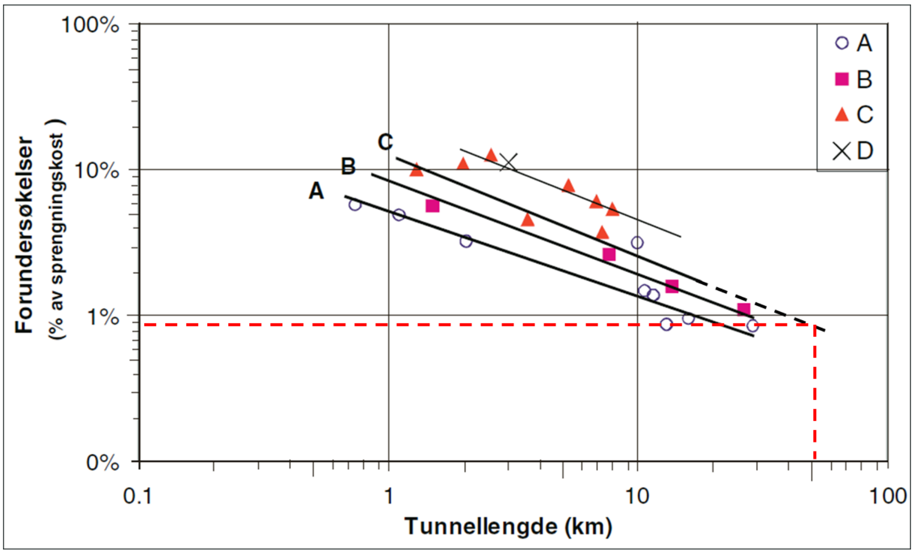


Figure 55: Recommended scope of preliminary investigations (% of blasting cost) as a function of tunnel length (x-axis) and investigation class (modified from Palmstrøm et al. (2003)). Blasting cost includes loading, charging, blasting inclusive rigging.

Two important aspects which may influence and give additional costs to the preliminary investigations are the high rock overburden and choice of construction method. Excavations with high overburden, like the Hardangerjøkulen tunnel, will have a higher uncertainty with respect to the geological prediction at tunnel level. The location under the glacier is also likely to increase the uncertainty, requiring increased effort for the preliminary investigations. The recommended scope of investigations are mainly based on the study of tunnels excavated by D & B. If mechanical excavation with TBM is chosen, this increases the demand of investigations before construction can start. Errors in the prognosis for full-face boring may give large cost additions and thus a reliable estimate is of major importance.

9.2 Preliminary investigations

It is recommended to use a combination of different investigation methods in order to get a satisfying and reliable prediction of the geological conditions along the tunnel corridor. This includes a study of existing material, field studies and laboratory testing:

- Study of basis material
- Inspection of near-lying underground constructions
- Engineering geological field mapping
- Drilling for defining rock depth
- Core drilling and/or exploration tunnel
- Laboratory analysis of rock samples
- In-situ stress measurements
- Geophysical explorations

As a first stage, geological basis material should be studied in order to assess the feasibility of the project. Suitable tunnel alignments should be identified as well as areas critical for construction costs and safety. For this purpose, existing material like geological and topographic maps, reports from near lying existing projects and aerial photos must be used. Field mapping is also relevant.

The next stage will be to go in-depth in order to understand and predict the geological conditions at tunnel level. Thorough field mapping complemented by study of basis material will give indications of distribution of rock types, rock mass quality, weakness zones, rock stresses and hydrogeological conditions. Measuring programmes for studying ground water level, water reservoirs and vulnerability of flora and fauna should be established. Identifying areas prone to settling is not very relevant as this is a non-populated area. Instead, focus should be put on not influencing the water balance, flora and fauna and the natural habitats existing here. For defining the rock depth above the tunnel alignment, drilling must be executed along the corridor. This will be a difficult task under the glacier and directional drilling or geophysical methods might be an option to assure the rock cover.

Due to the high overburdens core drilling is advisable in order to obtain a reliable prediction of the geological conditions at tunnel level. This is an method requiring expensive technical equipment and should therefore be used after careful consideration. With this method large depths can be reached, and also areas under the glacier. Logging of the core material will give information about rock type, degree of jointing and jointing characteristics, weakness zones and permeability. The investigations may be supplemented by laboratory tests of core samples, hydrogeological tests and inspection of the borehole by optical and/or acoustic televiewer. An alternative or supplement to core drilling are excavation of exploration

