THE PRINCIPLES OF NORWEGIAN TUNNELLING

Photo: Gunnar Kopperud

NORWEGIAN TUNNELLING SOCIETY

PUBLICATION NO. 26

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DISCLAIMER

FOREWORD

The Norwegian Tunnelling Society NFF is publishing this issue No. 26 in the English Language series for the purpose of sharing with international colleagues, and friends, the experiences of tunnel and cavern construction along with examples of underground use.

The purpose of Publication No. 26 is to demonstrate the content of Norwegian tunnelling by clarifying the principles of this approach, show-case breakthrough technologies and describe various ways of utilising the underground. We have named the publication 'The Principles of Norwegian Tunnelling'.

The current state-of-the-art is not necessarily the result of primarily favourable ground conditions. More so it is the result of a continuous development started by the mining industry 300 years ago, coupled with clear understanding of rock mass behaviour, brave and solution-oriented workers and engineers cooperation and mutual respect throughout the work site, to mention some important elements. This publication aims at telling more of this story.

In June 2017 the World Tunnel Conference and 43rd ITA General Assembly takes place in Bergen, Norway. The Norwegian Tunnelling Society is proud to be the host for this year's WTC, which is arranged under the slogan "Surface Challenges - Underground Solutions." The slogan coincides with that of the Norwegian Tunnelling Society itself.

On behalf of NFF we express our sincere thanks to the authors and the contributors to this publication. Without their efforts this distribution of Norwegian tunnelling experience would not have been possible. We sincerely hope that by reading the publication you will find useful information and maybe some good suggestions for use in your own projects.

Enjoy the reading!

Oslo May 2017

Norwegian Tunnelling Society, NFF - International Committee

Editorial Committee
Arnulf Hansen *Knut Garshol* *Rolf Blomberg* *Eivind Grøv (Chair)*

CONTENTS

WG TUNNEL SEALING

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01. INTRODUCTION

The Norwegian Tunnelling Society (NFF) has the great pleasure of hosting the World Tunnel Conference and the 43rd General Assembly of ITA, the International Tunnelling and Underground Space Association in Bergen in June 2017. The last time Norway was the host was in Oslo in 1999, and before that again it was in Oslo 1974 which was actually the inauguration of the series of annual WTC and General Assemblies. Norway is proud of its contribution to the international tunnelling industry and its communities in more than 50 member nations today.

In conjunction with the WTC2017 the NFF would like to demonstrate to the international tunnelling industry the different ways that underground space has been utilized in Norway since the 2nd World War ended in 1945. Tunnelling and underground space has been an important element in developing Norway to a modern and rich nation. This publication will be the 26th in the series of English language publications issued by the Norwegian Tunnelling Society and to some extent it summarizes the Norwegian tunnelling during the last 60 years.

The authors and the entire tunnelling industry in Norway wishes that this publication becomes a document that is worth while to be kept in the book shelves of members of the international and global tunnelling community. We do hope that the hard back becomes loose and that pages become creased and wrinkled following intense use. Our main goal with this publication as has been the case for all the previous ones; namely to communicate and share competence amongst the great variety of tunnelling activities globally. And hopefully YOU are also able to find something for your project, for your particular use or needs. By that we appreciate this publication as a useful one.

The World Tunnel Conference in Bergen is expected to welcome around 1500 participants and the General Assembly will encompass members from more than 50 member nations of ITA. The event has grown to become the most influencing conference and exhibition in the world of tunnelling and underground space. The Norwegian Tunnelling Society has delivered annually the last 15 years one of this English language publications bring these a unique topic every year representing the fore front of tunnelling activities in Norway. We do hope that many of the tunnelling professionals have one or more examples of these publications and that the spread goes beyond all sorts of barriers that may exist.

The Objective of Publication no. 26:
The objective of this publication is to describe the principles of Norwegian tunnelling as it has developed during the years after the Second World War when Norway transformed from an agricultural and fishing society into an industry-based nation which later on additionally developed into an oil and gas producer. One important element in this transition was the need of energy in terms of electrical power that in Norway was developed based on Hydropower. Norway is rich in high altitude water magazine opportunities and at the same time mother nature is generous with rain in the same areas, these are keys to develop Hydropower. To bring the water from a high altitude reservoir to the turbine was considered as a low cost alternative through tunnels, as steel and steel pipes in those days were expensive assets. And often the distance from the magazine to the turbine and from the turbine to the outlet were several kms or maybe even tens of kms. Tunnelling and underground space became a facilitating element in this transition. Several thousand km of tunnels were constructed and many hundred caverns to host power and transformer stations were built pushing the technology forward. And yes, the tunnelling industry today owes these pioneers and entrepreneurs a great deal for paving the way in tunnelling technology.

The objectives of the principles of Norwegian Tunnelling:
The main benefit of Norwegian tunneling is fast and safe tunnel excavation at affordable cost; or put in other words; time and cost efficient tunneling while maintaining excellent work safety, and high final quality without compromising required operational standard and design lifetime.

Another element is adaptation to actual ground conditions and follow-up of encountered rock mass and its behaviour in order to install best-suited rock reinforcement, whenever possible based on state-of-the- art technologies for sprayed concrete and rock bolts. Pre-grouting is the main method in controlling ground water. The overall approach and execution are associated with quality, cooperation, experience and innovation. These are key words in describing tunnelling and working underground in 2017. And a lot has changed since the pioneers and entrepreneurs of the 60'ies, 70'ies and so on when muscles counted more than computer added design, health and safety was an unknown topic and hand held pneumatic drill units were used.

In the following the history of underground activities in Norway is described. Then the elements that constitute the principles of Norwegian Tunnelling are contained in a separate chapter followed by some technological moon-landings. Finally a selection of projects and project types are disclosed. We do hope that you find this a useful taste of Norwegian tunnelling.

From our Reputation Programme:

Norwegian tunnelling
– pioneers in tailored solutions

The Norwegian tunnelling industry has built tunnels and underground facilities for more than 100 years. A rugged geography and the huge challenges associated with it have inspired us to solve even the toughest problems. Our recipe consists of thorough pre-investigations, long experience and advanced technological methods. This has made us one of the pioneers of the global tunnelling industry. First. Longest. Largest. Deepest. Always with a focus on safe solutions.

Much of our success can be attributed to ”the Norwegian way of working”. Well-proven technology, short decision-making lines and experts on site allow us to deliver even the most complex of jobs efficiently and effectively. We are an experienced bunch who deliver complete solutions. Population growth, extreme weather, economic progress and an increased need for safety pose enormous challenges for countries in Asia. Roads. Railways. Water. Energy. Rock installations. Flood control. These are all projects of vital importance to society. Surface areas are small, and project complexity great. Safety requirements are absolute.

You can rely fully on a partner with recent experience from such unique, ground breaking projects.

Quality - Innovation - Experience - Cooperation

02. INTRODUCTION TO THE HISTORY OF NORWEGIAN TUNNELLING

2.1 STATISTICS OF NORWEGIAN TUNNELLING AND DEVELOPMENT OF THE INDUSTRY

High mountains, long fjords and steep valleys. A demanding landscape and tough climate with abundant precipitation meaning infrastructure construction in Norway is a severe challenge. This became all the more clear as road and rail started to replace the sea links that were the most important communications in the previous centuries. Yet the landscape also holds great opportunities. Many hydroelectric projects in the second half of the 20th century formed the industrialized Norway.

The challenges of the topography have inspired Norwegian tunnelling engineers to become pioneers in management of projects that demand carefully tailored solutions. Norwegian tunnelling engineers have continuously looked for new methods, improved working procedures, new machinery and equipment and learned from colleagues within both the national and the international tunnelling community.

The tunnelling industry in Norway started in the 16th century in connection with increased metal mining activities, but tunnelling for hydropower, railways, roads and water supply/sewage projects brought the industry to the current level. Tunnelling in Norway includes the 24km long Lærdal tunnel, opened in 2000 and the railway link between Oslo and Bergen with 184 tunnels opened for traffic in 1909. Subsea tunnels are used for links to islands and crossing under fjords.

Norwegian tunnelling owes its origins to the 17th century mining. For centuries the mining industry formed the backbone of the Norwegian economy. Skills and experience gained in these mines with their cavernous spaces and complex geometry were important qualifications for Norway's many Hydro Power developments in the 20th century.

From an engineering geological point of view, Norway may be described as a typical hard rock province. The rock mass has been subjected to folding and faulting, which may have a great influence on the stability in tunnels and underground openings. Another complicating factor is the irregular stresses in the rock mass, caused by tectonic events and further resulting from the steep and irregular topography. Also high tectonic and residual stresses are encountered. The host rock is more or less intersected by weak zones, which may have an intense tectonic jointing, hydro-thermal alteration, or be faulted and sheared, constituting significant weaknesses in the rock and making the rock mass far from homogenous. These conditions may require rock strengthening measures.

The construction sector is among Norway's largest industries and the principles of Norwegian tunnelling has developed during the years following the 2nd World War when Norway transformed from agriculture and fisheries into an industry based nation which later on additionally developed into an oil and gas producer. One important element in this transition was the need of energy in terms of electrical power that in Norway was developed based on Hydropower. Norway is rich in high altitude water magazine opportunities and at the same time mother nature is generous with rain in the same areas, these are keys to develop Hydropower. To bring the water from a high altitude reservoir to the turbine was considered as a low cost alternative through (unlined) tunnels, as steel and steel pipes in those days were expensive assets. And often the distance from the magazine to the turbine and from the turbine to the outlet were several kms or maybe even tens of kms. Tunnelling and underground space became a facilitating element in this transition. Several thousand km of tunnels were constructed and many hundred caverns to host power and transformer stations were built pushing the technology forward.

The Norwegian tunnelling society has a tunnelling statistcs that dates back to 1971. The latest version is provided below which includes the years up to the end of 2016, from 1971. The trend is quite obvious looking at the different colours that represent various sector of industries or activities that are utilising the underground.

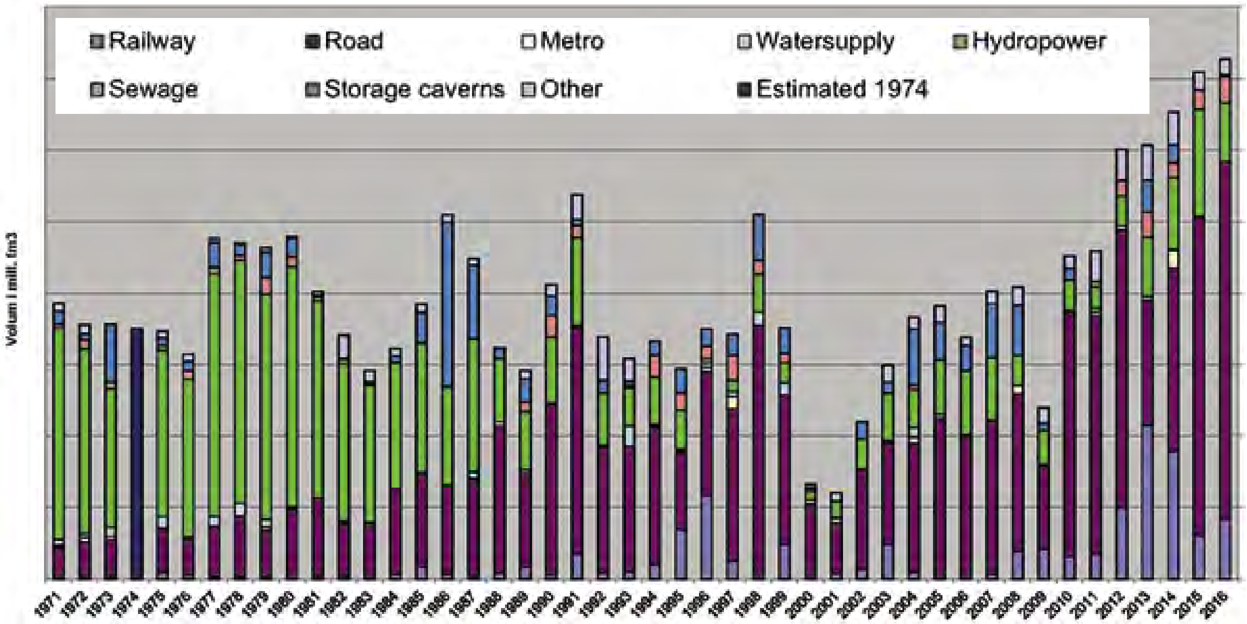


Figure 1. Development of Norwegian underground works (fm3=solid state on the vertical scale), a rough estimate suggests that every citizen in Norway has about 2 meters of tunnel. From the Norwegian Tunnelling Socity

In the graph above each horisontal line represents one mill m3 of solid rock, that means the record breaking year of 2016 reached marginally above 7 mill m3 solid rock in one single calendar year, whilst average annual production is around 4 mill3 solid rock during the years that the statistics has been collected. This does not include the mining industry which alone contributes with around 10mill m3 of solid rock on an annual basis.

The importance of the tunnelling industry for hydroelectric power development shows clearly in Figure 1 as the green coloured columns. Tunnels and underground caverns for the hydrolectric power development dominated into the eighties. Road tunnels were fading in as the hydropower development faded out and from the late eighties or early nineties road tunnels were dominating the tunnelling industry in Norway. Smaller in scale, but still a significant portion in some years is caverns and tunnels for the oil and gas industry, tunnels for water supply and sewage constitute a small portion. The darker blue colour is tunnels for rail way construction, this had a rather uneven use for many years, but grew significantly around 2012 and was the dominating use for tunnels in one year, 2103. Since 2013 road tunnels have been dominating again.

For the tunnelling technology and industry such long term high activity is a key in maintaining and building competence steady over several decades of continuous activity within the industry. This brings a stable work load, predictability for suppliers, consultants and contractors, owners with a long term perspective and a academia that provides education to a sustainable industry.

At present a rough statistics of Norwegian tunneling suggests the following rough figures reflecting the statistics in Figure 1:

- About 750 railway tunnels, 1000 road tunnels, 35 subsea tunnels, in total almost 3000km in length
- World`s longest road tunnel in Lærdal, 24.5 km long
- Of the world's 500 - 600 underground powerhouses 250 are located in Norway, > 4000 km of hydropower tunnels
- Some 60 unlined caverns for oil and gas storage (both chilled and pressurised)
- Clean water conveyance & storage and sewage water transport & cleaning in tunnels and caverns in all major cities
- Numerous civil defence and sports caverns, culminating with the Gjøvik hall
- TBM tunnelling 260km of mainly hydroelectric power tunnels leading to HP-machines with 32tons of trust and 19" cutters
- Wet mix sprayed concrete, high pressure rock mass grouting, risk sharing contract forms with equivalent time system to regulate time
- Some of the technologies developed during the development of Norwegian tunnelling industry remain state-of the-art internationally also.

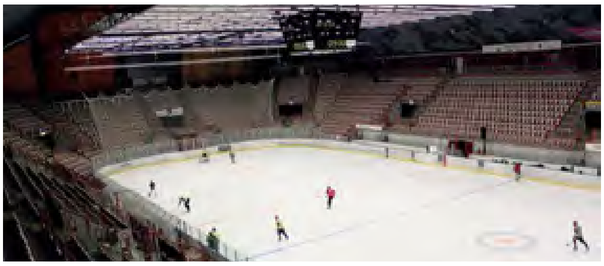
In the following table please find some historical highlights in the development of Norwegian tunnelling, not necessarily absolutely covering all and every aspect of this.

Year	Description
1623	Royal silver mines in Kongsberg, once Norway's largest enterprise, with more than 1000 km of mine shafts.
1882	World's very first hydroelectric plant in Senja.
1909	Bergen Railway connects eastern Norway with the west coast through 182 tunnels.
1953	Lyse Hydro Electric Power plant, the first in Norway to use an unlined shaft
1965	Norway was among the first to use and develop the wet-mix sprayed concrete.
1974	NGI's Q-System of rock classification developed at Norwegian Geotechnical Institute.
1983	Vardø sub sea road tunnel, Norway's first sub sea road tunnel 2890 meters long
1994	Gjøvik Olympic Mountain Hall, world largest cavern span 61 meters
1995	The Troll shore approch Tunnels with piercing at 175 metres below sea-level, for export/import of oil and gas.
2006	New Tyin Hydro Electric Power plant breaks 1000 m height limit for an unlined headrace.
2008	Eiksund, the world's deepest sub sea tunnel at 287 mbsl and 7765 meters long

These are some of the benchmarks in the history of Norwegian tunnelling, however as in the sports, world records are to be beaten and benchmarks are moved onwards continuously. However, those benchmarks that were first of its kind will remind to stay, like Neil Armstrong when he in 1969 became the first man on the surface of the moon, and some of the tunnelling benchmarks in Norway are first of their kind.

Norwegian tunnelers have a long series of pioneering projects on the drawing board. Here are a few of them:

Unique feature	Description
First	Stad Ship Tunnel - the world's first ship tunnel may commence in 2018. With a section of 1620 m2 and length of 1.7 km, it will offer safe passage through a notoriously exposed piece of coast with many shipwrecks.
Longest	Solbakk Tunnel, as a part of the Ryfast connection - the world's longest subsea road tunnel will be 14.3 km long when completed in 2018.



Gjøvik Mountain Hall

Vardø tunnel first sub sea road tunnel

Deepest	In Helgeland, the world's deepest road tunnel is being planned to descend 396 m.b.s, more than 100 metres below any other road tunnel in the world today.
Longest and deepest	Rogfast Road Link in Rogaland, when it opens for traffic in 2023, will then be the world's longest sub-sea road tunnel at 27 km, and among the world's deepest at 390 metres.
Norway's longest rail tunnel	The Follo double-tube Railway tunnel will be Norway's longest rail tunnel when completed in 2021. Most of the 20 km tunnels will be excavated by TBMs.

The Government of Norway aims to develop a modern transport system that will make traffic flow easier, faster and safer. According to the National Transport Plan for the years 2014 to 2023 a great number of projects are in the pipeline and there good chances that both railway and road projects will secure that these kinds of tunnels will be dominating the tunnel statistics in Norway in the future also.

Continue reading this publication that you have at hand and many of these projects, technical developments, bench marking projects and concepts will be further explained and detailed. The tunnelling industry in Norway has a bright future, a foresight that coincides with the future of tunnelling world wide. The solution for a better future for all people, a lower carbon foot print for the environment and thousands of jobs is under ground.

Photos and tables: Folder by the Norwegian Tunnelling Society on Surface problems – underground solutions.


Dynamic and accurate tunneling
No matter what your challenge is: A small tunnel or a large tunnel, a hydroelectric plant or a subway line, a utility tunnel for heating pipes or for any other purpose. We are specialists in underground tunnels!
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2.2 FROM FIRE SETTING TO FULL-FACE TUNNEL BORING MACHINES – THE NORWGIAN ROCK BLASTING MUSEUM (NFSM)

SUMMARY
The history of Norwegian rock blasting and tunnelling from the 17th century up to modern times is presented at the Norwegian Rock Blasting Museum (NFSM). The museum is located at Hunderfossen in the northern part of Lillehammer, approximately 200km north of Oslo. Planning of the museum started early 1990, and the museum was officially opened on June 19th 2004.

NFSM is unique in an international context. NFSM is a culture carrier that spreads construction history based on proud traditions. It takes you on a historical journey and shows the development of an industry characterized by solution-oriented attitude, hard work and excellent teamwork. The museum shows how we have contributed in building the country with industry, power plants, roads, railways and other infrastructure.

NFSM has a great potential and wants to be an arena for marketing and recruiting. It is well suited for different events like conferences, company meetings, seminars and networking.

THE HISTORY
The history started in January 29th 1990, a committee was appointed in the Hydropower Company Statkraft to identify plans for securing technical construction history. It was established an interim board on June 14th of that year. Relevant companies were invited to form a foundation and enthusiasts within the Norwegian Rock Blasting and Tunnelling Society NFF, started to collect equipment from different construction sites around the country.

Planning of the museum started in August 1991, and different companies within the Norwegian construction business were invited to participate. A foundation was established the 31. August 1992 with the following members:

- Berdal Strømme AS
- Dyno Industrier AS
- Eeg-Henriksen Anlegg AS
- Grøner Anlegg Miljø AS
- Norges Geotekniske Institutt
- Norsk Arbeidsmandsforbund
- Norsk Forening for Fjellsprengningsteknikk
- NSB Banedivisjonen
- Selmer ASA
- Statkraft
- Statens vegvesen
- Veidekke ASA

The process had started. Later on, Atlas Copco Gruveteknikk and AF Gruppen ASA became members and joined the team.

From the founders, an enormous effort was established on voluntary basis. Plans were prepared, and the timing was perfect. Released capacity from construction works towards the Olympic Winter Games at Lillehammer in 1994 was mobilized. Equipment and personnel available from the OL construction works were brought to the site at Hunderfossen. Supporters with political influence, not at least from the Labour Union and the Norwegian Public Road Administration (NPRA), were able to release funds from the Norwegian Government, and additional funds were made available from the Norwegian Hydropower Company, Statkraft.

Tunneling and rock excavation were performed by the construction companies Selmer ASA (now Skanska), Eeg-Henriksen Anlegg (now NCC) and Veidekke ASA. The tunnel and rock cavern were completed in 1993.

The completion of the museum, with minor rock excavations, rock support, civil works and installations, were performed by AF -Gruppen, Statkraft Anlegg AS and NPRA in cooperation. The Norwegian Rock Blasting and Tunneling Society, NFF, contributed with collection of equipment from different periods of the Norwegian rock blasting and tunneling history.

Thanks to the contributors and the enormous voluntary based effort from companies and enthusiasts, the museum was ready for official opening the 19th June 2004.

NFSM TODAY
At the opening in 2004, the Rock Blasting Museum was handed over to the Norwegian Road Museum (NVM), who is responsible for the daily operation and maintenance. The road museum get assistance with knowledge, equipment, voluntary effort and expertise from the following active board member companies of the NFSM Association:

- AF Gruppen AS
- Andersen Mekaniske Verksted AS, AMV
- Atlas Copco Anlegg og Gruveteknikk AS
- Bane NOR
- Entreprenørservice AS
- Hæhre Entreprenør AS
- Implenia
- Leonhard Nilsen & Sønner AS, LNS

- Multiconsult AS
- NCC Construction AS
- Norconsult AS
- NORMET
- Norsk Forening for Fjellsprengningsteknikk, NFF
- Norges Geotekniske Institutt, NGI
- Norsk Arbeidsmandsforbund, the Norwegian Labour Union
- Orica Norway AS
- Skanska Norge AS
- Statens vegvesen, the Norwegian Public Road Administration, NPRA
- Statkraft
- Sweco Norge AS
- Veidekke Entreprenør AS

This is an exclusive group, contributing with enthusiasm and considerable effort for operation and further development of the museum. The association is open for new members who are interested to contribute.

Board members of the NFSM Association are exposed with flags in the open area of the museum

The museum brings you through the history of tunneling and rock blasting, starting with fire-setting in the Kongsberg silver mines in 1623, construction of the railway line between Oslo and Bergen, opened in 1909 (with a cost equal to the entire national budget by that time), and construction of hydropower plants from the 1950'ies. Centennials of mining and construction of hydropower plants, roads, railways and other infrastructure through a landscape with challenging topography, has brought Norway in a forefront of conventional tunneling and rock excavation. Pioneering work, executed by devoted labour force and staff has developed and improved the expertise, skill and technology. Development of conventional tunneling and underground rock excavation is presented in a 240m long semicircular tunnel.

From the interior of the exhibition tunnel of the museum

Tunnelers at work at the tunnel face, Tokke Hydropower Project, and drilling equipment from early 1950'ies

Railbound loader for small cross sections

The open-air part of the museum shows equipment for open pit mining. Here, the biggest attractions of the museum are presented, the near 220 ton O&K loader, donated from the mining company Titania in 2013, and the 160 ton dumper truck donated by Sydvaranger Gruber. Another great donation, are the barracks from the E16 road project, donated by the Norwegian Public Road Administration, NPRA, transported to and installed by the contractor LNS. The barracks represent the future exhibition center at NFSM.

The near 220 ton O&K loader and the 160 ton Varanger dumper

Barracks from the E16 Project, the future exhibition hall

As part of the tunnel system at NFSM is the great Rock Cavern with restaurant facilities, ideal for social arrangements, company gatherings, meetings, conferences and seminars. The rock cavern has a capacity for hosting 200 persons. In 2014, the Norwegian Public Road Administration, celebrated its 150 years of operation, the same year NFSM its 10 year celebration and every year, the Association of NFSM has its General Assembly in the rock cavern. Besides, there have been several private and official arrangements and celebrations in the rock cavern.

NFSM AND ITS OPPORTUNITIES

NFSM has a great potential and many opportunities:

- NFSM is unique in international context
- The museum brings you on a journey through an interesting and unique construction history with proud traditions based on creative and solution-oriented attitude, hard and skilled workmanship and a good teamwork
- The museum shows how the rock blasting and tunnel construction business has contributed in building the country with industry, hydropower, roads, railways and other infrastructure
- NFSM is an arena for marketing and recruiting
- NFSM is an arena for network building
- NFSM is an arena for private and official events, courses, conferences and other arrangements

Table in the Rock Cavern ready for the guests

NFSM is located at Hunderfossen in the northern part of Lillehammer 200km north of Oslo, and is operated by the Norwegian Road Museum (NVM). The museum is normally open in the summer period from May throughout September. Groups are admitted off-season by appointment.

Young boys playing with an old dozer. Future blasters and tunnelers?

Every year, NVM arranges the «Family Day» at the museum. No doubt, this arrangement is a great source for inspiration among visitors, and also for recruitment to the rock blasting and tunneling business among young people. NFSM presents what the rock blasting and underground construction business is dealing with.

The Norwegian Rock Blasting Museum with the rock cavern, equipment and installations are unique in international context. With its special atmosphere, the Rock Cavern is excellent for social arrangements and events. For further information, please contact www.vegmuseum.no.

03. ELEMENTS THAT CONSTITUTE THE PRINCIPLES OF NORWEGIAN TUNNELLING

3.1 PRE-INVESTIGATIONS, ROCK MASS CLASSIFICATION AND LINING SOLUTIONS

3.1.1 Pre-investigations

3.1.1.1 Introduction

For any type of underground project, pre-construction investigations of high quality, well adapted to the geological conditions and the project characteristics are crucial. If the investigations are insufficient or inadequate, unexpected and in worst case uncontrollable ground conditions may be encountered, and poor quality and high cost will often be the result for the completed project.

Pre-construction investigation, often simply called pre-investigation, is therefore very important for evaluating the feasibility of the project and for planning and design. Among many other good reasons to focus on pre-investigation, the following outcomes are particularly important:

- Gives basis for analyzing stability and estimating rock support requirement.
- Provides input for evaluating alternative tunnelling methods and selecting equipment/tools for excavation and rock support.
- Provides a basis for predicting performance and capacities.
- Provides a basis for estimating time schedule and cost.
- Is important for assessing potential environmental impacts.
- Gives a basis for preparing tender documents.

If the pre-investigations are insufficient or of poor quality, reports and tender documents will not reflect a correct picture of the actual geological conditions. Conflict between contractor and owner due to "unforeseen geological conditions" will very easily be the result and in worst case the project may end up in court with more time lost and extra cost. Proper investigation is therefore very important for all aspects of the project.

The rock mass as material is in many ways complex and quite different from other construction materials such as steel and concrete. The rock mass is inhomogeneous and in many cases anisotropic, it contains complex structures such as folding and faults, and other factors such as rock stresses and groundwater are also strongly influencing the conditions. In addition, the planned project is located underground, while the pre-investigations mainly have to be carried out from the surface. This means that interpretation is required for estimating the conditions at the level of the planned underground project. Estimation of rock mass conditions based at the pre-construction stage is therefore often a difficult task, and experience is very important for a good result.

The engineering geological factors that need to be investigated for a planned underground project are mainly:

- Soil cover, particularly for portal areas and sections of potentially insufficient rock cover.
- Bedrock, with particular emphasis on rock type boundaries and mechanical character.
- Fracturing of the various rock types.
- Faults/weakness zones.
- Groundwater conditions.
- Rock stress conditions.
- Mechanical properties of rocks and potential gouge materials.

3.1.1.2 Investigation stages

Normally, the investigations are carried out in a stepwise procedure and linked with the progress of engineering design. The general ground investigation procedure for tunnels and underground excavations in Norway is illustrated in Table 1.

Pre-construction			During construction	During operation
Project conception	Feasibility study	Detail investigation		
- Basic knowledge of ground conditions	-Desk study of maps, aerial photos, reports -Field investigation of key points -Visit to nearby excavations	-Eng.geol. mapping -Geophysical investigations -Drilling -Sampling -Lab. testing	-Tunnel mapping Probe drilling -Monitoring (rock stress, convergence etc. -Sampling -Lab. testing	-Monitoring (extensometer etc.) -Quality control
=> Recognition major challenges	=> Preliminary design	=> "Final design"	=> Modification of design	=> Maintenance

Table 1. Main steps used for ground investigations for tunnels and underground openings.

The Norwegian Public Roads Administration (NPRA) is using a pre-investigation procedure based on four stages for planning and design of road tunnels (NPRA, 2010):
1. Feasibility stage, to provide the geological basis for evaluating the feasibility of the project.
2 .Overview plan ("oversiktsplan"), to give the geological basis for selection of alignment alternative. Cost to be evaluated within an accuracy of ± 25 %.
3. Zoning plan ("reguleringsplan"), to provide the basis for planning of the final alternative and the basis for estimating quantities. Cost to be evaluated within an accuracy of ± 10 %.
4. Tender documentation, including supplementary investigations, if required, for producing the tender documents.

The investigations, as shown in Table 1, are basically divided into two main stages, and are followed-up after completion of the project:
- Pre-construction phase investigations, or pre-investigations.
  Underground excavation has not yet started and information has to be collected on or from the surface.
- Construction phase investigations or post-investigations.
  As tunnels are excavated, the underground becomes accessible for inspection and sampling.
- Investigation and control during operation
  Surveillance and control of the completed project.

The pre-construction investigations can be divided into sub-stages as shown in Table 1, or as described for road tunnels above. Reports are written for each stage of the investigations, for large and complex projects normally several reports for each stage. In the following, the various investigation stages will be discussed in more detail.

3.1.1.2.1 Feasibility investigations
This initial stage is normally based on the designer's project conception study. The aim is to study the feasibility of the planned project, or to evaluate and reduce the number of alternatives based on available engineering geological information. This is in many cases very challenging. Important decisions have to be taken, often based on limited information. Experience from similar projects and sites may be very valuable here.

At this early stage, desk studies of available geological information, such as reports, geological and topographical maps (scale 1:5000) and aerial photos (scale 1:15000-1:30000) are carried out. During the following walk-over survey, certain key points of the actual area are investigated. Rock sampling for simple classification tests is often also done.

In the feasibility report, all collected information is presented and the different alternatives discussed. Plans and cost estimates for further investigations are presented, and any need for supplementary maps are made known. At this stage, an important decision has to be made as to whether or not to follow up with more expensive investigations.

3.1.1.2.2 Detail investigations
Based on the feasibility study report the client, often in co-operation with consultants, has to decide whether or not further planning should be carried out, and if so, what alternatives should be investigated. Additional air photos and better maps than used at the previous stage may be required. The engineering geologist normally needs air photos and maps that cover a larger area than is strictly necessary for the other planning purposes of the project.

The air photos and maps for the detailed investigation should be on a scale that is relevant for the actual problem. Air photos to scale 1:5000-1:15000 and maps to scale 1:1000 or 1:5000 are recommended as a basis. For important areas such as tunnel entrances, cavern locations and dam sites, maps of even larger scale are recommended.

At this stage of the investigations, a detailed engineering geological field mapping is carried out. The goal of this mapping should be to collect information about all factors that may cause difficulties for the project.

The results of the detailed surface investigations are collected in a detailed investigation report, which is often included as part of the tender documents. This report contains engineering geological descriptions, evaluations of construction and stability problems in the different parts of the project and an estimation of required rock support.

3.1.1.2.3 Construction stage investigations
During planning of underground projects, important decisions have to be taken regarding which investigations should be carried out before the start of excavation, and which may alternatively be postponed. When excavation has started and the tunnel can be entered, the possibilities of obtaining more and better information on the ground conditions improve considerably.

A high degree of flexibility and simple pre-investigations are recommended when it is possible to start construction phase investigations early in the construction period. Expensive pre-investigations, such as deep core drillings, may in many cases be replaced by cheaper probe drilling from the tunnel face during construction. Rock stress measurement is best done from underground openings, and is a good example of detailed investigation that may often be postponed until tunneling has started.

Detailed sub-surface investigation of course is not only delayed pre-investigation, but also a control and supplement of the pre-investigation. For underground works the pre-investigation report always has to be based on a certain degree of assumption. The sooner pre-investigation results are verified, the better the prognosis will be for the remaining part of the underground works.

3.1.1.3 Pre-construction investigation methods
The geological conditions of different sites may vary within wide limits. Each site has its own characteristics, and there is therefore no "standard investigation procedure" which will be the right one in all cases. When it comes to engineering geological investigations, flexibility is a keyword.

Many different investigation methods may be relevant for planning of underground excavations and the most common for methods for the pre-construction stage are:
- Desk studies, inspection of nearby excavations, site mapping, refraction seismic measurements, core drilling, seismic tomography, geo-electric methods, rock stress measurements and laboratory testing.

Since many field tests are quite expensive, their value should always be carefully weighed against their cost. If the geological conditions and/or the project design are complex, it is however never a good idea to try to save money by reducing the extent of ground investigation.

3.1.1.3.1 Desk studies
A lot of valuable information can be obtained already from a desk study, and spending time at this early stage of investigation on collecting, systematising and studying relevant background material such as topographical and geological maps, aerial photos and geological reports is generally a very good investment.

Good quality geological maps, like the ones in scale 1:50000 produced by the geological survey of Norway (NGU) gives a lot of useful information regarding the geological conditions, and makes planning of supplementary mapping much easier.

In regions which have been affected by glaciation like Scandinavia, including Norway, aerial photographs are particularly useful for identifying faults and weakness zones. Because the soil cover is in many cases very thin or non-existent and such zones have been eroded by glaciers and flowing water, they are often easily detected on a stereo-pair of aerial photographs due to the exaggerated vertical scale. Even a non-stereographic aerial photo shown in Figure 1 and satelite photos may provide very useful information for evaluation of the geology during early planning of the project.

Figure 1. Aerial photo illustrating the locations of several large faults in the area of a planned tunnel (scale approx. 1:15000).

Available engineering geological information from any previous underground excavation in the same area may be of great value, and should of course be included in the desk study. Particularly for urban tunnels such information is often available, and may provide a lot of useful data and experience for planning of new projects.

3.1.1.3.2 Field mapping

Field mapping is a very important part of the investigations, and is based on using simple tools like a geological compass, hammer, GPS and a notebook. The planning of field mapping is based on the results from desk study and during mapping, particular emphasis is placed on following factors:

- Rock type distribution/boundaries and mechanical character of the respective rock types
- Soil cover and weathering, if relevant.
- Joint orientation (strike and dip), spacing, continuity and character.
- Weakness zones, with special attention to zones that have been identified on aerial photos.

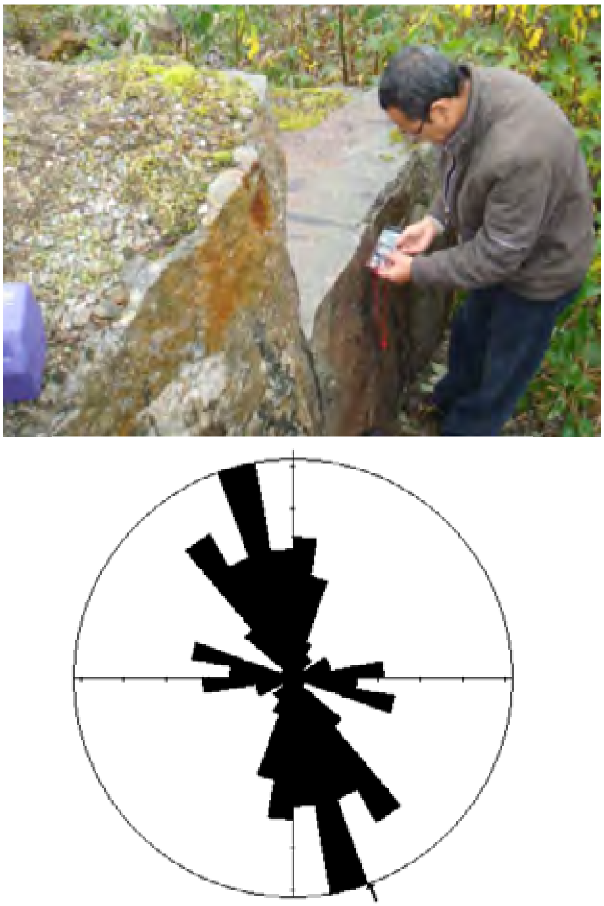


Figure 2. Mapping of joint orientation (strike and dip) and presentation of data in joint rosette.

Regarding rocks, emphasis is always placed more on character and mechanical properties than on sophisticated mineralogical and petrographical description. As part of the fieldwork, sampling is important for testing of properties that may greatly influence the degree of difficulty and the economy of the planned project, such as quartz content, rock strength and drillability. Great care must be taken so that the collected samples are representative.

Joint orientation (strike/dip) is most commonly presented as a joint rosette, see Figure 2. The joint rosette is a useful tool for evaluating the optimum orientation for an underground opening, and for evaluating the impact of jointing on stability. However, when a large number of joint measurements are to be studied, stereographic projection as shown in Figure 3 is often more useful. A stereo-plot gives much more detailed information on variations in dip than the joint rosette, and is also a better basis for identification of discontinuities with unfavourable orientation relative to the tunnel or cavern.

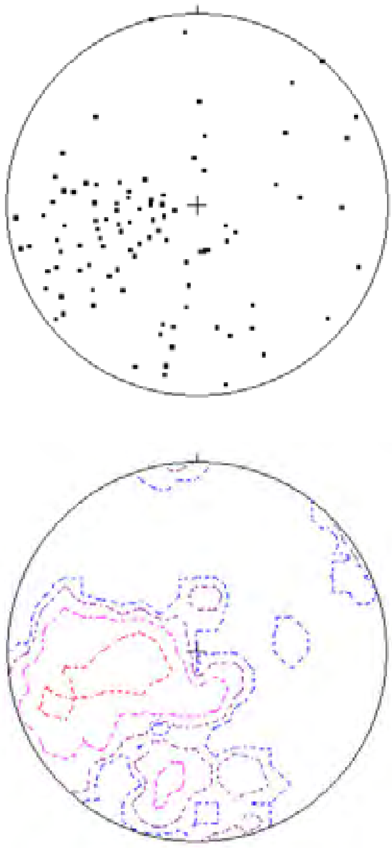


Figure 3. Presentation of the same joint data as for the rosette in Figure 2 in stereographic projection (equal area projection, lower hemisphere, with pole plot to the left and contoured plot with contours representing 1, 2, 4 and 8% densities to the right).

Regarding faults and weakness zones, the main objective of the field mapping is to check and supplement information that was collected during the desk study on strike/dip, width, character, etc. Shear zones may be identified based on the shearing/fracturing of the side rock and tensile zones based on the more massive side rock.

The results from desk study and field work are presented as engineering geological maps and profiles. As an example, map and profile for Meråker Hydropower Plant, based on mapping in scale 1:10000, is shown in Figure 4.

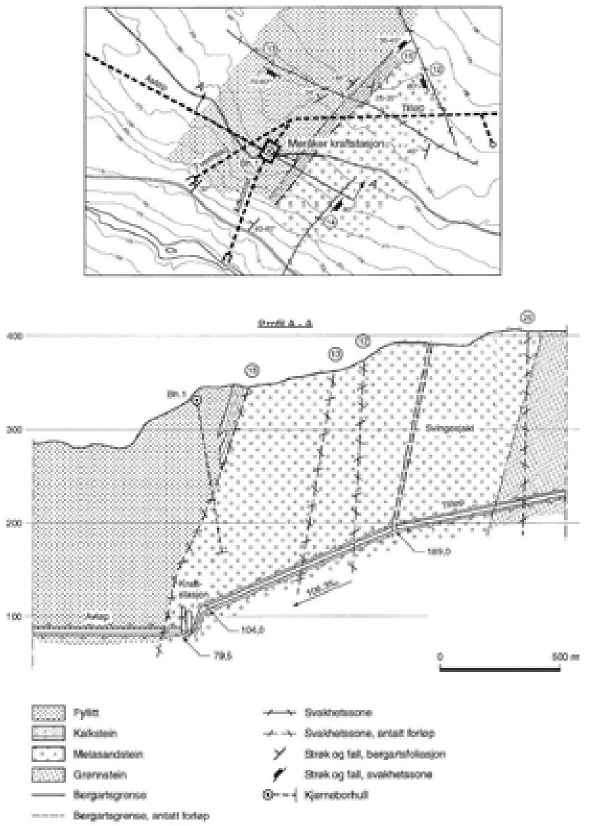


Figure 4. Engineering geological map and profile for Meråker Hydropower Plant.

In some cases the field work may also provide important information on rock stresses (i.e. based on exfoliation) and ground water conditions (i.e. in cases with karst, or based on rock mass character and fracturing characteristics).

Rock mass quality is often estimated based on rock classification, i.e. the Q-system. It is however important to be aware that the rock mass quality that can be observed at the surface is normally not fully representative of the subsurface conditions. Some of the parameters which are being used in the Q-system are also difficult to map at the surface. Great care is therefore taken in using classification data for support design at the pre-construction stage.

3.1.1.3.3 Geophysical investigation

Among the many geophysical methods which are available and may be relevant for pre-construction investigations, refraction seismic is most commonly used. In most cases it is used to log the thickness of soil cover and for evaluation of rock mass quality. An example of the use of refraction seismic combined with shallow reflection seismic (and core drilling) for planning of a subsea tunnel is shown in Figure 5.

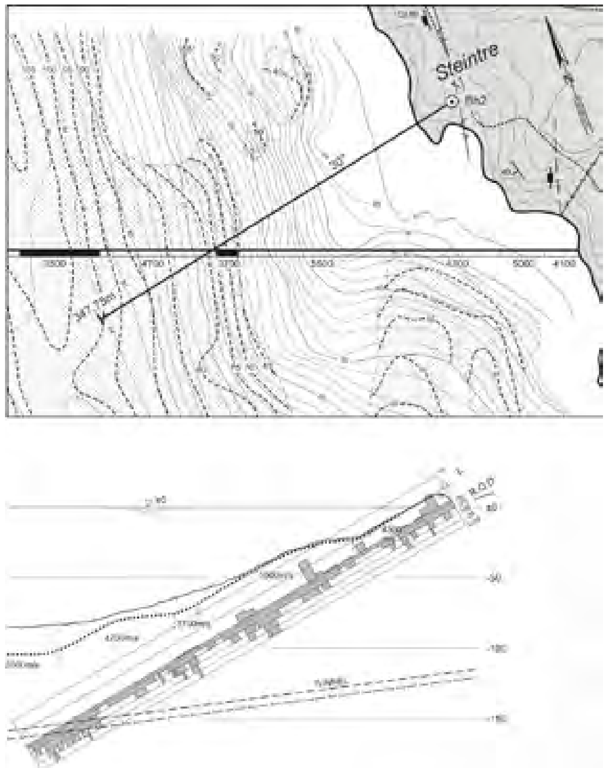


Figure 5. Investigation results for part of the Karmsund subsea tunnel.

The contour lines of the plan view in Figure 5 are based on shallow reflection seismic ("acoustic profiling") with signal source (boomer/sparker) and hydrophones placed at the surface of the sea. Thin lines represent the sea floor, and the more solid, stippled lines represent the bedrock surface below the sediments. The wide horizontal line represents the refraction seismic profile, with the seismic cable placed at the sea floor, and the digits represent the seismic velocities in m/s. Good quality rock masses below the water table have seismic velocities typically higher than 5,000 m/s, while the poor quality rock mass of weakness zones has velocities lower than 4,000 m/s. Based on the interpretation of the results from reflection and refraction seismic, two major weakness zones can be identified as shown in Figure 5 (black sections on the plan view, and represented by low seismic velocities of 3,500 and 3,700 m/s in the profile).

The seismic velocity gives very valuable information about the rock mass character. However, interpretation of the result is in some cases difficult. It is for instance not possible to decide whether a low velocity zone contains (swelling) clay or not. Distinct foliation or schistocity may also cause interpretation problems.

Seismic investigation methods also have limitations across deep clefts due to side reflection.

Seismic methods therefore do not automatically give high quality results in all geological environments.

Seismic tomography is used relatively rarely, but may be useful for instance in cases with subsea tunnels like in Figure 5, when geophones may be placed in the borehole, and shooting done from the sea floor.

One type of geophysical investigations which has become quite common for pre-investigation of underground excavations is geo-electric, or resistivity, measurement. Based on this method, the electric resistivity of the ground may be measured down to a considerable depth as shown in Figure 6. The main principle for interpretation is that fault/weakness zones, which contain water, will give a lower resistivity than the surrounding rock. Based on this, major fault zones may be identified.

The geoelectric method may in many cases be useful, particularly for mapping the orientation and character of weakness zones towards depth. The interpretation is however often uncertain.

Magnetometry is another geophysical method which has recently been introduced to investigate rock mass conditions for underground excavations. This is a method which in some cases has been useful for regional mapping of deep weathering.

3.1.1.3.4 Core drilling

While the methods described above are based on observing or monitoring from the surface, core drilling provides samples from the underground. In addition, the borehole itself may be used for many kinds of investigation and testing techniques.

Information from core drilling can be a valuable supplement to results from outcrop mapping, and is often combined with geophysical investigations. The former is illustrated by the case shown in Figure 4, where core drilling was used primarily to determine the boundary between phyllite and limestone, considered to be unsuitable for location of the powerhouse cavern, and the much better quality meta-sandstone.

For the subsea tunnel case in Figure 5, core drilling was used primarily to check the character of major weakness zones close to the shore, but also to estimate RQD-values, for permeability (Lugeon) testing and to

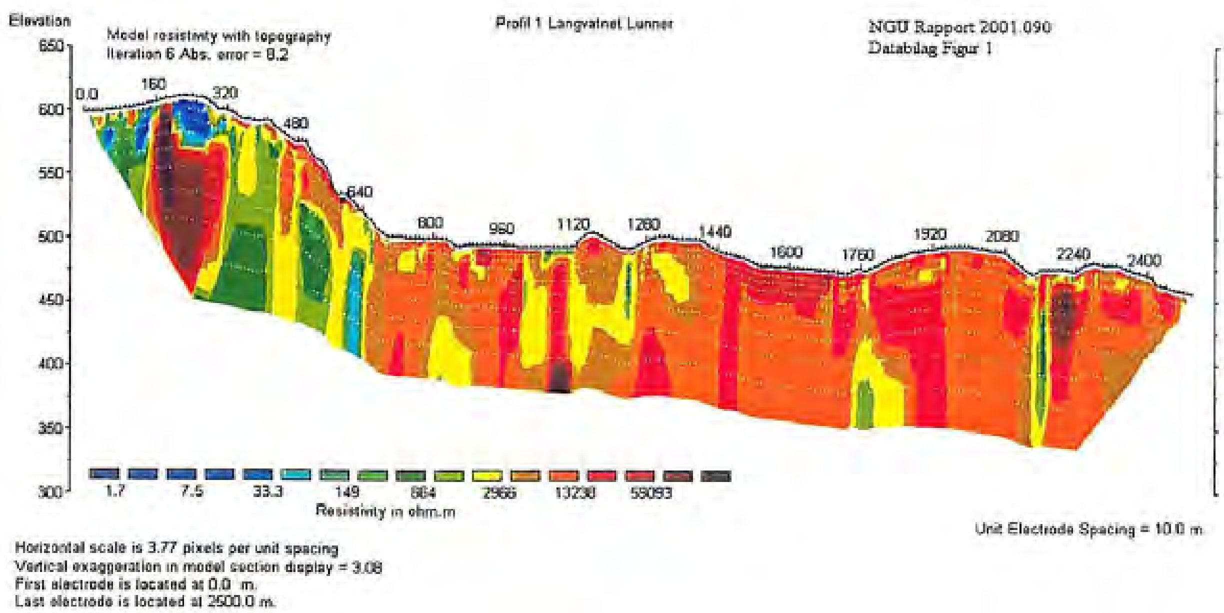


Figure 6. Results from resistivity measurement, with potential weakness zones (low resistivity) shown in blue colour (from Rønning, 2002).

provide rock samples and even samples of gouge material (including swelling clay).

In some cases, additional logging of the borehole is carried out based on geophysical methods or optical televiewer. The latter gives possibility of observing the walls of the borehole and making joint rosettes or stereo-plots from recorded joints, which may be particularly useful.

In many cases today, directional core drilling is used. Figure 7, shows a case where directional drilling of a 900 m long hole (BH-1) made it possible to detect a deep erosion channel in time to adjust the planned alignment, which illustrates the high value that such drilling may have.

By lowering the alignment 30 m, large stability problems were avoided, and considerable time and money were saved.

Considering the high cost of good quality core recovery, it is in most cases well worth spending a little extra to provide for good routine core examination and carefully prepared reports with high quality photographs of the cores before they are placed in storage.

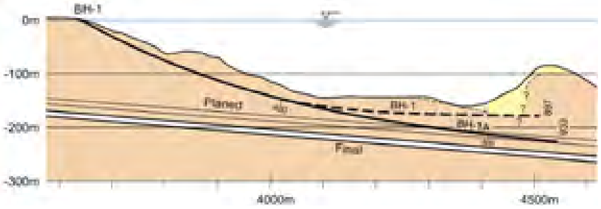


Figure 7. Directional drilling at the Bømlafjord subsea tunnel (from Palmstrøm et al).

3.1.1.3.5 Laboratory testing

Laboratory testing is often an important part of the investigation programme. The types of tests and extent of testing depends much on the character and complexity of the project. An overview of the tests most commonly used in Norway is given in Table 3.

Property/parameter	Method	Sample requirement
Mineral composition	Microscopy Differential-thermic analysis (DTA) X-ray diffraction analysis (XRD)	Thin section Powder Powder
Rock strength - compressive - compressive - tensile - brittleness	Uniaxial compressive strength test (UCS) Triaxial strength test Point load test Brittleness test	Drill cores (/cubes) Drill cores (rock), soil sample Drill cores(/irregular specimens) Aggregate (8-11.2 or 11.2-16 mm)
Rock elasticity - Young's modulus - Poisson's ratio - E_{dyn}	Uniaxial compression Uniaxial compression Sonic velocity	Drill cores Drill cores Drill cores
Discontinuity shear strength	Tilt test	Drill cores
Gouge material -mineral composition -swelling	DTA-analysis XRD-analysis Electron microscope Colour test See also swelling potential	Powder Powder Intact material Intact/powder
Drillability	Brittleness test Siever's J-value Abrasiveness	Aggregate (11.2-16 mm) Sawn specimen Powder (- 1 mm)
Blastability	Sonic velocity Point load strength Density	Drill cores Drill cores Drill cores/aggregate
Swelling potential	Oedometer test Hygroscopic properties Free swell	Fraction < 20 µm Fine fraction Fraction < 20 µm
Grain size distribution	Sieving Settling	Coarse and intermediate grained material Fine grained

Table 3. Common laboratory tests (based on Nilsen & Palmstrøm, 2000).