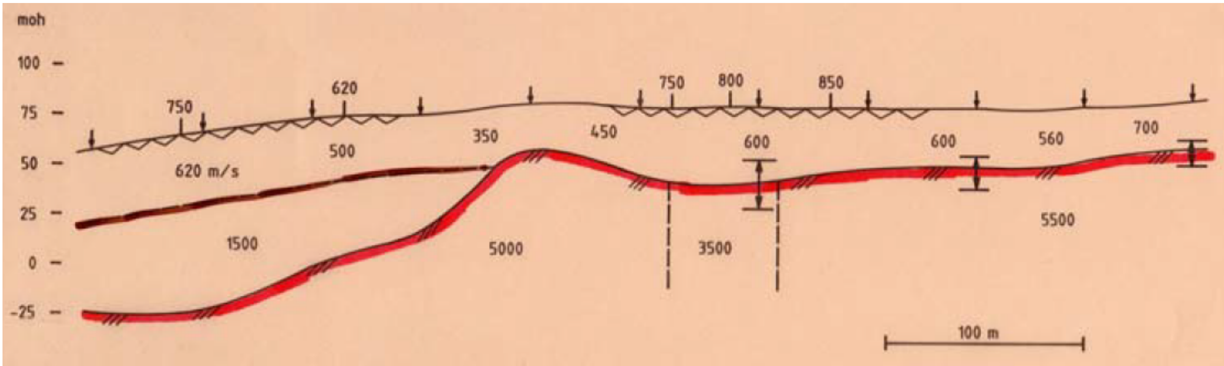


Figure 56: Example of geological profile based on refraction seismic investigations where rock surface (red line) and seismic velocities indicating a weakness zone (3500 m/s) are displayed (NGU 2012).

tunnel which will give even more reliable data. This is of course an expensive investment, but the costs might be justified if the tunnel is implemented into the finished project.

As a supplement to drilling and field mapping, geophysical explorations should be considered to get a more coherent picture of the geology towards depth. This is especially important as the tunnel is very deep in some parts. Different geophysical methods exists on the market, and dependent of the method these will give information about thickness of soil cover, weakness zones and ground water. Indications of rock mass quality may also be found. Relevant geophysical exploration methods are:

- Refraction seismic
- Seismic tomography
- Electromagnetic methods
- Resistivity method
- Borehole logging (optical and/or acoustic televiewer)

Also as a part of the planning phase, the material to be excavated must be evaluated with respect to quality, reuse and depositing. Maximum recycling, or reuse of the excavated material should be aimed at, and thus investigations and laboratory tests must be executed in order to assess whether the rock is suitable as for example concrete aggregate. Investigation of ground conditions for landfill sites must also be conducted.

9.3 Investigations during construction

Some types of geological investigations are only feasible during the construction phase of the tunnel as they can not be performed from the surface. These investigations may be regarded as postponed preliminary investigations:

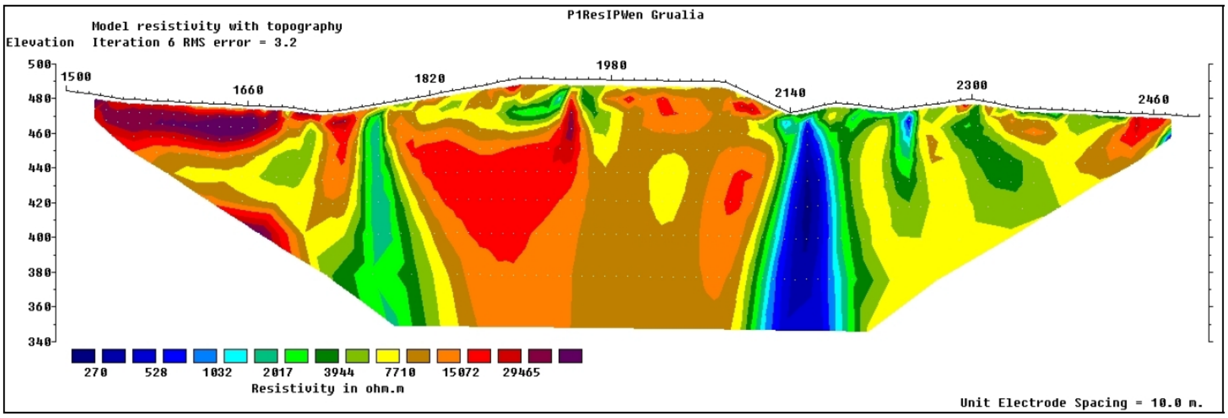


Figure 57: Resistivity model from the Lunner tunnel. Zones with low resistivity represent fractured rock (NGU 2012).

- **Probe drilling;** for requiring information about rock and water conditions ahead of the tunnel face.
- **Measurement While Drilling (MWD);** logging and recording of drill performance parameters during drilling of holes in the tunnel for predicting rock conditions and support requirements ahead of tunnel face.
- **In-situ stress measurements;** may be performed in holes drilled from inside the tunnel, including 2D and 3D overcoring method and hydraulic fracturing.
- **Sampling of gouge material for laboratory testing;** identification of swelling clay is especially important.
- Measurements of water leakages
- Monitoring of surface conditions above tunnel in order to prevent any disturbances to nature.