3.1.1.3.6 Rock stress measurements
Information about magnitude and direction of the principal stresses is particularly important for planning and design of deep seated underground excavations. Surface indicators such as exfoliation and core discing (intense parallel fracturing perpendicularly to the core axis) may give a warning of high rock stresses. However, full information on stress magnitude and direction can only be obtained by performing rock stress measurements. The rock stresses may vary considerably even within small areas, and it is therefore important to perform stress measurements in each individual case. Most easily, this is done by measuring underground, i.e. in an existing adit or at an early stage of excavation (which is commonly done, particularly for hydropower projects).

Two different methods for rock stress measurement are mainly used today:
- Triaxial measurement by overcoring.
- Hydraulic fracturing.

For detailed information about these methods, the reader is referred to the many examples of relevant literature.

3.1.1.4 Construction stage investigation methods
Even when very comprehensive pre-construction investigations have been carried out, there will still be some degree of uncertainty connected to the ground conditions. As discussed in Chapter 3.1.1.3 it is therefore very important that investigations are continued and supplemented during excavation.

3.1.1.4.1 Tunnel mapping
During excavation, continuous mapping of the ground conditions is very important for updating the interpretations based on pre-investigation, and for decisions regarding rock support. The emphasis on tunnel mapping has increased as the tunnel projects have become more and more challenging (urban tunnels, deep subsea tunnels etc), and in Norwegian tunnelling today it is common to allocate a special item for this in the tender specifications, often referred to as “the owners half hour”, which is intended for geological mapping and evaluation after each blast round.

This means that the contractor is compensated for the time which is used for mapping (which is not necessarily exactly half an hour – it may be more or less).

Continuous mapping at the face is very important also for documentation of the ground conditions of the project, and for planning of future maintenance. The mapping has to be done close to the face due to the extensive use of shotcrete in most tunnels today. Documentation based on the tunnel mapping should contain all geological factors that may influence the stability and conditions of the tunnel, such as rock type and character, jointing, faults, water leakage and potential rock burst problems, in addition to information about support work.

Examples of mapping forms and recorded actual data can be found in NBG 2008.

When the tunnel mapping and any other investigations are completed, a final report is made. Tunnel mapping logs are included in this report.

3.1.1.4.2 Probe drilling
In addition to the tunnel mapping described above, probe drilling as illustrated in Figure 12 is the most important investigation during tunneling. The probe drilling is carried out to collect information on the rock mass conditions ahead of the face, most importantly water inflow and faults/weakness zones. It is important that these factors are discovered at sufficient distance from the face for appropriate measures to be implemented for safe tunnel advance. Sometimes, probe drilling is also used to check the rock cover.

Normally, the tunnel jumbo with percussive drill is used for the probe drilling. Core drilling is sometimes used where particularly difficult rock conditions are expected. The extent of probe drilling is determined based on expected rock mass conditions, rock cover and previous experience on the project. The number of drill holes can be increased where zones of weakness or other poor rock mass conditions are expected, or where there is a risk of leakage or a need to check the rock cover. In subsea tunnelling, continuous probe drilling is always done.

The decisions regarding need for grouting are based on results from probe drilling. Rather than Lugeon testing as basis for decision about grouting, today it is normal to simply measure the volume of water ingress from the holes, and grouting is carried out if the ingress exceeds a pre-defined limit. For sub-sea tunnels this limit is commonly 10 l/min for 4 holes of 20 to 25 m length.

3.1.1.4.3 Measurement while drilling (MWD)
Over the last few years, there has been a considerable development regarding MWD (Measurement While Drilling) and DPI (Drill Parameter Interpretation), and this new technology is standard procedure today for road and railway tunnels in Norway. The principle of this method is that the instrumented drilling jumbo continuously monitors the main drilling parameters, and based on calibration and correlations with known

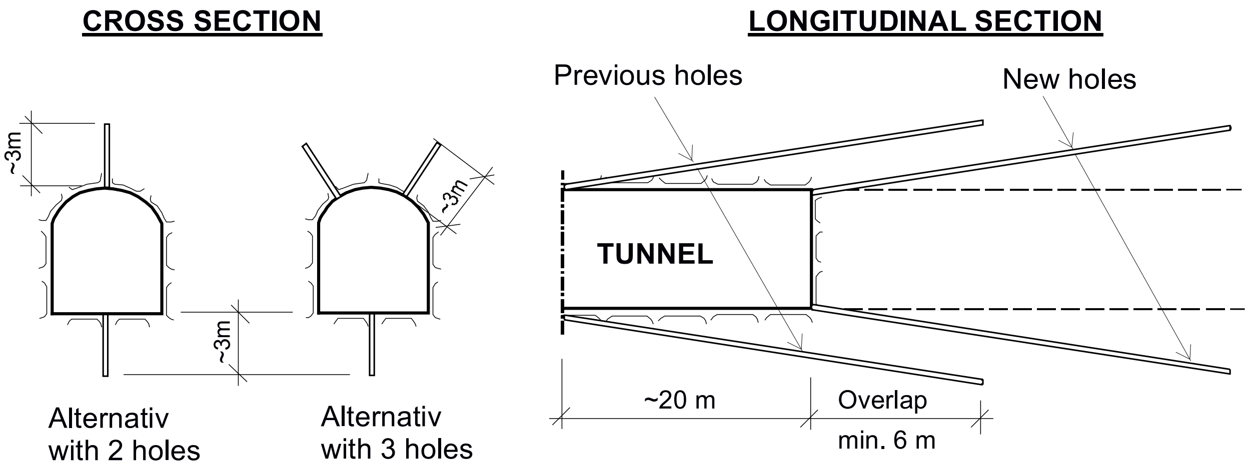


Figure 12. Examples of common layouts of percussive probe drilling.

rock mass parameters, prediction of factors such as rock strength, degree of fracturing and water inflow may be made.

3.1.1.5 Investigations during operation
For stability control of underground projects after completion, monitoring instruments such as extensometers are in some cases installed, in most cases for controlling the stability over a limited period of time, i.e. 2-3 years if no displacement is detected.

Quality control of rock support installations (i.e. potential crack development in shotcrete) are also carried out at certain time intervals, and based on the results from this control, maintenance is carried out when required.

3.1.1.6 Extent of investigation
The basic philosophy regarding extent of ground investigation is that this should always be governed by two main factors; 1) The degree of difficulty of the geology and the complexity of the project, and 2) The requirement for safety. For a rural, minor water supply tunnel in simple geology, for instance, the requirement for ground investigation is less than for an urban, high-traffic, large road tunnel in complex geology. This basic principle was defined in the Norwegian Standard NS3480, which in 2010 was replaced by the Eurocode 7, which is now also the valid Norwegian standard (Standard NO 2004). However, the basic principle of Eurocode 7 is the same as in NS3480, with a classification of Geotechnical Category (Project class in NS3480) according to "Degree of difficulty" and "Reliability class" ("Damage consequence class" in NS3480).

The commonly used Norwegian interpretation of Eurocode 7 classification in Geotechnical Categories is shown in Table 4. Based on this, a rural water supply tunnel in Norway most commonly will be a Category 1 project, while a road or subway tunnel most commonly will be a Category 2/3 or 3 project.

Reliability class	Degree of difficulty		
	Low	Medium	High
CC/RC 1	1	1	2
CC/RC 2	1	2	2/3
CC/RC 3	2	2/3	3
CC/RC 4*	*	*	*

Table 3. Classification of projects into Geotechnical Categories as recommended by the Norwegian Group of Rock Mechanics (NBG, 2012).

In accordance with Eurocode 7 and traditional Norwegian practice, a high geotechnical category implies:
- More investigation.
- More thorough planning.
- More control.

No exact recommendation regarding extent of investigation is however defined by the Eurocode 7, so this is basically to be decided by the owner.

To provide a better basis for defining the "proper extent of investigation", a recommendation was given based on a major Norwegian research programme with broad participation from the Norwegian tunnelling industry. Based on this system, a classification into Investigation Class is made based on evaluation of a) Level of engineering geological difficulty and b) Safety requirement for the project as shown in Table 5. The classification is similar to the one used for defining Project class in NS3480 (and Geotechnical category in Eurocode 7). Details on how to define the parameters a1-a3 and b1-b3

Investigation class		Level of engineering geological difficulty		
		a1. Low	a2. Moderate	a3. High
Safety requirement for project	b1. Low	A	A	B
	b2. Medium	A	B	C
	b3. High	B	C	D

Table 5. Classification of projects into Investigation Classes (based on Palmstrøm et. al, 2003).

are described in Palmstrøm et.al (2003). A water supply tunnel in simple geology typically will end up in class A, while a road or subway tunnel most commonly will end up in class C.

Based on critical review of a large number of completed underground excavations, when the engineering geological conditions and challenges were known, and it was possible to define what would have been the "proper" extent of pre-investigation, a recommendation for extent of investigation effort for future projects was defined as shown in Figure 14. The diagram includes all investigation classes in Table 5 (although only one case for class D, which is quite rare). The recommended investigation investment is given as percent of the excavation cost (which for a planned project is easier to quantify than the total construction cost). The relative cost, as shown in Figure 14, is higher for a short tunnel than for long tunnel, reflecting the higher relative mobilization costs for a short tunnel than for a long.

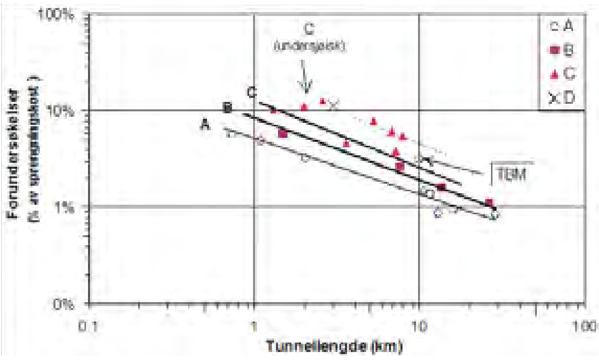


Figure 14. Recommended extent of pre-investigation as function of tunnel length for the respective investigation classes in Table 5. The excavation cost includes blasting, loading/ hauling and 25 % rig costs. After Palmstrøm et.al (2003).

As an example, for a 5.2 km long subsea road tunnel in difficult rock conditions, corresponding to investigation class C, the recommended extent of pre-investigations according to Figure 14 will be:

Blasting cost: 20000 NOK/m

Excavation cost: 20000 x 5200 x 1.25 = 130 MNOK

Recommended pre-investigation investment: 130 x 0.08 = 10.4 MNOK

This method is often used as a guideline for deciding what efforts should be spent on pre-investigations in Norway, and is recommended by the Norwegian Public Roads Administration to be used for road tunnels (NPRA, 2010). A similar diagram like the one in Figure 14 exists for investigations during tunnelling, but is not used to the same extent.

3.1.1.7 Summing up remarks

Pre-investigation tools are available for practically any kind of ground condition and any kind of site characterization. However, it is important to realize that even when great effort has been made in the pre-investigation, some uncertainty will still remain regarding the ground condition. Pre-construction site investigations therefore always have to be followed up by continuous engineering geological investigation during tunnelling. In many cases in Norwegian tunneling practice some of the detailed design is postponed so that results from construction stage investigations can also be included in the final evaluations (i.e. rock stress measurements, particularly for hydropower projects).

It is important to realize that the ground conditions may vary within wide ranges, and there is therefore no "standard investigation procedure" that will fit all types of conditions and all types of underground projects. The investigations for tunnels and underground excavations have to be designed according to the characteristics of each individual project, and should always be adjusted to:

1) The difficulty and complexity of the geological conditions.
2) The complexity and special requirements of the project.

The investigation should always be carried out in stages, and willingness for design modification as well as modifications of excavation and rock support is important for an optimum result of the final project.

3.1.2 Rock mass classification

3.1.2.1 Introduction

There are many different systems for rock mass classification and the preferences regarding which system to use will vary with geographic regions and personal views on what system that is most suitable for a given purpose. For natural reasons the Q-system that was developed by NGI between 1971 and 1974 (Barton et al. 1974) is the one mostly used in Norway. It is also a widely accepted system in world-wide tunneling and mining.

The Q-system has been revised and re-published several times, like the update based on 1050 project examples in 1993 and another 900 examples added in 2002. Some of the revisions also reflect developments in the use of fibre reinforced sprayed concrete and the increasing use of reinforced ribs of sprayed concrete (RRS).

The Q-system is applicable for two different primary purposes:

1. Classification of rock mass quality in relation to stability of underground construction, used either as part of surface site investigations and geological mapping, or as part of mapping of ground conditions during excavation. Note that in the last case, the Q-value will depend on the rock cover of the underground opening and may therefore be different to the Q-value recorded on surface (for the same type of rock).
2. Selection of rock support for an underground opening based on combination of the Q-classification of the local rock quality and the rock support diagram of the Q-system. This gives recommended support derived from the recorded support-example database of previous successfully executed solutions for similar rock conditions. The recommendations cover both immediate, temporary support and permanent support.

The Q-values generated based on geological mapping and rock classification from within underground excavations will give the most precise expression of rock quality. When the Q-system is used for on surface site mapping, classification of core samples or recordings inside boreholes, it becomes more difficult to establish accurately some of the parameters used to calculate the Q-value.

This chapter contains excerpts from the NGI handbook "Rock mass classification and support design" from 2015, which is accessible online in PDF-format from www.ngi.no. It is strongly recommended to download the complete document for any practical use of the Q-system. There is also an app available for use on a smartphone or tablet device.

3.1.2.2 Stability of the rock mass

During underground excavation it is very important to have a close visual observation of the rock surface in the whole tunnel periphery before the rock is covered by sprayed concrete. In addition to the visual observation, hammering with a scaling rod or a hammer will give important observation of deterioration of unstable rock giving particular sounds. Also small cracks, invisible from the invert, will be observed with a closer look. Altered rock may show the same geological structures as the original fresh and un-weathered rock, and may not be noticed when observed at distance. In order to have a close observation it is of outmost importance to have access to the face and crown by use of lifting equipment especially designed for this purpose. Rock mass stability is influenced by several parameters, but the following three factors are the most important:

- Degree of jointing (block size)
- Joint friction
- Stress

The degree of jointing, or block size, is determined by the joint pattern, i.e., joint orientation and joint spacing. At a certain location in the rock mass, there will, in most cases, be a joint pattern which could be well or not so well defined. Often joint directions exist systematically in rock masses, and most of the joints will be parallel with one of these directions. Near parallel joints form joint sets and the joint spacing within each set will usually show characteristic distributions. The joint spacing may be reduced considerably along some zones in the surrounding rock. Such zones are called fracture zones. Stability will generally decrease when joint spacing decreases and the number of joint sets increases. In soft rocks where deformation can occur independently of joints, the degree of jointing has less importance than it has in hard rocks.

In hard rocks, deformations will occur as shear displacements along joints. The friction along the joints will therefore be significant for the rock mass stability. Joint friction is dependent on joint roughness, thickness and type of mineral fillings. Very rough joints, joints with no filling or joints with only a thin, hard mineral filling will be favourable for stability. On the other hand, smooth surface and/or a thick filling of a soft mineral will result in low friction and poor stability. In soft rocks where deformation is less dependent of joints, the joint friction factor is less significant.

The vertical stress in a rock mass commonly depends on the depth below the surface. However, tectonic stresses and anisotropic stresses due to topography can be more influential in some areas. Stability of the underground excavation will generally depend on the stress magnitude in relation to the rock strength. Moderate stresses are usually favourable for stability. Low stresses are often unfavourable for the stability. In rock masses intersected by zones of weak mineral fillings such as clay or crushed rock, the stress situation may vary considerably within relatively small areas. Experience from tunnel projects in Norway has shown that if the magnitude of the major principal stress approaches about 1/5 of the compressive strength of the rock, spalling (strain bursting) may occur. When tangential stresses exceed the magnitude of the rock compressive strength, squeezing may occur. In other words; the anisotropy of the rock mass plays an important role when designing rock support.

3.1.2.3 The Q-system

The Q-value gives a description of the rock mass stability of an underground opening in jointed rock masses. High Q-values indicates good stability and low values means poor stability. Based on 6 parameters the Q-value is calculated using the following equation:

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

The six parameters are: RQD = Degree of jointing (Rock Quality Designation) Jn = Joint set number Jr = Joint roughness number Ja = Joint alteration number Jw = Joint water reduction factor SRF = Stress Reduction Factor

The six parameters are:

RQD = Degree of jointing (Rock Quality Designation)
Jn = Joint set number
Jr = Joint roughness number
Ja = Joint alteration number
Jw = Joint water reduction factor
SRF = Stress Reduction Factor

The individual parameters are determined during geological mapping using tables that give numerical values to be assigned to a described situation. Paired, the six parameters express the three main factors which describe the stability in underground openings:

RQD/J_n = Degree of jointing (or block size)
J_r/J_a = Joint friction (inter-block shear strength)
J_w/SRF = Active stress

3.1.2.4 Using the Q-system to determine rock support

Q-value and the six appurtenant parameter values give a description of the rock mass. Based on documented case histories a relation between the Q-value and the permanent support is deducted, and can be used as a guide for the design of support in new underground projects.

In addition to the rock mass quality (the Q-value) two other factors are decisive for the support design in underground openings and caverns. These factors are the safety requirements and the dimensions, i.e., the span or height of the underground opening. Generally there will be an increasing need for support with increasing span and increasing wall height. Safety requirements will depend on the use (purpose) of the excavation. A road tunnel or an underground power house will need a higher level of safety than a water tunnel or a temporary excavation in a mine. To express safety requirements, a factor called ESR (Excavation Support Ratio) is used.

A low ESR value indicates the need for a high level of safety while higher ESR values indicate that a lower level of safety will be acceptable. Requirements and building traditions in each country may lead to other ESR-values than those given in Table 7.

It is recommended to use ESR = 1.0 when Q ≤0.1 for the types of excavation B, C and D. The reason for that is that the stability problems may be severe with such low Q-values, perhaps with risk for cave-in.

In addition to the span (or wall height) ESR gives the "Equivalent dimension" in the following way:

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

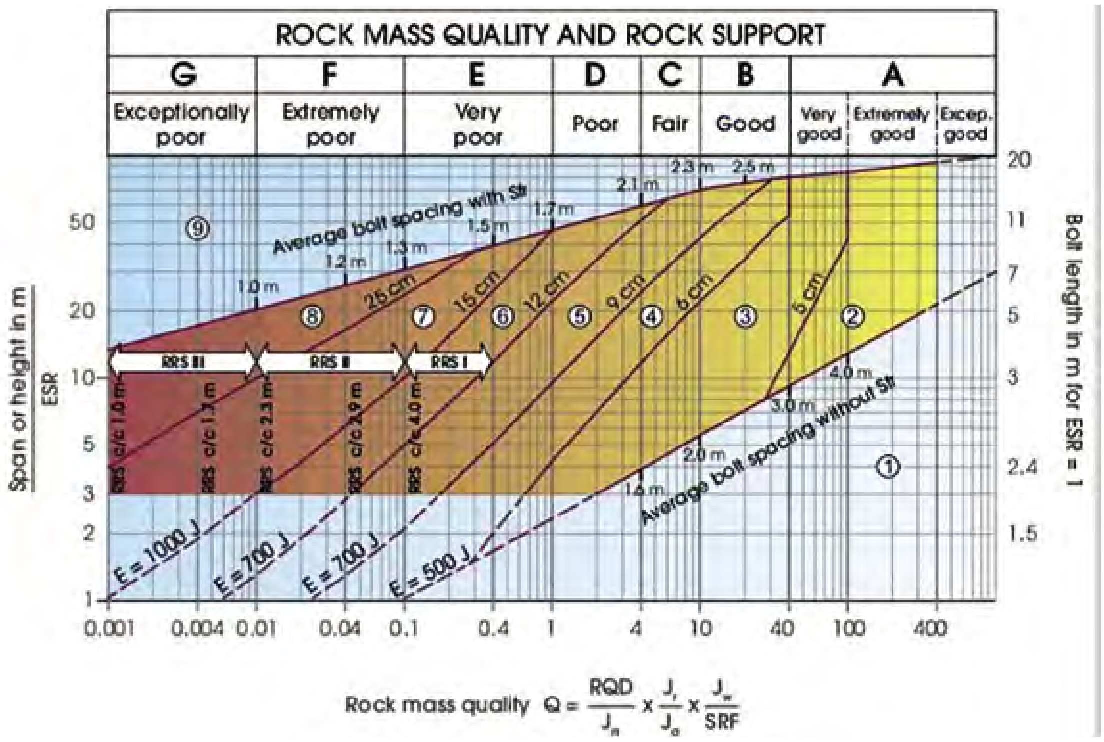
The Q-value and the Equivalent dimension will be decisive for the permanent support design. In the support chart shown below, the Q-values are plotted along the horizontal axis and the Equivalent dimension along the vertical axis on the left hand side.
The support chart gives an average of the empirical data from examined cases. In some cases the rock support represents a conservative magnitude of support, while in other cases cave in occurred during construction or years later, when the underground excavations were in service. For a given combination of Q-value and Equivalent dimension, a given type of support has been used and the support chart has been divided into areas according to type of support.

Please note that the chart is not divided into definite

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

support classes, but shown as a continuous scale both for bolt spacing and thickness of sprayed concrete. As the support chart is based on empirical data, it is able to function as a guideline for the design of permanent support in underground openings and caverns.

The support chart indicates what type of support is used in terms of the centre to centre spacing for rock bolts and the thickness of sprayed concrete. It also indicates the energy absorption of the fibre reinforced sprayed concrete, as well as the bolt length and design of reinforced ribs of sprayed concrete. Support recommendations given in the chart are general and in certain especially difficult cases, an increase in the amount or type of support may be relevant.



The thickness of the sprayed concrete increases towards decreasing Q-value and increasing span, and lines are drawn in the support chart indicating thicknesses. For positions between these lines the thicknesses will have an intermediate value. If deformation occurs, for instance caused by high stresses, reinforced concrete should be used in all categories.

Sometimes alternative methods of support are given. At high Q-values in the support chart, sprayed concrete may or may not be used. The mean bolt spacing in such cases will be dependent upon whether or not sprayed concrete is used. Due to this, the support chart is divided into two areas. The area defined as “Bolt spacing in fibre reinforced sprayed concrete” refers to bolting in combination with sprayed concrete. The area defined as “Bolt spacing in areas without sprayed concrete” indicates bolt spacing when sprayed concrete is not used. Recommended bolt spacing is more an expression of the quantity of bolts necessary rather than an exact recommendation for the spacing. The position and direction of each bolt should be based on an evaluation of the joint geometry. This is especially important in areas where the bolt spacing is large. In areas where sprayed concrete is not used, systematic bolting is not relevant, and there should always be an evaluation for the position for each bolt.

The length of the bolts depends on the span or wall height of the underground opening and to some degree on the rock mass quality. Recommendations for bolt lengths are given on the right hand side of the diagram, but some evaluation is necessary. In unfavourable joint geometry, longer bolts than recommended in the diagram will be necessary, and there is also a general need for increasing bolt length by decreasing Q-value.

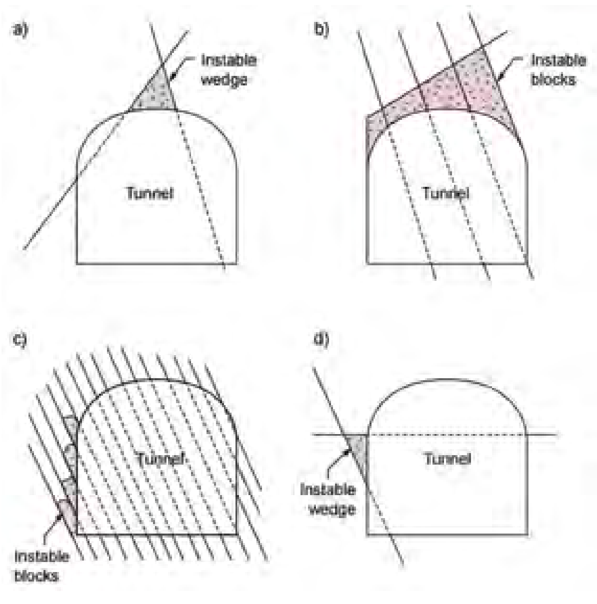
3.1.2.5 Additional comments on stability and rock support

A Q-value gives description and classification of a rock mass, and by using the support chart in Figure 7, one can design the general support methods and quantities needed for a particular Q-value. The Q-value and the support chart however, do not intercept every detail or all specific cases. The stability of single blocks is more or less independent of the Q-value. The specific rock support, i.e. location of single bolts is not taken into account by the Q-system. Faulty design of the rock support may lead to failure of single blocks even if the rock support is in accordance with the Q-system. When designing rock support it is therefore necessary to consider the joint geometry specifically. If the rock bolting is carried out before application of sprayed concrete it is possible to locate each individual block.

Some examples of unfavourable joint geometries that require special attention with regards to bolting are shown in Figure zx. In the crown of an excavation, joints with sub parallel strike direction to the length of the excavation but with variable dip directions may create unstable wedges. A combination of sub-horizontal and sub-vertical joints may require special attention because a sub-horizontal joint may intersect the rock mass just above the crown and may not be seen before failure. In such situations longer bolts than those recommended by the Q-system could be the solution. It is also recommended to adjust the directions of the rock bolts in such cases.

Inclined joints intersecting the walls in an underground opening could serve as sliding planes for unstable blocks. In such cases the stability of opposite walls may be quite different depending on the dip direction of the joints (Figure zx_c). If two intersecting joints form a wedge as shown in Figure zx_d, a similar situation will occur.

In some specific cases with $J_r = 3$, $J_a = 1$ and $RQD/J_n < 2$ in heavily jointed rock (almost sugar cube jointing), the Q-value alone may be give the wrong basis for rock support because the small blocks without cohesion may give reduced stability in spite of a relatively high Q-value. This may be compensated by increasing the SRF-value (as for a weakness zone) and using $J_r = 1$ (because of lack of joint wall contact).



3.1.2.6 General comments on rock mass classification and rock support design

As mentioned in the introduction about rock mass classification, the Q-system is one of many different ways of performing classification of rock conditions. However, the basic purpose of it all is to communicate in as specific as possible terms the quality of the rock in a tunnel or a cavern in hard rock. Bottom line is that immediate and permanent support must be installed and decisions about what to install must be taken on short notice.

Typically, all projects of underground excavation will have rock quality classes and linked rock support classes that have been pre-designed for use at the project. The rock quality class, established e.g. by the Q-system, can be used to decide on the rock support as shown above, but there are also other ways, like analytic calculations and numeric analysis. In really complicated and high risk cases, all useful tools will normally be employed to reach a conclusion.

Still, there will often be an element of uncertainty, depending on conservatism employed and often project specific political decisions of various types. The support recommendations presented by the Q-system may be taken as the final decision, but in Norwegian practice, it is quite normal to take it as one of many possible “recommendations” and to verify the final support choice installed by observation of performance. This is what is termed the Observational Method (OM), which is covered in more detail in Chapter 3.1.3.

The OM is part of Eurocode 7 and it is required in Norway to employ this Code to underground construction support design. The details of Eurocode 7 usage are currently under revision, but verification of sufficiency of support by observation (mostly meaning some level of instrumented monitoring) is central if OM has been selected as the design method under Eurocode 7.

3.1.3 The Observational Method (OM)

3.1.3.1 Introduction

The previous chapters have covered in some detail the preparations that are necessary before start of construction of a tunnel or cavern and it is all done to provide basis for, among other things, the selection of rock support design principles and the construction and support methods. The rock mass classification may start already at the pre-investigation stage and can be used also for systematic recording of rock condition data as excavation takes place. Furthermore, it is important to decide if the rock surrounding the excavation will be part of the load bearing structure, or if the rock is just seen as a source of load onto an installed support.

The normally applied principle in Norwegian tunneling is based on rock support being adapted to the local rock conditions, both regarding immediate support and for the permanent support. Also, the surrounding rock is taken into consideration as part of the overall structure that creates stability for the underground opening. Contrary to the approach in many other regions of the world, even the temporary support is required to satisfy quality and durability as specified for the permanent support. This way, all installed support can be integrated into the permanent support. This approach offers the advantage of saved time and materials and the use of one single over-conservative support solution for the whole tunnel can be avoided. A single support solution throughout would have to cover the worst condition encountered along the tunnel.

For civil construction tunnels, many different methods are used to decide on rock support solutions for given projects. Design typically has to cover both the immediate or temporary support case as well as the permanent support, and the latter may involve a time horizon of 100 years. Ground conditions for tunneling range from shallow tunnels through soil to deep seated tunnels through solid and massive hard rock. Some tunnels get lined with one single support solution that must cover all conditions for its full length, while the lining in most Norwegian tunnels gets adapted to the local ground conditions as they are encountered. When combining all the variables involved, like excavation method, immediate and permanent support, worst case single support solution or adapted solution and including many different design methods, the number of possible combinations becomes very large.

To a varying degree, all tunnel support design methods end up with an unknown factor of safety, as demonstrated by the fact that there are sometimes failures and collapses. Also, there are cases where the installed lining gets no load whatsoever. The ground conditions and stress situation along a tunnel alignment is typically so variable that it becomes impossible to accurately determine all the parametric input needed in support calculations. Also empirical methods will suffer from mapping and classification mistakes and subjectivity. Numerical methods are not any different and cannot possibly hold up the face progress while input parameters are measured and calculations executed for results to be used for initial support decision and execution.

In principle, all decisions about rock support solutions in tunnels carry uncertainty and anything pre-planned, but not yet excavated and installed, can only be considered a support prognosis. This will be the case irrespective of design methods and tools used to reach decision about what to install for different ground conditions. The rational approach is to face this reality and apply the Observational Method (OM) for *verification* of sufficiency of whatever has been installed for rock support. Rock 'support' in Norway is in most cases in reality rock reinforcement, e.g. installed inside the rock as rock bolts, or as surface reinforcement (e.g. sprayed concrete mostly with fibres). Actual rock support is normally either an in-situ cast concrete lining or back-filled concrete segments in a TBM tunnel. In any case, the tunnel stability depends on an interaction between the surrounding rock and the installed reinforcement and support and we will basically never know all the relevant parameters and mechanisms that play a role in this composite action. To circumvent this problem and use reality as a full-scale test laboratory, installed instrumented monitoring sections or other means of observation, can prove (or disprove) whether the tunnel is stable or not. If the real life observations are not satisfactory, this will allow mitigation by installation of added support. Without monitoring, unsatisfactory performance could go unnoticed and will sometimes cause a collapse.

It should be noted that the practical details of an OM approach must be adapted to the case at hand. As mentioned, the use of analysis tools for design of support solutions for expected ground quality classes will depend on the range of ground conditions expected, but also how complicated and possibly risky the underground construction is considered to be. It should be pretty obvious that a 15 m2 tunnel in good granite will be much simpler to design compared with a cavern system with large dimensions in poor rock and may be high stresses. Clearly, the same considerations apply when deciding what methods of observation will be necessary for verification of performance of installed support. For the mentioned small tunnel in good granite, observation can be limited to actually doing visual inspection, while the cavern system would typically require quite an elaborate system of monitoring devices. It may be claimed that there is in principle no distinction between visual observation and instrumented monitoring in this respect. In both cases the implemented steps can be seen as just different ways of executing 'observation' to satisfy the need for verification, but adapted to the requirements of the case at hand.

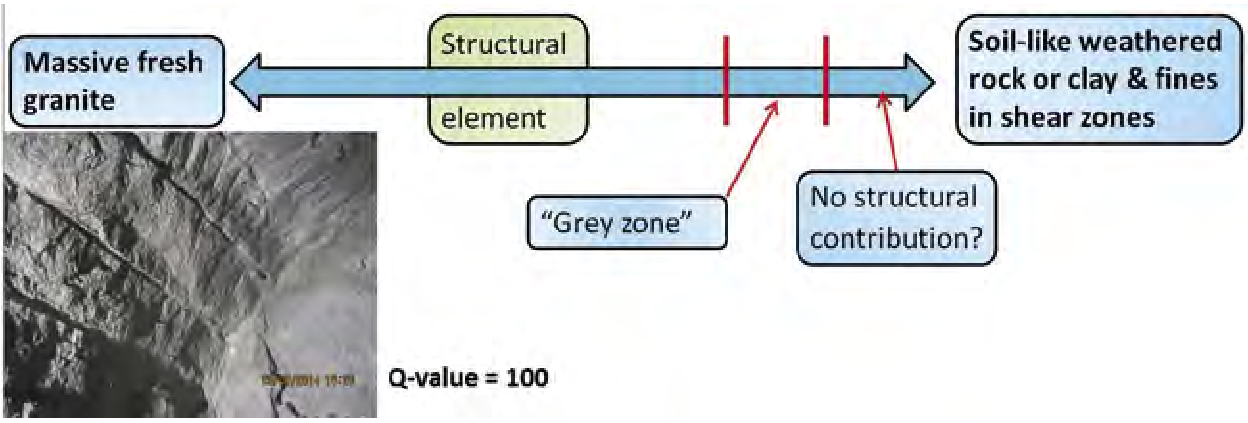
3.1.3.2 Some basic considerations

Even tunnels in generally good rock qualities will typically have to cross shear zones (faults) of sometimes extremely low quality. The situation can be illustrated as in the figure below.

In the situation to the left, no measures are needed since the rock is globally stable at almost any free span of underground excavation. All the way to the right, heavy support structures will be necessary and swelling clay content may add significantly to the loads that must be supported.

The dominating paradigm at the base of current rock support design in many countries is based on the approach needed for cases from the Figure below far right side. At this end of the ground quality scale, a heavy support solution is warranted, but it should not be extended too far to the left. In reality, most of hard rock cases, even with the normal jointing patterns and quality "flaws" of rock, the rock can be utilized as a structural element when designing and creating stability. This can be done by focusing on and installing rock reinforcement rather than rock support. Rock reinforcement will typically be some selection of different rock bolts or cable anchors installed in boreholes combined with surface reinforcement by fibre reinforced sprayed concrete. Even polymer based spray-on surface reinforcement is today available.

That satisfactory stability in the majority of rock qualities can be established using rock reinforcement only,



Ground quality and need for measures to create stability as outlined above, without heavy concrete lining with or without structural reinforcement, cannot be disputed. This approach is routinely being used for all kinds of temporary or initial support in drill and blast tunneling. Even if defined as temporary, it often takes months and sometimes years before the final lining, or permanent lining gets installed and lack of stability is very seldom an issue. Frequently, the heavy concrete lining is not necessary other than for reasons of durability, ground water control or esthetic requirements. An increasing volume of tunnel and cavern excavation is furthermore successfully lined by rock reinforcement methods with strict requirements on quality and durability for both initial and final support. The end result is often just 200 mm average thickness of reinforced sprayed concrete combined with permanent quality rock bolts working as permanent lining, thus replacing bolts and sprayed concrete defined as temporary, followed by additionally 5X as much in-situ concrete lining with double reinforcement.

One important reason for this conservatism in design of permanent rock support is that analyzing the stability case of sprayed concrete and rock bolts in a drill and blast tunnel is extremely complicated and can hardly be done accurately. For some designers there will be a feeling of lack of confidence. Rock bolts and a relatively thin skin of sprayed concrete on the undefined and variable geometry of the blasted rock surface is just part of the problem. Another important element is the frequent variation in rock quality along the tunnel, as well as variable rock stresses and ground water conditions.

3.1.3.3 The unavoidable practical approach

When first dealing with the initial support, the design methods have all of them serious limitations. Analytic and numeric methods suffer from:

- Inaccurate and missing input values and the validity of the geological model may be questioned.
- Approximations, simplifications and assumptions are used to be able to execute calculations and the validity of the mathematical model will sometimes suffer significantly.
- Tunnel advance is anyway far too fast to allow any per blast-round analysis for support selection by calculation result.

The empirical methods offer a simpler and faster approach, which therefore is a quite practical alternative, but:

- They are no better or worse than the cases included in the recorded data-base.
- Mapping mistakes and subjectivity are normal deviations that are hard to completely avoid.

Practically all drill and blast tunnels and other open face tunneling therefore end up managing the problem of selecting initial rock support more or less the same way:

1. First, identify the range of expected rock conditions along the tunnel.
2. Sub-divide this total range into a number of Ground Classes (typically anywhere from 4 to 10).
3. Design a "support" solution for each Ground Class. To reach conclusions for each Ground Class, any and all available design methods may be used based on preference and necessity and this work will naturally be much influenced by the complexity presented by the findings under item 1

.

Once tunneling has started the process continues by:

4. Mapping of the rock conditions in the tunnel, typically on a per blasting round basis to identify which Ground Class that applies for determination of the "support" solution to install as pre-designed according to item 3 above.

There are good reasons to ask why this approach may not be used also to design and decide on the permanent part of the support solution. After all, the temporary support evidently works very well, even for long periods of time.

In short, it can be summed up as a problem of overall structural analysis of the interaction between the rock material and the installed reinforcement under the normal geometric conditions, especially if used for a permanent solution. Verification of sufficiency can and often will be challenged.

On the other hand, the standard case of a full concrete lining with double reinforcement will provide a known load carrying capacity resulting from standard reinforced concrete analysis methods. As a side remark, this normal final lining approach of showing load-carrying capacity, is often disregarding that the actual load (if any) is still not really known. The only real difference to the temporary rock reinforcement approach is the typically resulting very conservative load-carrying capacity of the installed *support* structure.

3.1.3.4 Combination of immediate support and later complementary support

When using the OM as the permanent lining design method primarily based on sprayed concrete application and rock bolts, what ends up being the final and permanent lining may be constructed in more than one step. Even if the materials and processes involved placing the immediate support do satisfy the quality and durability requirements of the permanent lining, the fact that part of the final solution has been installed in one or more later steps, is sometimes raising concern that the end product is not one unit.

It may certainly happen that the immediate support gets subjected to high rock stress and some deformations before stability is reached, regardless if reached on its own or after additional measures have been placed. In extreme cases, it may be claimed that the immediate support has been damaged by elongation of bolts and probably some cracking of the sprayed concrete layer and that the immediate support therefore must be disregarded when considering the permanent lining. However, modern combination bolts are not suffering durability issues as long as they are not snapping from overload and deformation. Fibre reinforced sprayed concrete used under such conditions should use synthetic structural fibres and cracking will not cause fibre corrosion and loss reinforcing effect.

Another claim of concern may be that later layers of sprayed concrete may prevent a monolithic structure when looking at the overall sprayed concrete thickness. Provided proper surface preparation when applying later placements of sprayed concrete, the interlayer bond strength will be about 10 MPa or more and actually the same as when building large thickness in several passes during the same shift. This is the normal way of building the required final thickness of any sprayed concrete structure and it is not known to have caused any problems of practical nature or from testing of core samples from executed project.

At the end of the day, the strength of the OM is exactly that the verification by Observation (mostly monitoring) will be made on the support structure and the rock conditions as they are, so concerns like the ones described above are actually taken care of. Still, if conditions are really extreme in terms of large deformations ongoing for extended time, special considerations can always lead to adaptations that are normally not necessary. The integration of immediate into permanent support is still a recommended working principle for lowered time of construction and lowered overall cost, without sacrificing quality and durability.

3.1.3.5 Main elements of the Observational Method (OM)

1. Design rock support solutions for expected rock conditions (Ground Classes).
   - Use all necessary methods (analytical, numerical and empirical).
   - The case complexity dictates which methods to use.
   - The designed support solutions must be considered support prognosis at this stage and should not be taken as a design end-result.

2. Observe the support performance, or rather the performance of the surrounding rock and installed reinforcement interaction over time, while excavation is being continued.
   - The observation has the purpose of verifying the support prognosis.
   - Verification means that the observations get checked against the support prognosis. The design must contain estimated radial deformation against time with acceptance limits and levels of warning limits. Also other parameters may additionally be checked, also against specific criteria of acceptance or alarm.
   - Observation methods may range from simple visual checking in excellent hard rock conditions and small tunnels, to very complex instrumented monitoring and convergence readings in large caverns and complex and poor rock cases.
   - If the acceptance values from design (support prognosis) are not verified, then additional support measures need to be installed and the above steps must be repeated until verification has been recorded.

3. In case of local unsatisfactory performance and need for additional support, especially if a recurring phenomenon, then the available information must be fed back to the relevant part of the design to upgrade and adjust it to avoid further non-conformance under the same conditions.

4. Final Permanent lining Option
   - If all elements of the installed reinforcement has satisfactory durability to be defined as permanent, then verified stable by observation can be used for acceptance of the support as the final lining. However, it is of course possible to add another step after this, installing an additional and extra support for increase of the factor of safety. In most cases it will of course not be necessary to go to the extreme standard approach of adding double reinforced in-situ concrete lining. Some extra bolt pattern and possibly another layer of sprayed concrete would normally be enough.

By an adapted use of OM for the case at hand, many advantages can be listed:

- All factors influencing stability are covered, whether known or not known, since the “mountain” is being used as a full scale laboratory.
- Extensive rock sampling and parameter testing can be minimized.

- No scale-effect errors since the actual case is being observed and monitored.
- No errors from approximations and assumptions. Changes over time, like the effects of ground water flow and rock stresses are covered.
- The installed rock reinforcement and support gets adapted to the actual rock conditions, no more and no less.
- No expensive worst-case support installed for the whole tunnel..

The Observational Method is an accepted approach according to Eurocode 7 and offers final lining solutions adapted to ground conditions and therefore typically at lower cost. It may also be claimed that the advantages of OM will be enhanced in case of really complicated objects of underground construction. The more complicated (poor ground, system of caverns and other openings, sensitive neighbours etc.), the more unreliable will design methods depending on any kind of calculations be. In comparison, results from a proper OM-approach cannot to the same extent be questioned.

3.1.4 Reinforced sprayed concrete ribs

3.1.4.1 Introduction

When generally working with sprayed concrete for initial and immediate support, rock conditions may locally be very poor or even extremely poor. If such conditions were to be handled by fibre reinforced sprayed concrete just by increasing the layer thickness enough to smooth out an irregular contour in a drill and blast excavation to produce a defined arch, the required average thickness would become extreme. Such an approach would in many cases not be efficient regarding time and economy due to very large concrete quantities. If formwork and in-situ concrete is not used, then the alternative would have to include ribs at a typical spacing of about 1 to 2 m.

Ribs in combination with sprayed concrete may be constructed in many different ways and H-beam steel arches is frequently used, while the pre-formed lattice girders are more often installed when sprayed concrete is the main method of concrete placement.

Especially in drill and blast excavation, the actual rock contour will be at variable distance to the theoretical tunnel contour or net excavation line due to unavoidable overbreak. Locally this distance may be 0.5 to 1.0 m or more at locations of wedge fallouts and any pre-formed lattice girders or steel sets can only be produced to fit inside of the theoretical contour line. The overbreak will have to be backfilled with sprayed concrete, which requires a lot of extra concrete material and extra time. An alternative that mitigates at least part of this problem is the reinforced ribs of sprayed concrete (RRS).

3.1.4.2 Reinforced ribs of sprayed concrete

In sections with very poor rock mass quality (Q<1), reinforced ribs of sprayed concrete (RRS) in many cases is a preferred alternative to cast concrete. The ribs are constructed with a combination of steel bars (usually with a diameter of 16 mm or 20 mm), sprayed concrete and rock bolts, see Figure xw. When using steel bars of 20 mm diameter, the bars have to be pre shaped in order to gain a smooth profile. The thickness of the ribs, the spacing between them as well as the number of ribs and diameter of the steel bars has to vary according to the dimensions of the underground opening and the rock mass quality.

In cases where the Q-values indicate use of RRS, a 15-20 cm average thickness layer of fibre reinforced sprayed concrete normally has to be applied before installing the reinforced ribs. This layer has a function as temporary support as well as to smooth out the rock surface and reduce the sprayed concrete build-up that must be executed through and behind the installed steel bar reinforcement. The thickness of this layer is included in the total thickness of the RRS.

By monitoring and observation of RRS performance in recent case histories, there is a wide experience of RRS used in different rock mass conditions. A guideline for use of RRS in relation to Q-values and equivalent dimension of the underground opening is given in the support chart of the Q-system.

Note that in areas of extremely poor ground, overbreak may cause the actual rock contour to be located substantially outside of the theoretical excavation line. One major advantage of the RRS system compared with lattice girders or steel sets, is that the bar reinforcement will be attached to the sprayed rock surface where it actually is. The bent reinforcement bars will ride directly on the protruding contour points, thus minimizing the need to fill in depressions behind the bars and reducing the overall consumption of sprayed concrete to build the arch required by design. Furthermore, note that the pre-spraying for immediate support and profile smoothening must utilize fibre reinforced sprayed concrete (Sfr), while plain sprayed concrete must be used for covering the reinforcing bars of the rib. This is to avoid concrete build-up on the bars and poor compaction behind bars due to spraying “shadow”.

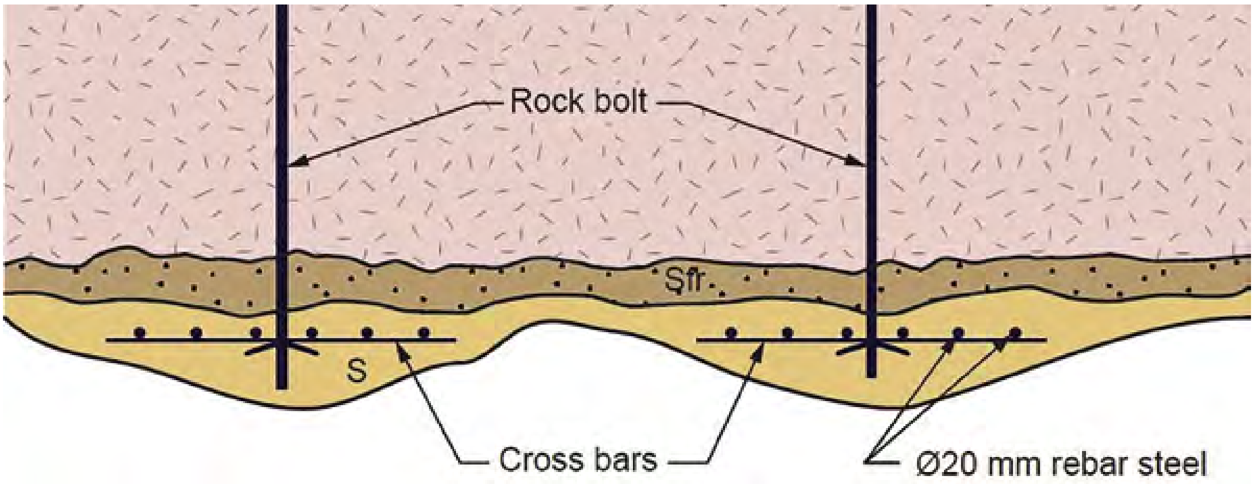


Figure xw. RRS construction principle, section parallel to tunnel centerline.

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- Rock stability assessments and reinforcement techniques
- Analytical and numerical analyses

3.2 RISK SHARING PRINCIPLES IN TUNNEL CONTRACTS

INTRODUCTION
Tunneling and underground excavation is related to handling of uncertainties, risks associated with geological conditions are significant and 'unexpected geological conditions' often claimed. This fact was brought to the table long ago, and ITA (the International Tunnelling and Underground Space Associ-ation) published its 25 Recommendations on Contractual Sharing of Risks in 1988 (Ref. 1). This public-cation constitutes an important asset to the tunneling industry in terms of being a guideline for those working with contractual issues, even today almost 30 years after being published. A more recent document is the ITIG code of practice for risk management of tunnel works from 2015 (Ref. 9). Various countries or geographical regions have their own view and tradition on risk sharing and tunnel contracts. In Scandinavia unit rates contracts are most commonly used with bid-build model, it shares risk between owner and contractor, owner retains the risk for geological conditions while contractor carries risk for performance efficiency. Rock strengthening is determined assessing rock mass quality encountered at tunnel face. Grouting is applied to stem water. Actual quantities may differ from the contract's BoQ but a flexible contract with remuneration is installed to enable adjustment of actual quantities. Variations of quantities involve a clause to adjust construction time.

Unit price contracts deals with expected and foreseen still 'unexpected geological conditions', if the 'unexpected' element results in variations in quantities. Work activities have quantities and 'standard capacities' for regulation of construction time. Variations in quantities are expected in tunnelling and hardly deserve the term 'unexpected'. If truly unforeseen geological features necessitate work not included in the BoQ, then the contract must be supplemented and can be said to be inappropriate. Fixed price contracts may not provide the intended predictable cost and is as such also inappropriate in the view of a risk sharing principle, once the geological baseline proves to be incorrect. 'Adjustable fixed price' contracts, combining unit rate and fixed price, may prove suitable.

Tunnelling bids and the chosen type of contract can be said to be a tool to allocate risk, and any bid for a tunneling project requires an estimate of the actual risk associated with a given project. Although, as the risk is inevitably closely related to the actual project and all underground or tunneling projects are unique, the actual risk for a given project is also unique and must be dealt with individually. According to a presentation by the FIDIC-ITA Task Group 10 (Ref. 2) on contract forms for tunneling and underground works, such works are related to risks that are difficult, if not impossible to estimate. In order to cover all risks reasonably, the price and therefore the cost of investment will inevitably rise, as the allocation of risk to one party only, is uneconomic. Read that as; allocating all the risk to the contractor using the typical EPC, Fixed price or similar type of contract that is commonly used around the world. Is this a sustainable way to arrange tunneling contracts? Bearing in mind that many tunneling contracts today often involves significant amounts of money at stake for all parties, also the greater society. In the opinion of the author it is not a sustainable solution and new contract formats are required. This relates also to the fact such contracts continue to be popular despite a history of failed or troubled projects, though they tend to work when costs are well known in advance. Such contracts may become the most expensive, especially when the risks or costs are unknown, as in tunneling projects.

According to the FIDIC Task Group 10 risk allocation and risk dependent costs can be shown in a simple way as in the following figure. In short terms it suggests that a fair risk allocation is likely to produce the lowest construction cost. The question to be asked must therefore be; how do we arrive at a fair risk sharing and the lowest construction cost for any given project, taking into account the uniqueness of the project and its particular risk aspects. A change in game plan may be required to arrange contract formats with an improved risk sharing.

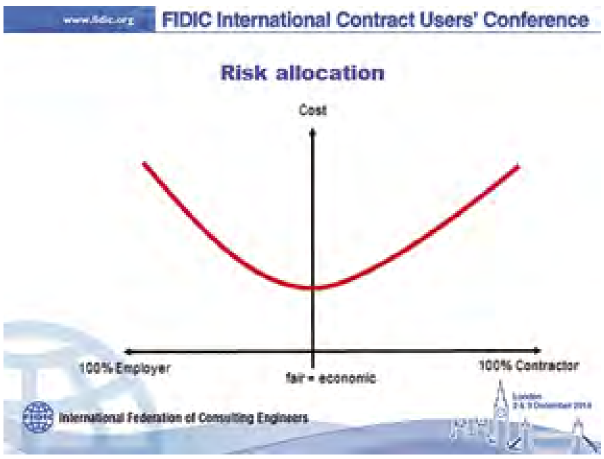

Figure 1. FIDIC-ITA Task Force 10 presentation of risk sharing to optimize the construction (FIDIC 2014) (Ref. 2)

Figure 1 shows a schematic way of identifying the most economically favorable risk sharing between an Owner and a contractor. In Figure 2 below a more detailed picture is provided on the same principle as in Figure

1.This paper aims at shedding light on the risk sharing principle being typically employed in Scandinavia, and in particular in Norway, it will also describe the various elements that builds up to such a risk sharing principle.