It is especially important to identify and and get detailed characterization of weakness zones as the excavation progresses. This allows for a better planning of rock support measures. Analysis of gouge material and especially swelling clay is necessary in order to apply sufficient and appropriate rock support. Detailed investigations during construction of the tunnel will be a check of the preliminary investigations, and important observations are jointing, rock mass quality, spalling rock and water ingress; all important aspects that the project needs to adapt.

10 Conclusion and recommendations

10.1 Conclusions

If the proposed 40 km long tunnel under the Hardangerjøkulen glacier for the high speed railway link between Oslo and Bergen will be built, several challenges will be met. Not only geological issues, but also constructional engineering and economical challenges are of major concern. Such a long and deep tunnel will face different geological formations with different mechanical properties, and thus different challenges exists related to depth and geology. The work will be a complex civil engineering system, to be designed under stringent requirements for safety during construction and operation.

Evaluating rock mass quality and potential geological challenges are of major concern during the planning stage of a tunnel. The condition of the geological formations which the tunnel must pierce will be decisive for the rock support concept and construction time and it will influence construction costs. The following conclusions can be drawn based on the study and analysis of geological conditions, geological challenges, time and construction costs for the Hardangerjøkulen tunnel presented in this thesis:

1. Review of the geology along the tunnel corridor has shown that the tunnel will be situated in two geological formations; Cambro-Silurian sedimentary rocks (chainage km 0 - 1.8) and Precambrian basement rocks (chainage km 1.8-40.2). The Cambro-Silurian rocks consists of phyllite, while the basement rocks mainly consists of different types of gneiss and granite. Phyllite is expected to be highly foliated and schistose with uniaxial compressive strength < 70 MPa. The basement rocks are expected to be massive, sparsely jointed and with uniaxial compressive strength of 100 - 200 MPa. Average rock overburden will be approximately 550 m, while maximum overburden is 900 m.

2. The tunnel will face different geological challenges. From the stability analysis, it is found that especially 3 circumstances might influence the stability of the excavation; 1) high water inflows under high pressure in fractured basement rocks, faults and weakness zones, 2) tunnel squeezing in weak phyllitic rock subjected to high overburden pressure and 3) spalling in massive brittle basement rocks subjected to high horizontal stresses. Spalling, or brittle failure is also found to be an issue estimated by numerical analysis in Phase^2. Swelling clay may be encountered in weakness zones.

3. Based on expected rock mass conditions along the tunnel corridor, the tunnel is categorized into tunnel classes with associated rock support concepts. If the tunnel is to be constructed by the conventional drill and blast method, estimated construction time for one main tube is approximately 9.5 years. Total costs, including both main tubes is estimated to be 9.9 billion NOK. If TBM excavation method is to be used, estimated construction time is 4.9 years per main tube. Total costs is estimated to be approximately 10.7 billion NOK including both main tubes. Based on this analysis it

seems that a TBM tunnel will have an advantage with respect to construction time, while drill and blast might giver lower costs.

Based on experiences from nearby projects the rock mass is expected to be generally suitable as a construction material. Conventional rock support concepts commonly used in Norway should be able to sufficiently support the tunnel, even where spalling, tunnel squeezing and water leakages are encountered. In addition to high costs and a long construction time, constructional engineering issues will also be related to the length and depth of the tunnel. Due to the depth of the tunnel, predicting the geological conditions are a difficult task and requires a great effort during the preliminary investigation stage. Access to the site is also limited due to Hardangerjøkulen glacier and the location in a protected landscape area. Concerns are also related logistical, operational and safety aspects which becomes more difficult in long tunnels.

10.2 Recommendations

It needs to be emphasized that this study is based on a limited amount of data, and the results should only be regarded as rough estimates. If the tunnel is ever considered to be built, this will require a great effort in documenting geological conditions, costs and construction time for evaluating the feasibility of the project. Especially important is the environmental aspect. The impact the tunnel excavation may have on the environment should carefully be evaluated. Supplementary research is therefore recommended for further documentation and reliability of the following:

1. Geological investigations in field and in-situ for a better understanding of the geology and increased confidence and reliability of potential geological challenges.
2. Geological investigations for increased confidence and accuracy of construction time and costs estimates. The link between geological uncertainties and construction costs should also be analyzed, which is an important aspect with respect to feasibility and risk in investment for the tunnel project.
3. Geological investigations for assessing the suitability of TBM versus conventional excavation method.
4. Environmental impact assessment for evaluating the surface environment's vulnerability and sensitivity to the tunnel.

References

AlpTransit Gotthard Ltd (2012), 'Gotthard base tunnel - official project website'.
URL: *www.alptransit.ch/en (21.03.2012)*

Askvik, H. (2008), *Berggrunnskart HARDANGERJØKULEN 14162, M: 1:50 000*, Norges Geologiske Undersøkelse (NGU).

Brattli, B. (2009), *Fysisk og kjemisk hydrogeologi*, Institutt for geologi og bergteknikk, NTNU.

Bruland, A. (1998), Hard Rock Tunnel Boring, PhD thesis, Department of Civil and Transport Engineering, NTNU.