RISK SHARING NORWEGIAN STYLE

By far, most underground projects in Norway during the last 50 years have been contracted as unit price contracts. During the hydropower boom in the 1960's through the 1980's, a contract concept was developed and applied that focused on risk sharing. During this period more than 150 mill m3 of solid rock has been excavated, with an average output from the tunneling industry at around 3.6 to 4 mill m3 of solid rock. One of the first papers published on this issue was Kleivan et. al in 1987. (Ref. 7). The risk sharing unit rate contracts address two main elements of risk:

- Ground conditions. The owner is responsible for the ground conditions. He 'provides the ground', and is also responsible for the result of the site investigations he finds necessary to do. If these prove to be insufficient, it shall remain his uncertainty and risk.
- Performance. The contractor is responsible for the efficient execution of the works. He shall execute the works according to the technical specifications. He is reimbursed according to tendered unit prices for the work actually completed. The construction time frame is adjusted based on preset 'standard capacities' ('time equivalents') for the different work activities, if the balance (increases minus decreases) of the work changes.

One important aspect of the geological risk for the parties is that at the time of preparing the geological summary report for the tender, and the contract documents holding project time schedule and BoQ, the Owner or the Owner's consultant shall provide the best estimate and judgement of the ground condition at the time the assessment is done. This was indeed an important message brought by Kleivan et. al in 1987. (Ref. 7) By this, the owner keeps the risk of increased cost if the ground conditions prove to be worse than expected; after all he chose the site location and instructed the investigation program. He will also earn the savings if the conditions are better than expected. The contractor keeps the risk of own performance. If he is less efficient than the norm set by the 'standard capacities', he may fall behind schedule and will have to catch up on his own expense to avoid penalties. If he is more efficient, he may finish earlier, save money and increase his profit, besides what he is hopefully earning within his unit prices.

The risk sharing principles ideally eliminates most discussions about 'changed conditions'. It becomes a matter of surveying the quantities performed, and the payment and construction time adjustment follow accordingly. The system is simply a balance sheet for time. This works well as long as the variations in ground conditions can be dealt with by just applying more or less of the work activities regulated by the tendered unit prices and the preset 'standard capacities'. This however assumes that all necessary work activities are included, which may not be the case if truly unforeseeable geological feature occurs. This system, its development and application was described by Kleivan et al. (Ref. 7) who coined the term NoTCoS – the Norwegian Tunnelling Contract System. In Figure 2 it is illustrated how this risk allocation produces the lowest cost possible in average for a number of projects. Note that almost 30 years ago it was assessed in a schematic way the implication of different contract formats, and their combined risk allocation on the two main parties in a tunneling contract.

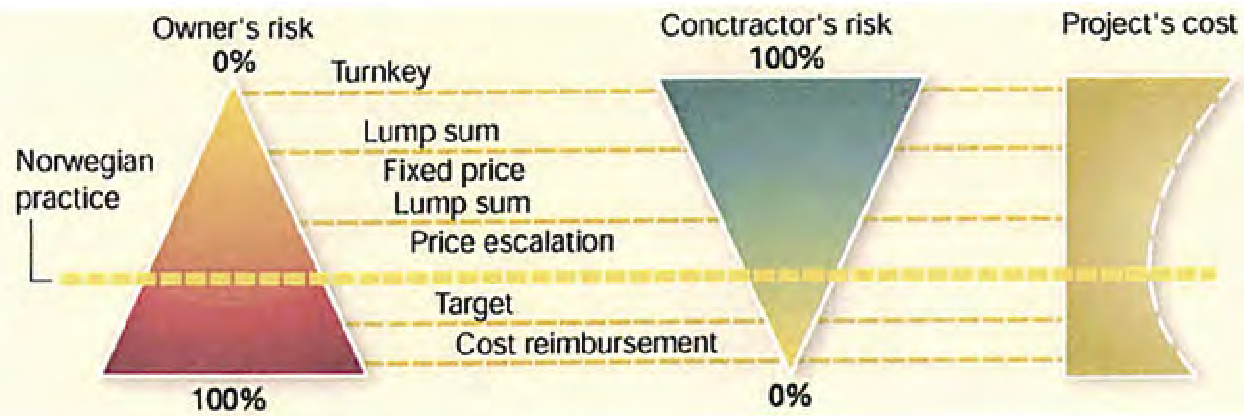

Figure 2. Risk allocation principles Kleivan et al. 1987 (Ref. 7)

CHARACTERISTICS OF UNIT PRICE CONTRACTS

The typical unit price contract in Norway is characterized by the following (Ref. 8):

- Geological/geotechnical report. This report is prepared for the owner based on the performed site investigations. It shall give a full disclosure of the information available, as recommended by the 1992 document by ITA. Traditionally it also contained interpretations, not being limited to factual data, but this practice has unfortunately been compromised by some of the larger public owners. It is a pre-requisite that all important geological features have been identified. The tenderers shall anyway establish their own interpretation.
- Bill of Quantity (BoQ). The quantities for all work activities, such as excavation, rock support, grouting, lining etc., as well as installations, are included in quantities according to the best expectations by the owner assisted by his advisors. Preferably, the owner shall refrain from tactical inflation of the quantities in order to get lower unit prices. Tactical pricing from the tenderers may occur, but can be discovered by analysis of the bids.
- Variations in quantities. The actual quantities may vary due to variations in the ground conditions. The contractor is reimbursed as per actual performed quantity and his tendered unit prices. The unit price shall remain fixed within a preset range of variation, for some contracts this may be set as high as +/- 100%.
- ‘Standard capacities’ (‘time equivalents’). Traditionally the 'Standard capacities' have been set by negotiations between the contractors’ and owners’ organizations. They may be updated concurrently with technology developments, but are usually kept from contract to contract over a period of a few years. As long as they are reasonably realistic, they provide a fair tool for adjusting the construction time and completion date if the balance of ‘time equivalents’ increases more than a specified amount. Typically the range which is included in the contract is +/- 3 weeks, and all regulations of the construction time beyond this range shall lead to a prolonged or reduced construction time.

For this system to work properly, some conditions are important to pay attention to:

- Owners and contractors. Both parties benefit of being experience with underground works and the site management teams from both sides must have the necessary authority to take deci-sions, allowing technical and contractual issues to be solved at site as they occur. This requires respect for each other's tasks. It needs to be said that the main owners such as the Norwegian Public Road Authorities, Railway Administration and the power development companies are multiple clients and owners of building underground facilities. 'One-time' clients to underground projects may have a different perspective on risks associated with tunneling works.
- Decision making. Of critical importance is the ability and authority of the representatives of both parties to take decisions at the tunnels face, especially with respect to primary rock support and ground treatment as pre-grouting etc. That again call for experienced, educated people through-out the project organizations of all parties involved; client, contractor and consultant.
- Acquaintance with the contract. If both parties are acquainted with the principles and details of the contract, discussions and agreements can be made expediently and with confidence as need arises. This is typically the situation when both parties are experienced from a number of similar projects.

A main advantage with this system is that the contractor’s incentive to meet the penalty deadline will be maintained, even if ground conditions get worse. Contractors have recently voiced as a disadvantage that their role is limited to performing the specified work for the owner without incentives to introduce innovative solutions by which the contractor could better utilize his special skills. Some owners do not ask for, or even allow, alternative solutions to be introduced. However, this is not a result of the type of contract being employed, but only to the way it is applied.

CONTRACT CLAUSES TO TACKLE VARYING QUANTITIES AND CONSTRUCTION TIME FOR ROCK SUPPORT MEASURES

As a part of Norwegian tunnelling important decisions are taken at the tunnel face, both related to the need for measures ahead of the tunnel face and support at the face. A possible consequence is that a considerable difference might occur between the stipulated quantities in the contract and the actual quantities as carried out. This is well taken care of in the typical contract formats that are applied, such as 'Prosesskoden' developed by the Norwegian Public Roads Administration and also used by the Railway Administration, whilst the standardisation bureau Standard Norge has its own 'Construction Contract'. To tackle this, the contract has defined “the 100 % rule” describing support:

- The unit prices apply even if the sum of actual quantities differs from the BoQ by up to ± 100 %.
- If the owner or the contractor wishes unit prices to be adjusted, prices are set by negotiation.
- The adjusted unit prices shall not differ from the contract’s unit prices by more than 20 %. Adjusted price shall be determined according to documented expenses.

These regulations take care of differing quantities that might occur due to changes in the en-countered geological conditions compared to those anticipated, but not the fact that varying quantities also have an impact on the contractor's available time towards the date of completion. To handle also the aspect of construction time a contract clause has been introduced that is called "the equivalent time principle" for adjusting the total construction time depending on the actually applied support methods and quantities. This is particularly related to tunnelling operations that are needed to secure a safe tunnelling but are hampering the tunnel advance:

- If the actual quantities for tunnel support vary in comparison with the contract's estimated quantities, the completion time is adjusted according to predefined standard capacities for the different operations, for example:
  - Manual scaling 1 hour/hour
  - Bolts up to 5 m 12 bolts/hr
  - Sprayed concrete (shotcrete) 6 m3/hr
  - Concrete lining 0,1 m/hr
  - Exploratory drilling and pregrouting 60 m/hr
- The total time for support measures is summed up in hours, both performed and described amounts from the bill of quantities.
- The difference (between accumulated values) is calculated
- The contractor normally has a tolerance for added support measures (typically a week per year of construction time)
- When this tolerance level is exceeded, the exceeded time value is calculated as shifts and days, which are added to the completion time.

These standard capacities resulted from negotiations between the contractor's organisations and representatives from the owners. The standard capacities reflect the state-of-the-art in Norway, based on equipment and methods being standard at a given point in time, and may not unconditionally be transferred to other countries. However, the equivalent time principle has proved to be a useful tool for sharing the risk for both owner and contractor. In combination these two clauses are useful tools to remove some uncertainty regarding risk in tunnelling contracts, meaning that the risk that the contractor has to carry is considered as fair. The owner must always bear in mind that risk has a price. In order to reduce the total construction sum, we must try to reduce the contractor's risk as well. No matter the type of contract chosen for a project, if the contractor is forced out of the contract, by termination, bankruptcy or something similar the ultimate risk taker would be the owner. In figure 2 above a classical risk principle is shown. In the long run it shows that the Norwegian contract practice based on unit rate contracts would in average produce the lowest construction cost.

INCREASED NUMBER OF COURT CASES

Despite the advantages and good track record of the typical unit price contracts in Norway, an increasing number of projects end up in litigation. This appears often to be due to:

- Inexperienced owners. The owner may be lacking experience with underground projects. Deviations from the expectations may put him 'off his feet' and the co-operation with the con-tractor deteriorates into contractual confrontations, instead of solving the problems as they arise.
- Insufficient funding for contingencies. The project may be based on too optimistic cost estimates. This could be by purpose to get approval from the authorities or by sheer lack of respect for the potential variations of nature.
- Public scrutiny. Public projects may be subject to criticism for any decision made during construction that deviates from the expected. The project management may prefer to stick to the letter of the contract in order not to be criticized, and allow disagreements to accumulate and be dealt with in court instead of using common sense
- Tougher profit requirements. The contractors, in order to survive in an increasingly competitive climate, focus on the economical result of their contracts. If a contract does not bring the planned profit by just performing the contracted work, it may be tempting to seek additional compensation in court.

SETTLEMENT OF DISPUTES

During the recent years basically all Norwegian contracts contain a clause stating that disputes that are not resolved at the construction site through ordinary meetings, are raised to a dispute resolution forum on a higher level. This forum includes representatives from the company management of both the owner and contractor. The representatives from both owner and contractor may agree to invite experts who may advise a solution. There is currently a drive in the tunnelling industry in Norway towards obtaining again solutions at the construction site to avoid disputes being brought to arbitration and court. Such solutions may involve both technical as well as commercial and contractual aspects.

In a couple of large projects, for instance the Bjørvika immersed tunnel in Oslo, dispute review boards have been appointed. Feedback so far suggests that the DRB's are playing an important role in resolving disputes. An additional effect is that the DRB's mere existence seems to have increased the willingness to reach a solution at the site meetings. If the dispute is not resolved by any of the chosen means, the ultimate solution still remains to forward the case to the court.

LESSONS LEARNED

In the articles by Blindheim and Grøv (Refs. 3 and 4) the authors conclude the following constitute lessons being learned by experienced parties, still these lessons would provide good input to all owners:

- Independent of the type of contract, it is important not to become too confident about the results or rather the interpretations from the site investigations prior to construction. It is necessary to rely on relevant and sufficient site investigations, still maintaining the respect for the potential variations of nature, both regarding variations of foreseen features, but also regarding the unforeseeable, the features that nobody expects. The systematic use of an independent project review, by a party not identifying itself with the project, is advisable.
- In unit price contracts, which normally allocate all or most of the risk for the ground conditions to the owner, it is easy to deal with large variations of quantities in a fair manner, as regulation mechanisms are built into the contract. If unforeseen features occur, for which there are no methods and quantities available in the contract, separate agreements need to be established, and cost reimbursement may be a suitable way to solve an intricate and difficult situation.
- Fixed price contracts, with all risk for ground conditions allocated to the contractor, may have an apparent predictability of cost, which may be attractive to the owner. However, this type of contract imposes risks on the contractor that may at best be difficult to quantify, at worst disastrous if the unforeseen or unforeseeable occurs. Such risks may become the owners problem, no matter the contract text, e.g. if the contractor is not able to bear the loss and complete the project.

REQUIREMENTS TO THE CONTRACT

The author believes that a suitable balance for risk allocation can be found, allowing a combination of the advantages of both unit price and fixed price contracts. It follows that the risk allocation must be specified in the tender documents, to the level of describing the geological features or the stabilization and ground treatment methods that are included in the contractor's risk. Not to forget: the contractor must be able to price the risks allocated to him. In developing such contracts, it may be useful to define success criteria for the project along these lines:

- Cost: The aim is to get the total cost as low as possible, including both the price for realistic tenders and the risks that remain with the owner. Predictability of total cost may come at a price.
- Compliance: The owner has to set the quality standards considering the life-time costs. Durable solutions are not for free. This also relates to other aspects such as compliance to general and overall standards and project specific specifications.
- Completion: Both parties have a strong economic interest to keep the completion date. The timely completion is probably the success factor that is most easily monitored by the public. The construction time can still be adjusted according to preset regulations.
- Confidence: The confidence in the outcome of a project is imperative for financing institutions and for the public as well, who in many cases are the users. This includes safety during and after construction towards hazards such as collapse, water flooding, and loss of the tunnel or of lives. In modern safety regulations the owner has an overall responsibility for safety, whereas the contractor maintains the executive responsibility.
- Control: The contractor needs to control (in the sense of ensuring) his performance. If this is done according to modern quality management principles, the owner may rest 'assured'. The owner may still want to survey the performance of the works, both with respect to quantity (progress) and quality.

CONCLUSIONS

In order to achieve success according to the above, the following contract requirements may apply:

- Incentives. By including incentives for the contractor, not only penalties, it is possible to stimulate focus on productivity, while maintaining quality and safety. Experience shows that in standard unit price contracts it may be tempting for the contractor to increase his production volume by applying more rock support than strictly necessary, especially if particular support measures are tactically priced. If he instead gets a bonus for early completion, and possibly also a compensation for saved rock support ('lost production'), this may turn around. The owner will then have to follow-up to ensure the sufficiency of the rock support for permanent use. The maintenance of safety during construction under such circumstances may be challenging, and requires experienced personnel for follow-up.
- Conflict solving. It is important to keep, or get back to, the problem solving at site instead of in the courtrooms. A tool to achieve this may be the use of advisory 'reference groups'. A key point is that such groups meet on a regular and frequent basis to monitor the works, before small problems develop into conflicts. In this respect a 'reference group' may have a different function than 'dispute resolution boards'

dealing with already materialised disagreements. The responsibility of such ‘reference groups’ should be defined in the contract. The personnel should be nominated by the parties and include professionals with practical tunnelling experience.

- Co-operation. Although it is frequently expressed in contracts that the parties have a duty to co-operate, as is the case with Norwegian contracts, this may not always come easy. It may be effective to stimulate this by focusing on the strong common interest in completion on time. However, other tools may also be used, e.g. ‘geotechnical teams’ to which co-ordination of geotechnical issues can be referred and disagreements about e.g. choice of rock support measures can be solved.
- Functional requirements. The use of functional requirements, rather than detailed technical specifications and work instructions, may stimulate innovation and development by the contractor. However, functional requirements are not easy to apply for rock works, and the result of many of the work processes does not lend itself to quality checking afterwards (e.g. grouted rock bolts).
- Regulations for ‘changed conditions’. As the inclusion of all uncertainties in a fixed price may result in a very high price, it may be beneficial overall to be specific about the risk allocation. A suitable balance may be found by identifying which features shall be included in the fixed price and which are kept as a risk of the owner, to be reimbursed by specified regulations. To include risk sharing clauses would be fully in agreement with the recommendations by the International Tunnelling Association (Ref. 1).

The experience shows that unit price contracts are suitable to deal with ‘unexpected geological conditions’, as long as the ‘unexpected’ element results only in variations in the quantities of work acti-vities. This means that all necessary work activities must have quantities and preferably also ‘standard capacities’ for regulation of the construction time. In fact, variations in quantities must be expected in any underground project, and such variations therefore hardly deserve the term ‘unexpected’. If truly unforeseen geological features necessitating work activities not included in the Bill of Quantity, the unit price contract must be supplemented by special agreement, usually some form of cost reimbursement. Fixed price contracts for underground projects, may not provide the intended predictable cost. Modified or ‘adjustable fixed price’ contracts, combining elements from unit price and fixed price contracts, may prove to be more suitable than fully fixed price contracts, and easier to handle than unit rate contracts.

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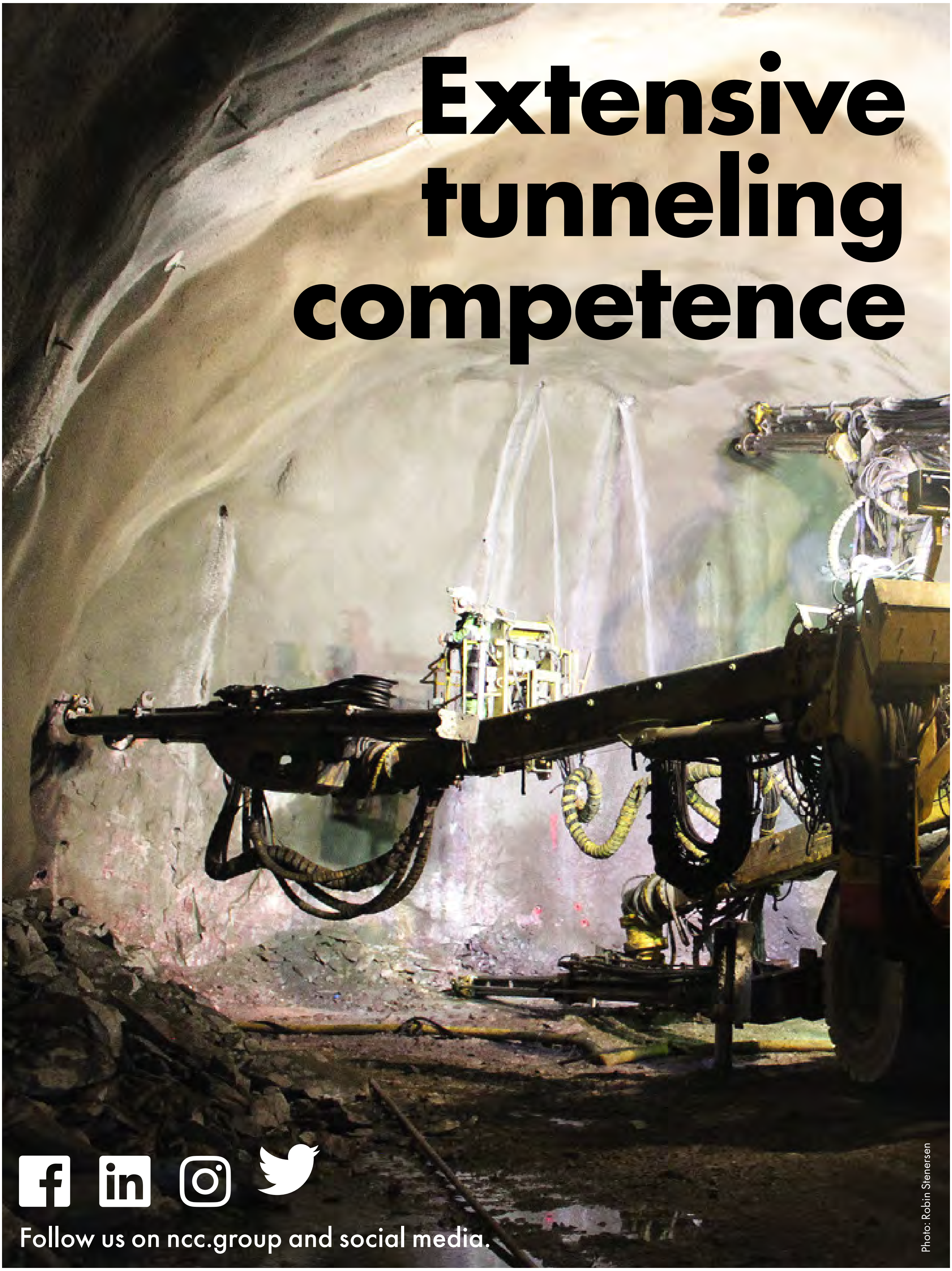
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3.3 DECISION TAKING

Geological Mapping
Crucial for the determination of the rock support and grout works is well mapped and documented rock mass conditions along with the tunnel excavation. Typically the rock mass is classified using quantitative classification systems such as the Q-system. New regulations in the construction contracts allow the geologists half an hour for mapping and classification of the rock mass following each blast round. The contractor is reimbursed for the possible hampering that is caused on the tunnel progress by the mapping. It is important that the mapping and classification, if possibly is done before the rock surface is covered by sprayed concrete (shotcrete) or by other means. The rock surfaces are inspected and geological features are registered along with classification in order to ensure that appropriate support measures are chosen. This process also ensures that a record of the geological conditions that were encountered during tunnelling exists. Such recording serves as geological as-built recordings and reference use, if for any reason a need arises in the future to document the conditions behind the support and inner lining.

Construction
A key element in the cost effective tunnelling is the Contractor`s performance during construction. Machines are becoming modernised with computer aided rigs (for drilling, bolting, shotcreting and grouting as well as a number of other activities). These could include:

- High capacity equipment, with multi-skilled workmen at the tunnelling face allowing high utilisation of the equipment .
- Adaptability to the actual ground conditions by careful following-up of the encountered rock conditions by mapping and classification for a best fit the of rock support measures.
- Observation of the ground behaviour by visual surveying and physical measurements if required fulfilling the intentions of the Observational method to ensure a stable tunnel profile.
- Installation of permanent rock support as close to the tunnel face as practically possible and advisable for the utilisation of the resources at the site.

Co-operation
The participants in underground construction have different objectives. However, in a broader perspective there are probably more common interests at the construction site than interest of conflicts. This includes such topics as:

- Respect for the different roles and values as tunnelling is a complex process and various skills are needed at the construction site.
- Constructive co-operation between the representatives of the involved parties.
- Experienced professionals participating in the decision making.
- Solve conflicts at construction site by negotiation after the technical issues have been settled.

Organization and Contract conditions
Norwegian Tunneling is recognized by an informal organization culture and fast decision making. This is often made by a better method description as named; The Observation Method. This word may be better and more correct, as it is linked to what is in facts taking place. This due to Contract conditions as described in chapter 12 and the Norwegian culture, where the respect for each representative is based on knowledge and know-how more than authority respect. It is with other word, based on equal respect of the Parties experience in all matters. The interaction between the Client, Contractor and Consultants are equally parties, but with different perspective, as visualized below.

Relation and interaction between Client, Contractor and Consultants.

HSE, Health – Safety and Environment, is in NMT and Norwegian traditions looked upon as a rather very important elemenst for running a tunnel and operate in a challenging environment, as what underground is. It is a respect for the safety of all people involved and also the long term stability of a tunnel when it is finished constructed, and the Client has signed it over. It is by looking at the HSE- situation based on defined areas where the responsibility is clearly defined.

- **The Contrcator** is responsible for the safety of his people and by then the total responsibility for safety on the construction site in all matters.

- **The Client**, who will be taking over the construction, object after it is finished, will have a saying in all matters. He will approve the rock support method.
- **The Consultant**, is the advisor for what is going to be approved, and he will with expereuince and requiremenst approve a solution, commonly agreed upon.

The Contractor alone is responsible for the work safety; he is responsible for his employee and undergo common regulations in Norway and by approved standards. By that it will always be a challenging situation to object against the saying of those responsible for the safety of the men working in the tunnel. Here is for sure where respect for each profession is needed.The Client is responsible for the permanent rock support and given advice from his the Consultanting Engineer. Regarding the excavation and rock support method this is often an issue openly discussed between all parties. The 3 involved parties discuss openly and feel free not to be bound by the tender document, as often experience is telling us that from a theoretically proposed concept, we have suddenly a different situation when we face actual rock support. The Client makes the final decisions based on input from the Consultants and the Contractor. During excavation of tunnels this is specially visualized at the face.

Typical discussion at face between Client, Consultants and Contractor.

Description of the different decision-making players in excavation and rock support in a typical tunnel project
The main subjects for decisions in a tunnel is linked to the following elements, but always related to engineering geology. That is why it is of the greatest importance that qualified engineering geologists are present on site at any time. Over the last years the requirements for educated engineering geologists have increased. The elements we are talking about are:

- General rock classification.
- Rock Support in general, where to use:
  - o Rock bolts,
  - o Length of rock bolts
  - o Shotcrete/ sprayed concrete, reinforced with steel fiber or not
- Full concrete lining on the tunnel face.
- Pre- injection, grouting at the tunnel face
- Or even shorten blast length for each blast, either reduce cross sections of length of the excavation round.

These elements are to be decided upon from those involved, and we look upon the situation at the tunnel face as a living situation, as rock as Norwegians see and look upon it, is a living structure, with variations from day to day, aling the alignment where the excavation takes place.

Client:

- Clients Project manager:
  The Clients Project manager, (The Engineer on site) is the Client's representative on site . He is there to look after that the project is running according to the contractual situation, and that all decisions are made in accordance with that. The challenge for tunnel work is all elements not accounted for, which require a fast decision, or consensus for where and how to go further. The Clients representative does not necessary need to be an experienced tunnel engineers, but some knowledge will be an advantage.

- Inspector:
  An Inspector might be following each shift and has often authority to make decisions at face. This person can be engaged through a consultant company.

Consultant:

- Engineering geologist/ Tunnel Engineer:
  The Engineering geologist, or Tunnel Engineer, will be following all excavation. They are responsible for collecting geological documents and assist the Client in deciding permanent rock support. Has also a key role in decision of excavation and pre-injection method.

Contractor:

• Tunnel worker:
The tunnel worker is normally skilled in rock- and machinery works and trained in a tunnel team. Each team consists of three to six members. This team is capable to perform all tasks excavating tunnels, including drilling, charging, blasting, excavating, scaling, install rock support, and to execute machinery maintenance and back face works. Each team has a Team Leader, who is responsible for work safety and temporary rock support. The Team Leader is also the blast manager and responsible for drill- and blast patterns.

• Foremen:
The Foreman follow each shift and I the first line manager by the Contractor. He is responsible for the daily HSE matters and makes sure that the works is executed according to the Contract and Clients instructions. Daily communicating with Clients Inspector.

• Site manager:
The Site manager is responsible for the entire production and makes production decisions for Contractor. Communicating with Engineering geologist and Client Project Manager.

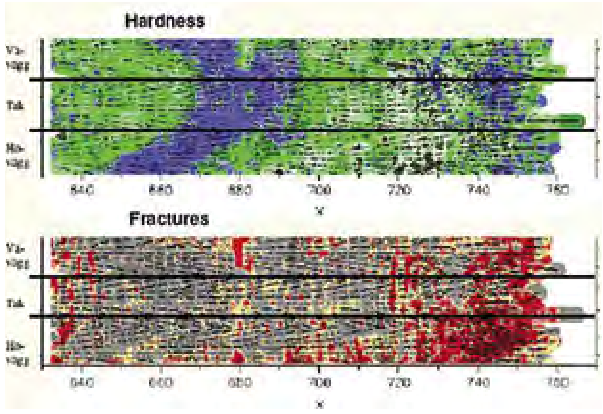
• Contractor Project manager:
Contractors Project manager has the overall responsibility for Contractors Contract obligations and communicate with Clients Project manager.

Basis for tunnel excavation – what kind of decisions are made at site

In Norway there is focus on good geological pre-investigations, good site investigation metghods and result. This will ben efit all åparties involved in a later decsission for how to perform rock excavation and rock support. This is done up to a certain level with the main purpose to give a the general description of the geological conditions, major weakness zones and give the basis for estimation of rock support and scope of pre-injection. The other purpose of the geological pre-investigation is to give vital information of the expected geological conditions. The contractor uses this information to calculate the unit prices and the duration of the excavation works.

Based on the geological report from the pre-investigation the Consultants work out a description of how the tunnel shall be excavated, including types and quantities of rock support and pre-injection. The Contractor gives unit prices based on this information. This is the basis before the excavation works starts.

During excavation the real geological conditions occur, and both parties will do their mapping for each blast. The inspector and/ or the engineering geologist do the classification, normally according to the Q-system and decide the permanent rock support. In addition all bore holes are mapped with a Measure While Drilling system, which the engineering geologist use in addition to the on face mapping both to decide permanent rock support and description of coming pre-injection, see figure .



Mapping by use of Measure While Drilling data.

The work rock support is decided by the Team Leader foreman, but normally the Inspector/engineering geologist, the Foreman and the Site manager discuss necessary rock support together with the Team Leader foreman they will try to implement the permanent rock support as a part of the temporary support. At an early stage is also usual to decide a Rock support classification system, where the size and the construction and the actual rock condition is considered and the Q-system used to give a guiding scheme to be used for planning of efficient rock support installation. We will underline that The Q- System can only be a guideline, as the actual conditions on the tunnel face , may change from day to day and not even fit into any Rock Classification systems. The Q- System is a very good tool for classify what might occur, but we underline, it is Not a “black and white” system to be executed in practical life. An example is given in figure xx. Usually a consensus of the level of rock support for normal rock conditions will be established between the involved parties and included in the rock support guiding scheme. This gives the tunnel workers good conditions for high productivity. The Norwegian tunnel worker is known as multidiscipline, hard workers which solve all tasks within the team. This result in high production and low manning cost although the Norwegian tunnel workers earn a lot above average.

Rock support classes	I	II	III	IV	V
Rockmass quality	Good	Intermediate	Poor	Very poor	Extremely poor
Class	A/B	C	D	E/F	G
Q-verdi	Q ≥ 10	4 ≤ Q < 10	1 ≤ Q < 4	0, 01 ≤ Q < 1	Q<0,01
Bolting in roof c/c and length	2,5m x 2,5m L=4m	2,0m x 2,0m l=4m	1,7m x 1,7m l=4m	1,3m x 1,3m l=4m *	Casted concrete
Bolting in wall c/c and length	Spotbolting L=3m	Spotbolting l=3m	2,0m x 2,0m l=3m	1,5m x 1,5m l=3m	
Shotcrete roof (mm)	80mm	80mm	100mm	150 mm + reinforced shotcrete arches	Special design
Shotcrete Wall (mm)	Scaling	80 mm 1,5 m over sole	80 mm 1,5 m over sole	100 mm	

Figure xx: Typical diagram for description of rock support given by mapping using the Q-system.

It is in special geological zones, as weakness zones, very poor rock conditions, if the pre-injection receipt doesn't give the foreseen result or if the geometry is challenging that the Norwegian Method of Tunneling is especially efficient. In these cases all experience and know-how is brought together to find the best solution for each case.

An example of an efficient interaction between all parties was given during construction of a sub-sea tunnel in Norway. The rock mass consisted of very crushed rock, described as extremely poor. The normal procedure would be to cast concrete at face, but instead they decided to pre-inject the rock formation, install spiling bolts and shotcrete arches and blast the cross section in two blasts, see figure.

At this location the Engineering geologist and the Contractors Site manager together decided double spiling bolts, perform pre-injection to stabilize the rock formation, to blast the cross-section in two blasts and use reinforced shotcrete arches together with spiling c/c 2 meters.

At a hydro power project one of the inlet tunnels had so high water inflow that it became a threat for feasibility to complete the tunnel. The water inflow was up 6000 liters per minute with a pressure of up to 40 BAR. A team from the Consultants and the Contractor including one of the skilled tunnel workers came up with a new idea for pre-injection with accelerated micro cement through a valve fixed to a pipe.

The newest indication for expected water problems on face, and in addition weak zones, is drill sink. Most drilling jumbos of today have an indicator to measure the drilling sink or speed of drilling a hole. This can indicate slopes, cracks or crushed/ weather rock. By that the observation by the drill operator and Team Leader on the Jumbo is of importance in judging what kind of rock conditions we might have in-front of us. By that a lot of the forthcoming rock support might be estimated here. And for sure the estimated injection volume, especially grouting, it be cement, micro fine cement of chemical grout.

Decision making at face – historical development
The Norwegian Method of Tunneling has been developed through more than 100 years. Technical development and improvement has been a vital of the Norwegian tunnel industry. Introduction of the Air leg, pneumatic and hydraulic drilling jumbos, introduction of dry- and wet-sprayed concrete and development of rock-bolts and other rock support products have been some of the achievements in Norway. Many in the Norwegian heavy Construction business have got their first work experience at agricultural farms or in the forest industry. The cultures in these businesses are recognized by hard work, thriftiness and inventiveness and this has been adopted into the Norwegian tunnel industry. The authors therefore assume that this is one of the reasons why the NMT is cost effective, has high productivity and continuous developing. Especially during the huge hydropower plant construction period from 1960 – 1980 the NMT developed a lot of know-how and experience to become a leading tunnel nation in the world.

Through traditions from a political life in the general society, it is also reflected over to how the Union of workers is corporate with the Society of contractors for how to work together. A tradition, developed over years, has given the Norwegian Society a weld of tools and traditions of how to work together. This respect for working life, also in the tunnel industry, with the professions of each involved party, has been a success of benefit for all. Through corporation, common respect of each party, a forum for discussions, where different arguments will be listen to. All parties have the same goal, a safe and quality constructed tunnel. This is in a small scale, what is taking place at the tunnel face, after a blast, often before the rock support takes place. The respect for each party's view benefits a total solution.

Under the influence of Norwegian Clients, by introduction of risk assessments, through the need for better safety along railways and high ways and through a better focus of HSE matters, the Norwegian tunnel worker role has changed. Their influences in the design and excavation method have slightly been reduced by introduction of requirements for planning, documentation and detailed control. Today their involvement is still very strong and includes other aspects as risk analyses and planning. The high-tech drilling jumbos have changed the work environment and today a Team Leadr need a lot of IT knowledge just to operate the machinery. Still is the need for high knowledge about rock conditions huge by the way tunnels are constructed in in Norway. The rock formation is the major part of the construction and must be supported by the right tools to be safe and cost effective.

The high tech industrial drilling jumbo will still not be able to take over the human experienced eye for how rock conditions are by looking at the tunnel face. It is the responsibility for humans, also long term stability that creates a situation that is a win- win for all parties. The Observation Method, which by practical seen will be, The Tunnel will be decided upon a long as the road is built. It is never the set given situation to be followed the next day, if ground conditions change. It is always the given situation on face that will need to be handled there at that spot/ moment. Economically, Norwegian traditions also believe that this will give the best result. It will befit all parties.

3.4 STATE OF THE ART TUNNELING EQUIPMENT

Introduction
The Norwegian tunnel contract have today a modern and up-to-date fleet of machines for tunneling. It’s common that they possess the machines and rent the machines out internally to the projects. They have in general good knowledge about the machines and they service them well to protect the investment, but more importent; safeguard high continuous production!
High efficiency is key to run a financially good project, but there are also many other causes for the selection of machines you will find on a Norwegian tunneling project. Health and safety is always a high priority. High level of automation, like use of rod handling systems on the tunneling drill rig, is very common. If you can use automation to reduce human interference you will for sure have less accidents. To have a small team working at the face that can perform several different tasks is another important cost saving factor. If you have the right type of equipment you can for sure manage with less hands. The clients demand for high quality and god reporting during the lifetime of the project is another strong factor that will influence the choice of machines and how well they are equipped.

Tunneling drill rigs
Tunneling drill rig with three booms and service basket are common to use on an infra-structure project with larger cross section. It is essential to cover the width and height in one setup and to navigate fast. A total station or profiler is used for navigation. You will also scan the profile to make sure you don’t have underbreak or overbrake. The profiler is operated by the operator. The hard rock demands use of high efficiency rock drills to get an acceptable penetration rate. The currency used is 1000 Volt. Almost all tunneling drill rigs have 20’ feeds to make a theoretical blast depth of almost six meters. The drill rigs have highest automation enabling several holes to be drilled without interference of the operator. You will save time, get god fragmentation and a nice contour. A good fully automatic rod handling system is important, the time spent on long hole drilling can be a big part of the total time spent for one cycle. Charging is done from the service basket on the tunneling drill rig. In the service basket you will have hydraulic and electrical connections for controlling the charging. The tunneling drill rig is also used for drilling the bolt holes, and the bolts are installed manually from service basket on the drill rig. Data like MWD (measure while drilling), drill plans and machine status are transferred wireless to and from the tunneling drill via the WLAN network installed in the tunnel.

Atlas Copco Boomer XE3 C

RCS5, Rig Control System from Atlas Copco

Concrete spraying rigs
The faster the contractor can apply the sprayed concrete, the better it is for the overall economy of the project. High capacity concrete pumps are used with the theoretical capacity up to 30 m3/h at 50 bar and a boom with large covering area to make sure you need only one setup. A truck is sometimes used as a carrier, if it is a road tunnel, so that you have the possibility to move on the road between the two faces. The spraying rigs are diesel hydraulically powered and onboard you have the compressor, tanks for accelerator and dosing system. The control systems are computer assisted for increased accuracy. The production data is logged and reported back to client. The sprayer are radio controlled by the operator from the ground. Some sprayers have the possibility to operate it from an elevated cabin on the sprayer boom.

Atlas Copco Meyco ME5

Picture showing AMV 7450 Hybrid shotcrete robot with operators cabin in operation.

Grouting

The grouting process can be a very time consuming part of the time spent on the tunneling cycle. A complete concrete factory built into a 20 feet container is normally used. In the container you will find four lines with separate piston pumps that deliver up to 100 Bar. Silo, mixer and agitator are also on board. The mixing capacity is the key when you have continuous injection, not only the capacity of the grout pumps. Control systems are used for quality control and operation excellence. Data are logged and reports handed over to the client to determent quantity and quality of the job done.

Trucks

A typical truck used for this task is a 6x4 or 8x4 truck. They usually have propulsion on 2 rear axles with hub reduction, and are operating on weights for a 6x4 at 26000 kg and for an 8x4 at 32000 kg. This is legal weights for normal road transport, but if the dump area is located within the construction area, the weights is often limited to the capacity of the dump truck. The capacity is normally about 11-12m3 for a 6x4 truck and 14-16m3 for a 8x4 truck. The most common engine-size for these truck is about 450-650 hp. The trucks used for this purpose is the same trucks that are used in all other construction sites, but in big projects that a transport provider is given a contract for 2-3 etc. years of tunnel operating, it would in some cases be wise to specify the truck with a stronger tipper body and thicker bottom and sidewalls because of the increased wear of tipping stone.

In tunneling work where space is no issue, it is also common to have a trailer after the truck in these kind of transports. Usually a 3 axle trailer for a 6x4 truck and a two axle trailer for a 8x4 truck. Then there is also a possibility to use a 6x4 tractor with a semi-tipper behind if there is sufficient space.

6x4Truck from Scania

Volvo wheelloader with side tilting bucket

Loaders
Loading equipment for use in tunnelling i Norway is mainly medium to large wheelloaders with side tilting buckets. This enables high loading capacity in tight spaces, such as a std road tunnel, without making specific loading areas with niches. The side tilting bucket enables the wheelloader to go in a straight line from the face and long the truck without turning.
Emptying of the bucket is done while passing the truck in one fluid motion, when the operator is skilled. A wheelloader with side tilting bucket is far more efficient in loading a truck than a wheelloader with a traditional bucket. The increase in loading efficiency lies between 25-50%.

For a 35-40t wheelloader the bucket size is around 5-5,5m3 and for a 50-60t wheeloader the bucket is around 7m3.

Due to the side tilting design and the required robustness such buckets have a mass from 6 to 11tonnes when empty and therefore requires additional counterweights or liquid filling of the rear tires of the wheel loader or both.

The normal operating life for a primary loader in tunneling is between 8-12.000 hours.

Working platforms, bolting rigs and concrete element erectors
Working platforms, bolting rigs and concrete element erectors are in Norway based on truck chassis in order to increase flexibility and reduce number of equipment used. All equipment is registered in order to move on public road and can therefore efficiently work on several faces using same equipment and personnel.

AMV 236-T Bolting rig used for drilling of bolts for membrane, PE foam and bolts for concrete elements.

AMV QR Working Platform with interchangeable platforms in order to change between various baskets and can therefore be used for scaling, charging, bolt installation, general tunnelling works , membrane installation and PE foam installation.

04. SOME EXAMPLES OF NORWEGIAN GROUND BREAKING TECHNOLOGIES

4.1 NORWEGIAN TBM TUNNELING – MACHINES FOR HARD, TOUGH AND ABRASIVE ROCK CONDITIONS

Overview of Norwegian TBM tunneling up to 2005

1 Introduction

A total length of about 258 km of tunnels with diameters ranging from 2.3 m to 8.5 meter has been excavated by TBM in Norway up to the beginning of the 1990-ties. The majority of projects include tunnels for hydroelectric power plants, water supply tunnels, sewer tunnels and road tunnels. All of the tunnel boring machines used in Norway in this period were of the Open Hard Rock Gripper type TBM.

In 1967 the very first fullface boring in Norway took place, executed by The Norwegian Hydro Power Board by boring of a 73 m long, 1.0 m diameter raise at Tokke Hydro Electric Project. In the beginning of the 1970's a couple of mines in Norway (Mofjellet and Sulitjelma) acquired equipment for boring of raises with diameters up to 1.8 m and length up to approx. 250 meters.

In 1972 contractor Jernbeton and the City of Trondheim entered into the first contract of fullface boring of a tunnel in Norway. The contractor leased a second hand 2.3 m diameter TBM and operators from a German contractor for boring of a 4.3 km long sewer tunnel.

Sulitjelma Gruber (Mines) became the very first owner of a TBM in Scandinavia. In 1974 a 3.15 m diameter Robbins TBM was bought for use on the initial tunnel section (4.5 km) of the main sewer system for Oslo City. Sulitjelma Mines was the first in the world to use constant cross section cutter-rings, which brought the hard rock tunnel boring technology another step forward due to increased rate of penetration and reduced cutter costs.

2 Probe Drilling and Pre-Grouting

On the Western Oslofjord Regional Sewage Project, nearly 40 km of tunnels with diameters ranging from 3.0 m to 3.5 m, comprehensive probe drilling and pre-grouting as well as post-grouting were required in order to avoid lowering of the water table and prevent damage to the buildings along the tunnel alignment. The drilling of probe-holes and holes for grouting on the first section were carried out by handheld equipment. It was recognized that on future similar projects it would be necessary to incorporate special equipment for probe drilling and drilling of grout holes on the TBM. For later tunnel contracts on the same project in 1976 and 1977, the owner made strict requirements for probing and pre-grouting. The contractors had to provide and demonstrate mechanized equipment and methods for efficient probing and pre-grouting. This became the most extensive probing and grouting program ever executed in connection with TBM operations anywhere in the world.

3 Prediction Model for TBM Boring

The Norwegian University of Science and Technology (NTNU), represented by the Department of Civil and Transport Engineering, has since the middle of the 1970's been a prime force for the TBM method and for the understanding and development of tunnel boring machines in hard rock. In cooperation with contractors, machine suppliers and tunnel owners the university has used the tunnels as full- scale laboratories and has in their project "Hard Rock Tunnel Boring" made a comprehensive collection and organized system of boring information, thus developed a prediction model for TBM boring. The model has formed the basis for better understanding and planning of full face boring projects and has given the contractors a good tool for detail calculation and scheduling for TBM projects. The prediction model is being used for planning and bid purposes on several projects abroad, as well. The prediction model has been internationally recognized by consultants, contractors and project owners.

In 2000 the university issued an updated and more comprehensive report on hard rock tunnel boring. This report (1-98) was prepared by Professor Amund Bruland and is part of his dr.ing.-thesis about hard rock tunnel boring.

Later, Nick Barton at NGI has presented a great amount of data from TBM boring and a prediction model based on the Q-system in his book ”TBM tunneling in jointed and faulted rock”.

In 2016 Francisco Javier Macias defended his Doctoral thesis at NTNU; “Hard Rock Tunnel Boring – Performance Predictions and Cutter Life Assessments” (The updated NTNU TBM prediction model). The PhD was linked to a joint research project named “Future Advanced Steel Technology for Tunnelling” (FAST-Tunn), which involved collaboration between Robbins, BASF, The Norwegian National Rail Administration, Scana Steel, BMS Steel, LNS Group, Babendererde Engineers and SINTEF/NTNU.

4 Tunnel Boring in Hard and Massive Rock Formations

Norway is generally considered to provide some of the toughest hard rock challenges in the world. With few exceptions, the first TBM projects in Norway started out in the softer end of the hardness scale, boring in greenschists, greenstone, shale, limestone, phyllites and micaschists. Later, tunnels in Precambrian rocks, granites and gneisses have been bored. The breakthrough for hard rock tunnel boring came in the period 1981-1984 with the accomplishment of the 8 km long, 3.5 m diameter transfer tunnel in Glommedal at Ulla Førre Hydro Electric Project. The area contained massive granite and gneiss formations with up to 210 MPa unconfined compressive strength. In fact, the rock on this project was so massive, that the NTNU predictor model was revised to include the fracture class 0 (zero). The Robbins TBM 117-220 worked for 2.5 years to cut through the massive rock on the 8,022 m diversion tunnel. The same TBM 117-220 has bored a total of 42 km of tunnel on five projects, all in hard rock formations.

5 High Performance TBM

In 1988, as a result of extensive hard rock tunnel boring in Norway through the seventies and eighties and the findings from comprehensive investigations and field studies of NTH-The University in Trondheim (now: NTNU –The Norwegian University of Science and Technology), The Robbins Company introduced in cooperation with Statkraft, the High Performance TBMs, using 19 inch (483 mm) cutters. The three first HP TBMs thus ever built, with diameters ranging from 3.5 m to 5.0 m, bored a total of 34 km of tunnels at Trollberget job site, Svartisen Hydro Electric Power Project. (Two other, second hand standard TBMs bored another 23 km of tunnels at the Svartisen Project). By this development, the cutters, TBM and the TBM Performance were taken to a new level, and gave the TBM industry an improved tool for boring hard to very hard rock formations.

The Main Bearings for the HP TBMs supplied to Svartisen are of the Tri-axial type. The change from Tapered Roller Bearings to Tri-axial Main Bearings has improved the utilization of the machines due to improved load characteristics and bearing life. Another big improvement is the wedge lock cutter housings that were introduced by Robbins at Trollberget job site for the first time. This new style cutter housing became the industry standard, leading to improved cutter life and less cutter changes. The development of the 19 inch cutters rated 312 kN/ cutter was another significant step forward in hard rock tunnel boring.

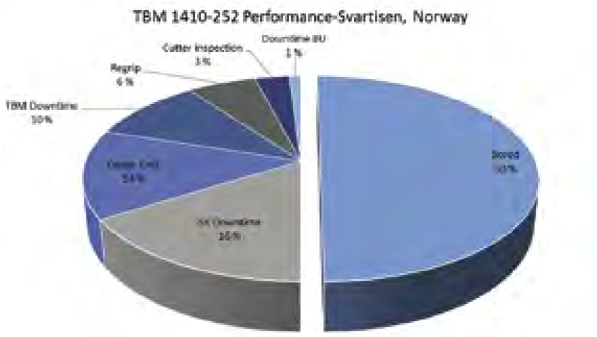


Figure 2 shows the performance of the 4.3 m diameter HP TBM 1410-252 after boring more than 10 km of tunnel at Svartisen project.

As a fact, the rate of penetration (ROP) increases exponentially with increased cutter thrust, while ROP increases linear with increase in cutterhead speed (rev per minute) up to an optimal rpm.

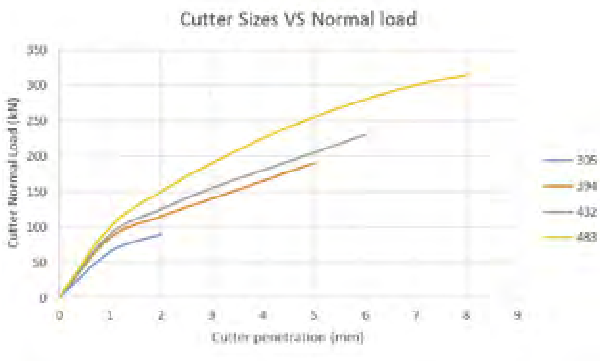


Figure 3 shows typical performance of disc cutters of four different diameters in hard granite

Experience from hard rock TBM boring with 19 inch cutters at the Svartisen hydro power project, shows that cutter penetration (mm/rev) increased in the range of 50-60 % by increasing the thrust from 250kN (17 inch cutter rating) to 312kN per cutter.

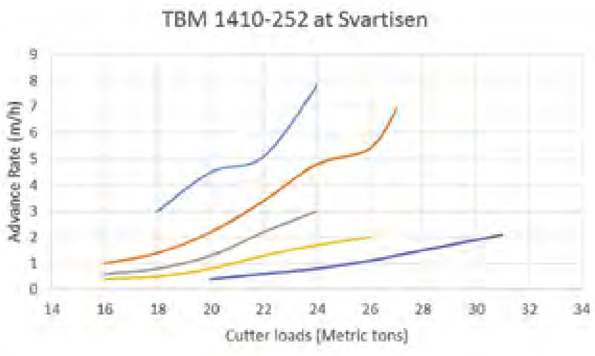


Fig. 4 shows actual performance of the 19 inch disc cutter on a TBM at the Svartisen project for different rock type and hardness.

The 20 inch cutter design is about to be the new industry standard for hard rock tunnel boring. A 20 inch cutter has the same cutter bearing load capacity as for a 19 inch cutter. However, the 20 inch cutter ring has 58 % more wear volume than for the 19 inch ring. Experiences from boring in hard, tough and abrasive rocks have shown a 20-25 % reduction in cutter changes. The 20 inch cutter ring tip has only an approx. 3% bigger “foot print” than for the 19 inch cutter which means a marginal less ROP. Using 20 inch cutters should therefore increase the machine utilization and the overall tunnel progress.

6 TBM Incline Shaft Boring
In the 1980’s, three pressure shafts; Sildvik HEP: 45 degree x 760 m x 2.53 m diameter, Tjodan HEP: 41 degree x 1,250 m x 3.2 m diameter, Nyset-Steggje HEP: 45 degree x 1,370 m x 3.2 m diameter, were bored by using Open Hard Rock Gripper type TBMs with Anti-Back-Slip System. The rock at Tjodan and Nyset-Steggje consisted of massive granite and granitic gneiss. Shaft boring of longer pressure shafts with TBM’s proved to be a very good alternative to conventional Drill & Blast Alimak-raising.

7 Record Performance
In August 1992 Merkraft, a joint venture of Eeg-Henriksen Anlegg AS and AS Veidekke, completed the boring of a 10 km transfer tunnel at Meråker Hydro Electric Power Project with a 3.5 m diameter Robbins High Performance TBM in less than 11 months. The tunnel boring was finished six months ahead of schedule. In the first full month of operation the TBM achieved the fastest start-up of any Robbins TBM on record by boring 1,028.9 m. Merkraft set outstanding national performance records along the way with the HP TBM working in geology ranging from hard, massive metagabbro, with UCS of 300 Mpa and greywacke and sandstone appearing as mixed face conditions to relatively soft phyllite.

• Best shift (10 hrs.)	69.1 m
• Best day (two 10 hr. shift)	100.3 m
• Best week (100 shift hours)	426.8 m
• Best month (430 shift hours)	1,358.0 m
• Average weekly advance rate	253.0 m

8 TBM Site Organization and Staff
Norway has long been recognized for its cost efficient tunneling. Some of the main reasons may be the low number of staff, crew flexibility and capability, and the use of modern and well maintained equipment. At Meråker, 16 men covering 3 shifts were employed, each working the regular 33.6 hours per workweek. This crew covered all operations including boring, rock support installation, mucking, work shop and cutter repairs. The TBM crew worked on a rotation system at the heading to improve teamwork. One operator controlled the TBM and the filling of trains from the cabin mounted on the back-up. One mechanic, one electrician and one locomotive driver handled all the other duties. The TBM site management included five persons, who also supervised the 5 km long 20 m2 Drill & Blast tunnel and the tunnel intake construction.

9 Norwegian TBM Contractors Abroad
In the 1970s and 1980s the Norwegian tunnel construction sector was one of the world’s leading players in the use of TBMs for hard rock tunnelling. TBMs were primarily used to excavate tunnels for hydroelectric plants but also in the excavation of sewage tunnels (including the 40 km long VEAS tunnel through Oslo, Bærum and Asker), in addition to a couple of road tunnels. When major hydroelectric projects were scaled down, the TBM excavation method lost ground in Norway. The last TBM project in Norway, the 10 km transfer tunnel at Meråker HEP, was completed in 1992.

Until year 2005 some of the Norwegian contractors with TBM experience were involved in several TBM projects abroad (Sweden, South Africa, Hong Kong, USA, India, Bolivia, Middle East, Italy), comprising of water tunnels for hydro power plants, pipeline tunnels, and tunnels for irrigation and water supply, totaling approx. 90 km of tunnels, all in hard rock formations.

The Norwegian tunnel construction industry has gradually lost some of its expertise in TBM operations. However, a few Norwegian consultancy companies have been involved in international TBM projects right up to the present day.

The Return of TBMs to Norway – A new TBM era

Reconstruction of the Røssåga Hydroelectric Power Plants

The two power stations Lower and Upper Røssåga in northern Norway some 200 km south of the Arctic Circle have been under a complete reconstruction and modernization.

Lower Røssåga with six existing units, each 43.5 MW started production in 1955, whereas Upper Røssåga with three units each 62.5 MW opened in 1961. Both hydropower plants are owned and operated by Statkraft Energi AS, a company within the Statkraft Group that is 100 % national Government-owned.

The reconstruction of Lower Røssåga allows for rehabilitation of 3 units, three units to be closed down, a new powerhouse equipped with one unit of 225 MW to be established. A major part of the civil work is tunnel excavation and includes a new 7.4 km headrace tunnel parallel to the existing headrace tunnel, and a new access tunnel directly from surface to the new underground power station. For the Upper Røssåga the reconstruction includes a new 4.6 km long tailrace tunnel parallel to the existing tailrace tunnel.

The upgrade project of the Lower Røssåga and Upper Røssåga combined, increased the capacity by 100MW and the annual production by 200GWh, from 1900GWh to 2100GWh, is expected as a result of the project.
The owner, Statkraft, first considered closing the existing power plants for a full rehabilitation and modernization in order to increase of the capacity. However, they came to the conclusion that they would benefit from building a new parallel power plant while the existing plants were in production - a three to four years stop in production would have been too expensive.

The lay-out of the new hydro power plant is very complex since the new tunnels and caverns are close to the existing tunnel system and the underground power station. Access and cable tunnels for the new power station have to cross existing tunnels. A shunting tunnel system between the existing headrace tunnel and the new headrace tunnel at Lower Røssåga has been established in order to increase water flow to the new power station for increased power production when needed.

The 170m long vertical pressure shaft was raise bored. The 6.6 m diameter raise is to date the largest diameter RBM bored shaft in Scandinavia.

The geological conditions are complex. The rock along the headrace tunnel alignment consists of mica schist, mica gneiss, limestone, marble, granite, granodiorite and quartzite. Marble sections with minor karstic features caused some water problems during tunnel excavation. The overburden is 200-300 metres. Rock stress problems were not encountered..

The civil works contract for both the Lower and Upper Røssåga projects were awarded to the Contractor LNS (Leonhard Nilsen & Sønner AS). The contract value was in the range of NOK 700 million, which includes TBM excavation of the headrace tunnel at Lower Røssåga and an option to bore the tailrace tunnel at Upper Røssåga after the completion of the headrace tunnel. Later contractor LNS decided to use the D&B method for the Upper Røssåga.

The project was originally tendered as drill and blast. However, once LNS submitted an alternative TBM solution, the project owner identified the benefits of the TBM excavation method, and asked for alternative TBM solution from all bidders.
The TBM alternative was found to be the best economical option. The option saved several kilometres of adit tunnels and they could reduce the tunnel cross section by almost 40 % due to increased water flow in the TBM tunnel, and by avoiding blasting there was reduced risk of damage to the existing headrace tunnel that was in full production. The evaluation of all the bids, lead to the contract being awarded to LNS.

The new headrace tunnel of Lower Røssåga was excavated at a slight uphill gradient of 0.02%, and its 500 metres long adit was as well TBM bored at a decline of 1:9 and a curve radius of 500 m, using a second hand refurbished 7.23 m diameter Robbins High Performance Main Beam Gripper TBM. The TBM has previously been used on the Karahnjukar Hydro Power project, Iceland.

The machine was equipped for probe- and grout-hole drilling as well as rock support work. The primary rock support consisted of two roof drills dedicated to rock bolting and mesh installations, the McNally roof support system using steel slats, and sprayed concrete systems. The initial plan for the headrace tunnel project was to assemble the TBM on the surface and walk it down to a starter chamber at chainage 450-500m at headrace tunnel level. After initial consideration it was decided to bore the 450m long access tunnel. The TBM was assembled at surface and walked to the start portal 60 m into the D&B section of the access tunnel.

The boring commenced in January 2014. After 150 metres of excavation with temporary muck handling behind the backup, the TBM stopped boring for installation of the tunnel conveyor. This is the first time a Continuous Conveyor System has been used in Norway for TBM operations. Also, for the first time in Norway the contractor LNS used special designed rubber wheeled vehicles (MSV-multipurpose service vehicles, with angled wheels perpendicular to the tunnel wall) for transport of personnel and material to the tunnel heading. Both LNS and Statkraft have high focus on health and safety. A refuge chamber for 8 persons with a stand up time of 36 hours was installed at the aft end of the BackUp.

The TBM, dubbed "Iron-Erna" after Norway's Prime Minister, Erna Solberg, was designed with 46 x 19-inch back-loading cutters, rated 312kN and has 3000 kW cutterhead power installed. The machine has Variable Frequency Drive with cutterhead rotation speed: 0 to 8.7 RPM.

For the TBM tunnelling the crews were working 24 h/day, six days a week. Two of the four crews were 12 days on site while the other two crews were 16 days off.

During the excavation of the access tunnel the TBM drive encountered extremely hard, tough and abrasive rock with quartz lenses and hard-bands of quartzite in the tunnel face.
The strength of the cores tested by SINTEF, show average UCS above 200 MPa. Some of the zones had a UCS up to about 300 MPa. In addition, the rock was massive with very limited fracturing.

The combination of unexpected hard rock, curvature, training of the personnel, and adjustment of TBM and conveyor, installation of conveyor cassette and D&B excavation of a 300m long tunnel from the TBM tunnel to the top of the pressure shaft caused the production to be slow. More stable production did not occur until the machine was at the start of the headrace tunnel.

After the initial very hard rock, boreability improved in the fall of 2014 and this, together with more skilled crews contributed to more efficient operation. The TBM achieved high monthly production rates for the remaining section of the tunnel with a best month of 835 m bored. The daily and weekly production records obtained were 56 m and 250 m respectively.

In February 2015 crews identified contamination in the main bearing cavity, which could have indicated main bearing damage. The TBM was stopped for further analysis, including probe camera inspections. The inspections gave no conclusive answers. The owner, contractor and the manufacturer needed to consider either doing a main bearing change or continuing to bore with the risk of a complete failure of the bearing and further damage to the TBM. After thorough considerations it was decided to change the main bearing. The main bearing was replaced and the machine started boring again end of March, less than six weeks after the decision to replace the main bearing was taken.

The TBM broke through into the intake shaft on December 10, 2015 – and the new power station went online on schedule in the summer of 2016.

The choice of Statkraft and LNS of using TBM for the construction of the headrace tunnel at Lower Røssåga project have brought TBM excavation expertise back to Norway after more than 20 years since the last TBM project. In some ways one may call it the start of a new TBM era in Norway.

Ulriken Railway Tunnel Project
On the Bergen line, the Norwegian National Rail Administration - NNRA (now: Bane NOR) has started construction of a new 7.8 km long single track railway tunnel for extending from single to double track between Bergen and the suburban town Arna. The alignment of the new Ulriken tunnel will be parallel and close to the existing tunnel.

In the early stage of the planning process, conventional drill and blast only was considered as excavation method for the new tunnel through the mountain Ulriken.
The existing single track tunnel has the most train movements per day in Norway. Should any extraordinary or unexpected stop occur in the tunnel due to construction work in the neighbour tunnel, this will have a great impact on the train traffic situation on the Bergen line.
After each blast round, inspection of the existing tunnel and possible rock support would be required throughout the construction period. Only short time-windows for inspections would be allowed for.

In 2013 NNRA decided to tender the project based on TBM excavation, as well. The idea was to let the construction market decide which of the excavation methods was the most beneficial for the project.

Prequalified contractors/ Joint ventures submitted their bids in February 2014. NNRA did receive quite a few bids for both conventional D&B and for the TBM excavation method. The evaluation team of the Ulriken railway tunnel project came to the conclusions that using TBM excavation method was the best technical and economical solution for the project. In May 2014, the negotiated unit price contract was awarded to Skanska-Strabag Joint Venture.

The contractor decided to use a second hand rebuilt Herrenknecht Open Gripper type TBM originally used at St. Gotthard railway tunnel project. The diameter was extended from 8.8 m to 9.33 m for the Ulriken TBM project. The new cutterhead was equipped with 62 ea. 19 inch cutting discs and prepared for use of 20 inch cutter rings. In the beginning of 2017 the contractor installed 20 inch cutter rings in 30 of the most demanding cutter housing positions. This is the first time 20 inch cutters have been used on any project in Norway.
In January 2016, the tunnel boring operation commenced at Arna job site from a starting chamber approx. 800 meters into the D&B section of the tunnel. As of end April 2017 the TBM had bored 5.1 km and breakthrough is expected in September 2017.

The Ulriken TBM tunnel. The rock mass consists mainly of Gneisses. Photo: Bane NOR

The Follo Line Railway Tunnel Project
Brief project presentation
The Follo Line project, a new 22 km double track railway under construction between the capital of Norway, Oslo and city of Ski, includes 20 km of twin tube single track tunnels of which approx. 18 km is to be excavated by four Double Shield TBMs in hard to extreme hard rock. Bane NOR (former Jernbaneverket) is the owner of the project and Acciona Ghella Joint Venture is the contractor. The Follo Line is the largest onshore infrastructure project to date in Norway and the tunnel would be the longest railway tunnel to date in the Nordic countries. The new twin-tube single track tunnel is being built for minimum 100 years life time and will have cross passages every 500 meters. The tunnel is designed for a train speed up to 250 km/h and is scheduled for completion at the end of 2021.

Geological conditions
The rocks along the tunnel alignment consist of Precambrian gneisses with bands and lenses of amphibolite and pegmatite, and in addition several intrusions. The rock mass is quite homogenous and competent with moderate jointing. The demanding massive, hard, tough and abrasive rocks have a UCS in the range of 100-300 MPa. The expected advance rate according to the NTNU Prediction model (Bruland, 1998) is 15.6m per day.

Figure 1: Face mapping on TBM S-980. Photo: Bane NOR

Role of the project owner
The Follo Line Project, developed by Bane NOR under commission from the Ministry of Transport and Communications, is a pilot project for a new contract model as well as new tunnel excavation methods for Norwegian railway tunnels. The use of EPC contracts, the use of drill and blast in combination with drill & split and the use of TBMs pave the way for innovation and knowledge upgrading. The project construction includes five separate EPC contracts and one contract for the signal system of which the EPC TBM contract is the largest.

Bane NOR is the owner of all of the TBM equipment at site until breakthrough. The contractor is then obliged to execute the buy-back. If the contractor goes bankruptcy during the tunnel construction period, then the equipment could be transferred to a new contractor JV.

The success of the Follo Line TBM tunnel project is highly dependent on the performance of the four TBMs. In order to cope with the hard rock challenges to be encountered at the project the most robust TBM design is a must. For hard rock tunnel boring a stiff cutterhead and a large diameter and high capacity main bearing capable to withstand extreme eccentric cutterhead loads, is crucial for the tunnel boring operation.

Invitation to tender (ITT) - TBM Requirements
- Hard rock double shield TBM
- Detailed specifications and design drawings prior to commencement of manufacture to be made available to Company
- All parts new and traceable
- Sealable ports in shield
- Core drilling system
- Retractable cutterhead
- Back-loading cutters
- Cutter monitoring system
- Main bearing L10 life: min. 15 000 hours (in compliance with ITAtech guidelines)
- Fully automatic guidance system
- Automated electronic data recording system – transmittal to surface in real time and available to Company at any time
- Gas detection system
- 12 bar water pressure resistance – static
- Water filling of excavation chamber
- Probing and pre-grouting drilling
- Equipment for MWD
- Mapping of probe holes by televiewer
- Number of grouting ports
- Number of drilling rigs
- None-flammable conveyor belts
- Automatic fire detection and suppression system
- Refuge chambers in compliance with ITA Guidelines -24 hours standup time

In addition there are another 40 requirements for the TBM and Backup system, Scope of work, TBM excavation and support, and OH&S.

Four of six pre-qualified international joint ventures submitted bids on the TBM section of the Follo Line tunnel. The international bidders had no or limited experience from tunnel boring in hard and tough rock similar to Norwegian bedrock consisting of gneiss and granites. The Follo Line organization have personnel with own experience from hard rock tunnel boring in Norway and abroad and who have been cooperating through many years with the Norwegian University of Science and Technology (NTNU), a world leading institution when it comes to hard rock tunnel boring. Therefore, the Follo Line organization was capable to, as well, evaluate the technical specifications proposed in the tenders and make their own judgements.

In the original tender the winning contractor Acciona-Ghella Joint Venture offered TBMs with technical specification well within the Invitation to Tender (ITT) requirements. However, in the revised tender the TBM supplier was changed and the TBM specifications were not up to the standard originally offered. Some technical specifications were changed prior to contract signing and further adjustments were performed during the final design of the TBMs.

Bane NOR is a state owned company responsible for the national railway infrastructure. Bane NOR has played a pro-active role in upgrading of the TBM technical specifications during the final design of the four 9.96 m diameter Double Shield TBMs for the Follo Line tunnel project (Figure 2). This includes improved Cutterhead (CHD) and Main Drive design which has resulted in increased stiffness of CHD and a larger diameter Main Bearing (MB) with extended L10 life.

Other important design criteria for TBM boring in hard to extreme rock conditions that have as well, been focused on.

- The Main bearing L10 life time was extended from 15,000 hours to min. 20,000 hours
- The weight of the cutterhead was increased from 230 to 265 metric ton
- The number of cutting discs were increased from 66 to 71 (70 tracks –the two outermost gage cutters are double tracking) thus resulting in reduced average cutter spacing to 71 mm.
- The cutterhead is equipped with 19 inch cutters, but the cutter housings are prepared for use of 20 inch cutters
- The inner and outer main bearing seal arrangement was modified to include pressurized seal rings for 12 bar water pressure resistance during emergency sealing of the TBM shield.
- The size of the Main bearing was increased from 6.0 m to 6.6 m diameter

Bane NOR requested a Third Party Verification of the Main bearing L10 life calculation for the proposed 6,000 mm diameter main bearing according to ITAtech Report No. 1 – April 2013, and for the design of the cutterhead (CH). Babendererde Engineers, Germany

	Details
TBM cutting diameter	9,960 mm with new cutters
Cutter size	19 inch wedge lock, back-loading
Number of disc cutters	4 center (x 2 discs) + 48 face + 15 gage = 71 cutting discs
Load per cutter ring	315 kN
Max. recommended CH load	
71 x 315 = 22,365 kN	
Weight of Cutterhead	265 metric ton equipped with cutters
Cutter monitoring system	5 cutter positions monitored (positions: 42, 44, 46, 48 and 50)
Cutterhead (CH) power	13 each VFD motors x 350 kW = 4 550 kW
Total power installed	approx. 6900 kW
CH rotational speed	0 – 6.06 rpm
Nominal torque	11,115 kNm @ 3.67 rpm
Max. overload torque	16,672 kNm @ 3.67 rpm
Main Bearing (MB)	3 axis roller bearing, 6,600 mm OD
Main bearing life time	> 20 000 hours according to ITAtech guidelines
Main Bearing Seals	Inner and outer sealing system, each: 3 lip seals + activatable seal ring for 12 bar static water pressure resistance
Total length, TBM + Backup	approx. 150 meters
Total shield length	14,415 mm
Total weight, TBM + BU	approx. 2,300 metric ton
Probe Drilling Equipment:	Two drill rigs with rod adding system for drilling up to 35 m long holes for probing and pre-grouting through 38 ports in gripper shield with 11 degree angle to tunnel axis and or through 8 openings in the cutterhead.

performed the 3rd Party Verification and proposed some design changes to the CH. In order to further minimize the risk of main bearing failure(s) during the TBM tunnel construction, the Follo Line management entered into an agreement with the contractor to enlarge the diameter of the MB to 6,600 mm OD, which gives a MB/TBM diameter ratio of 0.66.

Figure 2: Launching of TBM S-982. Photo: Acciona-Ghella Joint Venture

Logistics
One central TBM launching location

Figure 3: One central TBM launching location for the four TBMs

18.5 km of the 20 km long tunnel sections will be excavated by four TBMs, operating from one centrally located access point at Åsland, close to the main road E6 and with a limited number of neighbors in proximity to the rig area (Figure 3 and Figure 4). Two access tunnels, each approximately 1 km long, have been excavated from the main rig area and down to the location for the future railway tunnels so far. Additional auxiliary tunnels and two large assembly chambers have being constructed utilizing conventional drill and blast techniques as a preparation for the assembly and operation of the TBMs.

The first of the four Herrenknecht 9.96 m diameter Double Shield TBMs started the boring operation activities in September 2015 and by the end of the year all of the machines were in operation. The TBM excavation is expected to be completed by the end of 2018. Two TBMs are boring in the northward direction toward Oslo Central Station, and two TBMs are working in the southward direction toward the city of Ski, where they will be connected to a cut- and cover section.

The location at Åsland and the opportunities to develop a compact site arrangement, including all the necessary operations for the production of the 18.5 km long tunnels, provides great environmental benefits compared to excavation by drill and blast from several different access points. Continuous Conveyor belts, transporting the excavated material from the tunnels, will give a reduced number of vehicle and traffic movements.

Figure 4: Drone picture of the TBM site at Åsland. Photo: Bane NOR

Water tight segmental lining
Inside the tunnel, gasketed pre-cast concrete segments are installed in a closed ring to ensure rock support, as well as protection from water leaking into the tunnel. In addition invert segments are installed inside the segment rings.

The production of these elements is taking place at Åsland site and approximately 10% to 15% of the TBM spoil will be reused in the production of these concrete segments. Launching all four TBMs from Åsland, also enables reuse of spoil for potential future residential developments within the area. This will reduce the volume of traffic on public roads and pollution from vehicles.

Transport in the tunnels
Multipurpose Service Vehicles (MSV) is being used for transportation of concrete segments and other materials from surface storage area at site directly to the TBM headings. The shift-crews are as well, transported to/from the TBMs by use of MSVs.

The grout, both component A and component B, for filling the annulus gap between segment rings and tunnel surface is transported through pipelines from the grout plant at surface all the way to the TBM backup system.

Experiences
Rate of penetration and Advance rates

	ROP [mm/min]
Average	28.4*
Maximum	52.93
Minimum	14.94

* Average achieved for gross total thrust force 21,891 kN

Table 2. Rate of penetration (ROP), TBM S-981.

	Average [m]	Highest [m]
Day	16.9	28.0
Week	91.6	135.3
Month	366.2	524.8

Table 3. Advance rates, TBM S-981.

Cutter experiences
As a rule of thumb, the optimal tip width of the cutter rings for the given rock conditions, should be as narrow as possible in order to obtain good Rate of Penetration. However, the tip width of the ring should be sufficient to sustain the cutter loads needed to cut the rock efficiently without cutter ring chipping or "mushrooming".

At the Follo Line tunnel project the contractor is testing cutter rings with different tip width, but mushrooming is a problem. It is assumed that the Rockwell hardness of the ring is on the low side for hard rock tunnel boring. The tip width of the ring becomes so wide that penetration per revolution of CH is reduced and therefore the full potential of the TBM is not utilized.

In March 2017 the contractor started testing cutter rings from a different supplier in ten cutter housing positions on one of the TBMs. The following month a "full dressing condition"-test with complete cutter assemblies from the same cutter supplier was implemented. In order to find the best cutters for the project, the contractor has, as well, ordered cutters from a third cutter supplier for a full scale test.

Concluding remarks
The success of the Follo Line TBM tunnel project is highly dependent on the performance of the four TBMs. In order to cope with the extreme hard rock challenges to be encountered at the project the most robust TBM design is a must.

Bane NOR has played a pro-active role in specifying the technical TBM requirements, including eg. upgrading of cutterhead and main drive design, resulting in increased stiffness of the CHD, reduced cutter spacing and a larger diameter MB with extended L10 life.
Experiences gained so far show that these upgrades have proven their justification.

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4.2 NEW TYIN POWER PLANT. WORLD RECORD WITH 1030 M WATER HEAD ON UNLINED HEADRACE TUNNEL

New Tyin Power Plant Project was the largest hydropower upgrading project in Norway until 2004. It was a completely new power plant, replacing the old power plant utilizing parts of the old waterways. The project includes 21 km of new tunnels, excavation of the power station and other caverns. Water pressure on unlined tunnel is 1030 m, which is a world record. It is a typical Norwegian method of design to optimize location of power stations to minimize length of steel lining, using mechanical properties of the rock. The power station was moved further out (reduced length of access tunnel) due to higher tensions in the rock than presumed. This meant a considerable reduction in cost compared to original budget.

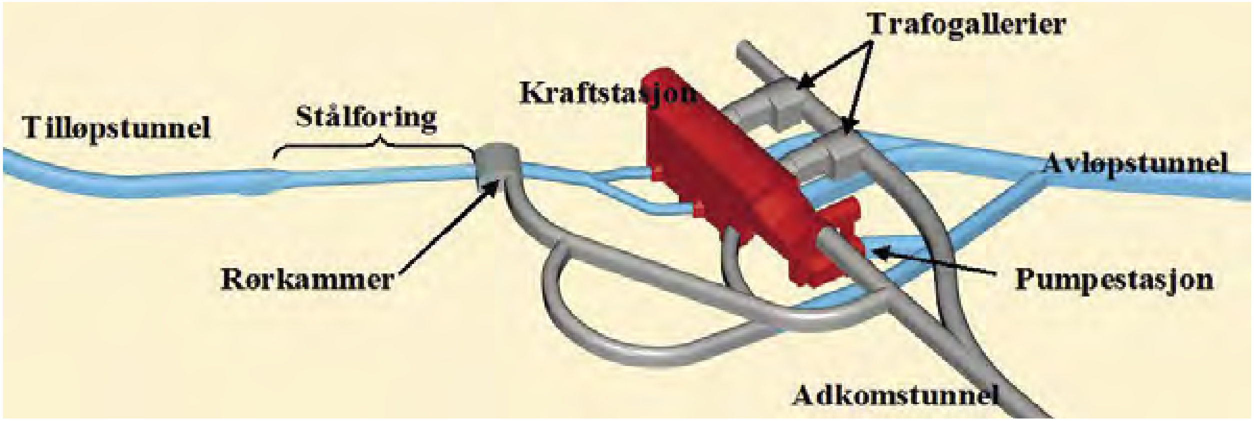


Figure 2. Power Station Complex; Tilløpstunnel = head race tunnel, Stålforing = steel lined section, Kraftstasjon = power station, Avløpstunnel = tail race tunnel, Adkomsttunnel = access tunnel, Trafogallerier = trafo caverns

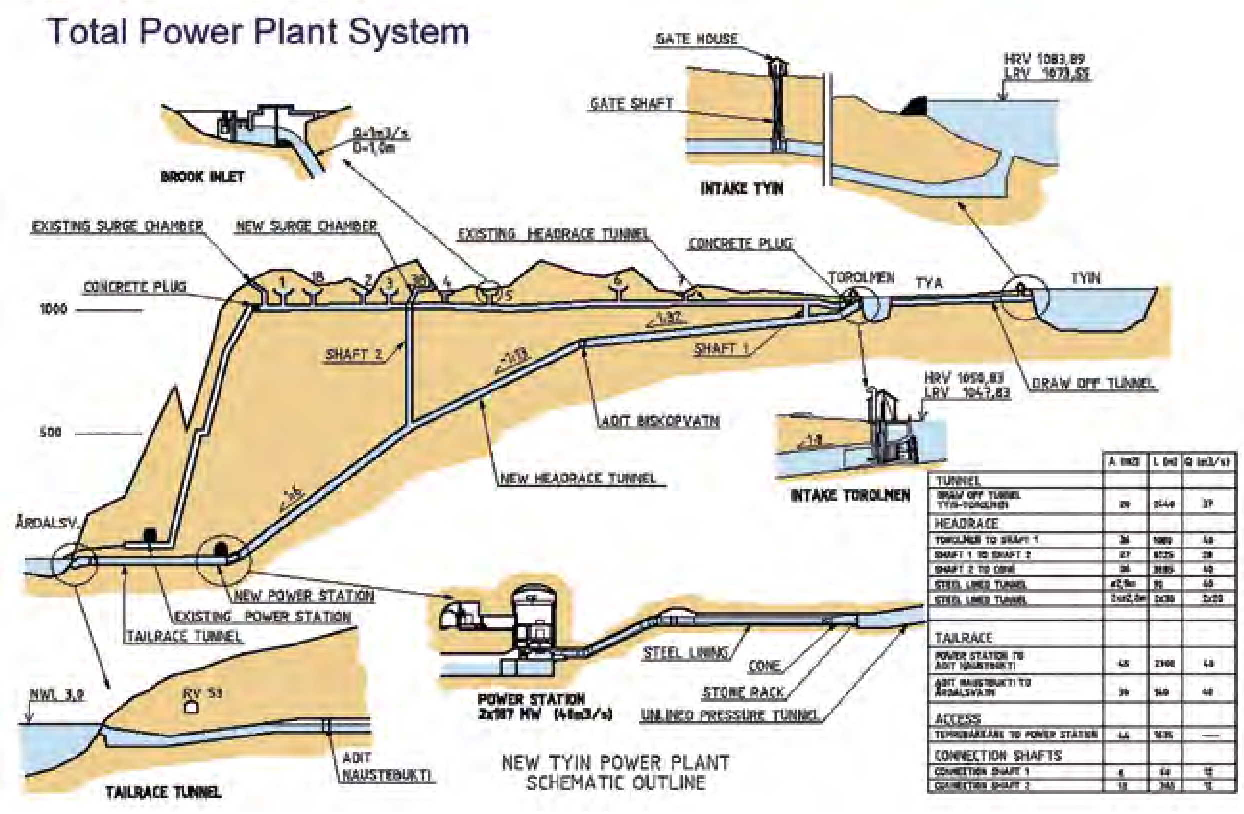


Figure 1. FSchematic outline of New Tyin, showing usage of old headrace, integrated into the new Power Plant, connection of old and new tunnel systems, with 400-meter vertical shaft working as connection between old and new headrace, combined with transfer tunnels, brook inlets and as surge chamber

LOCATION OF NEW POWER STATION AND GEOLOGY

Topography and Geology

Topography is rough with the main reservoir located at a plateau at approximately 1100 m.asl. The headrace is located along the Tyin Valley to an adit at Bishop lake. From Bishop lake, pressurised headrace tunnel continues through a distinct massif between the Tya valley and Rausdalen valley, down to the power station by Øvre Årdal at a few meters asl. Both valleys are deep canyons in the terrain down to main valley of Øvre Årdal, which is in the end of the longest Fjord at the Western Coast of Norway (Sognefjord).

The geology of the area is relatively complex, with pre-cambric and Cambro-Silur rock from Jotunheim/ Valdres plateau complex, which is folded over phyllite rock of Fortun/Vang complex. The foliation zone has a slight inclination towards Årdal Lake, which denotes that the main part of the underground structures is in the Joutunheim/Valdres complex.

The eastern part of the powerplant complex, the transfer tunnel from Tyin Reservoir to Torolmen intake reservoir is in the Fortun/Vang complex, mainly consisting of phyllite. The foliation strikes NNE with dip 25-30 towards east.

Towards adit Bishop lake, approximately 2 km from intake the geology consists of a transformed sandstone/quarzitic rock with distinct horizontal crack pattern.

Next 2,5 km is comprised by precambric, volcanic rocks mainly appearing as green coloured gneiss.

Lower part of the pressurised headrace tunnel, power station area and access/tailrace tunnel is characterised by a darkish gabbro, with frequent ores of light granite "Trondjemitt". The rock mass has little to moderate degree of fissures and cracking, with some undefined and variable foliation. A steep set of cracks with strike NS/NE, with some local variations gives the main direction of cracking pattern, besides the foliation cracking.

Three distinct fault zones cross the power plant complex. The fault zone that forms the Rausdalen valley, crosses the headrace tunnel 2.1 km upstream of the outlet. It consisted of a distinct set of cracks, but did not give any problems for tunnel excavations. The Laerdal fault zone, is a regional fault zone which can be followed form Joutunheimen plateau and almost to Voss (150-200 km). This fault zone crosses both the headrace tunnel and the surge shaft. Central part of the fault zone is approximately 0.5 to 1 m wide and contains swelling clay. The rock mass along the fault zone is crushed and converted and has a significant clay content.

Bergartssammensetningen i området er relativt kompleks, med prekambriske og kambrosiluriske bergarter fra Jotun-Valdresdekkekomplekset som er skjøvet over fyllittbergarter fra Fortun-Vangdekket. Skyvesonen har slakt fall utover mot Årdalsvatn, slik at hoveddelen av anlegget blir liggende i bergarter fra det øverste dekkekomplekset.

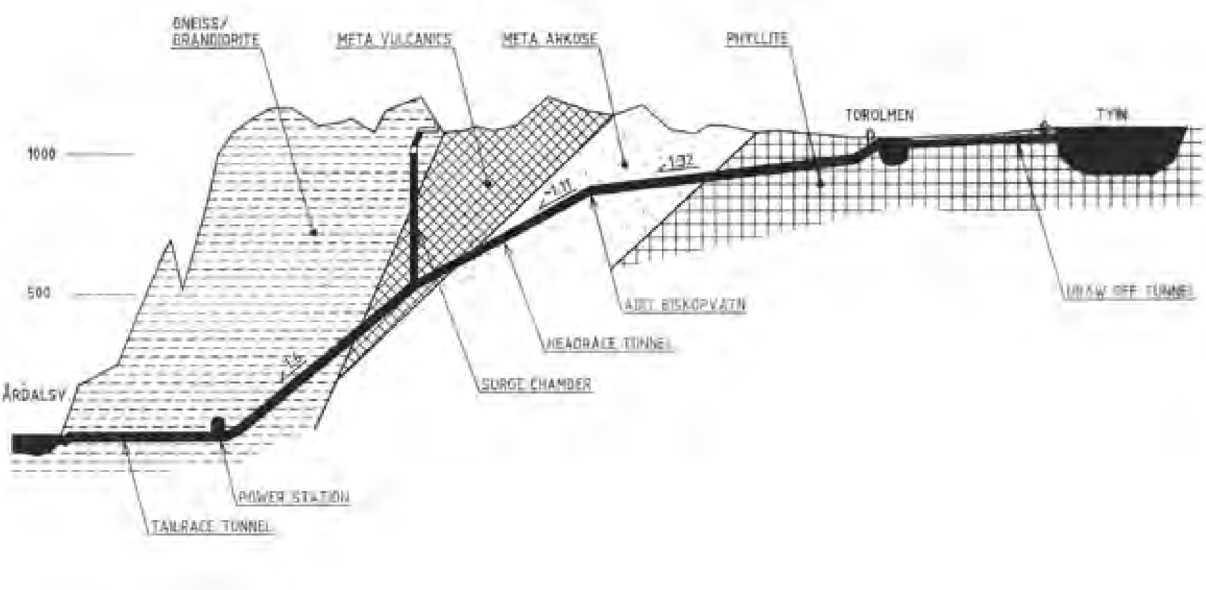


Figure 3. Geology composition along tunnel system

Preliminary Investigations
Field mapping was performed in 1997 to determine rock mass composition cracking and performance of weaker0 zones

3D element analysis of the mountain massif in between the Rausdalen and Tya Valley was performed to analyse the stress distribution in the rock and evaluate feasible design and preliminary location of power station and cone (transition from exposed rock surface to lined area).

Two core drillings were performed to analyse conditions of the rock, water leakage and stresses. The first hole was drilled to 535 m depth, approximately 500 m further out than the preliminary planned pressurised tunnel. This hole showed reduced stress after 300-meter depth. Hole number two was drilled to the area of the preliminary planned power station. Hydraulic split- and fracture testing from this hole suggested that the planned location had a safety factor between 1.0-2.4. It was still a degree of uncertainty attached to the measurements and testing. Hence, the feasibility study advised to include an uncertainty on location of the power station, e.g. include contingency for necessity of relocate power station further into the massif or extend steel lining to be within the required safety factors.

The typical Norwegian approach of include this risk into the basic and tender design as contingent was pursued, which includes final determination of exact/best location based on stress and split testing simultaneously with excavating the access tunnel.

Rock Stress Measurements in Access Tunnel
3D stress measurements and hydraulic splitting was done at several stations during excavation of access tunnel.

Table 1. In-situ stress measurements

Chainage	Measurement performed
1000	3D Stress measurement
1086	Hydraulic fracturing
1292	Hydraulic fracturing
1400	3D Stress measurement
1440	Hydraulic fracturing
1537	Hydraulic fracturing
1625	3D Stress measurement
1625	Hydraulic fracturing

The results are shown in the figure below

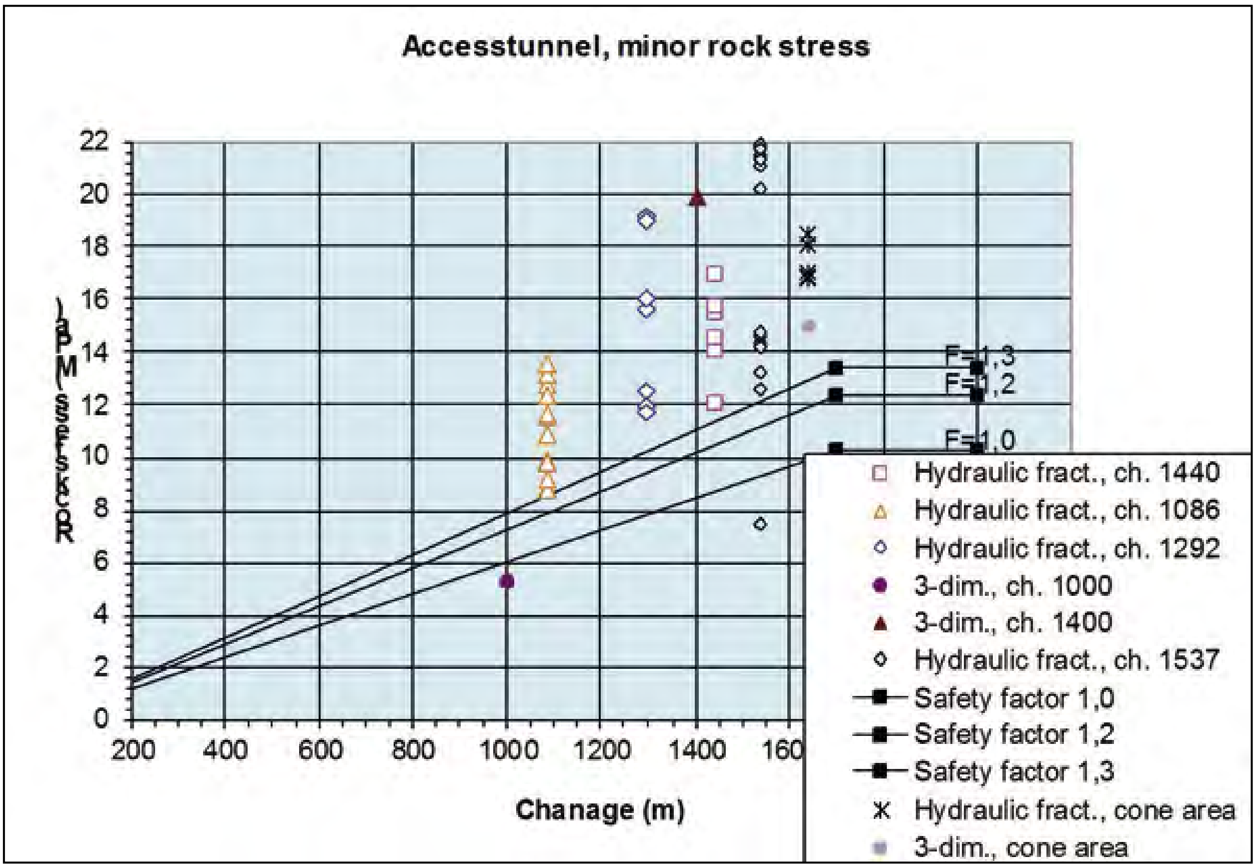


Figure 4. In-situ stress measurements along chainage numbers in the head race tunnel

First measurement at chainage 1000 showed lower values than expected, based on preliminary core drilling in preliminary phase. However, it was quite in line with the reduced stresses measured in the first borehole. This hole was located approximately in the same area as the access tunnel was passing. Anyways, this triggered a discussion in the panel of experts (with the one task of determining and decide location and direction of power house complex) on the location of power station in original design- possible need for moving power station complex deeper into the massif. Hence, inclination of access tunnel was slightly changed to make up to 400 m deeper relocation possible if later measurements should follow the same trend.

The successive measurements gave more optimistic results. Testing by hydraulic jacking at chainage 1086, 1292 and 1440, and a new stress measurement at chainage 1400, all showed strengths higher than originally expected.

The discussion turned around to moving the power station backwards (shorter access tunnel with all equipment much cheaper than headrace tunnel). The figure above shows the limits for safety factors 1.0, 1,2 and 1.3 for location of the cone. To cone-locations are illustrated: 1) Conservative location with cone at chainage 2000, and 2) cone location with 120 m shorter access tunnel (moving back power station complex). As can be observed it is two measurements below the safety limits which triggered intensive discussions, but in the end, it was concluded that one of the measurements was close to a local weakness zone with probable impact on the result, and that there was some uncertainty on the quality of the second measurement.

Finally, it was decided to move the station 120 m back than originally planned and perform a final 3D measurement in addition to hydraulic split in the cone area before final decision on the location. Possibility of extended lining was kept open in case of unsatisfactory results from measurements. However, very high strengths were measured at the cone area and confirmed the locations with a safety factor of at least 1.3.

Grouting of steel lined section
The rock mass in the cone area was characterised by limited cracks and fissures. The following grout procedure was performed:

1. Deep grout curtain:
   Four curtains as illustrated on figure. Two of the deep curtains was drilled wit 34 m holes, two with 17 m holes. Curtains were directed 45` with the tunnel. Each curtain consisted of 8 holes. After water loss testing showed that the rock mass was very impermeable, it was decided to go directly with epoxy grouting and avoid first round cement/micro-cement grouting as planned.

2 Contact grouting between rock and concrete:
   Contact grouting was executed for the entire length. The grouting is done through grouting hoses. A barrier was initially established at the upstream and downstream side of the lining, mainly with polyurethane. Thereafter, 11 loops of epoxy were grouted to ensure full contact between rock and concrete.

3. Contact grouting between steel lining and concrete
   This is performed with grouting through grouting hoses. A barrier of polyurethane initially established upstream and downstream in outer hose-loops, and thereafter epoxy in the inner three hose-loops

4. Control Grouting
   4 Curtains of 8 holes drilled from inside the penstock/steel lining through the concrete and 10 m into the rock. All holes grouted directly with epoxy.

The deep grouting only required 1979 kg of epoxy, as the contact and control grouting required 9000 kg.

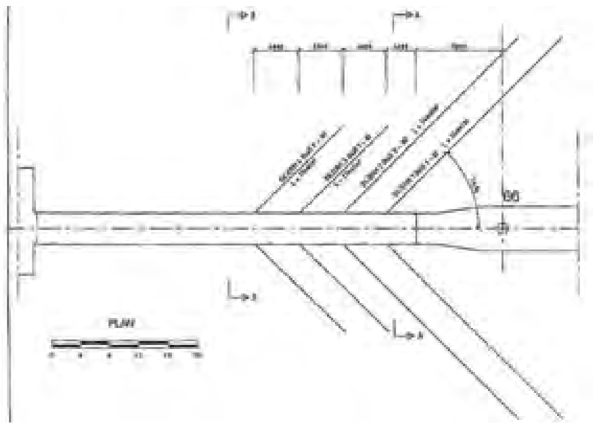


Figure 5. Illustration deep grout curtains

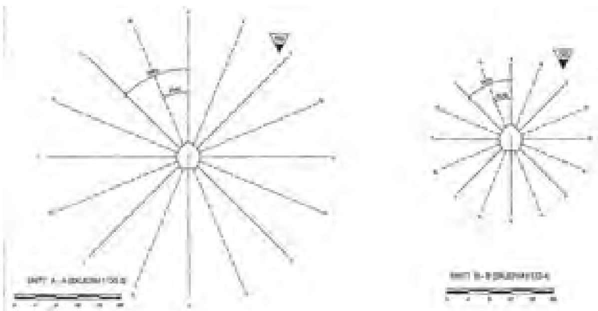


Figure 6. Illustration contact grouting between rock/concrete, concrete/steel

The grouting was very successful, after first filling the leakage was measured to 0.3 l/min in the chamber before bifurcation. The total cost for the grouting was in 2002 value around 0.5 MUSD.

Rock burst- stability support
The contractor experienced severe rock burst in the deeper parts of the tunnels/caverns. Tensions up to 50 MPA were measured in the power station area.

The rock bursting started early when excavating access- and tailrace tunnel, and it was swiftly arranged an effective system for temporary support. Swift application of fibre reinforced sprayed concrete with alkali free accelerator and systematic 3 m c/c 1.25-2 m polyester end anchored rock bolts in walls and crown. In both directions depending on the rock burst intensity. In this way, development of rock burst was "arrested" sufficiently early to secure continuous safe tunnelling.

Rock bursting was significant, especially in areas with the gabbroid gneisses and intrusive of light granite («Trondhjemitt»)

The rock mass conditions in the power station cavern was good, even with the high tensions. Temporary support consisted of 5-10 cm fibre reinforced sprayed concrete, systematic bolting in crown and walls c/c 2.0 m, mainly with 6 meter bolts due to tensions and height of walls. The additional permanent support, which supplemented the temporary support, included sprayed concrete up to 10 cm thickness (including previous layer thickness), and systematic 1,5x1,5 bolting in crown and walls, bolt lengths 4 and 5 m.

The support is based on numerical (FEM) analysis and observational method in the power station complex.

Swelling clay stabilisation
As expected form geological mapping, some fault zones in the tunnel showed content of clay, later confirmed to have swelling properties, especially Laerdal fault zone 2.5 km upstream power station and main fault zone Bishop lake, 7.3 km upstream power station.

Sonen i Lærdalsforkastningen var av noe begrenset bredde, men lå med slakt fall, slik at den påvirket tunnelen over en lengde på ca. 70 m. Denne sonen ble sikret ved å bolte fast 10 cm PE-skum mot leirsonen og 5 cm PE-skum mot leirinfisert sideberg langs sonen. Det ble lagt armeringsnett på utsiden av platene og deretter sprøyte alt inn med 40-50 cm fiberarmert sprøytebetong. Hele området ble boltet med 2,4 og 3 m lange bolter i mønster 1,5 - 2 m. I sålen ble det utført isolert sålestøp forankret med bolter.

Sonen ved Biskopvatn stod tvers på tunnelen og var en meget markert svelleleiresone med en bredde langs tunnelen på ca. 12 m. Her ble det valgt full utstøpning som permanent sikring. Det ble montert 10 cm PE-skumplater mot leiren i sonen. I sålen ble det utført isolert sålestøp.

Mindre soner med svelleleire ble sikret med isolasjon av selve leiren rundt hele profilet, støp i sålen og heng og vegger dekket med 20 - 50 cm tykk fiberarmert sprøytebetong i tillegg til systematisk bolting, c/c 2 m.

Summary location and performance of power station
The unique Norwegian approach and «design as you go» for high head underground hydropower schemes is based on optimising cost/time by utilising the properties of the rock mass itself as building materials, as well as conscious distribution of risks to manage the variations which may well arise with geological behaviour impossible to fully map in initial stages. Naturally, it was quite exiting with new Tyin, designed for world's highest unlined water pressure if risks were accurately addressed. Minor to non-existing leakage, a cheap grouting programme and savings on final design by moving power station 120 meter outwards made the project a success and is a proof of a design and execution approach which has removed e.g. the excessive costs and costs of time with excessive concrete lining. New Tyin may have been especially well suited as weak zones, fissures and cracks in the area for the power station were very limited, but this saved money from the original tight budget which may not have been realised if not allowing a certain "design as you go" component in the approach.

Design principles of unlined high pressure head race tunnels
The topographic conditions in Norway are favorable for the development of hydropower and more than 99% of annual production of electrical power in Norway is generated from hydro. Underground powerhouses and unlined tunnels have been constructed starting from the end of World War I in 1919. High-pressure unlined tunnels and shafts have been the preferred best solution for cost and time effective construction of hydropower. Thus the expensive lining with steel penstock and concrete embedment is avoided. From the valuable experience gained through the design, construction and operation of tunnels in the past 100 years in Norway, unlined pressure tunnels have been developed reaching 1046m hydrostatic head for Nye-Tyin Hydropower Project being a world record for its high hydrostatic head as exceeds 1,000 m in unsupported unlined pressure tunnel. Figure 1 shows the development of high head unlined tunnels and shafts in Norway.

The concept of constructing unlined pressure tunnel and shaft is based on the principle that the rock itself shall resist the water pressure during water transfer. It is essential that the minimum principal rock stress in the rock mass should be higher than the water pressure with regard to safety against hydraulic fracturing and uplift. A safe construction is highly dependent upon the design based on adequate vertical rock cover and side-cover near side valleys with regard to the rock mass quality.
At the very beginning when unlined high-pressure shafts and tunnels were introduced, an evaluation based on a simple equilibrium state of stress was used. The principle for this was that the weight of the rock cover above the tunnel should exceed the water pressure induced inside throughout the tunnel or shaft in-order to avoid hydraulic jacking and uplift.

The Norwegian rule of thumb design approach method has been widely applied for common case in which a tunnel is near to the side of valley. It considers both the overburden and side-cover to avoid the risk of hydraulic fracturing.

Snowy mountain design creation was also introduced in Australia as a consequence of that the side cover was found less effective compared to the vertical rock cover to secure that the tunnel or shaft has the required confinement.

Parallel to the revision of the rule of thumb, a new better simulation model was presented with the established standard design chart tools based on a finite element model (FEM) and the concept was that no-where along the unlined pressure tunnel and shaft should the internal water pressure exceed the minor principal stress in the surrounding rock mass.

Analysis using numerical models is one of the modern design approaches, which in use widely at present to carry out qualitative, stability and support analysis in addition to its vital use in the verification of the above deterministic design approaches. This chapter evaluates the possibility to develop ultrahigh head in unlined tunnels in general and particularly for the exemplified project for higher hydrostatic head based on the outcome of the empirical, deterministic and numerical analysis and recommendations have been proposed for further conclusions on this thesis.

The Nye-Tyin Hydropower project is located in Årdal, Norway. It has a hydrostatic head of 1047 m and operates at an installed capacity of 380 MW and discharges 2 times 20 m3/s with an average annual production of 1398 GWh.

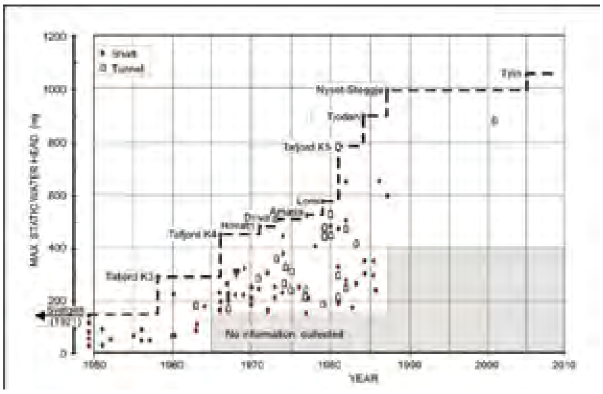


Figure 7. Development of Norwegian unlined high-pressure tunnels and shafts

The first and foremost requirement for a pressure tunnel or shaft is that leakages have to be avoided by providing adequate overburden which has a weight greater than the water pressure. This method was used in Norway before the rule of thumb method was introduced and in this paper, the method gives surprisingly a conservative result of the possibility to develop ultrahigh head in unlined tunnels/shafts up to static head of 1137 m with a factor of safety 1.3 for the exemplified project Nye Tyin HEP. In fact, this method does not consider side valley cover requirement for confinement, the topographic effect on stress, and the existence of considerable tectonic stresses.

The Norwegian rule of thumb has been widely applied in cases for tunnels and shafts sited in valley sides. It considers both vertical cover and the steepness of the slope of the adjacent valley side. The method assumes that, the minimum in-situ principal stress should exceed the water pressure at any point to avoid the risk of hydraulic fracturing. The analysis of this method indicated a static head of more than 1427 m with a factor of safety 1.3 being feasible to develop for the Nye Tyin HEP. The method accounts for gravitational rock stresses only but in many cases considerable tectonic and residual stresses also exist. Hence the effect of required increased confinement should be disregarded where the valley is undergoing tectonic extensions. This method of criteria for confinement is still used in prefeasibility studies.