Bruland, A. (2002), *Kompendium i anleggsteknikk; Fullprofilboring av tunneler*, Institutt for bygg- og anleggsteknikk, NTNU.

Dokken, A. (1992), Finsetunnelen, *in* 'Fjellsprengningsteknikk Bergmekanikk/Geoteknikk 1992'.

Edelbro, C. (2009), 'Different approaches for simulating brittle failure in two hard rock mass cases: A parametric study', *Rock Mechanics and Rock Engineering* **43**, 151–165.

Ehrbar, H. & Lieb, R. (2011), Gotthard base tunnel risk management for the wold's longest railway tunnel: Lessons learnt, *in* 'World Tunnel Congress 2011, Helsinki'.

Gong, Q.M. nad Yin, L., Wu, S., Zhao, J. & Ting, Y. (2012), 'Rock burst and slabbing failure and its influence on TBM excavation at headrace tunnels in Jinping ll hydropower station', *Engineering Geology* **124**, 98–108.

Goodman, R. E. (1993), *Engineering geology - Rock in engineering construction*, John Wiley and Sons, Inc.

Goodman, R. E., Moye, D., Schalkwyk, A. & Javandel, I. (1965), 'Groundwater inflows during tunnel driving', *Eng. Geol. 2/1* pp. 39–56.

Grimstad, E. (2006), Tunnelling under high rock stress in the Lærdalstunnel, *in* 'Fjellsprengningsteknikk; Bergmekanikk/Geoteknikk 1996'.

Grimstad, E. & Kvåle, J. (1999), 'The influence of rock stress and support in the depth of the disturbed zone in the Lærdal tunnel. a key to differentiate the rock support.'.

Gunleiksrud, O. & By, O. (1987), Utvidelse av Aurland kraftstasjon, *in* 'Fjellsprengningsteknikk Bergmekanikk/Geoteknikk 1987'.

Hansen, A. (2012), 'TBM vs. D&B - pros and cons', online.
URL: *www.nff.no (19.03.2012)*

Henriksen, H. (2000), *Berggrunnskart MYRDAL 14162, M:1:50 000*, Norges Geologiske Undersøkelse (NGU).

Hoek, E. (2009), 'Practical rock engineering', online.
URL: *www.rocscience.com/education*

Hoek, E. & Brown, E. (1980), *Underground excavations in rock*, Institution of Mining and Metallurgy.

Hoek, E., Carranza-Torres, C. & Corkum, B. (2002), 'Hoek-brown failure criterion - 2002 edition', *Proceedings of the 5th North American Rock Mechanics Symposium and the 17th Tunneling Association of Canada Conference* pp. 267–271.

Hoek, E., Kaiser, P. & Bawden, W. (1995), *Support of underground excavations in hard rock*, Taylor & Francis.

Hoek, E. & Marinos, P. (2000), Predicted tunnel squeezing problems in weak heterogeneous rock masses, *in* 'Tunnels and Tunneling International'.

Hoek, E. & Marinos, P. (2009), 'Tunnelling in overstressed rock', *Rock Engineering in Difficult Ground Conditions - Soft Rocks and Karst* pp. 49–60.

ISRM 1978 (1978), 'Suggested methods for the quantitative description of discontinutities', *International Journal of Rock Mechanics and Geomechanic Abstracts* **15(6)**, 319–368.

ITA (2009), *General report on conventional tunnelling method*, International Tunnelling and Underground Space Association (ITA), working group 19.

ITA (2010), *Long tunnels at great depth*, International Tunnelling and Underground Space Association (ITA), working group 17.

Jernbaneverket (2008), *Nytt dobbelspor Oslo-Ski, Vurdering av drivemetode for tunnelene på Follobanen; Rapport på utredningsnivå.*

Jernbaneverket (2011), 'Norwegian high speed rail assessment 2010-2012, summary of phase 2 works', online.
URL: *www.jernbaneverket.no/no/prosjekter/hoyhastighetsutredningen (16.01.2012)*

Jernbaneverket (2012), 'Prosjektering og bygging av tunneler', online.
URL: *https://trv.jbv.no/wiki/Underbygning/Prosjektering_og_bygging/Tunneler (15.03.2012)*

Johansen, P. (1976), Bergmekaniske målinger i maskinsal for Sima kraftverk, *in* 'Fjellsprengningsteknikk Bergmekanikk/Geoteknikk 1976'.

Johansen, P. (2007), Inflow criteria for a railway tunnel in the greater Oslo area, *in* 'Water Control, Publication No. 12', Norwegian Tunneling society.

Kalager, A. (2009), Nytt dobbeltsport oslo-ski - vurdering av tunnelkonsept og drivemetode, *in* 'Fjellsprengningsteknikk, bergmekanikk/Geoteknikk 2009'.

Kluver, B. H. & Kveen, A. (2004), *Berginjeksjon i praksis, Miljø-og Samfunnstjenelige Tunneler, Publikasjon nr. 104*, Statens Vegvesen, Teknologiavdelingen.

Kveldsvik, V., Erikstad, L., Holm, T. & Enander, L. (2007), *Planning of a 25 km long water supply tunnel in an environmentally sensitive area, NFF publication no. 12*, Norwegian Tunnelling Society (NFF).

Lieb, R. (2011), 'Experience in spoil management on conclusion of excavations for the Gotthard base tunnel', *Samoter 2011 - VeronaFiere (VR)* .

Løset, F. (1997), *Practical Use of the Q-Method*, Norwegian Geotechnical Institute (NGI).

Løset, F. (2006), *Norges tunnelgeologi*, Norges Geotekniske Institutt (NGI).

Martin, C., Kaiser, P. & McCreath, D. (1999), 'Hoek-brown parameters for predicting the depht of brittle failure around tunnels', *Canadian Geotechnical Journal* **36**, 136–151.

Martin, E. (1990), Rockburst imperil construction of Norway's largest underground power station, *in* 'Norwegian tunnels and tunneling, Publication No. 7', Norwegian Soil and Rock Engineering Association.

Myrset, Ø. & Lien, R. (1985), High pressure tunnel systems at Sima power plant, *in* 'Norwegian Hydropower Tunneling, Publication No. 3', Norwegian Soil and Rock Engineering Association.

Myrvang, A. (2001), *Bergmekanikk*, Institutt for geologi og bergmekanikk, NTNU.

Myrvang, A. (2012), Personal communication, May, 2012.

NGU (2012), 'Norges Geologiske Undersøkelse, official website'.
URL: *www.ngu.no*

Nilsen, B. & Broch, E. (2009), *Ingeniørgeologi-berg grunnkurskompendium*, Institutt for geologi og bergmekanikk, NTNU.

Nilsen, B. & Dahlø, T. (1994), 'A study of cases of instability in hard rock tunnels', *7th International IAEG Congress* .

Nord, G. (2006), TBM versus drill and blast, the choice of tunnelling method, *in* 'International Conference and Exhibition on Tunnelling and Trenchless Technology'.

Norsk Standard NS3420 (2008).

NTH-Anleggsdrift (1995), *Prosjektrapport anleggsdrif 2B-95; Tunneldrift, prognoser konvensjonell drilft*, NTNU.

Palmstrøm, A., Nilsen, B., Pedersen, K. & Grundt, L. (2003), 'Publikasjon nr. 101, miljø-og samfunnstjenelige tunneler; riktig omfang av undersøkelser for berganlegg'.

Panthi, K. (2006), Analysis of engineering geological uncertainties related to tunnelling in Himalayan rock mass conditions, PhD thesis, Department of Geology and Mineral Resources Engineering, NTNU.

Panthi, K. & Nilsen, B. (2007), 'Uncertainty analysis of tunnel squeezing for two tunnel cases from Nepal Himalaya', *International Journal of Rock Mechanics and Mining Sciences* **44**, 67–76.

Pompée, P. (2012), *Channel tunnel, project overview*.

Ramamurthy, T., Sharma, K., Gupta, A. & Baweja, M., eds (2010), *Manual on Rock Mechanics*, Central Board of Irrigation and Power.

Ramberg, I., Bryhni, I. & Nøttvedt, A. (2007), *Landet blir til - Norges geologi*, Norsk Geologisk Forening (NGF).

Rocscience (2012), 'Rocscience inc. homepage'.
URL: *www.rocscience.com (15.05.2012)*

Sam (1994), *Håndbok i fjellbolting*.

Sigmond, E. (1985), *Geological map of Norway - scale 1:3 mill.*, Norges Geologiske Undersøkelse (NGU).

Sigmond, E. (2011), 'Oversikt over geologien på Hardangervidda', online.
URL: *www.fylkesdelplan-hardangervidda.no (25.02.2012)*

Sigmond, E. M. O. (1998), *Berggrunnskart ODDA, M 1: 250 000*, Norges Geologiske Undersøkelse (NGU).

Sigmond, E. M. O. (2002), *Geologien på kartbladene Vinje 1514 3, Songavatnet 1414 1, Sæsvatn 1414 2*.

Statens Vegvesen (2010), *Håndbok 021 - Vegtunneler*, Statens Vegvesen Vegdirektoratet.

Stefanussen, W. (1999), Tunnelling in swelling and squeezing ground. M. Sc-course in Hydropower development, NTNU.