The Snowy Mountains for confinement requirement was recognized during the development of the Snowy Mountains project in Australia. The side cover is found less effective compared to the vertical rock cover to secure that the tunnel/shaft has the required confinement. This method has fairly similar results as the Norwegian rule of thumb method with some more conservative result on the side cover. Maximum water pressure of 16.77 MPa can be safe against hydraulic fracturing based on this method of analysis for the Nye Tyin, which is hydrostatic head of 1677 m with 1.3-safety factor. Both methods, Snowy Mountain and Norwegian rue of thumb reinforce that the pressurized water tunnel and shaft for Nye Tyin HEP is placed deep enough laterally and vertically. Hence, neither methods are actually considering the effect of tectonic horizontal stresses and topographic conditions on the stress regime. The comparison of depth of minimum cover specified by vertical, Snowy Mountain and Norwegian rule of thumb are in good agreement with the results, while the vertical criterion is not to the safe side relative to the numerical analysis. For preliminarily layout in terms of minimum requirements, it appears that the Norwegian and Snowy Mountain criteria are very useful tools.

Two-dimensional standard design charts based on the use of a numerical finite element model (FEM) has given a solution to compensate the completely neglected effect of tectonic horizontal stresses and topographic conditions on the above deterministic approaches. The basic principle of the finite element method for this purpose is to find the location, which for all parts of the shaft ensure that no-where along the unlined pressure tunnel or shaft should the internal water pressure exceed the in-situ minor principal stress in the surrounding rock mass. This method is a useful tool at the feasibility stage of the project and it makes it possible to find a preliminary location of the pressure tunnels and shafts; a location that in many case turns out to be the final one. The analysis based on this method indicated that the existing unlined pressure tunnel and shaft at Nye Tyin could handle a maximum static head of 1140 m with a factor of safety 1.3, which is a conservative result with respect to the above deterministic approaches. However, The result tends to be mesh-dependent and error can arise when selecting the boundary conditions of the domain of interest. Besides, using this analysis method will lead to errors, as the in-situ topography is very simplified and idealize with regard to the topography condition used in the model.

Finally, comprehensive 2-dimensional finite element calculation using Phase2 has been performed and the aim was to analyze the minimum principal rock mass stress situation in the vicinity of the pressure tunnel and to compare it with the induced water pressure inside the tunnel and shaft. The maximum static head that can be utilized for the existing unlined high-pressure tunnel and shaft at Nye Tyin hydropower project is 1097 m with 1.3 factors of safety. The analysis shows fairly good degree of correlation between the simulation results and the in-situ situation of the existing unlined tunnel. The author believes that the use of numerical analysis with profound knowledge of the geological conditions of the area will give best result in the design of ultrahigh head of unlined tunnels and shafts while having shortcomings of that, it requires rock mass parameters such as in-situ stress ratios that will be a problem to obtain, especially at the design stage when there is no in-situ measured rock mass stresses. The advantage of this analysis is that the influence to stress distribution of the valley side inclination, of major continuities, and rock with different properties can be as certain. Table 3 shows the summarized results of the different design approaches for their maximum static head that can be developed with safety of factor above 1.3 against hydraulic fracturing and uplift.

Design approaches	Max. Possible static head, (m)
First Design criteria	1137
Norwegian rule of thumb	1427
Snowy Mountain Creation	1677
Design Charts based on FEM	1140
Others	1197
Numerical Analysis	1097

Table 2. Comparisons and results of analysis
**A 1.3 factor of safety used and the actual project static Head is 1047 m*

The geological restrictions are the main challenges in the development ultrahigh head in unlined tunnels and shafts. It is essential to understand the geologic conditions along the tunnel alignment, relative to the hydraulic forces that will be applied during operation. Generally the rock should be sound and massive with a high intact tensile strength and low permeability in order to use unlined high-pressure tunnels and shafts.

Development of the design criteria for unlined pressure tunnels and shafts has proved the need for more rational and comprehensive design approach. Empirical and deterministic analyses have been established, and design charts prepared for use of preliminary feasibility studies. A numerical design approach has been developed for qualitative thorough design analysis.

Based on the outcome of these design approaches, the development of ultrahigh head of unlined tunnel and shaft is possible even for higher static head for Nye Tyin HEP as well as in general. All unlined sections should satisfy confinement criteria in order to avoid hydraulic jacking, unless the consequences of the hydraulic jacking have been reckoned acceptable. The unlined sections should be located only in rock masses, which are sufficiently durable and sound so as to satisfy long term requirements.

It needs to be emphasized that this chapter covers only the confinement requirements in general to develop safe ultrahigh head unlined tunnels and shafts against hydraulic fracturing of the main failure mode uplift of the ground surface for the exemplifying Nye Tyin hydroelectric Project. Even though various design approaches have been successfully done for an overall assessment of the possibility to develop ultrahigh head unlined tunnels, detail analysis including local fractures effect has not been assessed with respect to other fundamental mode of failures. These could be such as hydraulic jacking of joints or discontinuities, hydraulic shearing of joints and local crushing or blasting effects on the wall of the tunnel or shaft, as the water head becomes ultra high.

4.3 SUBMERGED TUNNEL PIERCING.

4.3.1 A NORWEGIAN SPECIALITY DURING THE LAST 100 YEARS

Underwater piercing of tunnels is generally connected to development of hydropower with the intention of utilizing for power production, lake reservoir lower than the natural outlet. Such tunnel piercings have been carried out for 100 years, but not exclusively connected to hydro- power development. Thus the first submerged piercing in Norway was the lowering of the lake Demmevatn located on the west side of Hardangerjøkelen in south west Norway and had nothing to do with hydropower development. In this case the water level was controlled and dammed by a glacier that served as a particular unreliable weir that might brake through any time and cause uncontrolled destruction to the below located Simadalen. A tunnel was excavated below the bottom of the lake and the blast was taken at 20 meters depth.

During the first half of the 20th century a great number of under water tunnel piercings were carried out at moderate depth in connection with hydropower development, and already before the last world war some hundreds were completed. There is little information about the methods that were used and how successful these blasts were. We assume that the contractors had there own way and a limited experience, some times a combination of good luck and good management. If something went wrong, they completed the job in the best possible way and nobody had any interest in publishing something that might be a blunder. The worst case for a contractor was an unsuccessful salvo without a brake through. To order people to enter the face was a big risk as no one could guarantee that the water would not brake through.

There are good reasons to presume that the previous investigations regarding rock mechanical and geological were less comprehensive than todays requirement when lowering of a lake reservoir is a topical subject. Some times serious landslides took place especially in places where marine clay appeared. In a particular case with marine clay it was suggested that the compensation cost for such slide damage amounted to the same cost as the constructional cost.

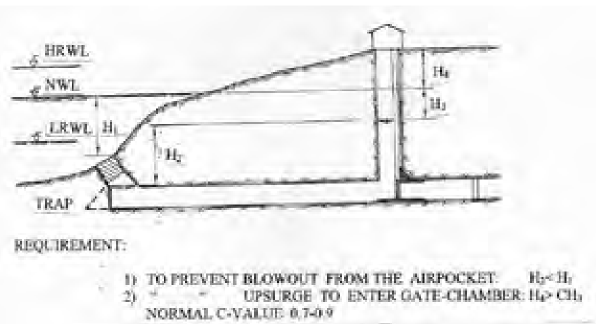
In this period the development of hydropower comprised mostly of medium to small power plants and although such damage cost could cause considerable financial deficit to the single power plant, such small accidents would have no effect to the national economy. It was generally understood that such under water tunnel piercing provided a cheap reservoir in most cases. The method also became known abroad as “The Norwegian Method”.

The rebuilding of the country after the last world war involved a large concentration on hydro- power development, which also called for an optimum exploitation of draw down reservoirs. The submerged tunnel piercing became more difficult as the bounds of experience were exceeded. Some unsuccessful cases made it clear that the physical processes involved in the piercing were not fully understood and this called for more research. SINTEF VHL, later named SINTEF-NHL had the capacity and competence to throw some light on the problems at the hydro-technical laboratory in Trondheim which had a reputation for hydro-problems connected to the development of hydropower. By means of physical scaled models it was now possible to study the flow condition inside the tunnel during the blasting process and if necessary introduce improvements to the design. The first physical model test was done in the beginning of the 1960`s and marked the start of a research programme that carried on as long as the hydropower part of the laboratory was in full activity.

As a result of this comprehensive research, it is justified to say that today we have knowledge to work out the design for any submerged tunnel piercing and execute the blast in a successful way, also with water depth which earlier was classified as a hazard. One should notice that the modern oil industry has profited from the experience and knowledge gained through the comprehensive hydropower development concerning the shore approach problem for the pipeline in the North Sea. The deepest submerged tunnel piercing ever done was in this connection at 200 m. The technique was also used for a number of cool water tunnels at the land-based oil terminals.

Since few of the tunnel blasts are identical with regard to water depth, geological conditions, rock quality, location of closing gate etc. a lot of different design for the submerged tunnel piercing were gradually executed. The most decisive factor turned out to be a system open to the atmosphere and a system with an air pocket isolated from the atmosphere. These two main systems are used considering that both systems may be used simultaneously in cases that comprise more than one single blast :
1) THE OPEN METHOD
2) THE CLOSED METHOD

1) THE OPEN METHOD
The characteristic with this method is that the plug and the tunnel has an open connection to the atmosphere through the gate shaft or a cross cut.



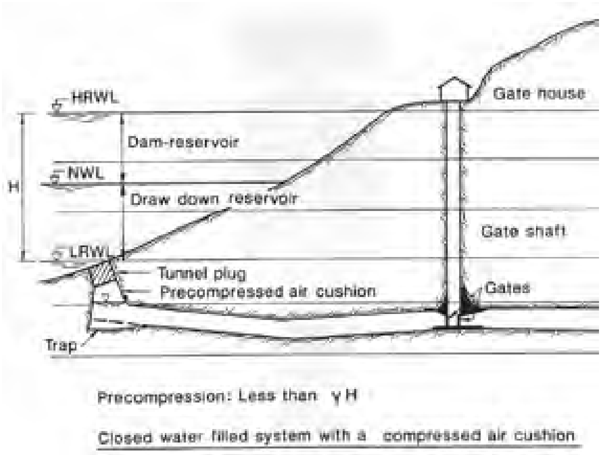
THE OPEN METHOD Fig. 1

The open method without water filling will set up high inflow velocities after the blast and complicate the design and construction of an effective debris trap. The upsurge in the gate shaft will usually be unacceptable unless measures are introduced. Water filling is therefore used in most cases. This must be done following particular criteria set up to avoid failure. One must make sure that the water level under no circumstances is covering the charge since explosive set off in water will cause destructive shock waves towards the gate. The filling level in the shaft must be sufficient to prevent the upsurge to enter the gate house, and at the same time not be too high so that the pressure in the air pocket, covering the plug, is lower than water pressure outside the plug.

These requirements are shown in Fig. 1 and since they may be in contrast to each other, they call for instrumentation to check the water level in the pocket at the plug and in the shaft. If the salvo by a mistake or ignorance is initiated in water which is coherently covering the gate, unacceptable damage may occur.

If the water is filled up in accordance with given guidelines the inflow velocities will be reduced and make the collection of the debris in the trap easier and comprehensive pre-calculation may not be necessary. On the other hand, if the situation for different reasons do not allow for a recommended water filling to reduce the water velocity, comprehensive pre-calculation or model test are required. The open method is adequate for a physical model test and this has been used frequently.

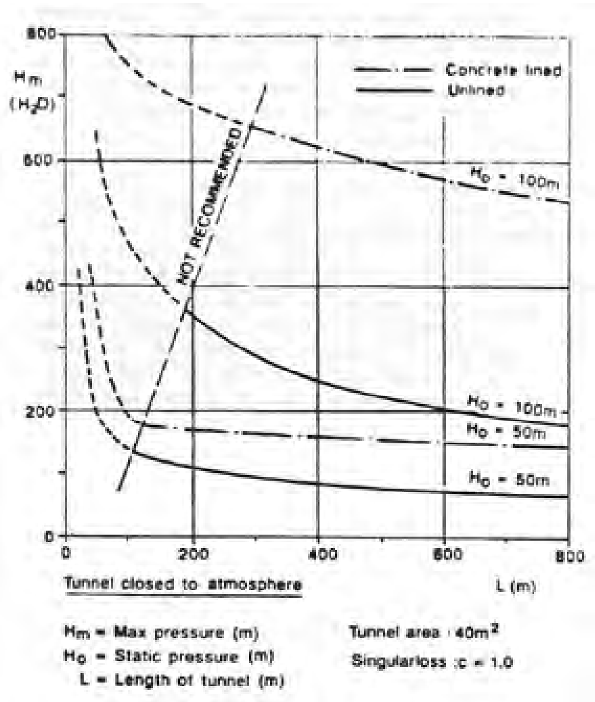
2) THE CLOSED METHOD



THE CLOSED METHOD. Fig. 2

The closed method is characterized by the air pocket at the plug being closed to the atmosphere and then compressed by the inflowing water after initiation of the salvo. Such a situation is more demanding and calls for comprehensive pre-calculation evaluation. If this is done by a competent person, this method is more flexible and safer than the open method and preferable in cases where the open method is unsuitable.
Pre-compressing of the enclosed air pocket make this method applicable and often the only recommended one especially when dealing with small air volume and high pressure. Pre- installation of pressure transducers to check the pre-compression and the water level is necessary.

The closed method can also be used without pre-compression of the enclosed air pocket and little or none water filling, but only if the closed tunnel is considerably longer than the water depth outside the plug. As an example, if the tunnel length is 10 times the water depth, the maximum pressure after the salvo will not exceed the static pressure by more than 10%.



THE CLOSED METHOD Fig. 3

Fig. 3 shows how the maximum pressure decreases with increasing length of the dry tunnel.It also shows the influence of another parameter, the tunnel roughness. It should be noticed that the maximum pressure in a concrete lined tunnel will more than double the pressure compared to an unlined rock tunnel.

The closed situation calls for advanced calculations which are complicated and must be done by a computer, otherwise they will be too time consuming. Besides, the computer model has to be calibrated as many empirical factors are involved in the model. These factors must be determined by measurements during the execution of closed method blasting. There are many factors that affect the maximum pressure in a tunnel closed to the atmosphere such as the external water pressure on the plug,the area and volume of the plug, the amount of dynamite, area and length of the tunnel, the tunnel roughness and not to forget the pre-compression. The system receives energy from the inflowing water and the charge. The different losses are,hydro-mechanical losses, heat transmission between water, air and rock walls,etc The difference between incoming energy and losses give information to calculate the maximum pressure.

The computer model that has been developed is calibrated based on full-scale measurements and have proved to be very reliable when provided with correct informations.The closed method is less suited for physical model-test than the open method as such model-tests require special remedial action. The reason is that the atmospheric pressure is a dominating factor in the compression face and should be scaled like the outside water pressure. This is very complicated. Other methods of approach have been tried to carry out physical model tests, but to day they are not to be recommended compared to the computer model.
The closed method has been applied to a lot of cases at different water depths. The deepest ever done was in connection with the oil activity in the North Sea. To solve the shore approach problem the pipe line from the sea was connected at 200 m depth to a tunnel from the land based terminal. In this case the closed air volume was limited and consequently a very high pre-compression had to be used. Both the dynamite and the detonators had to be adapted to the high pressure. In this case the pressure measured during the blast, closely corresponded to the pre-calculations. The uncertainty in such calculations is mainly connected to the estimation of dynamite-gas or probably an expansion of the volume outside the plug. The uncertainty in pre-calculation is less when a high pressure is used but is not considered to be determining factor. More important is that the plan of approach has been prepared thoroughly. If a high pre compression has to be used, one must make sure that the blasting equipment is available.

A rough description of a plan of approach regarding the closed method with a limited air volume is: Water- filling to protect the valve against debris from the plug, calculation of the remaining air volume and then choice of preliminary or final pre-compression. The salvo will now increase the pressure and if the probable pressure now is less than the external water column, a post-compression will take place caused by the inflowing water and give the maximum pressure in the air pocket. The pressure on the gate may be corrected according to the location of the gate.
Such calculations must be done by skilled people and if so the procedure is safe. Many measurements have been done during the execution of submerged tunnel piercings using both the open and the closed methods. The results have been used to correct the computer model and improve the other calculations that are necessary.

4.3.2 TROLL PHASE 1 – LANDFALLTUNNEL KOLLSNES

Background
In 1979, Norske Shell found Europe's biggest gas field in the North sea, 80 km northwest of Bergen. The Troll field development was planned for about 10 years,

before start of construction in 1990. A 6 years construction period was scheduled for the total development, giving 10% of the total need of gas in Europe when the gas production starts in 1996.

The yearly produced energy volume from the Troll field is about three times the total produced energy in all of Norway's water powerplants.

The Troll-field will produce gas in 50-70 years, where wet gas is pumped from the seabottom, through gas pipelines ashore to a gas treatment plant at Kollsnes. At Kollsnes the gas is being processed and thereafter exported to Emden and Zeebrügge, for further distribution into Europe.

Shore approach solution
Between the gas feild offshore and the gas treatment plant at Kollsnes, the seabottom are very uneven, especially the last distance towards land.

It was therefore decided a landfall solution with shore approach tunnels going 4 km out in the North sea, where 3 import pipelines and 2 export pipelines is going out on the seabed in vertical shafts at approximately 170 meter waterdepth.

An extreme challenge, giving both the Client and contractor challenges and limits not reached so far in the tunnelling history.

Shore approach tunnels
8 km of tunnels with cross-section between 50 and 110 m2, was driven in 21 months. The first 2 km was driven downwards 1:7 in granitic and amphibolitic gneiss, including several and difficult weakness zones containing active swelling clay, which necessitated voluminous injection and rock securing work.

Open zones was sealed with normal cement in combination with cemsil and mauring. Research work was performed to find the optimum combination to seal off the the most open zones.

Rock securing was performed with systematic bolting, sprayed concrete and a couple of zones was secured with full concrete lining.

The main tunnel has a low point approximately 250 meter below sealevel, where the tunnels continue upwards 1:100 ending in the piercing area 4 km out in the North sea. The last part of the tunnels was driven in more migmatitic gneiss, with few weakness zones and less need for rock securing work. The tunnelsystem ends in three vertical shafts going out on the seabed at 157.5, 161 and 168.5 meter water depth.

Preparation work before shaft driving
Prior to driving of the vertical shafts, necessary preparation work was performed.
A safety barrier in concrete was constructed in each shaft tunnel to stop an uncontrolled inleak of water during shaft driving.

Further, seismic examination, systematic core drilling and injection work was done in the shaft area to ensure that the optimal location was found, to identify the exact level and shape of the seabed and to avoid any uncontrolled inleak of water during drilling and blasting of shafts.

Finally, examination of existing soil masses on the seabottom was performed. It was observed that the thickness of soil sediments above rock where up to 4 meters, and these sediments had to be removed to increase the probability for successful shaft breakthrough and to avoid huge volume of sediments/clay to be stuck in the shafts after blasting of tunnel piercings.

The removal of sediments was performed after injection work with a submersible vehicle called SEMI-2 having 12.000 hp propels. The vehicle removed rock upto 3 tonnes from the seabottom.

Shaft driving
3 shafts with 35 m2 diameter and 25-35 meter length were driven with specially designed Alimak-equipment, until a piercing plug of 6-7 meters was remaining.

Preparation work after shaft driving
In the shafts after rock securing, a steel cone was installed 20-30 meters above the tunnel floor. This steel cone was machined to match the riser bundle containing the gas pipelines, after the tunnel piercings were successfully completed. Each steel cone weighed approximately 10 tonnes, and was installed with 2,5 mm accuracy using a specially constructed winch and sheave system enabling the steel cone to be installed without any persons in the shafts or below on the tunnel floor.

In addition to the steel cones, it was installed two steel constructions above the steel cone to ensure the correct rotation of the riser bundles during installation.

Finally several concreting lines with special built concrete locks were installed above the steel cone to resist the forces from the final blast, and to enable concreting between the rock walls and riser bundles after installation. The constructions in the shaft were designed to stay undamaged and resist the forces from the final blasts containing approximately 1500 kg of explosives, thereafter followed by the rock masses going passed the constructions and finally 20 bar of water pressure on the concreting lines with concrete locks/valves.

Final blasts
The requirements for the final blasts were completely different from historical experiences from the water power industry where no requirements to surrounding rock or installations nearby had to be considered, other than as much water as possible into the tunnel system.

The planning and engineering of the final blasts at Troll were started more than one year before execution, where the contractor and client in cooperation found the optimum way of designing and performing this 'world record'.

Three special requirements where especially challenging to solve:
• A riser bundle containing the gas pipelines and weighing approximately 450 tonnes shall be installed in the shafts after the final blasts. The final blasts must therefore be designed as careful blasting where the following had to be ensured:
  o no rock is left inside the contour of the blast
  o the steel and concrete constructions in the shafts cannot be damaged
• The final blasts must be designed and performed to ensure that the total volume of the masses (2 x theoretical volume) is safely transported to the steel cone which was only 46% of the shaft diameter. The final blasts must therefore be designed in a delayed sequence to ensure that:
  o the rock masses is not stuck inside the steel cone
  o no remaining rock above the steel cone is hampering the riser bundle installation
• After final blasts and riser bundle installation, the riser bundles shall be concreted in the shafts and the gas pipe installation shall be performed in dry conditions in the tunnel. The final blasts must therefore be designed and performed to ensure that:
  o the surrounding rock is tight and still sealed after blasting is performed
  o the concreting lines including valves/locks is undamaged with no leaks

The final blasts are drilled with a special built Nemek drilling rig, installed on a steel construction above the steel cone. About 230 bore holes with extreme tolerances, inclusive 8 no. of 6'' cut holes, were drilled per blast.

The explosives were specially designed by Dyno, and non-electric ignition were used for safety reasons, for the first time used for underwater piercings.

After drilling and charging of the final blasts, the shaft were partly water-filled, and the air volume between the water and the final blasts were pressurised upto 13 bar. All the three final blasts were successfully completed in February 1994 with the following result:
• No rock within the contour of the final blasts
• No damage to any of the steel constructions in the shafts
• No rock from the final blasts remaining above or within the steel cones
• No cracks or leaks observed in any of the shafts

Riser bundle installation
The 450 tonnes riser bundles were thereafter installed with the multi-vessel Regalia. The installation was performed using guide lines done to the steering construction in the shaft, and the landing speed was recorded to 0,05 m/s, well within the requirement of 0,11 m/s.

Finally the riser bundles was concreted in the shafts using underwater concreting especially designed for 200 meter water depth and long concreting lines pumped from the dry part of the tunnel behind the concrete plugs.

After concreting, the piercing area was emptied of water, the concrete plugs were removed, and the gas pipe installation could continue in the tunnel without a drop of water coming into the tunnel system in the piercing area.

4.4 GAS STORAGE AT -42 °CELCIUS

Status in 1996
In 1996 the company O. T. Blindheim as in Trondheim was engaged to prepare a study for one of their clients with the following object.

The study aimed to summarize the current state of storage of liquified gases in rock caverns based on the available material found in various sources of literature and references. The study aims to conclude some state of-the-art levels as far as development of such storages are concerned.

At that time, in 1996 in Scandinavia LPG storage facilities had been constructed and commissioned in Sweden, Finland and Norway. The host rock for most of these Scandinavian plants were crystalline rock types such as gneiss and granite. These storages were based on a partly chilled and partly pressurized concept, according to technology well known for almost 40 years. The pressure utilized varied from 2 bars to 11 bars, or 0,2 to 1,1 MPa. The temperature had been reported to be mainly in the range of approximately +10 °C.

Tests at that time had been performed with refrigerated storages with temperatures down to -100 °C. Due to various reasons these tests had failed. Limited experiences were available from fully refrigerated storages, however, in Sweden at that time one storage was designed for an operational temperature of -40 °C.

Consequently fully refrigerated storages in rock retained for LPG which requires a temperature of -42 °C. At such temperatures the product can be stored in uninsulated and unlined rock caverns, as most of the actual Scandinavian rock types have a moderate energy loss at this temperature level. Thus, the operation costs can be kept at a reasonable level. This coincides with the well proven approach in Scandinavia, namely to use the capacities of the rock mass, also in case of this kind of storages.

The study from 1996 concluded that although very few plants had been designed and constructed according to this concept and pot in operation, the technology to actually construct was considered being available. It was concluded that experiences could be obtained from various other types of projects utilising similar technologies that would be required for a LPG storage, such as:

- Pre-grouting techniques; sub-sea tunnelling, unlined storage of hydro-carbon products in rock caverns, radioactive waste deposits,

- Water curtain techniques; pressurised air-cushion surge chambers, unlined storage of hydro-carbon products in rock caverns,

- Cooling procedures; existing storage facilities of LPG, other types of cold storage,

- Rock mechanics; large underground openings, other types of cold storage, rock mass properties.

The current Norwegian technology for underground hydrocarbon storage in mined rock caverns was in 1996 based on the concept of shallowly seated unlined caverns with low pressure and/or moderate low temperature (>-42°). Three types of unlined rock caverns are used for gas storage: in porous formations such as depleted or abandoned oil or gas field and aquifers, in abandoned mines and in excavated unlined hard rock caverns. The storage temperature and pressure vary based on the product stored. Listed in Table 1 are the boiling temperatures of some LNG (Liquefied Natural Gas) and LPG (Liquefied Petroleum Gas) at the atmospheric pressure. The importance of this temperature level is seen in table 1 below.

Low temperature storages put in operation
Then in the late nineties and shortly after the turn of the millennium 4 projects were designed and built in Norway, see table below. Designers and the owners were pushing the limits in terms of utilizing the rock mass to build low temperature storages.

The Mongstad gas storage facility is located in west Norway and consists of two caverns: One is for liquefied propane and the other is for butane. The storage temperature in the propane cavern is reported at -42°C. The liquid propane, along with butane and naphtha, is being produced from natural gas liquids and condensate from the Norwegian North Sea Troll and Oseberg fields. The gas liquids and condensate are gathered and processed under under the Vestprosess partnership, which links three land-based facilities in western Norway.

For most existing LPG projects the storage pressure is

Gas	Methane	Ethane	Propane	I-butane	n-butane
Boiling Temp.[oC]	-162	-89	-42	-12	-1

Table 1. Boiling temperature of selected gases

Mongstad	1999	Gneiss	60,000	21×33×134	- 42	0.15	Reduced capacity
Sture	1999	Gneiss	60,000	21×30×118	- 35	0.1	No information available
Kårstø	2000	Phyllite	2 caverns, total 250,000	Approx. 20×33×190	- 42	0.15	No leakage
Mongstad	2003	Gneiss	60,000	21×33×134	-42 (propane) +8 (butane)	0.15	No information available

Table 2. Norwegian gas storages with low temperatures

not high, usually no more than a few bars. The major rock mechanics problems are associated with the thermal stress resulting from the low storage temperature. This tensile stress may or may not create thermal cracking in the intact rock, but it is definitely sufficient to open the joints that exist inevitably. Opening of joints and possibly thermal cracking will cause excessive boiloff which is one of the major reasons why some LPG storages have been decommissioned.

The shock thermal stress can be significant, take a propane storage in hard rock cavern as an example, if the storage and ambient temperatures are -41oC and 8oC, σ=7E-6/oC, E= 30GPa and ν=0.28, the shock thermal stress would be 14MPa.

4.5 GEOTECHNICAL DESIGN OF AIR CUSHION SURGE CHAMBERS

An air cushion surge chamber is an alternative to the traditional open surge shaft in hydro power plants, and has been used in Norway since 1973. The surge chambers have proven to satisfy the hydraulic demands, and have also shown to constitute an economic alternative that gives substantial freedom in the lay-out of the tunnel system, and the siting of the plant. Air leakage prevention is the major challenge when designing and constructing an air cushion. The paper shows how it is possible to handle this and other geotcchnicaJ aspects in an efficient way.

1 INTRODUCTION

Air cushion surge chambers are used as an econ· omic alternative to the traditional open surge shaft for damping of headrace tunnel transients from changes in powerplant loading. As illustrated in Figure I, the air cushion surge chamber is a rock cavern excavated adjacent to the headrace tunnel, in which an air pocket is trapped. The surge chamber

is hydraulically connected to the headrace tunnel by a short (< 100 m) tunnel.

The air cushion concept was originally introduced to improve the economy of a hydro·powerplant where a traditionaJ open surge shaft would be an expensive solution due to topographicaJ reasons (Rathe 1975). The air cushion solution also gives substantiaJ freedom in the lay-out of the tunnel system, and the siting of the plant. It is no longer necessary to maintain shallow, nearly horizontaJ, headrace tunnels for surge shaft economy (Figure 2a). Schemes which have used air cushions have tended to slope the headrace tunnel directly from the reservoir towards the power station as indicated in Figures I and 2b.

Air cushions are also favoured where the hydrau lic head of the headrace is above ground surface. In such cases, construction of an open surge shaft would require erection of a surge tower, which may be expensive and environmentally undesirable in comparison with an air cushion.

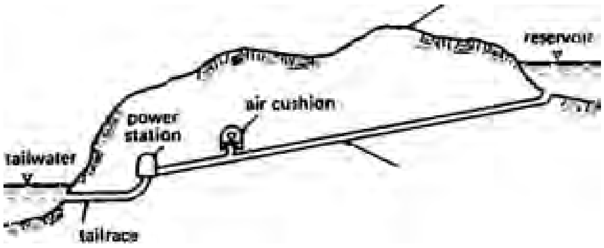


Fig. 1 Concept of powerplant with air cushion surge chamber.

2 HYDRAULIC REQUIREMENTS

The hydraulic design of air cushions follows the same principles as design of traditionaJ open surge shafts. Pressure surges in an open surge shaft system is according to the changes in water level in the shaft. In a system with an air cushion, pressure surges gives compression and expansion of the cushion according to ideaJ gas law. One should note that for normaJ surge periods (periods less than5 minutes) the air cushion responds adiabatically.

As the hydraulic design of a surge shaft is a question of finding the necessary water surface area in the shaft, the necessary air volume is the key factor for an air cushion. Traditional Norwegian design practice for high head plants, that allows pressure surges up to 10 to 15% above static, is also adopted for the air cushion sites.

To avoid air-escape from the surge chambers, they are located a few meters above the roof of the near-by headrace tunnel as shown in Figure 3. The volume of the water bed should be such that there is a high degree of safety against air escape from the chamber during unfavourable combinations of down· surge and surface wave action in the chamber. However, a major blow-out may be caused by possible failure of control equipment, improper gate operation or accidents (upstream blocking of the headrace tunnel). The mechanisms of such an event should be studied, and measures should be taken to ensure that the consequences of a possible blow-out would be tolerable. No blow-out has ever happened in Norwegian air cushion surge chambers. Normally the total surge chamber volume needed is 50% higher than the necessary air volume. The shape of a surge chamber is not a crucial point, except that the water bed response to surges may be problematic for very long caverns. Long caverns can be avoided by for instance giving the cavern a ring shape. More details about the hydraulic design of air cushions are provided in Goodall et al (1988).

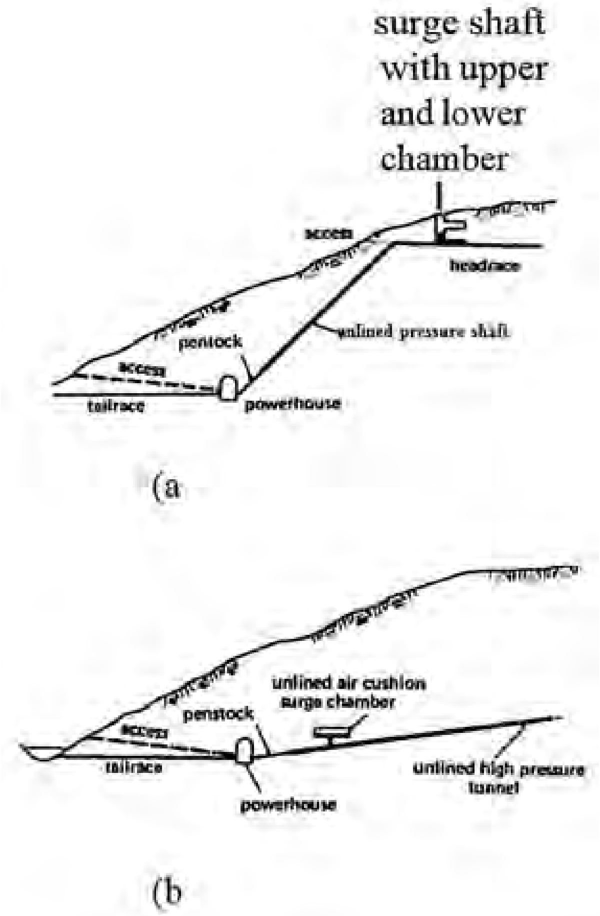


Fig. 2 Comparison of layouts for underground plant using: (a) open surge shaft; and, (b) closed air cushion surge chamber.

3 GENERAL FEATURES AND LAYOUT OF EXISTING AIR CUSHION SURGE CHAMBERS

A total of ten air cushions have been commissioned to date, all of them situated in southern Norway, as shown in Figure 4. General features of the air cushion sites are listed in Table I. The diagrams in Figure 5 show the cavern volumes and internal pressures. The first air cushion was commissioned at the Driva power plant in 1973, the latest one at the Torpa plant in 1989. As indicated in Table 1, air cushions have been used for power plants with capacity from less than 50 MW to more than 1200 MW.

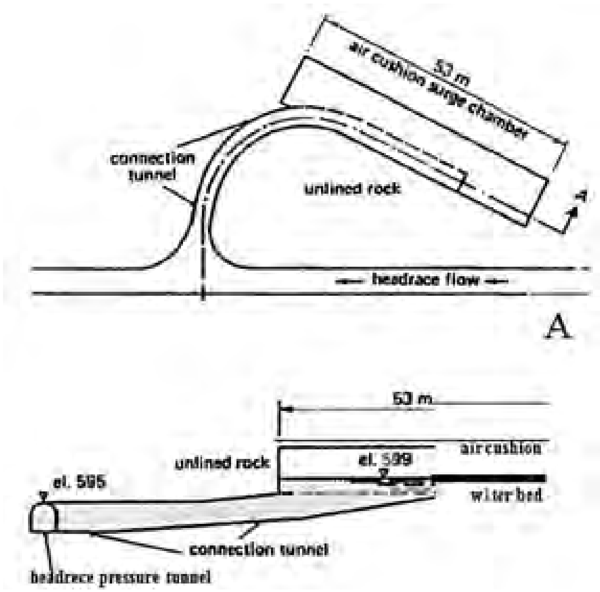


Fig. 3 Plan and profile of the Ulseth air cushion surge chamber.

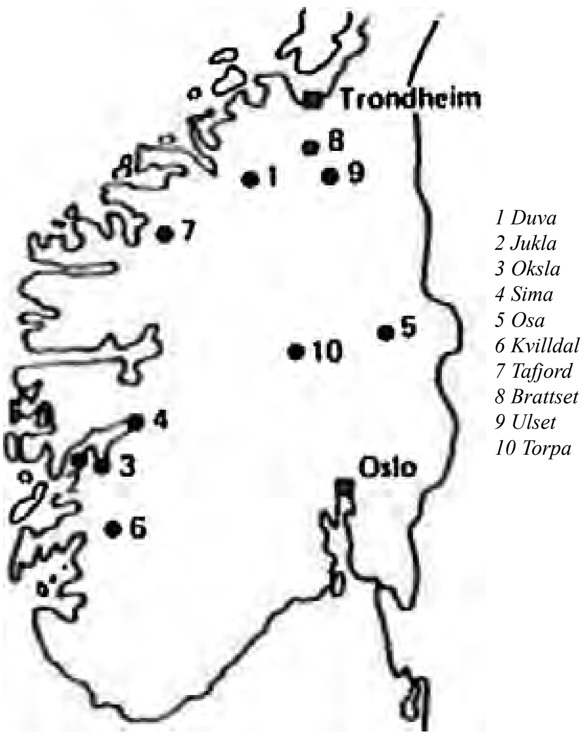


Fig. 4 Location of powerplants with air cushion surge chambers

An air cushion must be located within a limited distance from the turbine due to hydraulic reasons. In practice, distances up to more than 1000 m are found acceptable, at least at some of the sites.

Figure 6 shows two different layouts of the tunnel system in the area from the surge chamber to the power

Site	Comm issioning date	Power- plant capacity (MW)	Distance to turbine (m)	Conect. tunnel length (m)	Vertical cavern cross·sect. (m2)	Installed compres-sor capacity (Nm3/h)	Ratio between max. air cushion head and min. rock cover (m/m)
Driva	1973	140	1300	20	111	425	0.5
Jukla	1974	35	680	40	129	180	0.7
Oksla	1980	206	350	60	235	290	1.0
Sima	1980	500	1300	70	173	270	1,1
Osa	1981	90	1050	80	176	2320	1.3
Kvilldal	1981	1240	600	70	260-370	500	0. 8
Tafjord	1982	82	150	50	130	260	1.8
Brattset	1982	80	400	25	89	700	1.6
Ulsct	1985	37	360	40	92	360	I.I
Torpa	1989	150	350	70	95	940	2.0

Table 1. Features of air cushion sites.

station. Usually the surge chamber consists of one single cavern, but doughnut shaped caverns around a centre pillar have been used at the Kvilldal and Torpa sites (see Figures 7 and 9). The vertical cross-section of the caverns ranges from approxi mately 90 to 370 m2.

The ratio between maximum air cushion head and the minimum rock cover varies extensively from one site to an other. At the first air cushion, Driva, the minimum overburden is twice the air cushion head (in m of water column). At the most extreme site, To rpa, the over-burden is only half the air cushion head. The cavern volumes are generaUy less than the air cushions have pressures exceeding 4 MPa. the maximum operating pressure is 7.7MPa, equaloccupit'5 typically from 40to 80%of the cavern volume, which corresponds to a water bed thickness

Table 2 contains information about the rock types at the different project sites. Eight of the sites are located in various types of gneisses and granites, and the other two in and meta siltstone respectively. All caverns are essentially unlined, although some rock reinforcement, mainly in the form of rock bolts and shotcrete, have been used at a few sites. Each air cushion is connected to one or more compressors which are located in the access tunnels (Figure 6), or as at one site (Osa), at the ground surface above the air cushion. A system

Project name	Rock type	Natural rock Permeability* (m2)	Ratio between air cushion pres-sure and natural ground water pressure	Air leakage (Nm^3/h)	Air leakage (%/day)
Driva	banded gneiss	no data	0.6 - 0.7	0	0
Jukla	granitic gneiss	1x10-17	0.2 - 0.7	0	0
Oksla	granitic gneiss	3x10-18	1.0 - 1.2	< 5	< 0.01
Sima	granitic gneiss	3x10-18	0.8 - 1.2	< 2	< 0.01
Osa	gneissic granite	5x10-15	1.3	900/70**	11/1.0
Kvilldal	migmatitic gneiss	2x10-16	> 1.0	240/0***	0.2/0
Tafjord	banded gneiss	3x 10-16	1.8 - 2.1	150/0	5/0
Brattset	phyllite	2x10-11	1.5 - 1.6	11	0.2
Ulset	mica gneiss meta	no data	1.0 - 1.2	0	0
Torpa	siltstone	5x10-16	1.7 - 2.0	400/0***	2.0/0

Table 2. Air cushion fractures.
**to obtain hydraulic conductivity in m/s, multiply by - to7*
***before/ after grouting*
****without/with water curtain in operation*

of pipes and cables connects the air cushion to the compressor(s) and monitoring equipment. The compressors serve for commissioning of the plant. Second, the compressors serves two purposes: First, to establish the air cushion to compensate for air loss during operation A minimum and maximum air cushion volume is defined as a part of the hydraulic design. The air cushion volume is monitored by measuring the water bed level. All air cushions are equipped with at least two separate water level monitoring devices to safe guard against possible instrument malfunction . Air cushion pressure and temperature are measured directly only at a couple of the sites. The static air cushion pressure can be computed from the height difference between the reservoir and the cavern.

The temperature is found to vary only by approximately 5°C on a seasonal base due to changes in water bed temperature (which reflects seasonal temperature fluctuations in the reservoir). More details about air cushion monitoring can be found in Goodall et al (1988) and Kjørholt (1991). mentioned that none of these factors have turned out to be major challenges, neither for air cushion design nor operation. It is further important to note that no specific problems related to rock stability have been recorded at the air cushion sites.

The air loss from an air cushion may be due to both air dissolution in the water bed and leakage through the rock mass. The dissolution loss per year ranges from 3 to 10% of the compressed air (de pending on surge chamber geometry and pressure). This loss has no practical implication for the plant operation other than a need for supplementary air filling once or twice a year. The measured air leakage through the rock is listed in Table 2. Six of the air cushions have a natural air leakage rate that is acceptable. Three air cushions have no air leakage at all through the rock

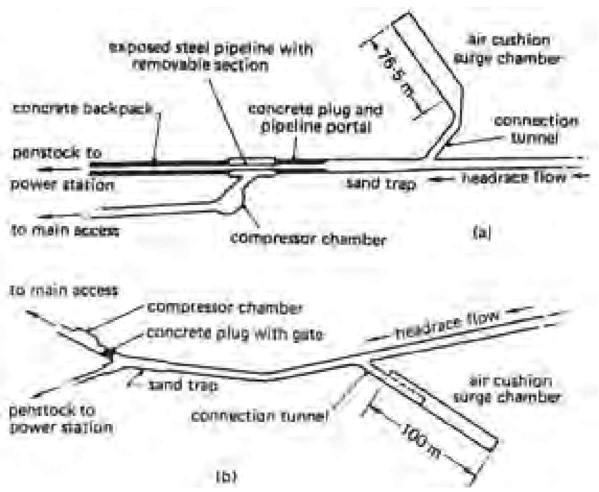


Fig. 6 Air cushion surge chamber arrangements.
(a) Oksla, (b) Brattset.

4 OPERATIONAL EXPERIENCE

4.1 General

A record of operational experience includes: Dynamic response of the air cushion as compared to computed behaviour
• Functioning of monitoring equipment
• Compressor operation Rock mass stability
Air loss and air loss prevention

Dynamic response, monitoring equipment and compressor operation are discussed to some extent in Goodall et al (1988). In this article it shall be mass. At four air cushions (Osa, Kvilldal, Tafjord and Torpa), the natural leakage rate was too high for a comfortable or economic operation. At these sites remedial work has been carried out to bring the leakage down to an acceptable level, see Table 2. One should also note from Table 2 that these four sites are located in the most permeable rock masses of all ten air cushions. Experience from the leakage prevention work at these sites are discussed below.

4.2 Leakage prevention work at Osa

Osa air cushion is located in the most permeable rock mass of all the ten air cushions, more than thousand times more permeable than the least permeable (Table 2). Although a significant cement and chemical grouting program was undertaken at the time of construction, the air leakage was measured to 900 Nm 3/h shortly after startup. After eight months of operation the plant was shut down for further grouting. The grouting was completed within three months. A total of 36 tons of cement and 5500 litres of chemical grout were injected. This brought the leakage down to 70 Nm 3/h, which is comfortably managed by the compressor plant, even though the leakage is the highest of all the air cushions.

4.3 Leakage prevention work at Kvilldal

At Kvilldal a major weakness zone passes within 50 m. of the cavern periphery and is probably responsible for a low natural ground water pressure in the surge chamber area, and thereby a higher air leakage rate than for a more homogeneous rock mass. Monitoring of the air cushion during the first year of operation showed an air leakage rate of 240 Nm3/h.

A water curtain consisting of 47 boreholes of 51 mm diameter was adopted to reduce the air leakage. These boreholes are kept pressurized with water at a pressure of 1.0 MPa above the air cushion pressure. A plan view of the surge chamber and the over-lying water curtain is shown in Figure 7. The intention of this first water curtain used at an air cushion was to limit the leakage only. However, the water curtain showed to be able to totally elim inate the air leakage through the rock mass.

4.4 Leakage prevention work at Tafjord

As at Kvilldal, the Tafjord air cushion was first com missioned without any leakage preventing measures undertaken. But even though the leakage at this site as somewhat less than at Kvilldal, the compressors installed to supply the air cushion did not have the sufficient capacity. The surge chamber at Tafjord was therefore out of operation from 1982 to 1990 (i.e the cavern was completely water filled). An attempt to grout a major fracture intersecting the cavern did not improve the leakage condition. Fortunately, the Tafjord air cushion is not crucial to the operation of the Pelton system to which it is con nected. The power plant has consequently been able to operate without a surge facility.

A water curtain was installed at Tafjord in 1990, partly as a research project. The curtain consists of 16 core drilled holes (diameter 56 mm) which covers both the roof and the upper part of the cavern walls as illustrated in the plan view in Figure 8. Also at this site the air leakage disappeared when the water curtain was put in operation at the design pressure (0.3 MPa above the air cushion pressure).

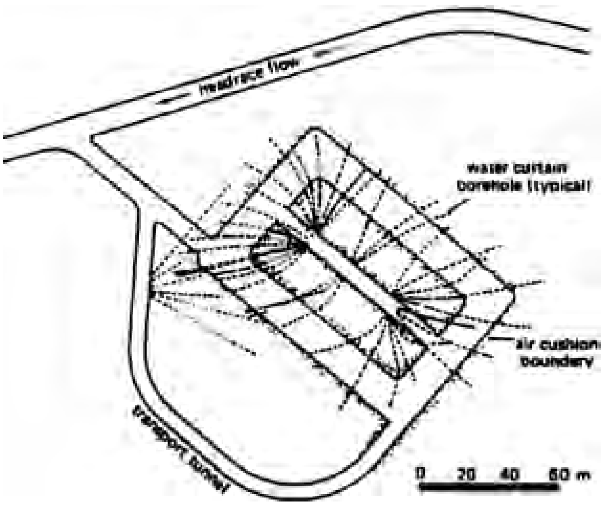


Fig. 7 Plan of Kvilldal air cushion surge chamber with water curtain.

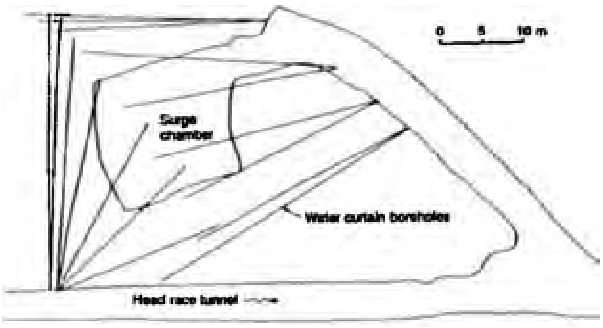


Fig. 8 Plan of Tafjord air cushion surge chamber with water curtain.

4.5 Leakage prevention work at Torpa

The Torpa air cushion is the only one where a water curtain was included in the original design. The water curtain consists of 36 boreholes (64 mm diameter}, drilled from an excavated gallery 10 m above the cavern roof (see Figure 9). In addition to the water curtain, grouting was undertaken during construction to 1mprove the rock condition.

As for the two other water curtains, no air leak age has been registered from the air cushion when the water curtain is in operation at design pressure 0.3 MPa above the air cushion pressure). To get an idea of the air leakage potential at Torpa, the water curtain was turned off for two days. This resulted in an "immediate" leakage rate of 400 Nm3/h. The leakage ceased as soon as the water curtain again was put in operation. The measured leakage rate corresponds very well with results from theoretical calculations.

To obtain the best rock stability possible, the long axis of the cavern are normally oriented so that it bisects the angle between the strike of the principal joint sets. More details about the engineering geology related to location and orientation of underground caverns can be found in Broch (1988).

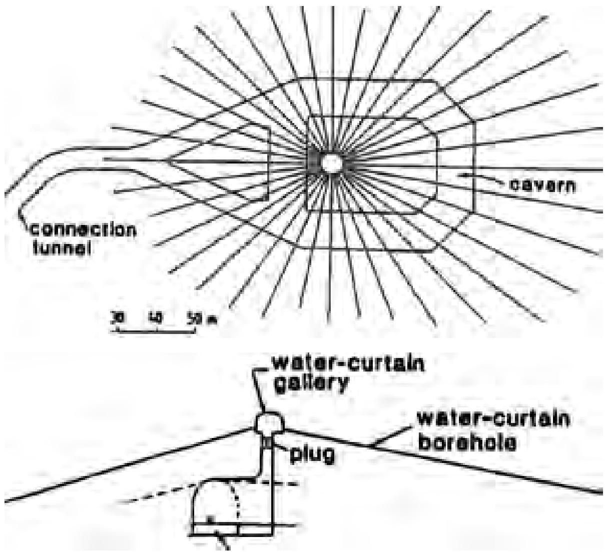


Fig. 9 Geometry of Torpa air cushion surge chamber with water curtain.

Air leakage through the rock mass has shown to be the most critical factor for a successful air cushion. The air leakage can be evaluated by use of the fol lowing equation presented in Tokheim and Janbu (1982):

$$Q_{gv} = \psi \frac{\pi K L P_o}{\mu_g G} \left(\frac{P_g}{P_o}\right)^2 \left(\frac{P_e}{P_o}\right)^2$$

where Qgw is gas leakage through the rock mass (m3/s at pressure P), K is rock mass permeability (m2), L is characteristic length of storage, µg is dynamic viscosity of gas, G is geometry factor, P0 is reference pressure, (normally atmospheric pressure, Pa abs), Pg is storage pressure (Pa abs), Pe is pressure at a plane isobar away from the surge chamber (normally ground surface with atmospheric pressure), ψ is a factor describing the relative leakage reduction due to the presence of groundwater. The factor ψ varies between one for "dry" rock, and zero if there is a positive groundwater pressure gradient towards the air cushion.

5 ENGINEERING GEOLOGICAL DESIGN OF AIR CUSHION SURGE CHAMBERS

5.1 General

The location of a surge chamber is limited by hydraulic constrains to be within approximately one km from the turbine. The challenge for the engineering geologist is to find the best location within this area. The most important factors are avoidance of rock masses with poor stability and high permeability. It is of course also essential to ensure that the rock mass has the sufficient capacity to withstand the internal pressure, in the same way as for the nearby unlined headrace tunnel.

The final location of the surge chambers in Norway is based on mapping and tests carried out from the headrace tunnel. The tests in question are permeability measurements and hydraulic jacking. Core drilling is done to verify the rock quality of a selected site.

Engineering geological mapping includes first of all mapping of:

- Rock type
- Strike, dip and frequency of rock joints, fracture zones and other discontinuities Rock mass permeability on the basis of water inflow
- Fracture roughness and infilling

The above equation shows that for a given surge chamber geometry and pressure, the air leakage depends on the surrounding ground water pressure and the rock mass permeability.

To obtain a non-leaking air cushion without introducing leakage preventing measures it is necessary that the natural groundwater pressure at the air cushion site (to be interpreted as the ground water pressure before construction of the surge chamber) is significantly higher than the air cushion pressure. In this way there will be a positive ground water pressure gradient (note pressure gradient) towards the air cushion during operation (ψ=O), which is the criterion to avoid an outward air leakage flow. This is discussed in detail in Kjørholt (1991).

If the natural ground water pressure is insufficient to totally prevent air leakage, an air leakage according to the above equation should be expected. For ratios of air pressure to natural ground water pressure less than unity, both the permeability and the ground water pressure will play a significant role for the leakage rate. In cases where the air pressure exceeds the natural ground water pressure, the permeability will be the dominating factor for the leakage rate.

5.2 Air leakage prevention

If the estimated or experienced leakage from an air cushion exceeds the acceptable lcve remedial actions can bring the leakage down below this limit. There are mainly two actions in question: Grouting and installation of a water curtain. Our experience is that the cost benefit ratio for a water curtain is considerable more predictable than for grouting.