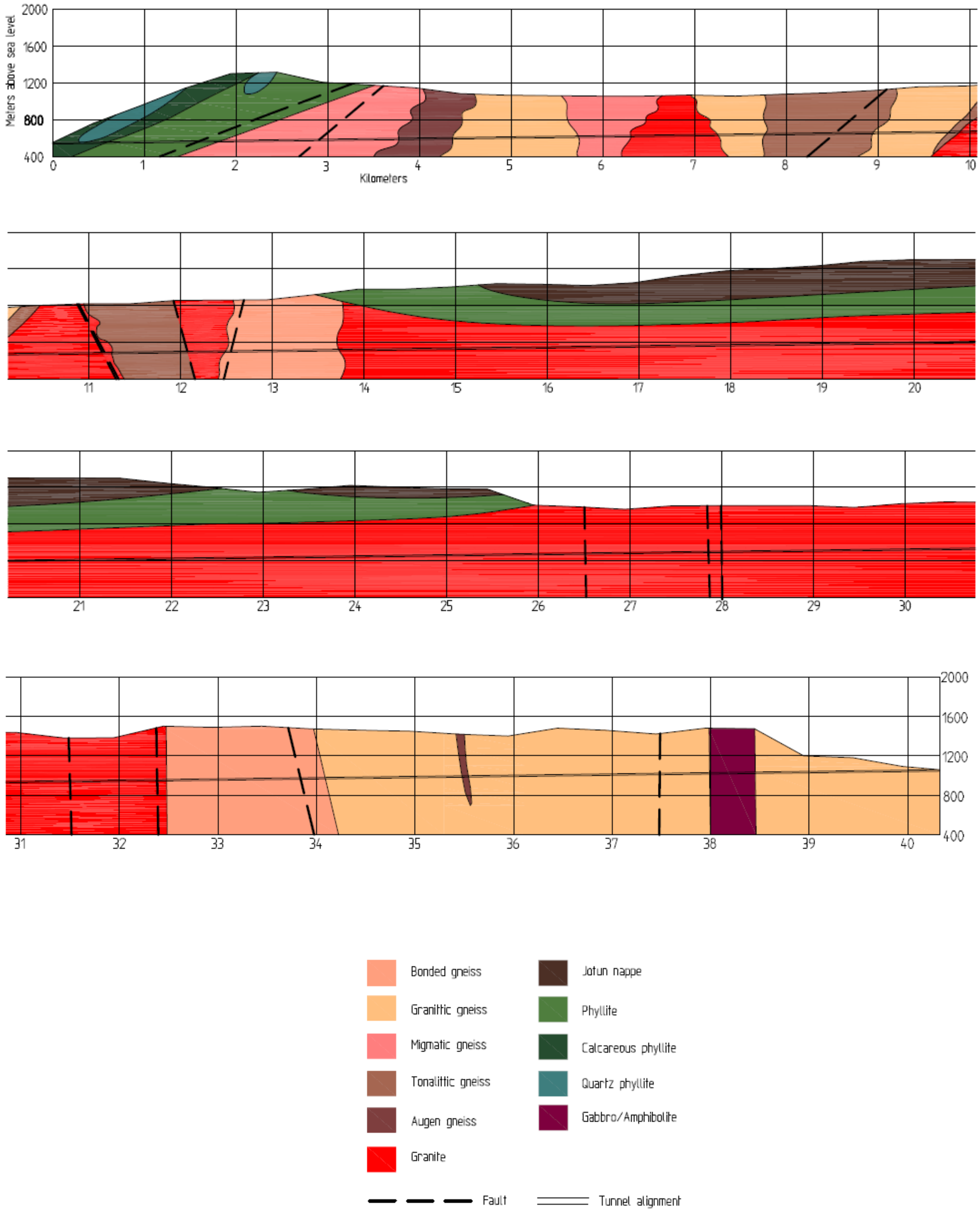
Sweco (2011), 'High speed rail assessment 2010-2012, phase 3 - corridor west', online.
URL: *www.jernbaneverket.no/no/prosjekter/hoyhastighetsutredningen*

Unterschütz, P. (2004), *The New Gotthard Rail Link*, AlpTransit Gotthard Ltd.

Vegdirektoratet (2007), *Håndbok 025 Prosesskode 1 Standard beskrivelsestekster for vegkontrakter, hovedprosess 1-7*, Statens Vegvesen.

Zare, S. (2006), *Report 2C-05 Drill and Blast Tunnelling Costs*, NTNU Department of Civil and Transport Engineering.

Appendix A: Geological profile



Appendix B: Construction time and costs analysis, conventional tunnelling

Table B–1: Support concept tunnel class A, conventional tunneling

Rock support concept tunnel class A		
Rock support concept	Systematic bolting c/c 2m	
	80mm shotcrete	
Number of bolts pr. meter	7	bolts/m
Volume shotcrete pr. meter tunnel	2,9	m3
Probe holes, length 24m, per 20m	4,0	holes/cross section

Table B–2: Support concept tunnel class B, conventional tunneling

Rock support concept tunnel class B		
Rock support concept	Systematic bolting c/c 1,5m	
	150mm shotcrete	
	Spiling bolts c/c 300mm in hanging wall	
	Armed shotcrete ribs c/c 3m, boltet systematically c. 1,5m	
Number of bolts pr. meter	10,4	bolts/m
Volume shotcrete pr. meter tunnel	5,3	m3/m
Number of spiling bolts pr meter	10,0	bolts/m
Armed shotcrete ribs	0,3	rib/m
Probe holes, length 24, per 20m	6,0	holes/cross section

Table B–3: Support concept tunnel class C, conventional tunneling

Rock support concept tunnel class C		
Rock support concept	Systematic bolting c/c 1m	
	200mm shotcrete	
	Spiling bolts c/c 200mm, roof and walls	
	Armed shotcrete ribs c/c 2m, boltet systematically c. 1m	
	Concrete lining in weakness zones	
Number of bolts pr. meter	23,4	bolts/m
Volume shotcrete pr. meter tunnel	7,1	m3/m
Number of spiling bolts pr meter	23,0	bolts/m
Armed shotcrete ribs	0,5	rib/m

Table B–4: Pre-injection grouting concept, class B and C

Pre-injection grouting works			
Tunnel Class	B	C	
Number of injection holes per grout curtain	23	38	holes
Distance between grout curtains	30	15	m
Injection hole length	24	24	m
Injected grout per injection hole	1000	1000	kg
Injected grout per grout curtain	23000	38000	kg

Table B–5: Estimated weekly advance rates for the two main tunnels in Hardangerjøkulen tunnel

	Capacities		Estimated					Estimated total time consumption		Remarks
	Quantity	Unit	Quantity				Unit	Hours	Weeks	
charging, blasting, ventilation, loading hauling and scaling)**			Class A	Class B	Class C	SUM quantity (A+B+C)				
80 m2, standard blasting round	60,2	m/week	18 300	17100	4800	40 200	m		667,8	
SUM excavation elements									**667,8**	
Rock support, probe drilling and injection grouting works at face *										
Spiling bolts(6-8m)	4	bolts/hour	0	171000	110400	281400	bolts	70350	696,5	
Drilling of probe holes (24m)	50	m/hour	87840	123120	0	210960	m	4219	41,8	Incl. in the grout curtain
Drilling of injection holes	60	m/hour		235980	291840	527820	m	8797	87,1	
Injection works (pumping time)	20	hours/curtain		570	320	890	curtains	17800	176,2	
Waiting time for injection works	3	hours/curtain		570	320	890	curtains	2670	26,4	incl. set up and hardening time
Bolts, end anchored	16	bolts/hour	128100	177840	112320	418260	bolts	26141	258,8	
Post-grouting of bolts	20	bolts/hour	128100	177840	112320	418260	hours	20913	207,1	All bolts will be grouted
Shotcrete incl. Cleaning	9	m3/hour	52155	89775	33840	175770	m3	19530	193,4	
Armed shotcrete ribs incl. bolting	0,2	ribs/hour		2850	2400	5250	ribs	26250	259,9	
Concrete lining	0,15	m/hour			200	200	m	1333	13,2	
SUM rock support, probe drilling and injection grouting								**198004**	**1960,4**	
Total working time at face per main tunnel [weeks]									**2628,2**	
Average weekly advance rate per working face, incl. sealing and rock support works [m/week]									**15,3**	
Total construction time per main tube [weeks]									**438,0**	

*Ref. NS 3420-F, vedlegg A, cross section >40 m2

**Ref. Prosjektrapport anleggsdrift 2B-95; Tunneldrift, prognoser konvensjonell drift

Table B–6: Estimated costs for the two main tunnels in Hardangerjøkulen tunnel

	Unit prices		Estimated quantities					SUM [NOK]	Remarks
	Unit	Cost	Quantity				Unit		
Rigging and operation (HP1)	RS	15 %					RS	***571 431 705***	
			Class A	Class B	Class C	SUM quantity (A+B+C)			
Tunnel length			18 300	17 100	4 800	40 200	m		
Tunnel works (HP3)									
Probe drilling and injection grouting									
Probe- and injection holes	NOK/m	90	87 840	359 100	291 840	738 780	m	66 490 200	
Up/down rigging for injection	NOK/curtain	3200	-	570	320	890	curtains	2 848 000	Cement based injection grout, per grout curtain
Delivery of grout including additive components	NOK/kg	3	-	13 110 000	12 160 000	25 270 000	kg	75 810 000	Material costs
Injection works	NOK/hour	3500	-	11 400	6 400	17 800	hours	62 300 000	20 hours/grout curtain
Sum probe drilling and injection grouting								**207 448 200**	
Excavation elements									
Without restrictions, incl. loading and hauling to local depository	NOK/m3	300	1 464 000	1 368 000	384 000	3 216 000	m3	964 800 000	Standard round cycle
Sum excavation elements								***964 800 000***	
Tunnel rock support									
Rock bolts, length 4m	NOK/bolt	700	128100	177840	112320	418 260	bolts	292 782 000	End anchored, tensioned and fully grouted
Spiling bolts at face	NOK/bolt	1500	-	171 000	110 400	281 400	bolts	422 100 000	Length 6 m, ø32mm
Shotcrete	NOK/m3	3500	52 155	89 775	33 840	175 770	m3	615 195 000	Fibre reinforced shotcrete
Armed shotcrete ribs	NOK/rib	85830	-	2 850	2 400	5 250	ribs	450 607 500	
Concrete lining	NOK/m	50000	-	-	200	200	m	10 000 000	
Sum tunnel rock support								***1 790 684 500***	
Water and frost control (HP34)	NOK/m2	900	428 220	400 140	112 320	940 680	m2	***846 612 000***	
Unforeseen costs	RS	15 %						***571 431 705***	
TOTAL COSTS	**Sum costs incl. rigging, operation and unforeseen costs [NOK]**							**4 952 408 110**	
	Total cost [NOK/m]							**123 194**	
	TOTAL COSTS PER ROUTE KM INCLUDING BOTH MAIN TUNNELS [NOK/km]							**246 388 463**	
	TOTAL COSTS INCLUDING BOTH MAIN TUNNELS [NOK]							**9 904 816 220**	