By grouting, the permeability of the surrounding rock mass will be reduced, and the air leakage will decrease correspondingly. Experience shows that grouting can only reduce the permeability to a certain extent, and can not be used to totally eliminate the leakage. Practical experience indicates a leakage reduction by maximum one order of magnitude if the grouting takes place ahead of excavation.

Grouting after excavation is generally less effective than pre-grouting. The working principle of a water curtain is to increase the ground water pressure around the air cushion artificially. A water curtain consists of an array of boreholes above, and some times also along the sides of the air cushion. These holes are connected to a water pump that maintains a permanent water pressure in all these holes, Figures 7, 8 and 9 show the water curtain design at the three air cushions which have such an installation. If the water curtain pressure is high enough to establish a pressure gradient towards the air cushion in all potential leakage paths, no leakage will take place at all, as stated above. Guidelines for design of water curtains to obtain complete air "tightness" are provided in Kjørholt (1991).

Water curtain operation is a matter of keeping the water pressure in the boreholes at a given level. To obtain this, it is necessary that the water curtain pump operates continuously. Two types of pumps have been used, triple-plunger pumps and centrifugal pumps. Plunger pumps are used if the water has to be pumped from low pressure, while centrifugal pumps have been used in the cases that the headrace have been used as water supply.

At Kvilldal both types of pumps have been used. A plunger pump was used for the first three years until a failure occurred in the supply line for the water curtain. Since it was believed that this failure may have been caused by vibrations introduced by the pump, the plunger pump was replaced by a centrifugal pump. Torpa also uses a centrifugal pump, while Tafjord has a plunger pump mainly because of the high pressure level.

Experience has shown that if the pump stops, an air leakage will develop. How fast the leakage devel ops can vary significantly, and is a question of over burden, air cushion pressure and the structure of the rock joints. At Kvilldal it takes several months to approach the stationary leakage level experienced before the water curtain was installed. Tests at both Tafjord and Torpa showed that full leakage was reached within few hours. At all sites the leakage stops "immediately" after the water curtain has been put in operation again.

6 CONCLUSIONS

Air cushions have proven to be an economic alternative to the traditional open surge shaft for a number of hydro power plants. Experience shows that the hydraulic design should follow the same principles as for an open surge shaft. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

Air leakage through the rock masses is the major challenge when designing and constructing an air cushion. A certain leakage may for economical rea sons be accepted. If, however, the leakage exceeds a given limit, both grouting and the use of water curtains are possible actions. Experience has shown that grouting will reduce the leakage to a certain extent, while a water curtain is able to eliminate the leakage through the rock.

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4.6 GJØVIK UNDERGROUND SPORTS HALL WITH 62 M SPAN

Introduction

Norway has a long tradition of building large rock caverns for different purposes. During the 1970s, a series of studies, including some in situ testing, were initiated to investigate the feasibility of underground siting of nuclear power plants. The focus, at that time, was on the need for a reactor containment cavern with a hemispherical domed arch of at least 50 m diameter [1]. In the late 70s, another research project regarding large rock caverns was established. It was a prefeasibility study for a large rock cavern at Liåsen, close to Oslo, with a very similar size to the Gjøvik rock cavern [2]. The planned cavern was to be 60 m wide, 127 m long and 20 m high. The project would give the Norwegian rock society an advantage internationally by demonstrating their talent for underground construction. Two sketches are presented in Figure 1 and show three tunnels above the cavern. These tunnels were planned to be used for rock support and instrumentation.

In Seoul 1988, the IOC-president Juan Antonio Samaranch announced that Norway had been awarded the 17th Olympic Winter Games to be held in Lillehammer in 1994. Two ice hockey halls were needed for the games. Gjøvik already had an underground swimming pool that had been completed in 1974. The experience from the early investigations regarding large caverns in Liåsen and the presence of an underground swimming pool in the rock massive where the Gjøvik cavern could be located, gave the idea and the boldness to recommend that the world's largest underground cavern hosting an ice hockey arena be built. Thanks to the prefeasibility study at Liåsen, the Gjøvik cavern project could start directly with a detailed design. The first sketches were actually drawn on a napkin at a dinner in 1989 [3]!

The Gjøvik rock cavern

The Gjøvik Olympic Mountain Hall was excavated between 1991 and 1993. With a span of 61 m, a length of 95 m, and a height of 25 m, it is the largest man-made rock cavern for public use in the world. The total exca-

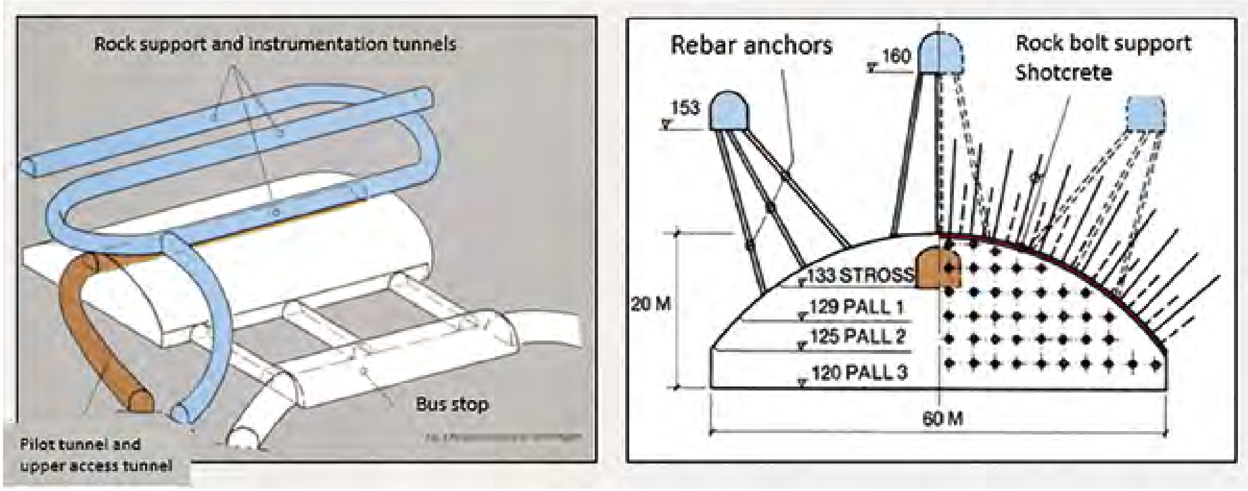


Figure 1. The Liåsen project [2].

vated volume was 140 000 m3. The width of the cavern is not the only exceptional feature of this project, equally interesting is the fact that the rock cover varied between only 25 m and 55 m, i.e. the overburden throughout the underground cavern is far less than its span.

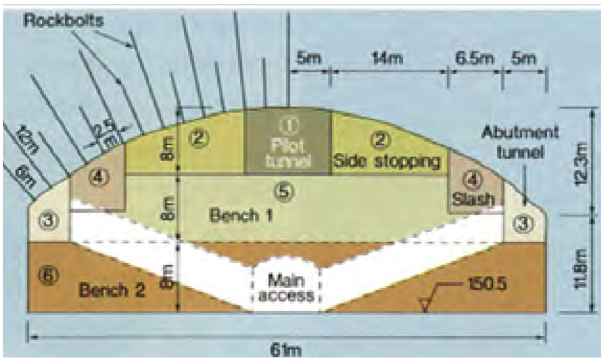


Fig. 3. Cross section of the Gjøvik Mountain Hall [4]

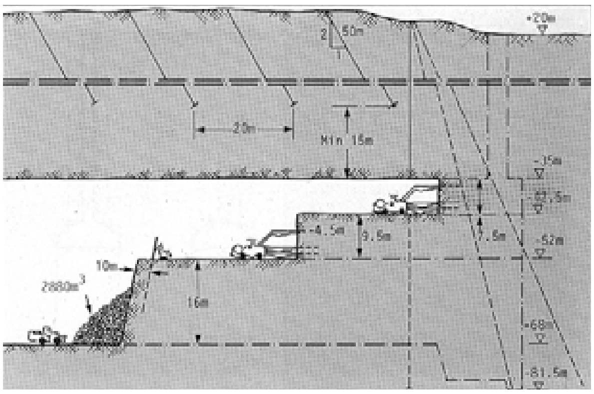
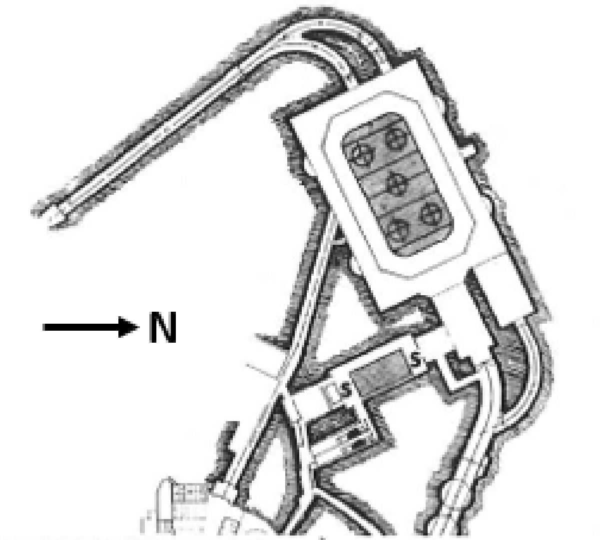


Fig. 4. Excavation sequence Gjøvik Hall (Photo: NGI)

The rock at the site is a Precambrian gneiss. The rock has a network of tectonic micro-joints, which are often filled or coated with calcite or epidote, creating a well jointed rock mass with an average RQD of about 70. The rock joints are typically persistant, with moderate to marked roughness, and normally without clay filling, i.e. positive characteristics when considering large spans. The Q-value is typically 30 for the best and 1 for the poorest quality rock mass, with 12 as an average value.

Experience from large span mining chambers in Norway indicated that an important prerequisite to obtain stable large span caverns without heavy rock support was a sufficiently high horizontal rock stress. Therefore, at a very early stage of planning, insitu rock stress measurements were made from an existing tunnel. The results showed a major horizontal stress of the order of 3 - 5 MPa, with an E-W orientation. At a depth of 25 - 55 m the vertical stress due to gravity is less than 1 MPa, indicating that the horizontal stresses are generated by geological processes (tectonic stress). This was verified later by additional tests, conducted in several rounds, including both over-coring and hydraulic fracturing in vertical boreholes drilled from the surface above the hall. In this investigation the major horizontal stress direction was N-S. Based on these findings, it was decided to proceed with the investigations.

With reliable in-situ stress values from the stress measurements, numerical modelling was carried out, using various BEM, FEM, UDEC and FLAC codes. The final conclusion was that a stable and virtually self supporting 62 m span could be constructed under the given geological and rock mechanics conditions. The maximum roof displacement was expected to be in the range of 5 – 10 mm. A key element of this entire process was



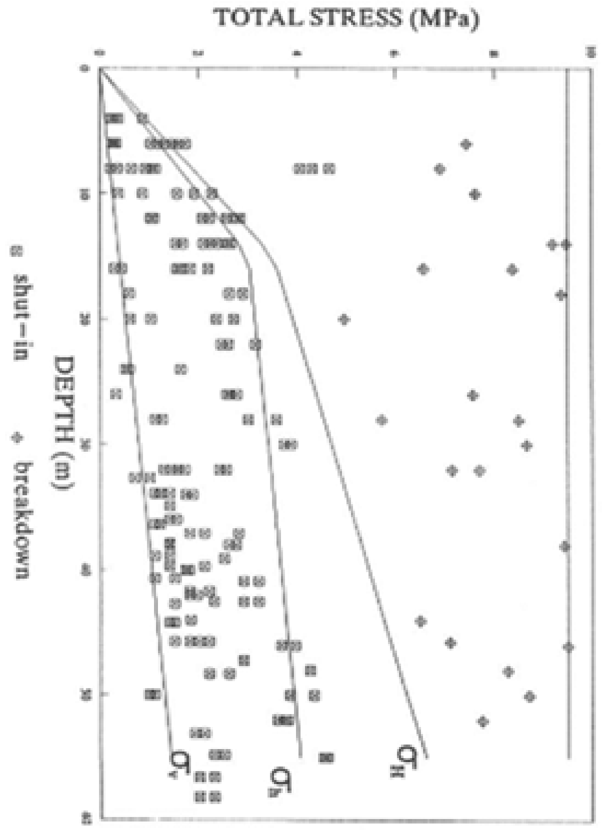


Figure 2. Left: Plan sketch of the cavern, Right: Results of rock stress measurements using hydraulic fracturing and joint jacking [1].

the results of the in-situ stress measurements. Adjacent to the location of the proposed main cavern hall area, an underground swimming pool had already been constructed in 1974, thus the geology of the area was well known, and later this facility was included in the new construction. In addition, there was an underground telecommunication centre in close vicinity to the proposed Gjøvik hall. To monitor roof deformations a number of multiple position borehole extensometers (MPBX) were installed. Figure 6 shows the cavern layout and the position of seven, 3-anchor (position) MPBXs placed in boreholes drilled from the surface (marked E1-E7), and three placed in boreholes drilled vertically upwards in the cavern roof (marked S1 – S3). In addition, surface precision levelling was carried out on top of the three centre-line extensometers.

Readings were taken regularly throughout the construction period. Figure 6 shows typical readings for the central extensometer E4, with A1 being the anchor close to the cavern roof. After the full span was excavated in about 100 days, the deformations show a decreasing trend until they stabilised completely after some 300 days. The maximum displacement was less than 4 mm. By adding the readings from surface and roof extensometer and the surface levelling, the maximum displacement was estimated to be about 7 mm. This is well within the predicted values from the different numerical models. To check the roof stresses, 2D in-situ rock stress measurements were carried out mid-span close to the S1, S2 and S3 extensometer locations. They all showed compressive roof stresses in the range of 2 - 5 MPa, which are good indicators of stable conditions in the immediate vicinity of the roof.

The investigations clearly indicate that the roof "globally" is a self-supporting structure. The roof is, however, systematically supported by 6 m fully grouted 25 mm rebar bolts in a 2.5 m x 2.5 m grid, where every fourth bolt is substituted by a 12 m cable anchor. The rock surface is also supported by a 100 mm thickness of fibre reinforced shotcrete.

Eight rebar bolts were instrumented by strain gauges, and the load change was monitored as the span was increased from 10 m, through 37 m to the final span of 61 m. Only three bolts showed any indication of being loaded. This happened close to the roof surface, with very moderate load in two cases (10 kN and 15 kN), while the third one showed 87 kN, which is about 40% of the yield load.

To check the performance of the fibre reinforced shotcrete, strain gauge rosettes specially made for concrete were installed at four locations. The readings indicated only very low tensile stresses, which are probably due to shrinkage of the shotcrete. The strength of the bond of the shotcrete to the rock was tested by a direct pull test on drill cores containing the intersection. The average tensile strength of the bond was 0.85 MPa, which was regarded as satisfactory. The main purpose of the shot-

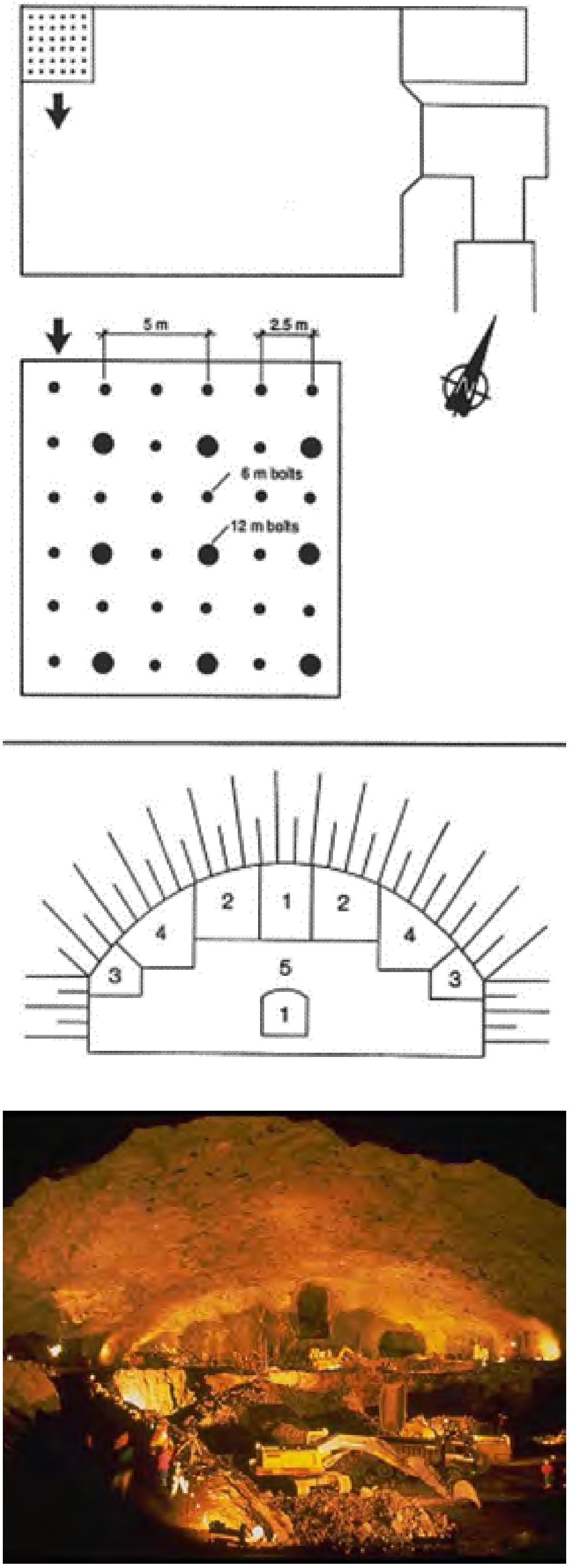


Figure 5. Left: Rock support pattern [1], Right: Construction of cavern [5].

crete seems to be to bond the rock surface together, preventing smaller rock volumes from loosning and falling. Based on this, it may be concluded that the need for the systematic pattern of 6 m and 12 m bolts and cables is questionable [7]

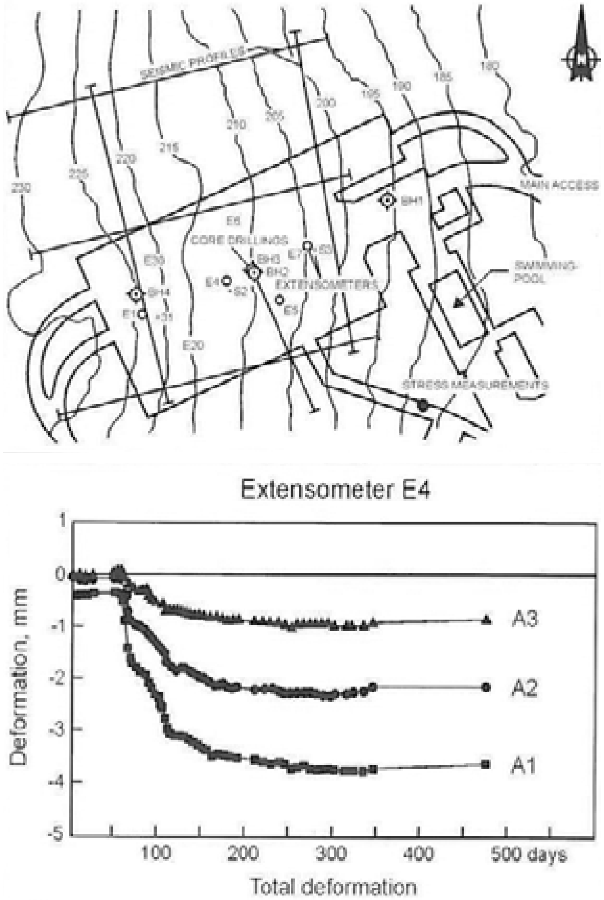


Fig. 6. Left: Cavern lay-out and borehole extensometer locations, Right: Typical readings of surface extensometer [6]

A systematic 2.5 m x 2.5 m pattern with 3 – 4 m fully grouted rock bolts, combined with 75 mm to 100 mm shotcrete will do the job even for a 50 m to 60 m span. However, horizontal stresses of sufficient magnitude are necessary to establish the global stability of the roof.

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4.7 SUB SEA TUNNEL PROJECTS IN HARD ROCK ENVIRONMENT IN SCANDINAVIA

This chapter describes the experience from Nordic sub sea tunnel benchmark projects, with emphasis on sub sea road tunnels excavated in bedrock. More than 25 such tunnels have been built in the Nordic countries, representing a total length of more than 100 km, and with the majority of the projects located in Norway. The completed projects include tunnels with length up to 7.9 km and depth below the sea level down to 264 m. All these tunnels have been excavated by drill and blast. Important issues concerning investigation, planning, design and construction are described, and important lessons learned from these projects are discussed. Finally, plans for potential future sub sea tunnel projects are presented, representing tunnel lengths of up to 24 km and depths below sea level down to 400m. Also sub sea tunnel projects in other Nordic countries will be presented as they have be designed and constructed according to Norwegian principles.

1 INTRODUCTION

In Norway, more than 30 sub sea road tunnels have been built since the Vardø tunnel was officially opened in 1983. In addition eight sub sea tunnels have been built for the oil industry as shore approaches and pipeline tunnels, and another eight for water supply and sewerage. All these tunnels are excavated entirely in bedrock by drilling and blasting (no submerged culverts), with a strong reliance on probe drilling and pre-grouting, and with drained rock support structures (Ref.1).The Bømlafjord tunnel is presently the longest at 7.9km. The Hitra tunnel is so far the deepest at 264m below sea level; the Eiksundet just recently completed construction will reach 287m depth. These tunnels have successfully replaced many congested ferries on the stem roads and connected island communities to the mainland. In total, this represents no less than a new era in coastal communication and development. A record breaking 27km long sub sea road tunnel below a wide open fjord exposed to hard weather is at the planning stage.

The Norwegian sub sea tunnel concept has gradually been implemented in other Nordic countries. The first of these was Iceland, where a 5.8km long tunnel was built below Hvalfjördur and opened for traffic in 1998. The Hvalfjördur tunnel is located in an area prone to seismic activities and risk assessment of the seismic hazard was necessary to gain the confidence of both the financing institutions and the public.

In the Faroe Islands, further south in the North Atlantic, the first sub sea road tunnel was opened for public in 2002: the Vága tunnel (4.9km). Construction commenced in 2004 for a second tunnel, the Nordoya tunnel (6.2km) and opened for traffic in mid 2006. At present 2 more projects have been contracted under one contract worth 2,5 billion DKK where excavation started up in early 2017, i.e. the Eysturøy tunnel being 11,3km and the Sandøytunnel with a length of 10,5km, a major project in a small community. Sub sea road tunnels enable highly desired improvements of the road network reducing the number of ferry connections and vitalising local businesses. A widely scattered population of 50,000 welcomes these tunnels, which on a local scale are 'major' projects.

The concept is now spreading further. Similar sub sea road tunnels are under elaboration on other Atlantic islands; Greenland, Orkney and Shetland, and on Åland in the Baltic sea. Two similar tunnels have been constructed in China and one in Korea recently.

This paper presents the experience gained from completed tunnel projects in Norway, Iceland and the Faroe Islands, with main focus on investigation strategy, construction methods and tunnelling guidelines, and based on hard rock.

However, for construction purposes the rock mass must be considered as a construction material. Throughout Scandinavia, general rock mass conditions are favourable for such utilisation. The geological setting is dominated by igneous rock types such as granite, together with metamorphic rocks of various types and origins like gneiss, shale etc. and in some places basalt The host rock is more or less intersected by weak zones, which may have an intense tectonic jointing, hydro-thermal alteration, or be faulted and sheared, constituting significant weaknesses in the rock and making the rock mass far from homogenous. These conditions may require rock strengthening measures.

Fig.1 Excavation for the portal of the Nappstraumen sub sea road tunnel

The host rock in Scandinavia general varies from poor or very poor to extremely good rock quality according to the Q-system. The zones of weakness can exhibit great variation in quality, their Q-classification ranging from "extremely poor" rock mass at the lower end of the scale, to "good", with width extending from only a few centimeters to tens of meters. The stand-up time of many of these zones may be limited to only a few hours.

2 COMPLETED NORWEGIAN PROJECTS

The road tunnel projects are located on the trunk roads along the coast, replacing often congested ferry connections, and on 'side-roads' establishing ferry-free connection from the main land to island communities. A complete list of the projects is given in Table 1. In order to give an idea about the typical environment of these projects, the early stage of excavation for the Nappstraumen tunnel, crossing under very rough waters in Northern Norway, is shown in Figure 1. The interior environment of the more recent 3-lane Oslofjord tunnel is shown in Figure 2.

Fig. 2 The 3 lane Oslofjord tunnel with artistic illumination effects for driver's comfort.

No water seepage visible due to cost-effective pre-grouting and water shielding (Ref. 3 (from Norwegian Public Roads Administration, 2002a)).

The projects are all financed with toll fees, with the toll priced typically 20% above the ferry fee. The toll portion could be around 40%; the government contributes the remaining 60%, partly by the capitalized ferry subsidies. As the local population pays a significant part, the local community has to approve the plans. Most often the projects are initiated locally. Pay back time is usually around 15 years, depending on extra cost and traffic development.

The Public Roads Administration approves the plans, including requirements to maximum gradient, minimum rock cover, safety equipment etc to ensure a consistent approach to safety and to adapt the standard to the expected traffic volume. Guideline regulations are implemented to keep costs at a reasonable level. If cost levels are allowed to escalate, it would mean that fewer connections could be established.

Norwegian road tunnels are classified in six classes labelled from 'A' to 'F' according to tunnel length and the Annual Average Daily Traffic (AADT), see Figure 3. Focus in Norwegian road tunnels is particularly on the AADT, rather than the tunnellength which is common in many other countries.

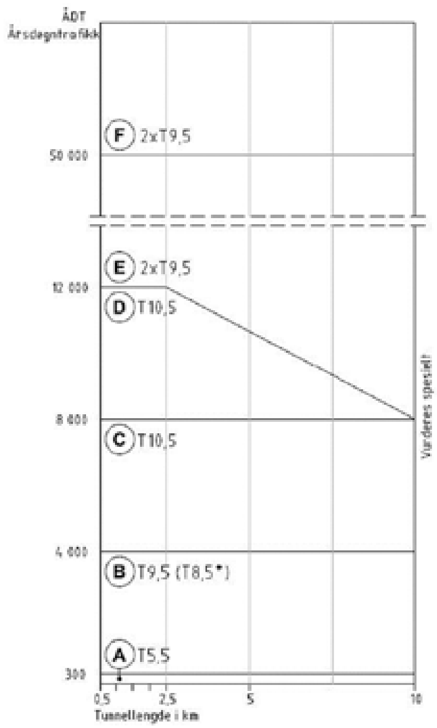

Fig. 3 Tunnel Classification according to the Norwegian Public Roads Authorities. "T" refers to tunnel width in metres (ÅDT = Annual Average Daily Traffic)

The required installations are also related to the tunnel class. Depending on length and traffic volume, this may include:
- Emergency lay-bys at regular intervals, with fire extinguishers and telephones. Longer tunnels may have turning niches for trucks (semi-trailers);
- Electrical supply: high voltage supply from both tunnel entrances with transformers along the tunnel, supplemented by emergency power;
- Ventilation by reversible jet fans providing longitudinal ventilation. Maximum air velocity is 7m/s for two way traffic and 10m/s for one way tubes. In case of fire, the air velocity shall not be less than 2-3.5m/s (5MW/20MW) to allow smoke control;
- Illumination divided into nightlight, transition and daylight zones;
- Communication: emergency communication for rescue operations, as well as radio reception and cellular phone coverage along the tunnel. The tunnel operator can interrupt the radio stations with emergency messages;
- Control system by a programmable logic control (PLC).

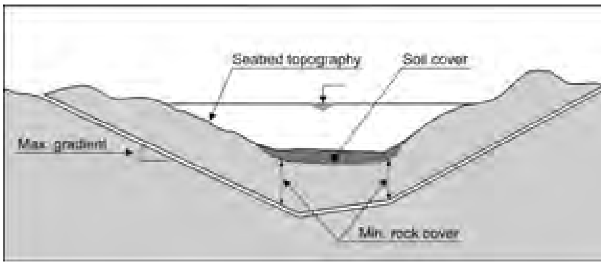

Fig. 4 Typical section of a sub sea tunnel with critical parameters for the design

The design fire load is 5MW for AADT(10) <10,000 and 20MW for AADT(10) > 10,000. These figures are under revision in view of recent fire tests showing very high effects of ordinary truck loads.
Pumping of remaining water inflow is always included. A water sump is located at the low point with capacity to store at least 24 hours of allowed inflow (typically 300 litres/min per km or less).
The different requirements are gathered in a standard issued by the Norwegian Public Roads Administration (Norwegian Public Roads Administration 2002a). The standard is based on the experience gained from almost a thousand km of road tunnels, including also sub sea road tunnels. Any deviation from the specifications in the standard must be approved by the Directorate of Public Roads.
Present cost levels for sub sea road tunnels, including installations, vary between USD 6,000 and USD 10,000 per meter tunnel (in 2005), depending on whether it is two or three lanes (the latter often used in the upslopes). All tunnels so far, except one, have one tube.
Contract types have, with one exception, been the traditional Norwegian unit rate contract (Refs. 4 and 5). This includes payment according to experienced quantities of excavation, rock support, probe drilling and grouting and other waterproofing, which takes care of most of geological risks with respect to variations in rock mass quality. Notably, the contract also includes 'standard capacities' which allows automatic adjustment of construction time according to the experienced quantities. This provides for a risk sharing between the owner and the contractor which is especially suitable for sub sea tunnels. The owner maintains the risk for any 'surprises'; after all he has decided the extent of the site investigations.
In addition to the road tunnels, several sub sea tunnels have been built by the oil industry for oil and gas pipelines, and some for water supply and sewerage. Key data for some of the most significant are shown in Table 2.

3 DESIGN PRINCIPLES FOR SUB SEA TUNNELS

3.1 Site investigation strategy

Besides normal geological surveys on both sides of the fjord, and on any adjacent islands, the site investigations rely heavily on seismics in the first stages. Acoustic profiling will first cover a large area to determine the most suitable corridor, then extensive refraction seismic surveys to select the best alignment and to provide information about soil deposits above the bedrock and about weakness (low velocity) zones in the bedrock. If possible, directional core drilling as illustrated in Figure 5 is used from shore to the critical deepest points of the alignment, which typically also could be the location of major fault zones. Core drilling from drilling ships has been applied in a few cases, if other methods were not feasible, or the results in doubt. Such drilling is seldom cost effective and not always conclusive; if feasible it may be better to plan for more directional core drilling.

The costs for the site investigations typically amount to 3-7% of the construction costs.

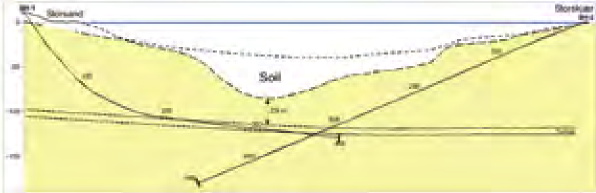

Fig. 5 Conventional and directional core drilling applied for investigation of critical part of the Oslofjord tunnel (based on Palmstrøm et al. 2003).

	Project	Year open	Main rock types	Cross section, m2	Total length, km	Min. rock cover, m	Low point, m b.s.l.
1	Vardø	1982	Shale, sand-stone	53	2.6	28	88
2	Ellingsøy	1987	Gneiss	68	3.5	42	140
3	Valderøy	1987	Gneiss	68	4.2	34	145
4	Kvalsund	1988	Gneiss	43	1.6	23	56
5	Godøy	1989	Gneiss	52	3.8	33	153
6	Hvaler	1989	Gneiss	45	3.8	35	121
7	Flekkerøy	1989	Gneiss	46	2.3	29	101
8	Nappstraumen	1990	Gneiss	55	1.8	27	63
9	Fannefjord	1991	Gneiss	54	2.7	28	101
10	Maursund	1991	Gneiss	43	2.1	20	92
11	Byfjord	1992	Phyllite	70	5.9	34	223
12	Mastrafjord	1992	Gneiss	70	4.4	40	133
13	Freifjord	1992	Gneiss	70	5.2	30	130
14	Hitra	1994	Gneiss	70	5.6	38	264
15	Tromsøysund	1994	Gneiss	2 x 60	3.4	45	102
16	Bjorøy	1996	Gneiss	53	2.0	35	88
17	Sløverfjord	1997	Gneiss	55	3.3	40	112
18	North Cape	1999	Shale, sand-stone	50	6.8	49	212
19	Oslofjord	2000	Gneiss	79	7.3	32	134
20	Frøya	2000	Gneiss	52	5.3	41	164
21	Ibestad	2000	Mica schist, granite	46	3.4	30	112
22	Bømlafjord	2000	Greenstone, gneiss, phyllite	74	7.9	35	262
23	Skatestraumen	2002	Gneiss	52	1.9	40	91
24	Melkøysund (priv)	2003	Gneiss	50	2,3	36	63
26	Halsnøy	2008	Gneiss	50	4.1	45	135
27	Eiksund	2008	Gneiss, gabbro, limestone	71	7.8	50	287
28	Atlantic Ocean	2009	Gneiss	85	5.8	45	250
29	Finnøy	2009	Gneiss	62	5.7	50	200
30	Rya	2011	Garnet-mica schist/gneiss	67	2.7	24	87
31	Karmøy	2013	Gneiss, greenstone/-schist, phyllite, meta-sandstone	92	8.9	45	139
32	Knappe	2015	Gneiss	2 x 85	6.4	13	29
33	Kvernsund	2017	Gneiss	62	3,3	50	127
34	Hundvåg	(2019)	Gneiss, phyllite	2 x 67	5,6	45	95
35	Ryfylke	(2019)	Gneiss, phyllite	2 x 62	14	43	291
36	Rogfast (planned)		Ophiolite, phyl-lite, gneiss, granite, green-stone	2 x 75	27	50	392

a) The only tunnel with two tubes b) Assumed rock cover from site investigations, proved to be lacking at deepest point, see text

The established practice of site investigations has proven to be reliable, but exceptions have occurred. In the Oslofjord tunnel, despite of an extensive program of seismics, directional core drilling, hole-to-bottom seismics and seismic tomographic interpretations, the glacial erosion along a known depression along the bottom of the fjord proved to be much deeper than interpreted and left the tunnel without rock cover over a short section. This was detected by probe drilling during construction; a by-pass tunnel was prepared to allow continued tunnelling under the fjord. The soil filled section was frozen (at 120m water pressure) and excavated through (Ref 6). Figure 6 demonstrates that if the core hole had been placed above the tunnel alignment, not within the cross section, the eroded channel could have been avoided.

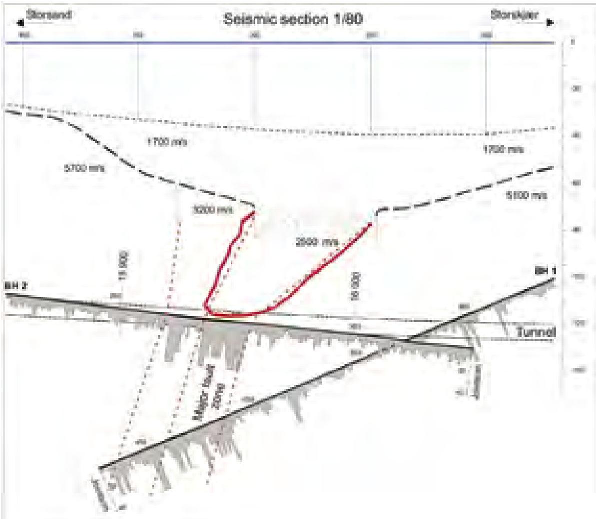

Fig. 6 The eroded channel at the deepest point of the Oslofjord tunnel (based on Ref. 7).

A similar situation was close to occurring at the Bømlafjord tunnel. A 900m long directional core hole towards a low point in the bedrock (not the deepest) hit moraine where rock was expected. This was checked by further directional core drilling and the tunnel alignment was adjusted (from 7.0 to 8.5% slope) to pass in the bedrock below the moraine deposit.

The length of the tunnel, and therefore the cost, is to a large extent decided by the maximum depth, the minimum allowed rock cover at the critical point(s), and the applied maximum slope. The allowable slope has typically been between 6 and 8%, depending on the Annual Average Daily Traffic (AADT). Slopes up to 10% have been used for low traffic tunnels on side roads. At present the maximum inclination is 5%.

The requirement to rock cover has basically been the same for all the road tunnels built until 2002, i.e. minimum 30-50m. A rock cover of less than 50m could earlier be accepted when detailed site investigations demonstrated fair rock mass conditions (taking into account the typical occurrence of fault zones at the deepest point). This is left much open to interpretation, and rock cover less than 20m has been used, but then typically restricted to shallow waters and for good rock conditions (Ref. 8). In some cases, the economic feasibility of a low traffic tunnel project depends on the minimum rock cover being cut to a safe minimum (Ref. 9).

Basically, the rock cover can be looked upon as including an rock mass arch of sufficient bearing capacity (considering the water pressure), a margin for undetected variation ('surprises'), and a margin for 'reaction time' should a fallout occur. The latter proved useful in the Ellingsøy tunnel (Ref. 10), where a cave-in started in a blasting round through a fault zone and developed upwards at a rate of 1m/h. It stopped however after 10m. Due to such incidents, and a couple of other 'surprises', the Norwegian Public Roads Administration now (since 2002), unless fair rock mass conditions have been proved, insists on a minimum rock cover of 50m.

Smaller cover has to be approved by the Directorate of Public Roads, and is checked by independent review.
As always in tunnelling, much effort is put into avoiding 'surprises'. Many so-called unexpected geological conditions are indeed foreseeable. But they may be more difficult to check out due to the sub sea conditions. A certain remaining risk has to be considered, even after significant and relevant site investigations. This is why risk control during planning and construction becomes important. For the Frøya tunnel, an external team of experts provided an independent risk assessment (Ref. 12). This is now recommended for all sub sea tunnels. Continuity in planning and investigation should always be aimed at to ensure that interpretations from early phases are brought forward to the detailed design and construction phases.

3.2 Excavation, probing, grouting and rock support
All sub sea tunnels in Norway have been excavated by D&B, as illustrated in Figure 7. This method provides great flexibility and adaptability to varying rock mass conditions and is cost effective. The 6.8km North Cape tunnel was considered for TBM, but the risks connected to the potential water inflow were considered too large. In hindsight, this would not have been critical, as the main problem proved to be thinly bedded rock causing stability problems in the D&B drives, which would likely have been less in a TBM drive.

The most difficult rock mass conditions often occur in fault zones along the deepest parts of the fjord. Any uncontrolled major water inflow will have severe consequences. Major water in-bursts have been avoided so far.

Fig. 7 Drill and blast excavation in difficult rock mass conditions in the Frøya tunnel where extensive shotcreting and concrete lining were required.

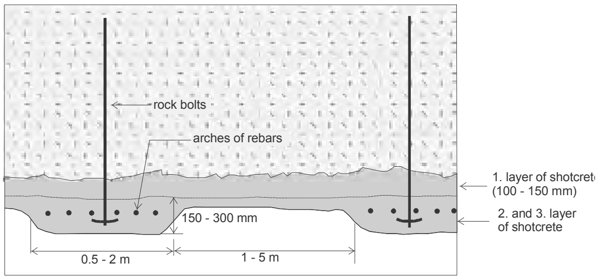

Fig. 8 Typical design of shotcrete ribs used at Frøya and other Norwegian sub sea tunnels.

The systematic percussive probe drilling by the drilling jumbo is the single most important element for safety. By applying criteria related to inflow per probe hole on when to pre-grout, the remaining inflow can be controlled and adapted to preset quantities for economical pumping, which is normally 300 litres/min per km. Follow-up at the tunnel face by well qualified engineering geologists (and rock engineers) is of great importance.

All rock support structures are drained, whether they are made of cast-in-place concrete (mostly horseshoe) lining, sprayed concrete ribs (see Figure 8) or sprayed concrete. Sprayed concrete is dominantly applied as wet mix steel fibre reinforced. Extensive testing demonstrates that, if the thickness of the sprayed concrete is above a minimum of 60-70mm, and the concrete quality is good (C45), corrosion of steel fibres is not a problem.

The use of sprayed concrete has increased over the years from 0.7-1.0m3/metre tunnel to about 1.5-2.0m3/metre tunnel (Ref. 12Norwegian Public Roads Administration, 2002b). This reflects the increased demands to detailed stability and reduced maintenance.
Rock bolts have extensive corrosion protection. In the Eiksundet tunnel, the multiple corrosion protection provided by the CT-bolt, by hot-dip galvanising, epoxy coating and cement grouting applied on both sides of a plastic sleeve, provides excellent corrosion protection on the sub sea sections. For the different tunnels, the average number of rock bolts has varied from 1.5 to 7 bolts/metre tunnel.

3.3 Experience from the deepest tunnels (>250m)
The three deepest tunnels, Hitra 264m, Bømlafjord 260m and Eiksundet 287m have not experienced any special problems. Grouting against water pressures of 2~3MPa can be efficiently achieved with modern packers, pumps and grouting materials.

Fig. 9 Andersen Mek. Verksted high pressure grouting rig.

Grouting pressures up to10MPa are today quite common with modern grouting rigs as shown in Figure 9.

4 OPERATIONAL EXPERIENCE
4.1 Water leakage and water handling
Water ingress at time of opening has varied from 20 to 460 litres/min per km, depending on the local conditions. The normal target upper level is 300 litres/min per km, which is achieved by pre-grouting. The remaining water seepage has stayed constant or reduced by self-sealing (up to 50% reduction) in all tunnels. A curious problem in some tunnels is the development of algae in the drainage water and the pump sumps. The algae population seems to expand to a certain level, collapse and then expand again. If exposed onto the driveway, which shall not normally happen, the algae make the asphalt slippery. A number of installations have to be replaced periodically ('re-investment'), including pumps, drainage pipes, electrical installations and water/frost shield-

ing. The tunnel environment is normally quite corrosive, in sub sea tunnels in-leaking saline water makes it even harsher. The recommended and experienced lifespan for installations may vary from 15 to 40 years. Steel quality shall be corrosion resistant. Typical costs for re-investment, maintenance and operation for several tunnels have been 65-130 USD/metre per year, or 1-1.5% annually of the initial investment. Costs for electricity amounts to 25-50% of the annual maintenance and operation cost, with ventilation taking the highest share. For the water & frost shielding, several solutions have been tried: Corrugated aluminium sheets with rock wool insulation; Polyethylene foam, sprayed concrete fire protection; GPR sandwich segments; Aluminium or steel panels with insulation; Pre-cast concrete segments. Some of the aluminium sheets have shown too little resistance to corrosion and to the air pressure impact loads from passing trucks.

4.2 Accidents

Statistics related to accidents and road tunnel safety is available from an extensive study performed in 1997 including almost 600 road tunnels (Directorate of Public Roads, 2002). The tunnels were divided in zones: 50m outside and inside the portal, the next 100m and the middle part of the tunnel. A special study was performed for 17 sub sea tunnels opened before 1996. Of these 9 were longer than 3.5km and all had an AADT <5000.
The mean accident rate for the 17 sub sea tunnels was found to be 0.09 accidents per million vehicle-km per year, which is comparable to the accident rate in the middle zone in the 1997 study. This rate is lower than on open roads in Norway. In the tunnels with 3 lanes (an extra lane in the steep slopes), the accident rate was 0.07. The accident rate for the Tromsøysund tunnel with two tubes was as low as 0.05. The tunnels with a slope of 9-10% had an average rate of 0.18, whereas in tunnels with slope 8-8.5%, the rate was 0.06. The highest accident rate of 0.45 was recorded for the Hvaler tunnel. There is a pronounced overweight of young drivers in the accidents.

4.3 Fires

Until 2002, only 3 fires had been reported in sub sea tunnels in Norway: in the Vardø tunnel (1993), the Hitra tunnel (1995) and the Oslofjord tunnel (2000), none of theme involved personal injury.

5 SECTIONAL COMPLETION FOR APPLICATION IN LONG TUNNELS

5.1 Sectional completion for application in long tunnels

In long tunnels there is a need of managing and utilising the construction time in an optimum way so as to ensure the tunnelling work will be completed as per the contracted construction time (Ref. 16). The main dominating and time consuming activity in the tunnelling is the excavation of the rock tunnel. In long tunnels there is often a need of establishing additional adits to divide the tunnel into several equal working areas. By opening up such adits the work connected to the tunnel excavation may take place at a number of working faces simultaneously. Long tunnels may be defined as tunnels exceeding 10km in length. However, in certain projects, such as sub sea tunnels it might be impossible to open additional accesses/adits than one at each end of the tunnel. Therefore, this paper will also include an example of a sub-sea road tunnel. A solution with several adits has been utilised for long headrace tunnels in the hydroelectric power industry. For the construction of road and railway tunnels the situation may call for other solutions. Recent Norwegian road tunnel projects are shown in table 1 below and experiences gained from these tunnelling projects will be briefly described together with different ways of establishing a sectional completion.

5.2 Reference projects on sectional completion

5.2.1 Sectional completion in Folgefonn tunnel

The Folgefonn tunnel is excavated in typical Precambrian gneisses. The construction contract was a traditional Norwegian unit rate contract. There were two contractors, one at each of the entrance. For this tunnel a working pattern was established as follows:

- Blast and excavate the tunnel and the ditch simultaneously.
- Install all equipment in the ditch such as pipes and manholes in sections of 1000 -1500m.
- Install all permanent rock support preferably at the tunnel face, or at least before the installation of the ventilation duct.
- Utilise the excavated rock as road embankment and reduce the need of replacing.
- A temporary asphalt layer to be laid allowing transport to take place on an covered surface.
- Installation works, except rock support, were not allowed to take place closer than 400m from the tunnel face.

The contract did not include any requirement for a sectional completion, and the contractor could choose his construction process. The asphalted road surface enabled the possibility of using heavy vehicles for transport, and together with a high speed this produced an effective transport. The asphalted road surface also produced a better Health and Safety environment in the tunnel.

No	Project	Year completed	Main rock types	Cross section, m2	Total length, km	Min. rock cover, m	Lowest level, m below sea
1	Frierfjorden, gas pipeline	1976	Gneiss and claystone	16	3.6	48	253
2	Kårstø, cool-ing water	1983	Phyllite	20	0.4	15	58
3	Karmsund (Statpipe), gas pipeline	1984	Gneiss and phyllite	27	4.7	56	180
4	Førdesfjord, (Statpipe)	1984	Gneiss	27	3.4	46	160
5	Frølandsfjord, (Statpipe)	1984	Gneiss and phyllite	27	3.9	55	170
6	Hjartøy, oil pipeline	1986	Gneiss	26	2.3	38 (6 m at piercing)	110
7	Kollsnes (Troll), gas pipeline	1994	Gneiss	45-70	3.8	7m at piercing	180
8	Kårstø, new cooling water	1999	Phyllite	20	3.0, 0.6	a)	60, 10
9	Snøhvit, water intake/ outlet	2005	Gneiss	22	1.1/3.3	a)	111/54
10	Aukra, water intake/outlet	2005	Gneiss	20/25	1.4/1.0	5/8 (5.5 at piercing)	86/57

a) No information

Table 2: Some main Norwegian sub sea tunnels for water, gas and oil.

No	Project	Year of completion	Main rock types	Cross section, m2	Total length, km	Minimum rock cover, m	Lowest level, m below sea
1	Hvalfjördur	1998	Basalt	55/65	5.8	38	165
2	Vága	2002	Basalt	65	4.9	30	105
3	Nordoya	2006	Basalt	65	6.2	35	165

Table 3: Overview of tunnels completed and in Iceland and the Faroe Islands.

PROJECT	Folgefonn tunnel	Bømlafjord tunnel	Lærdal tunnel
Type of tunnel project	Road tunnel	Sub sea road tunnel	Road tunnel
Tunnel length	11 km	7.5 km	24.5 km
Tunnel width	8 m (dual lane, single tube)	11 m (triple lanes, single tube)	9 m (dual lane, single tube)
Number of exits/entrances	2	2	2 + 1 adit
Number of working faces	2	2	4
Maximum length of tunnel face	Appr.6 km	Appr. 4 km	Appr.
Tunnelling method	Drill & Blast	Drill & blast	Drill & blast
Construction time (mobi-lisation to opening of the tunnels)	May 97 - May 01	September 97 - December 2001	July 95 - November 2000

Table 3. Reference projects sectional completion

5.2.2 Sectional completion in Bømlafjord tunnel
The Bømlafjord sub sea tunnel has a maximum descend on both sides of the fjord of 8,5%, and 5,5% in its middle part. The geology consists of various Precambrian metamorphic rock types. The project was split into two tunnelling contracts. The intention of both contracts was that a sectional completion should be aimed. However, only one of the contractors followed this principle, thus it became easy to compare the differences and identify the benefits of the sectional completion procedure. The following work was associated with the sectional completion:

- The contractor completed sections with length of approximately 1000 m.
- Due to the poor quality of the rock all blasted rock needed to be replaced, and the sectional completion included a complete re-establishing of the road embankment.
- The completion included rock support, ditches, drain pipes, man holes, cable canals and electrical/fibreoptic cables.
- The completion included also a first layer of asphalt.

The Owner, the Norwegian Public Roads Administration, expressed that the solution with the sectional completion was advantageous. The following negative aspects were associated with the one tunnel face that was constructed without following the concept of sectional completion:

- A permanent sandfilter facility had to be established outside the tunnel to clean drainage water due to large production of fines.
- The material in the road embankment of the temporary road in the tunnel was crushed due to the load from the heavy traffic.
- Additional ventilation fans due to large amounts of dust and exhaust air.
- "Dirty" working conditions affected negatively the Health and Safety aspects in the tunnel.
- Frequent local replacement of road embankment to to maintain construction traffic.

5.2.3 Sectional completion in Lærdal tunnel, the longest road tunnel in the world

The Lærdal tunnel is 24.5 km long with a 9 m wide profile and a maximum inclination of 3%. The tunnel has one adit that allowed additional tunnel faces to be opened for tunnelling. The project was split into two main contracts for the tunnelling work. There were no contractual obligations for a sectional completion of the tunnel. However, after just a few km's tunnelling from one side, the main contractor and the Owner agreed that sectional completion would be beneficial for the project. It was agreed that the contractor should establish a sectional completion in accordance with the following:

- Most of the permanent rock support was installed at or close to the tunnel face.
- The lower part of the road embankment was replaced every week in accordance with the tunnel progress of the week to be able to have an appropriate transport surface.
- Work was going on continuously with the construction of the ditch and with the installation of drainage pipes, cable canals and cables.
- Twice a year the permanent road embankment was established in addition to the first layer of asphalt. This was the permanent road construction, but approval was granted for use in the construction period.

It is important to avoid un-necessary interruption of the tunnelling advance, and thus planned stand still in the tunnelling works were used to complete the work with the permanent road embankment. This was typically done for example in the ordinary vacation periods.

5.3 Summary of Norwegian experience with the concept of sectional completion

It has not been possible to quantify the actual reduction in construction time that was achieved by using this concept. Effective tunnelling is very much dependent on constructing in good and robust routines and procedures to enable a smooth and even performance. Any interruption in these procedures will cause an immediate loss in performance. It is therefore important to plan any activity that may hamper the excavation work with care to avoid interruption. The studied projects clearly indicate positive experience with the sectional completion procedure, agreed by contractors and Owners. Sectional completion does impose a significant challenge to the contractor in respect of planning and performing a logistic that goes beyond the typical drilling, blasting and hauling routine.

The Owners indicate that they have significant time saving using the sectional completion. Time saving has an economical affect on both the Owner and the contractor. There are also clear benefits associated with the Health and Safety aspects. Health and Safety aspects are immaterial values and are very much related to the reputation of the project. The contractors valued aspects associated with transport and hauling, in additions to the time saving, as advantages from sectional completion. The contractors listed also the improved Health and Safety aspects as important improvements.

The time needed for hauling and transport is normally quite dominating for the tunnel progress. It has been shown that transport on an asphalt paved surface, and particularly in tunnels with steep gradients is more effective and better as far as Health and Safety aspects are concerned than transport on an ordinary gravel surface. Asphalt pavement in the tunnel reduces also significantly the maintenance work of the road in the tunnel itself, cleaning tunnel walls/roof and all equipment was also reduced. The tear and wear on the transport equipment was reduced as well as the fuel costs. For the contractors savings might be materialised by the fact that they have a shorter period of time with manning and site operation. Traditionally, the construction time was longer, but with less man power and equipment at the site simultaneously, as the work was taking place in batches. Implementing sectional completion, the manning and demand of equipment at site is higher, and the every day production is increased.

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4.8 WET MIX SPRAYED CONCRETE AND FIBRE REINFORCEMENT REPLACING MESH

4.8.1 From dry-mix to wet-mix
As is generally known in the tunneling industry, sprayed concrete has developed into one of the most important methods available for rock support within drill and blast excavation or any other open mode excavation system. It is fair to say that close to all tunnels of this kind under excavation today will be more or less 100% covered by sprayed concrete for immediate support and partly for permanent support.

Going back 30-40 years, the situation was quite different for a number of reasons. One change that really stands out when comparing the 70-ties and 80-ties with today, is the almost complete change internationally from the totally dominating thin-stream dry-mix sprayed concrete to thick-stream wet-mix sprayed concrete. All change is difficult and frequently will be met by opposition and as everybody who experienced the changes knows, it took decades and involved many different aspects of the technology.

It will be outside of the scope of this publication to present a complete historical overview of this interesting development, but it may be useful to point out some of the primary reasons that caused the change from ‘dry’ to ‘wet’ sprayed concrete. Equally interesting is to note that in Norway, the market switched from 100% dry to 100% wet in a matter of just 10 years starting from the beginning of the 70-ties. This was a first-in-the-world development and at the time completely unique to Norway.

When dry-mix was dominating, application was typically done by manually handling the spraying nozzle. This was extremely heavy work and would seldom allow more than about 1-4 m3 sprayed per hour and the typical rebound would be 30-40%. Hydraulic nozzle manipulators (often called ‘robots’) were developed to increase the possible output, first in Sweden and then also in Norway. However, with higher output, the problem of dust development with the dry-mix method became even worse.

When trials with wet-mix finally succeeded at offering this method as a practical alternative, it quickly became clear that the use of a robot was required to allow any increase of output. The wet-mix method is based on thick-stream transport (pumping the wet concrete all the way to the nozzle) and the hose and nozzle became too heavy for manual spraying. To allow the use of maximum size aggregate to 8 – 10 mm, the hose and nozzle diameters were also larger than what was required for dry-mix. So one change pushed the other in the sense that the robotic application then finally allowed much higher output and the higher output with only 10% rebound or less produced vastly improved economy. Rather than 1-4 m3 sprayed per hour with a 40% loss of material, the output would rather be 5-10 times higher and at only10% rebound.

4.8.2 Working environment and safety
Another important factor favouring the wet-mix was working environment and safety. The amount of dust produced using dry-mix was a constant problem and wet-mix offered a substantial improvement. This was an important health issue for people working in the tunnel and reduced dust also improved the operator’s ability to observe the production process due to improved visibility. With the switch to robotic application came also a huge safety improvement. Since sprayed concrete could be placed by remote control, nobody would be exposed to unsupported ground at any stage. Especially when traversing poor ground sections, sprayed concrete would always be the first step and drilling and placement of rock bolts would be executed as the second step. Also installation of lattice girders, reinforced ribs or even steel sets could be executed as second step under protection of the sprayed concrete.

4.8.3 From mesh reinforcement to fibre reinforcement
The next major development of the sprayed concrete technology was the use of fibre reinforcement. Traditionally, sprayed concrete would be reinforced by steel mesh when necessary, typically in the form of 2 by 5 m panels consisting of cold drawn steel wire of 3 to 5 mm thickness placed at 100 mm spacing in both directions. The panel weight would be about 4-5 kg/m2 of mesh.

The practical work procedure when using mesh reinforcement caused a number of built-in problems:
- Mostly, the process involved applying a first layer of sprayed concrete, then installing the mesh and finally spraying the cover layer, which meant 3 different operations resulting in an overall low output and efficiency.
- Sometimes, operators would place the mesh directly on the rock and spray all the concrete afterward. This approach would require a lot of concrete build-up locally in depressions behind the mesh to cover it and with seriously poor concrete compaction due to spraying shadows behind the mesh.
- On blasted rock contours, the mesh could not be made to follow the contour, causing more concrete consumption than necessary to be able to cover the mesh.

- Where individual panels of mesh had to be overlapped, spraying through 3 and sometimes 4 layers of mesh would make it impossible to ensure concrete compaction due to shadow effects and premature concrete build-up onto the mesh. Poor quality concrete and corrosion problems would be the result.
- Most steel mesh qualities were produced from cold drawn wire, which resulted in extremely low elongation before failure. In sprayed concrete subjected to high stresses and deformations, the failure energy capacity of the support would be very low. This is exactly opposite to what is aimed at when reinforcing sprayed concrete for rock support.
- Obviously, the installation of mesh would have to take place under partly supported ground, or even unsupported. The work safety would many times be questionable.

Experiments with steel fibre dosage into sprayed concrete started already in the 70-ties, but it quickly became clear that the combination with dry-mix was not a good approach. Due to the thin-stream transport where all constituents are transported through the hose to the nozzle by high volume fast moving compressed air, meant that a large amount of excess air would have to be evacuated sideways upon impact on the rock substrate. This air evacuation and the high-speed impact caused at least 50% of the fibres to rebound and fall to the ground. Later, when also synthetic fibres became available, the rebound problem linked to thin-stream transport became even more serious.

What turned out to be a much better solution, as first commercially demonstrated in Norway, was the use of steel fibres in wet-mix sprayed concrete by mixing the fibres into the concrete at the batching plant and spraying the reinforcement together with the concrete. Again, the Norwegian sprayed concrete market in tunneling switched from 100% mesh reinforcement to 100% fibre reinforcement in a matter of 10 years, between about 1975-1985. This was made possible by the fact that wet-mix concrete pumps, hose diameter, nozzle diameter and the robotic equipment could handle the fibre addition without problems.

The problems listed above linked to the use of mesh could all be avoided by employing fibre reinforcement and additionally the following advantages were achieved:
- The failure energy of reinforced sprayed concrete was much improved by using fibres as demonstrated by numerous instrumented full scale tests.
- All cross sections of sprayed concrete would contain fibres, thus ensuring that even without knowing the distribution of tensile and compressive loads, the reinforcement would be effective everywhere. (Mesh would for sure sometimes not be in the location of tensile stress).
- The minimum average thickness of sprayed concrete with mesh properly covered would typically be at least 150 mm. Any necessary thickness could be chosen when using fibres, without over-consumption of concrete.
- Substantially improved uniformity of concrete quality parameters and no corrosion problem.
- Much improved work safety could be offered by avoiding mesh installation underneath partly- or unsupported ground.
- Much higher overall production output and drastically improved economy.

4.8.4 In summary

In the world-wide underground construction industry of today, a major part of the sprayed concrete volume is executed by the wet-mix method and mostly by robotic equipment. When reinforcement is used, the dominant solution is fibre dosage. What has changed from the 80-ties and 90-ties is that synthetic fibres are now an alternative to steel. The choice depends on site specific requirements and economy.

Sprayed concrete is now used both for immediate support and permanent support and there are many different aspects of the process that will be important in each case. Besides what has been discussed above, sprayed concrete has also seen major development regarding:
- Accelerators have developed from highly caustic (aluminate) versions that are very negative for final strength and terrible for health and safety, through versions of modified sodium silicate to today a range of alkali free products (AFA). There is a range of different AFA versions, which is necessary because of the wide variation in cement types, but generally, fast reaction and marginal reduction of final strength is offered. This allows building of large thickness in one set-up, even using high output robotic equipment.
- The remote control 'robotic' spraying is mostly managed by visual observation by an experienced nozzle operator. However, several versions of automatic help systems now exist, that e.g. will accurately keep the optimal nozzle distance from the rock surface and/or ensure jet direction perpendicularly to the rock surface. There are also fully functional actual robots available, which means full-automatic spraying of pre-defined and pre-scanned areas of rock surface to a defined average concrete thickness. The whole operation can be completely computer controlled without any human intervention.

Today, the sprayed concrete method can deliver the quality and durability required, provided that the contract, specifications and control mechanisms are properly defining what has to be done, how to do it and what results are required.

4.9 HIGH-PRESSURE PRE-EXCAVATION GROUTING FOR WATER INFLOW CONTROL IN TUNNELS AND CAVERNS

Utilization of the underground space in urban areas has become a preferred option for governments and planners around the globe. In general there is a degree of uncertainty related to planning and construction of tunnels and caverns. Rock mass quality and geological conditions may be determined by drilling of probe holes and interpretation of available geological information. The positional water inflow into tunnels and caverns is always a substantial risk to the construction program and construction cost. This paper will present a model on how water inflow into tunnels and caverns may be predicted and most important how this may be counted for in the program and budget for projects in urban areas were water table draw down may cause settlements and major liability risk and cost implications. Identifying the risk and cost related to water inflow control in the planning stage is a preferred option compare to mitigations like post grouting. This paper will outline some alternative pre excavation models related to the designed water inflow for tunnels and caverns during construction. International project references will be used to verify the method.

1 Introduction
In the past 15 years or so grouting practice and technology in Scandinavian countries have evolved in order to meet the new demands such that some long-held 'rules of thumb' regarding what is possible or practicable now appear to be outdated and grouting practice has become far better documented. This is particularly the case in Norway where state-of-practice grouting methods have become highly developed. One important element of tunnelling in urban areas or elsewhere where a strict requirement applies for water control is the technique of rock mass grouting. Norway is one of the countries globally that has been the driver for this development whilst on the other hand the technology is mainly empirical based.

Underground construction and utilization of the underground space in urban areas has become common in mega cities and capitals around the globe. Predicted water inflow in tunnels, lack of knowledge and insufficient contractual tools to handle such project implications are the most frequent reasons for cost and program overrun in tunneling projects worldwide. By utilizing pre-grouting knowledge and technology we claim that the tunneling industry has a powerful technique to reach predictable costs and construction programs.

Pre- grouting has developed from being the tunnelling activity with low status and limited contractual attention to become the key performance indicator for all parties involved in urban tunnelling projects. Urban tunnelling history and frequent contractual disputes is the best evidence that this technique will benefit future projects.

The authors claim that most underground projects would benefit from increased awareness of the risk related to water inflow that cause construction implications as well as ground settlements caused by ground water draw down in the vicinity of the tunnel and cavern.

Pre-grouting has reached a high-tech level in material technology as well as to the equipment at hand. It is utmost important that the industry is capable of utilizing the technique correctly as it is required, in project specific situations. The rock mass itself is often an excellent barrier, having a significant capacity with regards to its impermeable and tightness characteristics, but owing to its nature, cut by cracks, joints and discontinuities it is not homogenous and its characteristics can vary greatly within short distances.

Tunnelling may cause a drawdown of the groundwater level resulting from the excavation process. The allowable amount of water inflow to the tunnel is governed by practical limitations related to the excavation process and pumping capacity. This applies to tunnelling in remote areas without strict regulations on groundwater impacts, and in projects without particular requirements for a dry internal environment.

2 Water inflow predictions and allowed water inflow
Water inflow prediction is an important part of successful water inflow control. There are several methods of quantifying the expected water inflow into a underground facility like a tunnel or a cavern. The main parameters that will influence the water inflow are listed below:
- The size of the water source
- The head of water above the tunnel
- The horizontal separation between the water source and the tunnel or cavern
- The recharge to the water source
- The degree of joint openness in the rock mass

Investigation drilling will produce information about the geological conditions and the rock mass classes. Permeability testing and Lugeon values will provide indications of the local permeability in the vicinity of the investigation drill hole. But this information is difficult to utilize in an accurate way in the calculation of the actual water inflow capacity.

The main purpose for water inflow prediction is to verify the risk related to the underground excavation works. In international contracts this calculations will be a part of the Geotechnical Baseline Report (GBR).

The consequences of the water flow prediction are related to the allowable water flow into tunnels and cavers in the construction phase or for the life time of the underground facility.

To specify the allowed leakage into an underground facility is the responsibility of the project owner. But the limits often need approval from authorities like Geotechnical office or similar. In general all developers are responsible for any damage to property or environment caused by temporary ground water drawdown or lowering the ground water level on a permanent basis. Monitoring the ground water levels in short and long term perspective is important though often neglected I urban projects.

It has become the international common practice to quantify allowed water inflow to tunnels by how many litres of water per minute that may flow into a 100 m length of tunnel.

The water inflow if often mentioned as Limit of Residual Inflow Rate (LRIR). Typical values are given in the table below.

Leakage measured in litre/ minute/100 m	Type of construction	Type of environment
2 – 5	Urban tunnels	High sensitivity to settlements
5 - 10	Urban tunnels	Sensitive to settlements and fauna and flora
10 - 30	Urban tunnels and caverns	Moderate sensitivity
> 30	Rural tunnels	No particular sensitivity

Table no. 1 Typical allowed water inflow rates into tunnels

To specify inflow rates below 10 liter/minute/100m will add 50 – 70 % on the excavation cost for the tunnel compared to an un-grouted tunnel. The time and cost related to grouting are not a linear function to the LRIR. Weekly excavation rates will be reduced 10 % - 60 % by systematically probe drilling and pre-grouting depending on the specified LRIR.

The most important benefit and outcome of a reliable water inflow calculation are to make an assessment of the probe drilling regime and to include the right cost and schedule implications for pre-excavation grouting to the required inflow level the construction budget and program.

The historical way of ignoring water inflow issues in the GBR, leaving the contractual risk for handling water inflow to the contractors will only benefit the legal side of the construction society.

3 The grouting approach

There are two options of sealing off water inflow into tunnels by grouting. To treat the rock mass surrounding the tunnel and ahead of the face with a material the fill up and block all cracks, joints and openings is generally referred to as pre-grouting or pre-excavation grouting. Water control is achieved by probe-drilling ahead of the face followed by pre-grouting of the rock mass. A standard drill pattern may be used along with grouting materials at reasonable cost and volumes. This method is predictable in terms of time and cost implications. The actual influence on tunnel cost and progress will vary with the LRIR, but in general the excavation rates decrease and the actual excavation cost will increase by 10 to 60 %

The alternative to pre-grouting is post grouting. Post grouting is the general term for all grouting related to stopping water inflow post excavation of a tunnel or cavern. Due to the fact that the water has free flow into the tunnel this method become very difficult. The number of drill holes will increase and open joints and running water will make it difficult to keep the grouting material inside the rock mass. Accelerated grout and foaming water reactive grout may be used, but the author does not know any cases that claim to have undertaken cost efficient post grouting. But there are a lot of case stories of the opposite. In general the cost and time spent for post grouting to the same LRIR will increase the actual excavation cost by 50 to > 200 %

This said post grouting could be an option to seal the last few drips in urban tunnels with a low specified LRIR but this requires a systematically pre-grouting regime applied during excavation of the same tunnel.

Water inflow measured in litre/minute/100 m length of tunnel	Type of grouting	Recommended inflow criteria in probe holes (approx 25 m length)
0 - 15	Systematically pre grouting	Probing not applicable
15- 30	Pre-grouting initiated by measured water inflow in probe holes	1 – 2 ltr/min for single holes 3 – 6 ltr/min for all holes
> 30	Probe drilling	Individual evaluation

Table no. 2 Inflow limits for systematically pre-grouting

The inflow criteria from probe holes need to be in line with the LRIR for the actual tunnel. Some experience show that the water inflow from probe holes is related to the total water inflow if no grout applies.
A common calculation used for setting water inflow trigger levels from probe holes when grouting will be required is:
PQSs = P1 x 3 or PQST = Pt x 2

PQs = Probehole trigger level for grouting measured as water inflow from one single probehole

PQT = Probehole trigger level for grouting measured as water inflow from all probe holes

P1 = Water inflow in one probehole 30 min after drilling

PT = Water inflow in all probe holes 30 min after drilling

Pre-defined grouting criteria will govern the progress of the tunnelling works. The tunnel will not be allowed to advance until these criteria have been met. Probe drilling ahead of excavation has become an integrated part of modern tunnel excavation. A minimum of 2 probe holes ahead of the face at all time will lower the risk for unforeseen water inflow as well as progressing into unforeseen faults and adverse rock mass conditions. In areas highly sensitive to groundwater fluctuations probe-drilling and pre-grouting shall be executed continuously along with the tunnel advance, e.g. such as every 15 – 25 m and with a specified overlap between each round according to project specific requirements. A pre-grouting round typically includes grout holes in the circumference with a spacing of 0.8 – 1.2 m depending on the specified LRIR. Grout holes are drilled in a specified pattern to create a trumpet shaped barrier in the rock mass. A normal pattern is 24 m long holes at an angle of 12 – 14 degrees off centre to create a 5 m barrier. Typical overlap ahead of the face of grouted rock mass would be 5 – 8 m depending on LRIR and the actual water head.

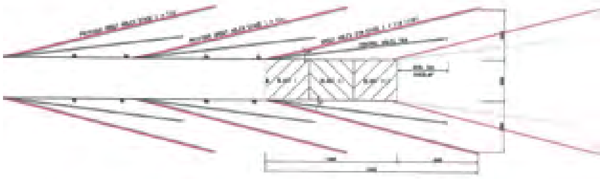

Figure 1. Typical systematically pre-grouting pattern in small cross sections

The primary purpose of pre-grouting is to establish a zone around the tunnel periphery with reduced permeability. The impervious zone ensures that the hydrostatic pressure is relocated from the tunnel periphery to outside of the pre-grouted zone. The water pressure acting on the tunnel contour and the tunnel support can be close to nil in drained tunnels. In addition, pre-grouting may have the effect of improving the stability in the grouted zone within the rock mass, a secondary effect that is an issue of concern still not fully documented. The pre-grouting technique has been particularly important for the successful construction of sub-sea tunnels with an indefinite source of water above the tunnel and thus strict focus on keeping water inflow control.

Figure 2. Photo showing pre-grouting in a small cross section tunnel adit

The length of grout holes may reach a max. of 20 to 30 m. Longer grout holes are not recommended due to lack of drilling capacity of such long holes and the grout pressure at the far end of the holes will be reduced. Drilling accuracy shall be within 5% to secure even distribution of grout materials. The pre-grouting scheme must cover the complete 360 degrees of a tunnel and include specifications for control holes and success criteria for the grouting work.

4 Grouting materials

Rock mass grouting has reached a technical level where the equipment has become quite industrialized with highly efficient, multi hole units that are capable of providing a wide range of grout pressures. A variety of grout materials are available utilizing knowledge from concrete technology to improve the material properties.

Micro Cement (MC) has become the main grouting material for urban tunnelling. MC is characterized by no bleeding at water cement ratio below 1, medium Blaine value, low D95 and fast setting, characteristics which are important for a successful grouting. Stable MC grout will penetrate well into all fissures > 0.2 mm and will not migrate deep into the rock due to its fast setting properties. Using stable MC, dual stop criteria is applied for achieved predefined 1) grout pressure or 2) grout volume per drill hole. This allows sound predictions and limits the material consumption. MC may be accelerated by introducing an alkali free accelerator to the grout line. This technique is utilized in situations with very open fissures or when running water appear on the face. Accelerated MC has to a certain extend replaced the use of PU. MC is superior in terms of performance and cycle times and should constitute the main material in lower LRIR classes.

Ordinary Portland Cement (OPC) used to be the traditional pre-grouting material for tunnels. OPC is characterized by substantial bleeding, low Blaine value, high D95 and slow setting. The limitations in OPC penetration properties may only partially be compensated by excessive drilling and a very high grout pressure. OPC may be modified by adding Silica Slurry to the mix, but this will only reduce bleeding and will not have any significant effect on penetration or setting time properties. When using OPC the "grout to refusal" principle shall be used in every hole. Even if the cost of OPC is only 20 % of the cost of MC the total cost of drilling and material consumption will be in the same range as for micro cement. Cycle time would be 20 – 50 % longer than for Micro Cement. In relaxed LRIR classes OPC would still be an option as the main grouting material.

Colloidal Silica (CS) is a valuable supplement to cement based grout in the lower LRIR classes. CS is a liquid mineral based grouting material and is different to micro silica slurry. CS contain of two components with a viscosity like water and may be accelerated to set in the range of 1 – 60 minutes, it has long term stability and durability and will penetrate all fissures with openings that may cause water inflow. SC has a cost of 300 - 400 % of MC but volumes will substantially be reduced compared to MC.

The grouting procedure shall aim at completing the grouting work in one grout round. Another key aspect of grouting is to closely focus the grouting works to a limited area surrounding the tunnel periphery in the range of 5-10m. Penetrating deep into the rock mass with the grout should be avoided. A thorough knowledge of the hydrological characteristics of the rock mass is required for the planning of a pre-grouting scheme, to choose the appropriate grout materials, grout pressure, number and length of grout holes, and grout strategy. However, an important input to the grout design will also be the experience obtained for the particular project, and monitoring results of water inflow to allow for modifications of the grouting scheme.

The principle of the grouting trumpet is described above. It is important that the trumpet covers the full circle surrounding the tunnel including grout holes in the invert. For a typical grout sequence the grout holes in the invert would be the initial holes then the holes in walls follow and finally the holes in the roof.

The use of high pressure grouting has shown to be effective in good rock mass conditions and in situations with rather impervious rock. Hydraulic fracturing can even be applied to improve the effectiveness of the grouting. The use of grouting pressures up to 100 Bars has become quite common in conjunction with OPC. By utilizing MC a more moderate grout pressure of 50 – 70 Bar above actual water head may be applied It requires that strict compliance to the stop criteria is executed throughout the work. However, in poor rock mass conditions care must be used to avoid a too high grout pressure, which could cause a lengthy and consuming grout effort, and harm to the tunnel surroundings as injecting into neighboring houses and road/railway bases in the close vicinity of the tunnel work.

5 Organisation of grouting works

With all respect to Project Specifications on grouting techniques and methods, a grouting scheme may only be successful if the organizational and contractual aspects are well prepared.

The efforts involved in the grouting scheme includes: definition of criteria for allowable inflow, establishing requirements for probing and pre-grouting, employ adequate drilling and grouting equipment, establish a competent site team of managers, engineers and tunnel crew and last but not least follow-up of residual inflow and surface monitoring. To be able to run this grouting schedule as well as optimizing the performance without sacrificing the established requirements, a significant organizational effort is needed at site, including the co-operation of the contractor and the engineer as well as project owner and authorities issuing construction permits. During this co-operation, the geological base line and related requirement for water inflow control will be outlined in LRIR for sections of the tunnel. Procedures shall be agreed and authority to adapt to the varying rock conditions need to be delegated to the dedicated grouting staff. Method Statements shall give all necessary information to the crew undertaking pre-grouting works. Flow charts and drill patterns specifications and grout mix design are other vital parts of the Method Statement.

6 Risk and Contractual issues
Water inflow and related pre-grouting works required to avoid settlements and ground water draw down in the vicinity of the tunnel is heavily influencing the project cost and schedule. Ignoring this fact and leaving these issues as contractor risk and design responsibility compensating water inflow control as Lump Sum is the ultimate way to cause program delay and post project claims and disputes.

The project owner needs to be directly involved in control of water inflow and costs associated with the efforts to achieve the specified LRIR. The only way this can be secured is by employing resources with relevant experience with Project Specifications for pre-grouting. The PS shall define inflow requirements for the different sections in the tunnel. Drilling and grouting equipment capacities and minimum performance requirements shall be defined in the PS as well as intended material to be applied.
The most efficient way for the engineer and owner to secure control with performance and cost related to pre-grouting is to obtain qualified estimates in the Bill of Quantity at the tender stage and to re-measure all grouting related activities during tunnelling excavation. A guideline for compensation units would be as follows:

- Probe drilling ahead of tunnel face – re-measured and reimbursed by drill meter
- Drilling for grouting – re-measured and reimbursed by drill meter
- Grout packers - re-measured and reimbursed by pc
- Grout materials – re-measured and reimbursed by kg for all materials
- Grouting time – re-measured and reimbursed by hours used for grouting

To compensate for actual consumption of time and materials for grouting may sound risky for many project owners. However, specifying minimum capacities on machinery and by setting minimum contractual production rates the owner has tools making him capable to control volume and cost of pre-grouting works.

7 Some examples of tunneling projects in Norway
In Norwegian tunneling pre-grouting was introduced in the early seventies to manage high water inflow in hydro power tunnels. Pregrouting superior to post grouting became the common method for reducing water inflow to a level that would not affect the tunneling progress rates. The latter being the main focus at that time. With the increased number of tunnels sub-sea and in urban areas sensitive to ground water draw down the development of the pre-grouting concept boosted. Improvements in methodology and development of improved drilling and grouting equipment along with utilization of new grouting materials was initiated by clients and contractors. New guidelines were established and contracts amended to make the risk involved in pre-grouting visible and subject to reimbursement by re measuring time and material quantities related to water inflow control.
The Norwegian concept by utilizing drained sprayed concrete lining without waterproofing measures makes pre-grouting the only mean for water inflow control. The design life of infrastructure tunnels in Norway is 100 years with the first 50 years as a maintenance free period in terms of installations. Water inflow control by pre-grouting is designed for the full lifespan.

Project	Type of tunnel	Length of tunnel	LRIR L/min/100m	Final L/min/100m
T-baneringen	Metro	1,7 km	7 – 10	< 7-10
Jong-Asker	Railway	4,5 km	4 – 16	2,5
Tåsen Tunnel	Highway	0,9 km	10	13
Svartdal Tunnel	Highway	1,5 km	5	<5
Storhaug Tunnel	Highway	1,25 km	3 – 10	1,6
Lysaker/Sandvika	Railway	5,5 km	4	<4
Eiksund Tunnel	Subsea	7,7 km	30	<30
Oslofjord Tunnel	Subsea	7,3 km	30	<30

Table 1 Limit Residual Inflow Rates (LRIR) in a selection of Norwegian Infrastructure Tunnels

All reference tunnels have permanent sprayed concrete linings and drained internal water and frost insulation linings. Inflow monitoring systems are installed and water inflow is normally decreasing as a function of time.

8 Pre grouting superior to post grouting

Pre-grouting, prevent is the preferable method to post-grouting, or cure. Post-grouting being a method applied subsequent of the tunneling excavation is often an intricate, time consuming and costly process (more than 20 times the cost of pre-grouting) and the result of post-grouting schemes may be rather uncertain and variable. It may at certain occasions and in the extreme low LRIR classes become a supplement to the pre-grouting.

The main advantages of the pre-grouting concept versus waterproofing and concrete lining are as follows:
• It provides a flexible approach.
• It utilizes the capability of the rock mass, with respect to both its self standing capacity and impervious characteristics.
• It is cost effective.
• It reduces the construction time.
• It fulfills the requirement for strict water control in urban tunneling, and also for tunneling projects focusing on the practicality of water pumping during excavation.
• It provides full documentation of all activities.

Probing and pre-grouting is an integrated part of D&B tunneling. With systematic probe drilling ahead and full access to a dry face due to the overlap, almost all possible situations in terms of stopping water from entering the tunnel may be handled in an efficient way. Efficient in terms that pre-grouting becomes a part of the normal excavation procedure being repeated at every 3-5 blast rounds, it is a repetitive procedure and the work is planned and integrated in the process advancing the tunnel.

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4.10 BLASTING TECHNIQUES

Introduction
Norway is a country with a long and proud tunnelling history. The country's geography and topography are challenging. It is long stretched and has mountainous areas and deep fjords. In the past, and in the present, these geological formations are barriers against effective trade and logistics. A way to overcome these barriers has been to build tunnels, both through the mountains and under the fjords (the latter also in the bedrock).

As early as the 16th century tunnelling technology was used in underground metal mining in Norway. However, it was first in the late 19th century the first infrastructure tunnels were developed. These were railway projects. The railway link between Oslo and Bergen with 184 tunnels, with Gravhalstunnelen 5311 m as the longest, opened for traffic in 1909 after 15 years of work. Through the 20th century hydro power tunnels overlapped the railway hegemony and were the most important technology pusher. As technology developed and tunnels relatively became less expensive for the society the use of the underground has turned more and more versatile, and the variety of civil underground use is widened. Road tunnels have been the biggest contributor to the Norwegian Tunnel Statistics the last 30 years, however other uses are also important: Storage caverns, public halls, metro, sewage etc. Tunnelling under the deep fjords emerged during this period. Not to forget is the unique knowledge the Norwegian tunnelling industry has in under water tunnel piercing. Mostly hydro power related (water transferring tunnels) and landing of oil and gas from the offshore drilling platforms.

Looking at the drilling and blasting techniques, methods and equipment in the Norwegian tunnelling industry, these are relatively united comparing all the contractors working here. The article presents a compilation of general blasting techniques.

Drillplan design
The most common drillhole diameter in the Norwegian tunnelling industry today is 48-51 mm. 64 mm is used in special situations.

There are a lot of different cut designs used in the Norwegian tunnelling industry, depending on factors like geology and drilling machinery, but also the individual blaster's experience plays an important role. It's said that each blaster has its own the cut design. Anyway, it is a variation of the parallel cut, which is dominating more or less 100 %. Fan cuts and V-cuts were common before, but as the blast lengths increased and longer drilling booms and rods were used, these methods became ineffective.

Normally the blast holes closest to the reaming holes (uncharged holes) are placed in a distance 1.5- 2.0 times the reaming holes' diameter (between 15 – 25 cm). This is geologically dependant. The reaming holes are often 102 mm.

The standard drilling length is 5.3 m. In the larger road and railway tunnels also 6.2 m rods are used when

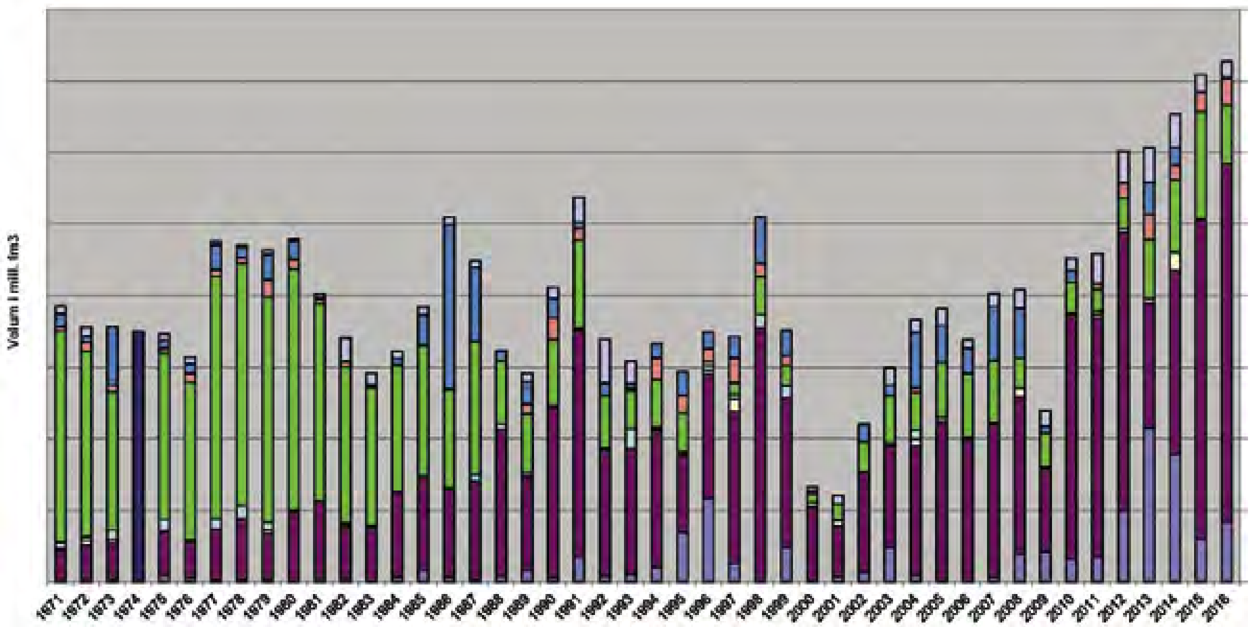
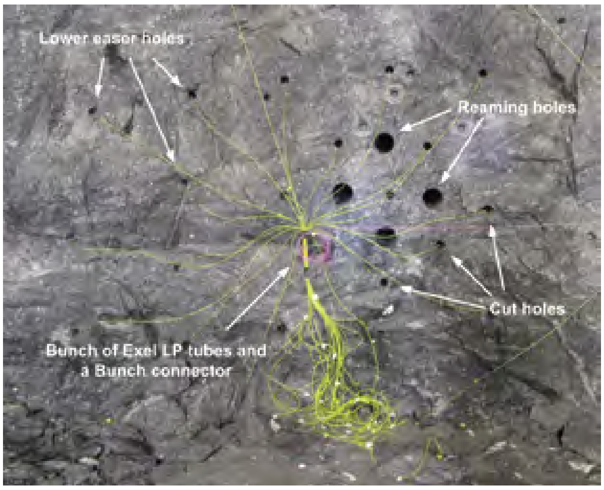


Figure 1 Norwegian tunnel statistics 2016 (NFF, www.tunnel.no).

the geology accepts this. If geology is poor the drilled length must be reduced to get proper pull-out. Tests with blast rounds up to 12 m has been tested, however not successful regarding productivity (ref NTNU, Rønn).



Picture 1 Parallell hole cut with four reaming holes, 102 mm, and in total 9 cut holes. Exel LP (yellow tubes) assembled and hooked up with an Bunch connector (pink tube).

The design of the full blast is normally based on experience and empirical burden and spacing parameters. Every project has its own optimal drill plan, but some standard numbers are used to estimate and plan the first blasts. See table 1.

Type of hole		Burden, V	Spacing, E
Contour	Good blastability	0.8 - 1.0 m	0.7 - 1.0 m
	Poor blastability	0.7 - 0.9 m	0.6 - 0.9 m
Row nearest contour			
	Good blastability	1.0 m	1.1 m
	Poor blastability	0.9 m	1.0 m
Invert hole			
	Good blastability	1.0 m	1.0 m
	Poor blastability	0.8 m	0.8 m
Easer	Good blastability	$F_s = 1.8\ m^2$	
	Poor blastability	$F_s = 1.3\ m^2$	

Table 1 Typical values for burden, spacing and drilling pattern for 48 mm blast holes.

Most drilling rigs in the Norwegian market can be drilled automated with digital drill plans and screens in the operator cabin. One or two operators can handle drilling rigs with 3 or 4 booms. Three booms are most common.

The hole distance in the contour and the inner contour are often set by the builder in the tender descriptions 60 - 90 cm. To get as smooth blasted wall and roof as possible the offset and eccentricity angles are kept at the practical minimum needed for the space of the boom

Picture 2 Drilling rig with 3 drilling booms and one basket. Atlas Copco (www.atlascopco.com)

and drill hammer. Normally the offset is about 10 to 15 cm and the eccentricity at the bottom of the holes should not exceed 30 cm to 40 cm. Increased offset makes possible less eccentricity. These numbers are geology dependant, and substantial drill hole deflection will influence these empirical values.

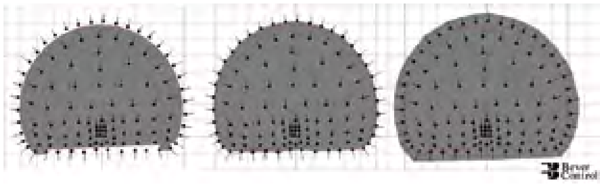


Figure 2 Typical BeverControl drill plan and drill report. Lofast tunnelling project. Left: Theoretical blasting cross section (63.12 m2).Middle: Collaring cross section (68.43 m2). Right: Cross section at the bottom of the holes due to eccentricity (85.03 m2).

Explosives

The use of cartridge explosives and ANFO is hardly used in the Norwegian tunnelling and underground industry anymore. Since mid 1990'ies bulk emulsion explosives are applied, which also permits mechanical loading. Cartridge explosives or readily made, or bagged ANFO have one common property, they are classified as explosive goods, class 1.1 D. They must therefore be stored, transported, and treated according to laws and regulations in force. Bulk emulsion explosives are not classified as explosives, but classified according to ADR as oxidizing agents, class 5.1 UN, or correspondingly class as for pure ammonium nitrate.

The use of the SSE system (Site Sensitized Emulsion) means that equipment and raw materials may be transported and stored at the construction site with no concern about explosives regulations. This implies not only a large simplification of the logistics, but also a rational handling, which totally results in great savings regarding costs of transportation and storage.

Environmental issues and safety properties have become a significant issue in blasting works, and this often determine the choice of explosive type. In conventional tunnelling it is important that the toxic component blast fume level is kept as low as possible. It is the concentrations of NO2 and CO which represents the greatest health hazard, though production of respirable dust is an additional health risk from the blasting process.

Experiences from monitoring Norwegian construction sites show that the concentrations of the mentioned gases have been dramatically reduced when using emulsion explosives compared to ANFO and cartridge dynamites (Figure 2). The bulk emulsion explosives are more efficient during the charging process. The time for ventilation break after blasting is considerably reduced with use of emulsion explosives in tunnels.

Emulsion explosives are water resistant, unlike ANFO, and all holes can therefore be charged with the same product, dry or water filled. Cartridge explosives are only used under very special condition together with emulsion, e.g. running water. High productivity is obtained at all times.

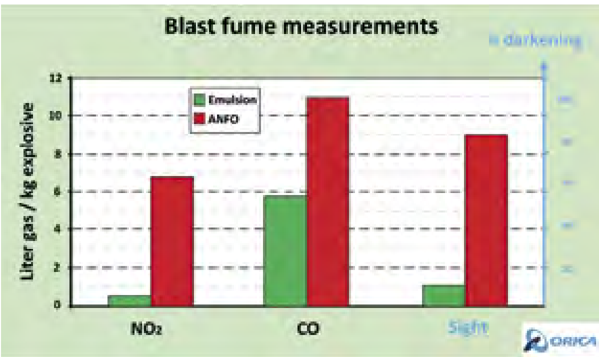


Figure 3 Blast fume measurements in Swedish test tunnel. (Petterson 1995).

Charging Method
In the Norwegian tunnelling industry a small emulsion charging unit (Handiloader) is used to serve the tunnelling faces with explosives. The emulsion system (SSE-system) consists of tanks for the emulsion matrix and the chemical sensitization compound. The Handiloader is normally mounted on the platform of a small lorry provided by the contractor. The power supply (electricity and hydraulics) is provided by the drilling rig. The tank capacity of the Handiloader varies from about 1000 to 1800 litres, which is enough for most tunnelling blasts.

The SSE-system gives high charging capacity and is normally used for all holes in the face. Compared to blasts with ANFO and manual cartridge charge in the contour the productivity difference is clear. Emulsion charging is 40 % faster, and less people is needed at face (Zare 2007). The SSE-system is also flexible regarding charge amount, as the charge strength and length can be adjusted easily. Correspondingly the charge strength can be changed by the amount of the gassing agent. Charge length can be set by the operator for every single hole.

Picture 3 Handiloader mounted on the platform of a lorry. El- and hydraulic power hoses connected to the drillrig.

Initiation system
In tunnel blasting it is essential that the delay time between the intervals is long enough to avoid "stack-up" and time confinement. In standard tunnelling in Norway non-electric detonators with long period delays are used (e.g. Exel LP series) to get the correct timing. The delay timing affect the round pull-out, muck pile heave, vibrations, back-break and contour quality amongst other things. Non-electric systems are fast to connect.

To get a good blasting result it is of outmost importance that the cut breaks well. Drilling accuracy and timing is crucial. The shortest interval time designed in parallel hole cuts is normally 100 ms (50 ms is available). This is sufficient to get rock displacement and avviod stack-up. Normally the parallel cut holes are finished in 600 – 800 ms. Further the lower easer holes are initiated, and successively the upper easer holes are initiated. The inner contour comes before the contour at the end. The floor holes may be blasted successively with the lower easer holes, or as the last interval. The latter is done to lift the muckpile for better loading conditions. Maximum round delay time is usually 6000 ms to 9000 ms. Due to variation in the pyrotechnic elements, the detonators with the longest delays (>2000 ms) has increments of 400 or 500 ms to get 100 % certainty of correct detonation sequence.

The non-electric tubes are assembled in bunches and they are normally initiated by a 5 g/m detonating cord. The ends of the detonating cord are taped together so that it creates a full circle. This gives double security for initiation of the tubes.

Picture 4 Tie in sequences. Exel tube bunches (upto 20 tubes) hooked up with detonating cord around the face.

Damage zone control and over break

In the late 1990'ies national administrative regulations prohibited the use of detonating cord (>40 g/m) in underground blasting. This was decided due to many incidents of undetonated cord in the muckpile and several uncontrolled detonations of the cord during loading and crushing of the muckpile.

The SSE system, introduced in tunnels in 1995, also brought a mechanical retracting unit. This unit made it possible to leave a string of explosives in the hole and charge the contour hole with the blast-technically correct energy concentration.

To be able to meet the demands for smooth blasting and the cracking depth the contour charge is normally reduced by 75 % and the inner contour charge by 50 %. To ensure proper detonation the string charge should not be lower than 350 g/m. Cut, easer and floor holes are normally charged 100 % with emulsions.

The retractor unit is calibrated together with the Handiloader and it retracts the hose according to the amount of explosives set to be loaded in the hole. Different retracting speeds give less or more explosives in the hole.

The SSE needs a primer to detonate. Normally a 25 g booster is used. This product fits the hose inner diameter and it is put in the hose before charging of each hole. When the operator pushes the remote control fixed to his arm, the emulsion pump starts. The holes will then be charged through a preset charge amount. The contour holes has a preset delay time of the retractor unit makes the system leave a bottom charge that fills the hole and surrounds the primer which is pushed out of the hose by the emulsion, and ensures a proper initiation of the emulsion string.

The quality of explosive strings in the contour and the inner contour holes are to some extent influenced by the

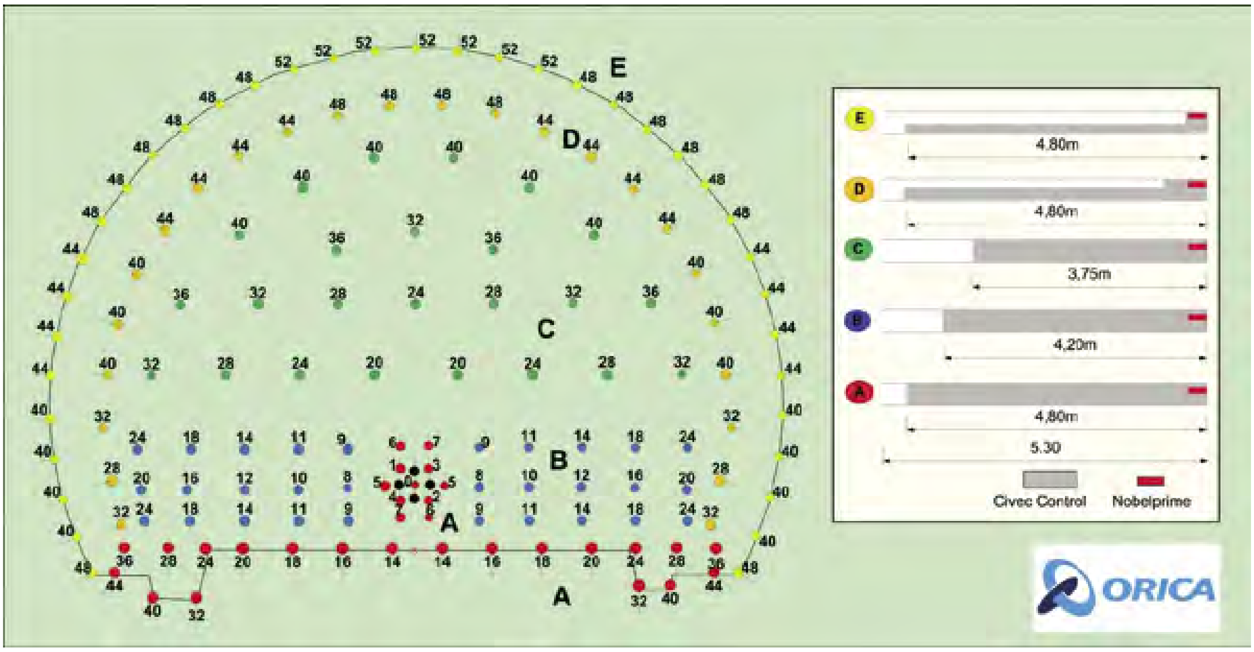


Figure 4 Typical blast design (T9.5 highway tunnel) with parallel large hole cut and string loading in the contour and the inner contour. The colouring of the holes represents different charging. The numbers show the Exel LP intervals. The blast holes are out of scale.

operator skills and the geology and water conditions. The operator may overrule the settings of the Handiloader, and incorrect amount of explosives may be charged. In unfavorable geology the string is extra vulnerable for cut-offs from neighboring holes or flowing water.

Vibration constrains

Urban tunnelling has been very relevant in Norway the latest three decades, and is more and more challenging due to closer distances to residents, infra structure and other underground constructions. Environmental issues has become more and more prominent with vibration constrains as the most important.

Different methods are used to reduce the vibrations. First hybrid-initiation of the tunnel face is performed to get one-hole initiation. (This is theoretical but it gives good results). Surface connectors are used to divide the cross section in several areas, and holes with the same numbering are getting different timing due to the face delays.

If a full blast hole length give too high simultaneous explosive charge the round length has to be reduced. Not less than 2 m. The final vibration reduction step is dividing the cross section in several blasts.

Alternatively, but not very common in the Norwegian tunnelling industry it is possible to use string loading for all holes in the blast. Tighter drill pattern is then required and consequentially longer drilling time is experienced. However, the total productivity may be increased due to longer rounds and less unproductive time (rigging and evacuation time).

Also the use of electronic detonators is an alternative. In Sweden, our neighboring country, the use of electronics is used excessively to reduce and control the vibration influence generated by blasting. The public projects owners in Sweden are the drivers of this trend. The excavation of larger and larger cross sections with 200 to 300 holes gives the electronics an advantage as the time intervals are "unlimited" with precise time delays of 1 ms intervals up to 20000 ms and maximum 800 dets per blast. Blasting with non-electric detonators in these large cross sections, the face has to be divided in several blasts to meet the vibration constrains. The authors think that this will be the situation in the Norwegian tunnelling industry as well within a short period of time.

Tunnel project implementation

The Norwegian Public Road Administration (NPRA) and the Norwegian National Rail Administration (JBV) are the two biggest builder organizations in Norway. Both use the NRA's Handbook 025 and the Norwegian Standard NS 3420, as guidelines for tunnelling management and tender descriptions. For vibration limits and calculations NS 8141 is used.

Process Code 32, Tunnel Blasting in Handbook 025 describes in detail some of the requirements for the contractor. Extracts from the process code text are as follows (unofficial translation):

- General contour: Hole distances in contour cc 0.7m and distance inner contour (burden) max cc 0.9 m. The maximum effect of a contour hole charge shall not exceed 3.0 GW/m.
- Alternative contour: Hole distances in contour cc 0.5m and distance inner contour (burden) max cc 0.7 m. The maximum effect of a contour hole charge shall not exceed 2.2 GW/m.
- Bottom charge of contour and inner contour shall not exceed respectively 200 g and 400 g (dynamite equivalents).

NS 3420 gives amongst other things functional requirements for the contour. Contour classes are described as follows:

Class 0	No rock allowed inside the planned contour line.
Class 1	Some underbreak allowed at maximum 0.15 m inside the planned contour line.
Class 2	Some underbreak allowed at maximum 0.5 m inside the planned contour line.
Class 3	No contour requirements.

Within the requirements described the contractor is usually responsible for the drilling, charging and initiation plan. The contractor is normally instructed to report every blast to the builder administration before every blast. This may be electronically or by paper. To control the blasting result, the contractor has to document a scanned profile. This is done by scanners mounted on the drilling rig, often simultaneously with bolting, or "manually" by a surveyor employed by the contractor. This is performed after scaling, and before applying shotcreet. The digital information may be implemented in digital 3D models and used as documentation of tunnel geometry and also for further time planning.

Profile scanning of blasted surface with Bever 3D Win Profiler. Scanning time is typical 5 minutes for a 6 meters round. The data is recorded for documentation to the client and saved as “as built” tunnel geometry.

Comparing the Bever Control drilling report, shown earlier, and the scanned profile a measure of the blast induced over break can be made. This is important when looking at different charging methods to improve the tunnelling contour.

In some project contracts the builder is entitled to do rock surface inspections for half an hour, before the shotcreet is applied. Based on this documentation the permanent ground support is decided.

Research and development

Since the first tunnelling projects in the late 19th century there has been a continuous development of equipment and methods in the Norwegian tunnelling industry. The Norwegians shall not take credit for all developments made the last century, but they have been early users of new technology and been participants in many development projects. Some of the main development steps of the blasting technology in the Norwegian tunnelling history are:

- Electric dets (Mid 1950'ies). HU-type detonators from 1970.
  Safer initiation, bigger rounds, improved blasting technique and increased productivity than use of black powder cord initiation.
- ANFO bulk explosives (Mid 1970'ies)
  Mechanized loading gives reduced charging time and higher productivity than use of manually loading with cartridge explosives.
- Nonelectric dets (Mid 1980'ies)
  Safer and better working environment than electric detonators as electronic machinery can be used at face. Longer rounds and improved blasting technique with even more delay intervals. Better vibration control. Increased productivity.
- Automated drilling (Late 1980'ies)
  Drilling time reduces and less crew at the face gives higher productivity.
- Emulsion bulk (Early 1990'ies)
  Faster charging, easier logistics and better working environment due to less toxic gasses.
- String loading (Mid 199'ies) Contour holes are mechanically charged and gives improved contour quality control and overall higher productivity
- Electronic tunnel detonators (2010)
  Increased vibration control, better contour control.

"Need for speed" has been and "fast advance" have been a motto through the history. The latest years we may see that this focus has been a little too important, and that the quality of the final tunnel is not the best. The pendulum is about to turn as increased focus on contour quality and longer tunnel life is more prominent in the eyes of the builder organizations.