Appendix C: Construction time and costs analysis, TBM

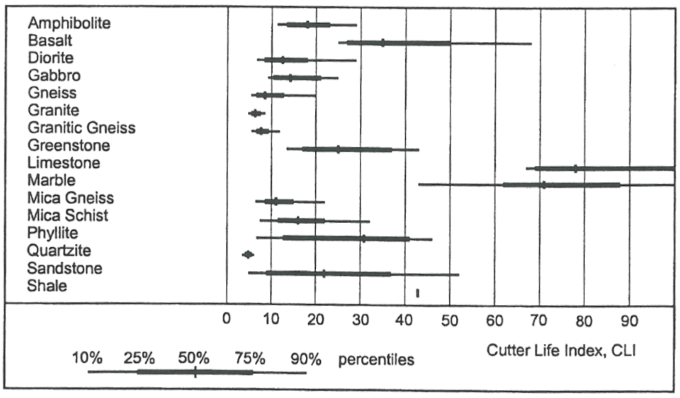


Figure C–1: Registered CLI for different rock types (Bruland 2002).

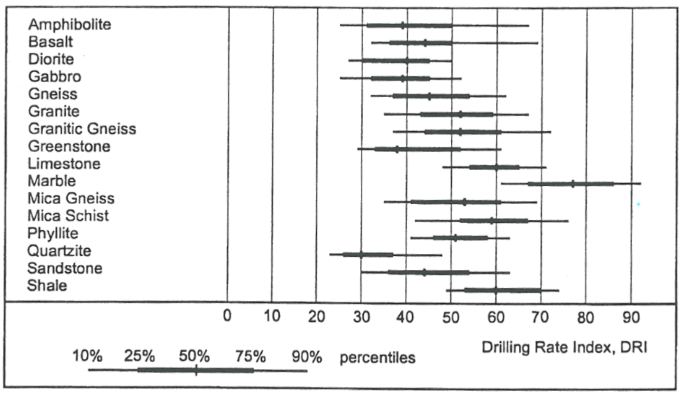


Figure C–2: Registered DRI for different rock types (Bruland 2002).

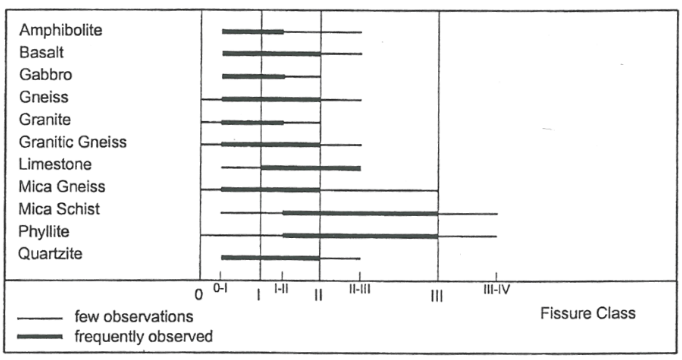


Figure C–3: Registered degree of jointing for different rock types (Bruland 2002).

Table C–1: Estimated construction costs using TBM. The costs are based on the assumption of 2 adits and three tunnel sections à 13 400 m.

	Unit	Quantity	Unit price	SUM
Rigging for tunnel	RS		15 %	213 180 600
Tunnel boring	NOK/m	13 400	30 100	403 340 000
Rock support	NOK/m	13 400	160	2 144 000
Injection grouting	NOK/m	13 400	300	4 020 000
Concrete lining	NOK/m	13 400	74 000	991 600 000
Water and frost control	NOK/m	13 400	1 500	20 100 000
Unforseen	RS		10 %	142 120 400
Total costs per tunnel section [NOK]				1 776 505 000
Total costs [NOK/m]				**132 575**
Total costs incl both main tubes [NOK/km]				**265 150 000**
Total costs incl both main tubes [NOK]				**10 659 030 000**