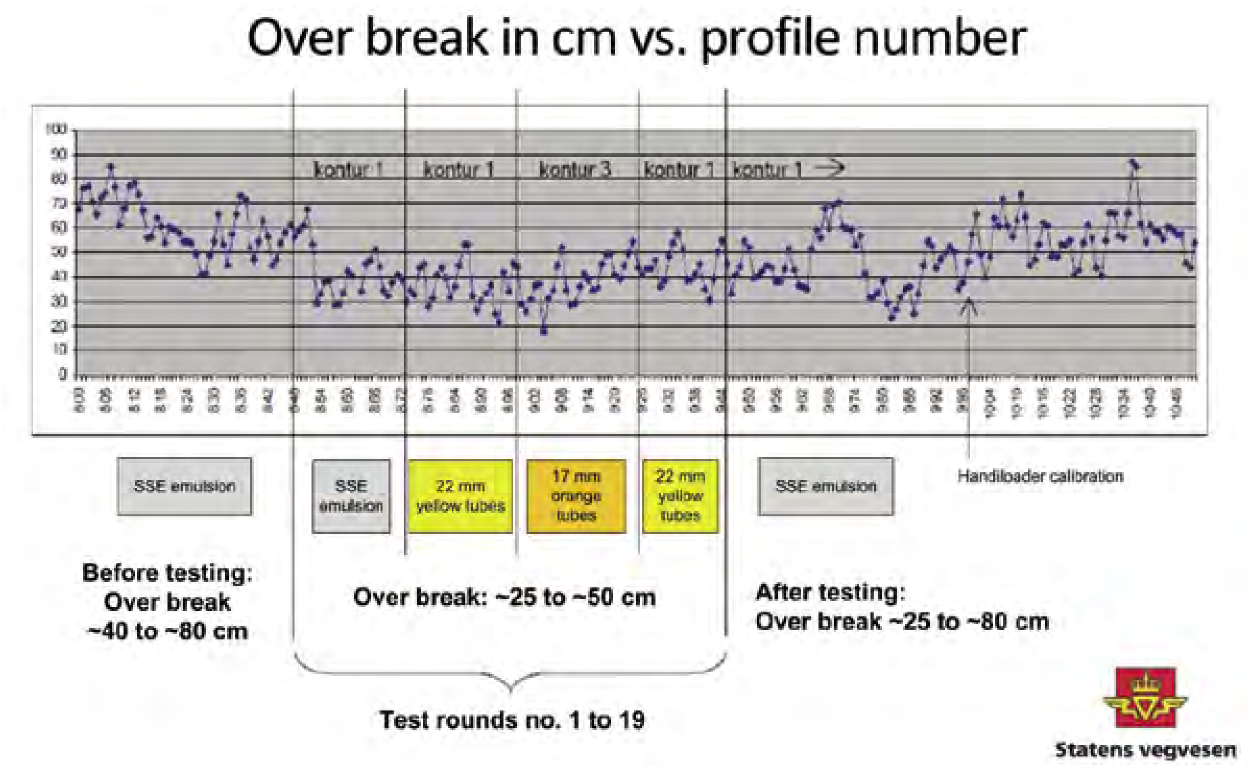


Figure 5 Results from tunnel contour testing in the Eikrem tunnel. (NPRA 2011).

The NPRA has included in situ testing in some tunnelling project contracts the last decade to get more information about this trend. The most important result regarding contour quality and over break control is the drilling accuracy. If the drilling is poor, neither explosives choice, charging method nor the initiation system will compensate for this.

A short summary of the results shown in figure 3 shows that the over break was reduced in the testing period, compared to normal routines before testing. The tests show no significant difference between emulsion or cartridge explosives, or different drilling plan ("Kontur 1" and "Kontur 3). The main conclusion was that the operators in the testing period was extra accurate when drilling the contour holes and during the charging of the emulsion string. Improvement after testing compared to before shows the positive learning effect of the testing amongst the workers.

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05. SURFACE PROBLEMS – UNDERGROUND SOLUTIONS; TUNNELS AND UNDERGROUND SPACE FOR THE SOCIETY

5.1 TUNNELS FOR HYDROPOWER PROJECTS

- lessons learned in home country and from projects worldwide.

1. INTRODUCTION

It is natural to start this paper by giving a brief description of the development of the hydropower industry in Norway, and in particular concentrate on the underground aspects. This is presented in Chapter 2. One special lesson learned from the Norwegian hydropower projects is that it is possible to replace the standard ventilated surge chamber by an unlined rock cavern operating as an air cushion. This is a technology that can be used for storage of compressed gas and will be discussed in Chapter 3. Having been involved in different ways on hydropower projects in many countries around the world, the author also includes some lessons learned from some selected projects. In Chapter 4 some samples of problems caused by special types of rock masses and stress conditions in water conveying tunnels are discussed.

2. THE DEVELOPMENT OF UNDERGROUND HYDROPOWER PROJECTS IN NORWAY

Topographical and geological conditions in Norway are favourable for the development of hydroelectric energy. The rocks are of Pre¬cambrian and Paleozoic age, and although there is a wide variety of rocks, but highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rocks.

More than 99% of a total annual production of 125 TWh of electric energy in Norway is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant, (Broch, 1982). In fact, of the world's 600-700 underground powerhouses, one third, i.e. 200, are located in Norway. Another proof that the Norwegian electricity industry is an "under¬ground industry" is that it today has 4000 km of tunnels. As the dotted line in Figure 1 shows, during the period 1960 - 90 an average of 100 km of tunnels was excavated every year.

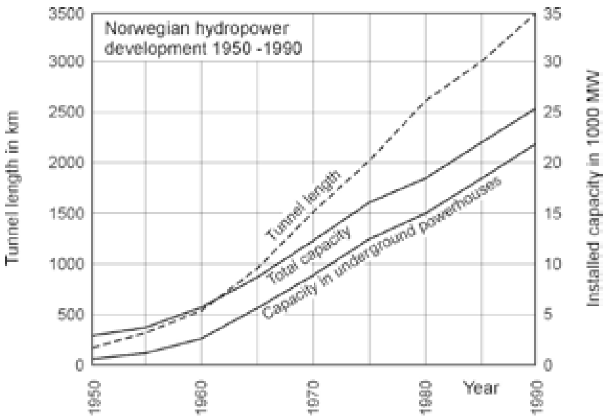

Figure 1. The development of Norwegian hydroelectric power production capacity and the accumulated length of tunnels excavated for the period 1950 -1990.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience was gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today, (Edvardsson & Broch, 2002). Example of an underground powerhouse from the early 1950s is shown in Figure 2. In this case a concrete building has been constructed inside a rock cavern. The powerhouse has in fact false windows to give people a feeling of being above ground rather than underground.

Figure 2. The Aura underground hydropower station, commissioned in 1952

Later people became more confident in working and staying underground, and powerhouses were constructed with exposed rock walls, often illuminated to show the beauty of rock such as demonstrated by two powerhouses commissioned around 1970 and shown in Figure 3.

Figure 3. Åna-Sira (left) and Tafjord K-5 (right) underground powerhouses

Some special techniques and design concepts have over the years been developed by the hydropower industry. One such Norwegian speciality is the unlined, high-pressure tunnels and shafts, (Broch, 1982B, 2000). Another is the so-called air cushion surge chamber

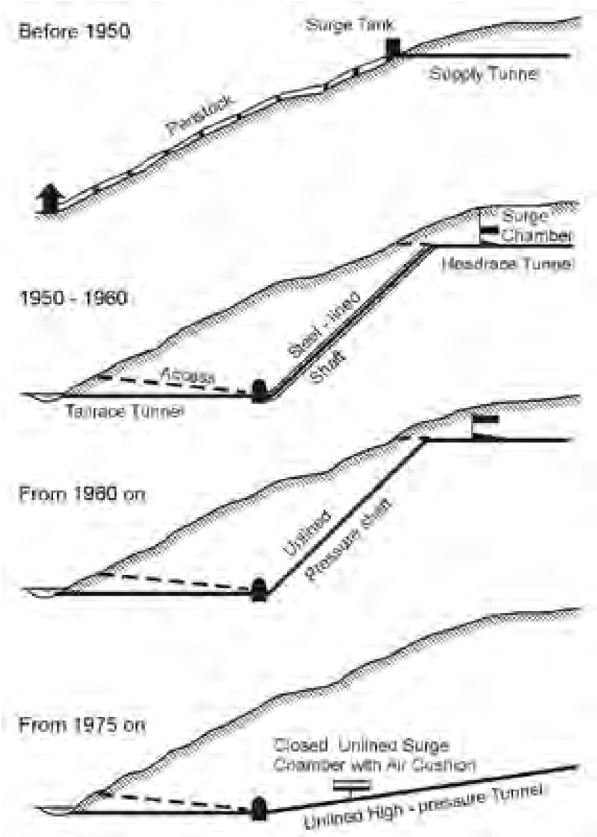

Figure 4. The development of the general layout of hydroelectric plants in Norway

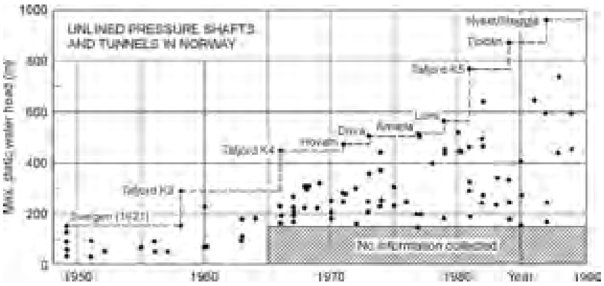

Figure 5. The development of unlined pressure shafts and tunnels in Norway.

which replaces the conventional vented surge chamber, (Goodall et al., 1988)

Most of the Norwegian hydropower tunnels have only 2 - 4% concrete or shotcrete lining. Only in a few cases has it been necessary to increase this, and in these few cases only a maximum of 40 - 60% of the tunnels have been lined. The low percentage of lining is due not only to favourable tunnelling conditions. It is first and foremost the consequence of a support philosophy which accepts some falling rocks during the operation period of a water tunnel. A reason¬able number of rock fragments spread out along the headrace or tailrace tunnel floor will not disturb the operation of the hydro power station as long as a rock trap is located at the downstream end of the headrace tunnel. Serious collapses or local blockages of the tunnel must, of course, be prevented by the use of heavy support or concrete lining where needed. Normal water velocity in the tunnels is approximately 1 m/sec.

During and after the Second World War, the underground solution was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent reduction in costs, underground location very soon came to be the most economic solution, see Figure 4. This gives the planner a freedom of layout quite independent of the surface topography. Except for small and mini-hydropower stations, underground location of the powerhouse is now chosen in Norway whenever sufficient rock cover is available.

When the hydropower industry for safety reasons went underground in the early 1950's, they brought the steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. In 1958 at the Tafjord K3 hydropower station a completely unlined shaft with a maximum water head of 286 m was successfully put into operation. This gave the industry confidence in this time and money saving solution. As Figure 4 shows, new unlined shafts were constructed in the early 1960's and since 1965 unlined pressure shafts and tunnels have been the conventional Norwegian solution. Today almost 100 unlined high-pressure shafts or tunnels with water heads above 150 m are success¬fully operating in Norway, the highest head now being more than 1000 m. Figure 5 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

3. AIR CUSHIONS

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant, (Rathe, 1975), (Goodall et al., 1988). The bottom sketch in Figure 4 shows how this new design philosophy influenced the general layout of a hydropower project. The steeply inclined pressure shaft, normally at 45o, is replaced by a slightly inclined tunnel, 1:10 - 1:15 grade. Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. ¬This compressed air acts as a cushion to reduce the water hammer effect on the hydrau¬lic machinery and the waterways, and also ensures the stability of the hydraulic system. Ten air cushions are now in operation in Norway, and compressed air with pressure up to 83 bars, equalling a water head of 830 m, have been successfully stored in unlined rock caverns. These air cushions may also be regarded as full scale test chambers for storage of gas in unlined rock caverns.

The first containment principle for storing of air or gas in unlined rock caverns is that any internal storage pressure must be sustained by the minimum in-situ rock stress (σ3) to avoid hydraulic splitting.

Secondly, the ground water pressure and the gradient of the water seepage towards the caverns provide the containment. The rock material itself has in most cases an insignificant permeability. Hydrodynamic control by the groundwater is the main principle of containment for storage of air in unlined rock caverns. In some cases, the hydrostatic head from the natural ground water may be sufficient. In other cases, one ‘assists’ the natural ground water by infiltrating water into the rock mass around and above the caverns, by ‘water curtains’. These are established by drill holes from the surface or designated infiltration galleries. Normally, the requirement to the hydrostatic head will be the dimensioning factor for the cavern elevation. The Norwegian Explosives and Fire Safety Authority requires a safety margin of a minimum 20m water head above the water head corresponding to the storage pressure.

Thirdly, if the rock mass is more permeable than desirable, grouting is performed to ensure safe operation (permeability control). This reduces the overall inflow of water into the storage volume, reduces pumping costs, and ensures a high gradient close to the cavern contour, increasing safety against air leaking out of the cavern. As a rule, the grouting needs to be performed as pre-excavation grouting of the rock mass. Post-excavation grouting should only be allowed as a supplement after pre-grouting; it is not a substitute for pre-grouting.

Table 1 shows the main data for ten air cushion surge chambers built in Norway, (Kjørholt et al, 1992). Remarkably, the caverns range in size from 2,000m3 to 120,000m3 and have operating pressures of 1.9MPa to 7.8MPa, serving the need for surge dampening for power-plants with installations from 35MW to 1240MW. Note the increasing trend to greater ratios between water head and rock cover over the years, indicating the increasing confidence.

Project	Commis-sioned	Main rock type	Excavated volume, m3	Cross section, m2	Storage pressure, MPa	Head/ cover *)	Experience
Driva	1973	Banded gneiss	6,600	111	4.2	0.5	No leakage
Jukla	1974	Granitic gneiss	6,200	129	2.4	0.7	No leakage
Oksla	1980	Granitic gneiss	18,100	235	4.4	1.0	<5Nm3/h
Sima	1980	Granitic gneiss	10,500	173	4.8	1.1	<2Nm3/h
Osa	1981	Gneissic granite	12,000	176	1.9	1.3	Extensive grouting
Kvilldal	1981	Migmatitic gneiss	120,000	260-370	4.1	0.8	Water infiltr. necessary
Tafjord	1981	Banded gneiss	2,000	130	7.8	1.8	Water infiltr. necessary
Brattset	1982	Phyllite	9,000	89	2.5	1.6	11Nm3/h
Ulset	1985	Mica gneiss	4,800	92	2.8	1.1	No leakage
Torpa	1989	Meta siltstone	14,000	95	4.4	2.0	Water infiltr. necessary

Table 1: Overview of main data for compressed air storage, including air cushion surge chambers
**) Ratio between maximum air cushion pressure expressed as head of water and minimum rock cover*

As shown in Figure 4 typically air cushion surge chambers are located adjacent to the headrace tunnel within a limited distance from the turbines. However, in some cases distances exceeding 1000m have been acceptable. This provides a large flexibility in the location of the chamber in the best available rock mass. The chambers are in most cases designed as a single cavern, but in two cases (Kvilldal and Torpa) they have been given a doughnut shape. All chambers are unlined with a minimum support of rock bolts and sprayed concrete, as minor rock falls during operation are accepted.

The air loss from the chambers may be due to both air dissolution into the water bed below the air cushion (annual losses for typical caverns are 3-10% of the air volume), and leakage through the rock mass. Three chambers have no leakage at all through the rock mass and six chambers have acceptable losses (within reasonable compressor capacity). Three chambers (Osa, Kvilldal and Tafjord) showed natural leakage rates that were too high for economical operation. This necessitated remedial measures. For Osa, extensive post-grouting reduced the leakage to an acceptable level. For Kvilldal, where the leakage probably was resulting from a near by weakness zone, a water infiltration curtain was established that totally eliminated the leakage. For Tafjord, it appears that hydraulic splitting took place during the first filling due to unusually low minor principal stress conditions considering the rock cover. Repair attempts by sealing of the split joint failed. The plant was operated for some years in 'tandem' with another plant without its own surge chamber. In 1990-1991, a water curtain was installed, and the air leakage disappeared when the curtain was put in operation with pressure 0.3MPa above the air cushion pressure.

For the tenth chamber, at Torpa, a water curtain was included in the original design, and installed from a designated gallery above the chamber as shown in Figure 6. During construction the rock mass around the doughnut shaped chamber was pre-grouted. Extensive rock stress measurements were performed with a variation of results; some indicated the minimum rock stress to be as low as the storage pressure at 4.4MPa (Kjølberg, 1989). Without the infiltration running, the leakage rate was 400Nm3/h; with the curtain in operation at 0.2MPa above the air pressure, there is no measurable leakage.

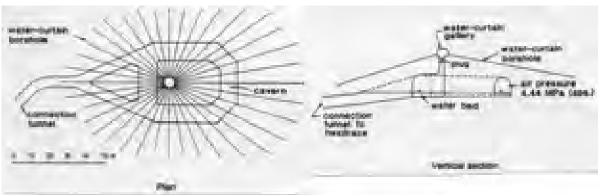

Figure 6. Air cushion surge chamber with water infiltration curtain at Torpa (Kjølberg, 1989)

The experience from designing and operating unlined air cushions confirms the following (Blindheim et al., 2004):

- Thorough geotechnical investigations to obtain relevant information about the hydro-dynamical and rock mechanical conditions are required.
- The rock cover must provide sufficient rock stress to avoid hydraulic splitting of the rock masses by the air pressure.
- The groundwater level should be maintained during construction with the use of water infiltration curtains, unless location is possible in very favourable rock mass and a high groundwater level.
- Water infiltration is an effective means for maintaining the ground water level, and thus the confining effect of the hydrostatic head (hydrodynamic control). Used in combination with pre-grouting, excessive water consumption can be avoided. Water infiltration curtains have successfully been installed in areas of groundwater drawdown.
- Systematic pre-grouting is necessary if strict requirements to tightness need to be satisfied (permeability control). High pressure pre-grouting of the rock mass with micro- or ultrafine cements minimising the remaining water inflow, ensures tightness by controlling the gradient close to the contour, and provides operational safety and economy. Grouting of concrete plugs needs special attention.

4. LESSONS LEARNED FROM WATER TUNNELS AROUND THE WORLD

Tunnels designed and constructed for carrying water are special in the way that during the construction period they are full of air, often dry air with high velocity because of the ventilation system, while in operation mode they will be filled with flowing water. Dry rocks are normally stronger than wet rocks, and some rocks may even contain minerals that start swelling and expanding when exposed to water. Also gouge material in faults and weakness zones intersected by tunnels often contains swelling minerals. In some tunnels and shafts for hydropower projects the stresses in the periphery of the tunnel may vary with changing water head in the tunnel. Thus there are several geological/topographical factors that need special attention for tunnels designed to convey water. In the following subchapters some selected cases from the author`s involvement in projects around the world will be discussed.

4.1 Tunnelling in "crazing" basaltic rocks.

The 45 km long headrace tunnel for the Muela hydropower station in Lesotho, also referred to as the Transfer tunnel, goes through basalts for its entire length, Broch, (1996). This basalt is of Jurassic age and overlies the Clarens sandstone. It dominates the highlands of Lesotho. In the tunnel area the rock is in general hard and strong with a uniaxial compressive strength of between 85 and 190 MPa. The entire length of the Transfer Tunnel was very successfully excavated with 5 m diameter TBMs. Record breaking advancements rates were obtained.

Initially, some 91 % of this tunnel was expected to fall into a rock support class requiring no more than spot bolting. It was also impressive to see the quality of the finished TBM tunnel shortly after it had been bored. Rock falls were only observed in a few areas of very high rock overburden where the rock was clearly overstressed. These areas were supported with rock bolts and wire mesh. The tunnel was in general very dry, in fact over long stretches it was dust dry.

However, as time went by some cracking and "sloughing" of the rocks was observed in the few wet places along the tunnel. This was also typically observed along the invert where water from the boring process flows constantly. A phenomenon known as "crazing" was observed. This is a form of rock deterioration or weathering which occurs in highly amygdaloidal basalts. Studies revealed that this is caused by the reaction of two mineral types occurring in the highly amygdaloidal basalt. When in contact with water, smectite minerals in the characteristic amygdales or matrix of the basalt swell causing the rock face to disintergrate, see Figure 7.

Figure 7. "Crazing" due to weathering in amygdaloidal basalts in the Transfer tunnel in Lesotho.

In addition, active zeolites, in particular laumontite, caused fine fracturing in weak, highly amygdaloidal basalt. Both these conditions caused deterioration, ranging from very minor weathering of soft minerals to the sloughing of large slabs or weakened rock. Degradation was, however, not wholly confined to highly amygdaloidal basalts, although it was in this type of rock that almost all the areas of the more severe weathering occurred.

Having identified the nature of the problem, many solutions were considered. One immediate suggestion was the application of a protective skin of shotcrete. It was surprising to learn that the relative cost of this obvious solution was higher than the conventional in-situ concrete lining. There were also some concerns about the long term durability of the shotcrete in this high pressure water tunnel.

A comprehensive system for the evaluation of the quality of the rocks along the tunnel was made. The prime factor was a weatherability classification which was developed locally. The intention was to identify the places where concrete lining was needed. The major problem turned out to be that at any place along the alignment, the cross section of the tunnel was intersected by at least two horizontal basalt flows. Even though one or two of the flows were of good quality, very often a basalt flow of poor quality intersected the tunnel, and thus concrete lining was needed for this.

It is also an economical fact that the concrete lining procedure cannot be stopped without costs. In fact a 300 m long section of the tunnel was the minimum length of good rock which was needed to stop lining. Thus the task of the tunnel geologist was not any longer to identify the poor rock that needed support, but to find 300 m or longer sections where they could guarantee the long term stability of the rock. The final conclusion was that the whole 45 km long Transfer Tunnel needed lining.

4.2 Tunnelling in friable sandstone.

4.2.1 CASE 1 – Guavio hydropower project

On November 7, 1983 the excavation from the downstream adit of the 5.25 km long, 65 m2 tailrace tunnel for the Guavio Hydropower Project (8 x 200 MW) in Colombia, had reached Station K4+ 567 as shown in Figure 8 , Broch, (1996).

The lower part of the tailrace tunnel goes through the so-called Une-formation, which belongs to the Cretaceous era, i.e. approximately 100 million years old, - see Figure 8. The formation is dominated by aeolian sandstones, some of them with a rather high porosity and low diagenesis, which means that they are poorly cemented and have a uniaxial compressive strength as low as in the order of 10-20 MPa. During the drilling of a probe hole from the centre of the tunnel face, water under high pressure was struck at a depth of 25 m. The leakage increased rapidly to 7 l/sec (420 l/min), and fine sand started coming out of the borehole. During the following day two slides occurred. From these two areas up to 40 l/sec and 70 l/sec of water was pouring out for a couple of hours. After one day a total 350 m3 of sand

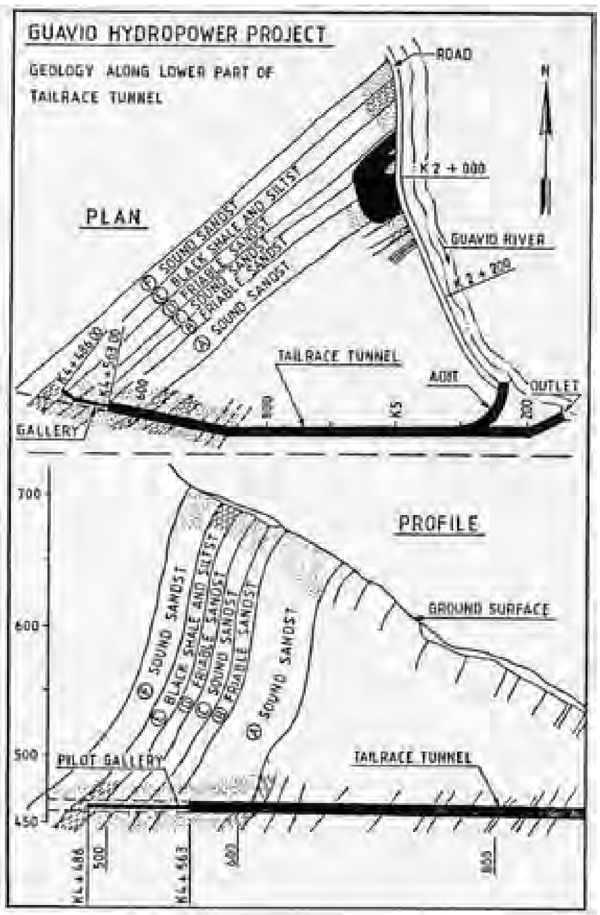

Figure 8. Geology along the lower part of the Guavio tailrace tunnel.

had been flushed into the tunnel, and the tunnel face had moved 4 m from Station 4+567 to Station K4+563. It was then decided to block the tunnel face with concrete.

During the following months several attempts were made to reduce the ground water level above the tunnel as this is normally the most effective way of solving stability problems in friable sandstones. The result was a number of small inflows of sand. By early February a total of 5000 m3 of sand had flowed into the tunnel, and it was obvious from all attempts that it was impossible to reduce the pore water pressure below 20 bars, i.e. 200 m water head. (The annual rainfall in this part of the Colombian jungle is as high as 4 - 5000 mm). It was therefore decided that the rock mass ahead of the tunnel face should be stabilised with a grouting procedure, and that a 3.5 m diameter pilot tunnel should be driven through the unstable zone. This was very complicated and time consuming work. In spite of all precautions several slides or inflows of sand occurred, so when the whole 77 m long pilot tunnel after 15 months was finished, a total of 15000 m3 of sand had flowed into the tunnel.

Typical for these water and sand inflows was that they started as small water leakages followed by the inflow of sand which eroded the drill hole and thus increased the capacity of the hole, which again allowed more water and sand to flow into the tunnel. It was also commonly observed that the inflows had a pulsating character. After strong inflows which could last for some hours, the inflow decreased for some time and then increased again. The most serious water inflow was as high as 400 l/sec. To cope with this, the tunnel face had to be blocked with a bulk head, and the rock mass was re-grouted before new excavation could start.

The excavation diameter for the final tunnel was 8.5 m. Several possible solutions for the excavation of this tunnel were evaluated, among them freezing. A method based on grouting and drainage through radial drill holes was, however, chosen. This is shown in Figure 9. A 6 m thick ring of grouted rock outside the final tunnel was established. The maximum distance between the grout holes was only 1.5 m in the middle of the ring. The grouted ring was then drained through holes which where 1.0 m shorter than the grout holes and had a spacing of 3.0 m. Grouting was done in three stages starting with cement/bentonite, followed by a thick silicate - mix and then a thinner mix as the final stage. All drilling was done through blow-out preventers and all drain holes were equipped with filter tubes. A large number of piezometers were installed to monitor and control the pore water pressure.

In addition to the 3000 m3 of grout used for the pilot tunnel 2250 m3 of cement/bentonite and 6250 m3 of silicate were used for the radial grouting. This gives 11500 m3 grout for a 77 m long tunnel, or 150 m3 per m tunnel. All grouting was completed by May 1986.

Final excavation of the main tunnel was done by the use of a roadheader. The upper half of the tunnel was excavated first and preliminary secured. The excavation was done in 1 m steps, followed by the installation of heavy steel ribs at 1.0 m spacing. Reinforced shotcrete was applied between the steel ribs. Final support includes a circular concrete lining.

This 77 m long section of the tailrace tunnel for the Guavio Hydropower Project was completed three and a half years after the first serious inflow of water and sand. Fortunately it did not delay the completion of the project as the difficulties were met at an early stage in the construction, and it was possible to speed up the excavation of the tunnel from the upstream side.

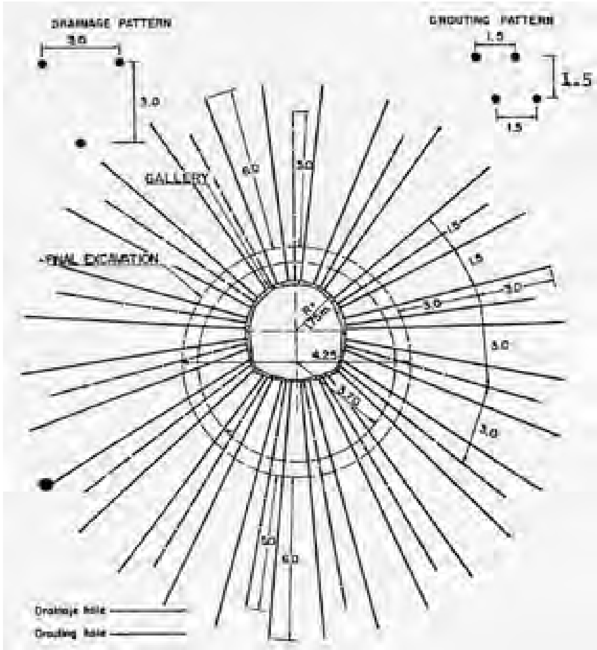

Figure 9. Pattern for the radial grouting and drainage for enlargement of the pilot tunnel of the Guavio tailrace tunnel.

4.2.2 CASE 2 – Delivery tunnel, Lesotho Highlands Water Project

The Muela hydropower station, which is part of the Lesotho Highlands Water Project, as well as Muela Dam and the Delivery Tunnel South are all in the so-called Clarens sandstone, which is a very uniform sandstone, partly of aeolian origin. It is quite similar to the Une sandstone in Colombia, but somewhat older (Jurassic) and stronger. The rock is however also friable, but is not subjected to high pore water pressures like in Guavio, and the stability in the powerhouse is very good.

The Delivery Tunnel South was excavated by a 5 m diameter TBM. Only minor stability problems were encountered during the tunnel boring process. After several weeks overstressing phenomena were, however, observed in the sidewalls of the tunnel. These phenomena were locally called “dog-earing” and are shown in the picture in Figure 10.

Figure 10. “Dog-earing” in sandstone due to high vertical stresses in the Delivery Tunnel in Lesotho

The “dog ears” developed slowly, but consistently. A full concrete lining was finally needed to stop further spalling. Measurements of the uniaxial compressive strength (UCS) and the vertical stress, showed that overstressing always occurred where the ratio was lower than 2.5. There were indications that time dependent overstressing might occur even for ratios up to 4.0. These stress induced spalling phenomena are rather different from the violent rock bursting that is observed in the Norwegian hard, crystalline rocks.

4.3 Tunnel collapses in shales – the Chingaza Project in Colombia

In the early 1970s the need for fresh water in the city of Bogota in Colombia was increasing, and the Chingaza project was started. After a construction period of approximately 10 years the project was completed in 1982. 38km of tunnel had been excavated through different types of sedimentary rock of Cretateous and Tertiary age. The longest tunnel, which is 28.4km and has a diameter of 3.7m, was filled with water for the first time in September 1983.

A short time after the tunnel had been put in operation, the capacity started decreasing. In the beginning of January 1984, after four months of operation, it was obvious that the tunnel was about to become blocked. The water flow had almost totally stopped, and the tunnel was taken out of operation. After a complicated clean-up operation, more than 40 fall-outs and slides from the tunnel periphery were observed. Some of them had completely filled up parts of the tunnel.

In Figure 11 cross sections of the upper 15 slides are shown. The majority of the fall-outs occurred in the Fomeque-formation. This formation is dominated by shales with interbedded siltstones and limestones. The rocks are partly folded and sheared along the layers. In some places the deformation has resulted in slicken-sided joints, and in other places crushing and fracturing.

Of the 4250m tunnel through the Fomeque-formation, 2000m was lined with a shotcrete layer of 5 to 15cm thickness, 2200m was lined with circular unreinforced concrete, and 64m was covered with four short steel linings. All the fall-out and slides occurred in the shotgrete lined parts, despite the fact that the shotcrete layer had been applied several years before the tunnel was filled with water, and no clear signs of weakening or cracking had been observed.

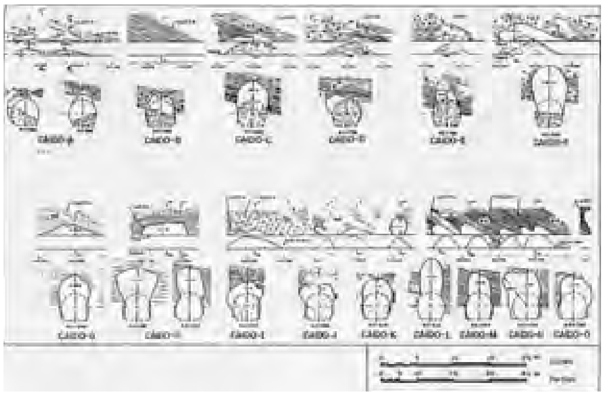

Figure 11. Cross sections of the slides A to O in the Chingaza tunnel

In a preliminary study several hypotheses regarding the reasons for the fall-outs in the Fomeque-formation were discussed. In the actual area the tunnel had been ventilated for several years before before it was taken into operation. Since the fall-outs only occurred after the tunnel had been filled with water, it seemed obvious that wetting of the rocks had to be an important factor affecting the stability. Water causes a reduction in the rock strength, but it seemed unlikely that this alone could explain the cracking of the shotcrete and the many fall-outs and slides from the tunnel periphery.

In order to understand what had happened rock samples from the slides were collected and analysed with respect to mineralogy and texture and swellability. The results from these analyses are discussed in detail in Brattli and Broch (1995), and the interested reader is referred to that paper. In this paper only the conclusions are presented. Based on observations in the tunnel, evaluation of the swelling tests and the observed reactions of the rocks when submerged in water as well as the mineralogical/petrographical analyses, the following explanation of the slides in the Chingaza tunnel is suggested:

i) Failures occurred in the Fomeque-formation where the tunnel periphery was lined with shotcrete, normally of 5 to 15cm thickness. The tunnel had in the actual area been ventilated for several years before being filled with water. As a shotcrete lining is not water or air tight, it is believed that an extensive draining and drying-out of the rock masses along the tunnel periphery took place.

ii) The drying-out process gave rise to local contraction stresses in the rocks near the tunnel periphery. This resulted in a heavy microfissuring along the bedding and shear planes of the shales, while for the stronger and more massive siltstone very few new fissures were created, if any.

iii) The microfissuring of the shale not only reduced the general strength of these rocks, but more importantly caused a considerable increase in the permeability and the exposed rock surface area. Some time (a few days or weeks) after the tunnel had been filled with water, the water would have penetrated through the shotcrete and entered into the numerous fissures in the shale.

iv) The increased water content will in itself cause a reduction in the strength of the rock mass. It is, however, unlikely that this alone can explain the slides, fall-outs and cracking of the shotcrete. Stronger forces seem necessary to initiate this process.

v) When water enters into all the new microfissures in the shales, the exposed and partly dehydrated illite/ssmectite minerals will adsorb water and start swelling. The swelling pressure measured on discs of the shale in the laboratory clearly indicate pressures of a magnitude that easily cracks a normal shotcrete lining and thus initiates the fall-outs and slides.

vi) The swelling of the illite/smectite minerals in the strongly fissured shale is therefore believed to be the initiating factor causing most of the fall-outs and slides in the Chingaza tunnel. The problems have been enhanced by the reduction in rock strength as a result of the many cracks and fissures caused by the draining and drying of the rocks during the long construction period.

5. CONCLUDING REMARKS

As shown in the described examples with weak and unstable rock masses, headrace and tailrace tunnels for hydropower projects in such conditions need to be properly supported by concrete or shotcrete linings. There are, however, lots of cases around the world where hydropower tunnels are excavated in rock masses that are only slightly affected by the water. In such cases considerable cost savings can be made by reducing the amount of lining to an absolute minimum. The cost of lining a meter of tunnel is often in the order of two to three times the cost of excavating the tunnel. And to put it frankly: The water does not care if there are some minor rock blocks along the tunnel floor, - and a rock trap at the end of the headrace tunnel. This has been demonstrated through decades of successful operation of several hundred unlined hydropower tunnels in Norway.

With a good understanding of the rock stresses in the planned area for the underground powerhouse, Norwegian experience has also shown that it is possible to convey the water to the powerhouse through unlined highpressure shafts and tunnels with water heads more than 1000m. Steel pipes or steel linings are only used for the last 25 – 75m dependent on the water head. It is basically a question of putting the powerhouse and thus the shaft deep enough into the hill side so that the rock stresses along the shaft at any point is greater than the internal water pressure. Avoiding installation of a steel lining means not only considerable direct cost savings, but also saving of construction time for a part of the project that often is on the critical path.

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5.2 UNDERGROUND STORAGE FOR HYDROCARBON PRODUCTS

Introduction to hydrocarbon storage
Storage of hydrocarbon products such as crude oil and liquefied gas is a necessary link in the process of transporting and distributing these products from the oilfields and to refineries and then on to the consumers. Appropriate storage volumes along this distribution line increase the availability of the product and the timing of the supply from pipelines, terminals and refineries. This is certainly in the interests of the consumers. This paper focuses on underground storage of hydrocarbons. It is acknowledged that sub-surface solutions have been utilised for other purposes too, in oil and gas projects, such as shore approaches, pipeline tunnels, slug cathcers etc., but such facilities will not be discussed in this paper.

During the post war era in Norway, sub-surface storage of oil and gas became important for strategic purposes. Various options for underground storage were considered but Norway was short of suitable alternatives to rock caverns. During this period utilisation of underground openings had seen a significant growth in Norway, particularly due to the development in the hydroelectric power sector, where an increased number of projects utilised underground alternatives for waterways, pressurised tunnels and location of hydropower stations and transformer rooms. The Norwegian tunnelling industry developed techniques and methods to improve the efficiency and quality of underground works. A comprehensive experience base was established which became important when the underground storage of hydrocarbons was introduced. In the figure above it is shown how the development of underground utilisation took place in Norway, shifting the tunnelling industry from hydropower projects to oil and gas storage and then towards the current use for infrastructure purposes.

In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development. Underground oil and gas storages mainly utilise the following capabilities of the rock mass: a) It's impermeable nature, b) It's stress induced confinement, c) It's thermal capacity and d) It's selfstanding capacity.
Why to go underground with oil and gas storage in Norway? In the following this paper presents the rationales and motivations for underground oil and gas storage in Norway, further it presents the development of underground oil and gas storages documented with factual data and case stories as well as presenting the basic principles for the establishing these storage facilities.

Aboveground storage
The most common way of storing hydrocarbon products has been, and still is in clusters of aboveground steel tanks, as tankfarms. Typically they are found in the near vicinity of airports, close to harbours and ports, in connection to industrialised areas, at electric power plants and of course in the surroundings of refineries and terminals, and finally at natural gas treatment plants. The close proximity to the production line and the users are the main reasons for such locations. The natural resource of suitable rock mass for underground storage may in many countries be in short supply and the best (and maybe the only) solution may therefore be above ground tankfarms.

Figure 6. Typical tankfarm

The negative elements of such tankfarms are significant, particularly with regards to protecting the environment. In addition aboveground tanks are vulnerable targets to hostile actions such as sabotage and war. Further they are aesthetical undesirable and demand large land tracts of land which can often be utilised in a better way as in many densely populated areas surface space is becoming a scarce resource.

Going underground
During and shortly after WW II
In Norway, the first underground hydrocarbon storages were excavated during the Second World War, designed for conventional, selfstanding oiltanks. Later, being located underground was basically for protective purposes during the cold war era. One project of such kind is located at Høvringen, near the city of Trondheim in central Norway, where

ESSO is operating underground steel tanks, whilst one other storage is located at Skålevik, and is operated by BP. Following on from these first projects was underground hydrocarbon storage in steel lined rock caverns, designed and built in accordance with for example Swedish fortification standards. This concept implies in brief a steel lining with concrete backfill of the void space between the steel lining and the rock contour. One such project is located in Hommelvik outside Trondheim and is operated by Fina. This project provides the supply of gasoline to the nearby airport. The above described projects were commissioned almost a half a century ago, and are still being in operation.

In the sixties, following experience from the hydroelectric power development, the confidence in unlined tunnels and caverns grew, and the first unlined hydrocarbon storage project was initiated. Concept developments took place in other Scandinavian countries at the same time, however, in Norway unlined pressure shafts had been in use for some time in the hydroelectric power development and the importance of sufficient in-situ rock stress to prevent hydraulic splitting of the rock mass was recognised as an important success criteria. Also the techniques of pre-grouting of the rock mass to stem or reduce water leakage started to be developed during this period. Adding to this, caverns with large cross-sections were already in use as hydropower stations. Thus, the Norwegian tunnelling industry was prepared and technically ready for the new challenge of unlined hydrocarbon storage in rock caverns.

Typically storages facilities during the cold war era were supply storages prepared for war time operation. They were in general owned by the Ministry of Defence but were often operated in peace time by the commerical oil companies.

The Ekeberg Storage
In 1966 construction work commenced on the Ekeberg storage facility located close to Oslo, the Norwegian capitol. The project was designed and constructed as an unlined rock storage, and in 1969 oil filling commenced. This storage facility was later expanded to include new storage caverns. The Ekeberg storage introduced a design concept, which in general has been applied for later similar storages in Norway. A storage facility in rock was concluded as being the best solution for a fuel storage in the Oslo area, being well secured against acts of war and sabotage. The Ekeberg storage is located adjacent to the Sjursøya Terminal, see figure below, in the ridged area on the land side of the terminal.

Figure 7. Ekeberg crude oil storage and Sjursøya terminal (city center of Oslo to the left)

The project was extended with a second stage some ten years after the commissioning of the first stage, when the Ekebergtank entered operation. The Ekebergtank is used for storage of jet fuel and gasolin. Both phases of the Ekeberg storage have been constructed based on unlined cavern storage. In the bottom of each cavern there is a waterbed with water being pumped from the adjacent sea, the Oslofjord. The caverns are situated well below the sea level, with the deepest point at 45m below sea level.

The typical size of the rock storage caverns in most recent projects indicates a cross-section of appr. 500m2. In practical terms this means that the caverns cannot be excavated in one blast round, but must be split into a top heading and several benches. As can be seen for the Sture and Mongstad crude oil storages, the caverns are close to 20m wide and 33m high, and this has become a typical cross sectional area for such caverns.

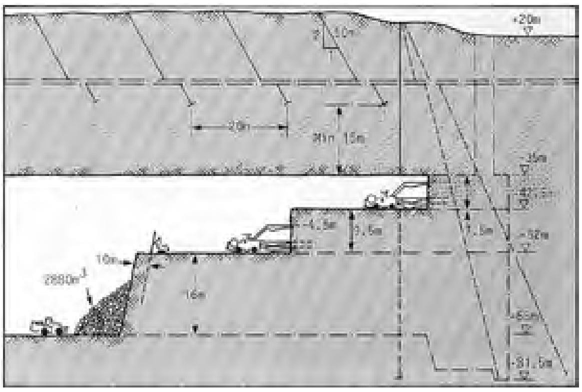


Figure 8. Typical excavation sequence; top heading and lower benches and with artificial groundwater infiltration from surface.

Project	Year of Completion	Main rock type	Width x height, m	Temp. ºC	Pressure MPa	Experience
Kristiansand, Skålevik	1951	Gneis-granite	Ø=32 H=15	40	0,1	No problems reported
Høvringen, Trondheim	1955	Quartsdiorite	Ø=32 H=15	40	0,1	As above
Sola, Stavanger	1960	Mica schist	Ø=15		0,1	Corrosion, decommissoned
Ekeberg I	1969	Granitic gneiss	12x10		0,1	No problems reported
Mongstad	1975	Meta-anorthosite	22x30	7	0,1	Some water leaks
Høvringen, Trondheim	1976	Quartzdiorite	12x15		0,1	Water curtain has been added
Herøya	1977	Limestone	10x15	8	0,1	Leak between caverns
Ekeberg II	1978	Granitic gneiss	15x10	60	0,1	Some blockfalls
Harstad	1981	Mica schist	12x14	7	0,1	No problems reported
Sture	1987/1995	Gneiss	19x33 ~1.000.000m3			
Mongstad	1987	Gneiss	18x33 1.800.000m3			No problems reported

Table 1. Norwegian crude oil storage facilities and refinery caverns for hydrocarbon products

Gas storages
In 1976 Norsk Hydro constructed an unlined rock storage for propane at Rafnes (close to Herøya) in Southern Norway. This project included a pressurised storage with an operation pressure of 7 atm at normal rock temperature and with a volume of appr. 100.000m3.
The storage at Herøya is excavated in Precambrian granitic rock with its roof 90m below the sea level. The rock mass is practically impermeable, but due to jointing and few minor weakness zones, there was a need for grouting. The technique of pre-grouting was applied to prevent leakage of water into the caverns and to control potential fluctuations of the groundwater level. The design criteria required a hydraulic gradient towards the cavern to be greater than 1, a figure which was recognised as being safe. During excavation it was experienced that the water level in some observation wells above the cavern showed a general fall in the groundwater level and it was decided that water infiltration holes needed to be implemented. Piezometers were installed in some of the observation wells to monitor the water gradient immediately above the cavern roof.

The experience during the first projects of this kind was that it was difficult to maintain the surrounding groundwater level without establishing a system for water infiltration of the rock mass. Since these first projects the potential of loosing control of the groundwater has been the governing decision whether or not to install water infiltration systems for groundwater compensation. The correctness of this approach might of course be questioned, and it has, still it is considered as good engineering practice by Norwegian engineers and plant operators to do so. And, as a consequence ´water curtains´ have been included in all of these projects, the extent and layout of the infiltration systems may, however, have varied. In most cases though, the infiltration systems have been installed and entered into operation prior to the excavation work. A very typical attribute of these projects is their shallow location, which necessitated an artificial groundwater infiltration.

Then, in 1986 the first chilled storage facility was constructed in Glomfjord. This included a water infiltration curtain from a gallery above the doughnut shaped cavern. The project was cooled to a temperature of -33oC. To be able to reach the designed temperature in a chilled storage, different methods have been applied for the freezing process itself. One method that was often used previously was the direct cooling by introducing the product directly into the cavern. However, during the most recent years an improved method has been commonly applied that includes a 2-stage freezing process. Typically air cooling takes place until the 0-isotherm has reached a certain depth

Project	Commissioned	Main rock type	Storage volume, m^3	Width× height× length, m	Temp. , ^0C	Pressure, MPa	Experience
Rafnes	1977	Granite	100,000	19×22×256	~ 9	0.65, tested at 0.79	No leakage
Mongstad	1989	Gneiss	3 caverns, total 30,000	13×16×64	6-7	Up to 0.6	No leakage
Mongstad	1999	Gneiss	60,000	21×33×134	- 42	0.15	Reduced capacity
Sture	1999	Gneiss	60,000	21×30×118	- 35	0.1	No information available
Kårstø	2000	Phyllite	2 caverns, total 250,000	Approx. 20×33×190	- 42	0.15	No leakage
Mongstad	2003	Gneiss	60,000	21×33×134	-42(propane) +8 (butane)	0.15	Under construction
Mongstad	2005	Gneiss	90.000	22x33x140	6-7		Construction start 2005
Aukra	2007	Gneiss	63.000/ 180.000	21x33x95 21x33x270	6-7	0,2	No reports

Table 2: Overview of main data for petroleum gas storage
**) All with propane; Mongstad 1989 also stores butane and Sture 1999 stores a propane/butane mixture. Mongstad 2005 will be naftalene, Aukra 2007 will be condensate*

in the rock mass, say in the range of 3-5m. This enables necessary inspections to take place inside the cavern allowing qualified personnel to inspect for any defects and instabilities that may exist and rectify these whilst still working in a non-hazardous environment. Then the final cooling stage takes place during storage of the product itself.

Statoil is also involved in the Stenungsund propane caverns project in south-western Sweden. The total volume of 550.000m3 is stored underground at -42ºC.
The latest gas-storage facility commissioned in Norway was the propane cavern at Mongstad in 2003. This cavern was actually the 27th rock cavern excavated at the Mongstad Terminal for underground storage of oil and gas. This latest project was built to replace a cavern that had lost appr. 30% of its storage capacity due to rock fall into the cavern, subsequent water ingress and ice being formed. Despite such a negative one-time event the operator maintained his confidence in this storage concept. Due to the favourable costs associated with underground storage the operator finds advantages with such concepts connected to the operation and maintenance, and also to the safety aspect. There are no reported cases in Norway of negative environmental impact caused by such underground facilities.

This latest extension at the Mongstad refinery is a 60.000m3 underground storage facility for propane. It is designed for an operating temperature of -42ºC. The cavern is formed like a ´flat lying bottle´ with a concrete plug being located almost at the bottle-neck and with the pumping arrangement in the far end of the cavern. The maximum height of the cavern is 34m and it has a width of 21m, whilst the length is 124m.

Figure 9. A typical cross-section of an underground oil and gas storage cavern with straight walls

The concept of underground storage

The concept of unlined oil and gas storage in use in Norway follows the main principles and methods as outlined below:

Permeability control and hydraulic containment

The methods for controlling leakage from an unlined underground storage consist mainly of 1) permeability control and 2) hydrodynamic control (or containment). In the figure below it is schematically shown.

By permeability control it is meant that leakage control is achieved by maintaining a specified low permeability of the rock mass. This can be achieved by locating the rock caverns in a rock mass that has natural tightness sufficient to satisfy the specified permeability. However, the rock mass is a discontinuous media and the presence of joints etc. governs its permeability. Permeability control can be preserved by artificially creating an impermeable zone or barrier surrounding the rock caverns by; a) sealing the most permeable discontinuities in the rock mass by grouting; or b) introducing a temperature in the rock mass which freezes free water and filling material in the rock mass; or c) a combination of both methods.

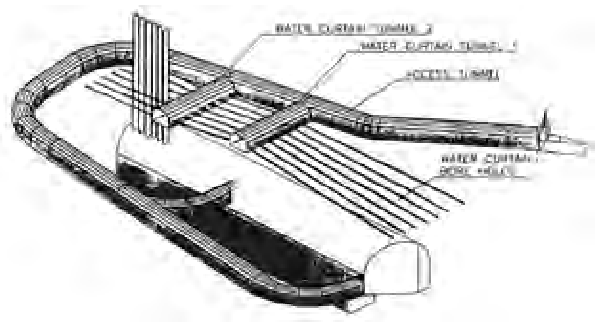


Figure 11. Water infiltration Sture

By hydrodynamic control it is meant that there is groundwater present in discontinuities (joints and cracks) in the rock mass and that this groundwater has a static head that exceeds the internal storage pressure. In practical terms it means that there is a positive groundwater gradient towards the storage, or the rock cavern. In general, sufficient groundwater pressure is obtained by a) a deep seated storage location which provides the sufficient natural groundwater pressure, or b) by an additional artificial groundwater such as provided by ´water curtains´ and similar arrangements.

In the invert the crude oil is normally floating on top of a water bed. The water bed could either be fixed or variable, depending on the discharge pump arrangement to be used. An important element in the hydrodynamic confinement is related to the following up of the groundwater level surrounding the storage facility. It would normally be required to install a number of monitoring wells to monitor groundwater levels. The concept of unlined pressurised storages evolved during the hydroelectric power development that took place in Norway during the 1960's to 1980's. See table 3. Both permeability and hydrodynamic control was applied in compressed air storage projects.

According to regulations issued by the Norwegian Fire and Safety Administration (DBE)DSB the natural groundwater or the overpressure resulting from water curtains shall be 2 bar higher (20m water column) than the internal storage pressure, for oil and gas storage facilities.

It is possible to combine the two methods of leakage control and apply a combination of both hydrodynamic control and permeability control, this has been termed as a double barrier. However, and in a rather general manner; hydrodynamic control may be the preferable method in situations with a substantial internal storage pressure, whilst permeability control may be preferred in situations with low/atmospheric storage pressure. For storage facilities where the hydrodynamic control has been applied excessive water from the artificial water curtain may be allowed to enter the storage facility and thus the product stored must tolerate the presence of water before it is separated, collected and discharged from the storage.

One typical example of the application of hydrodynamic control is air cushion chambers in hydropower schemes where air is compressed and the ´water curtains´ constitute the containment. On the other hand, a typical example where permeability control is applied is crude oil storage facilities.

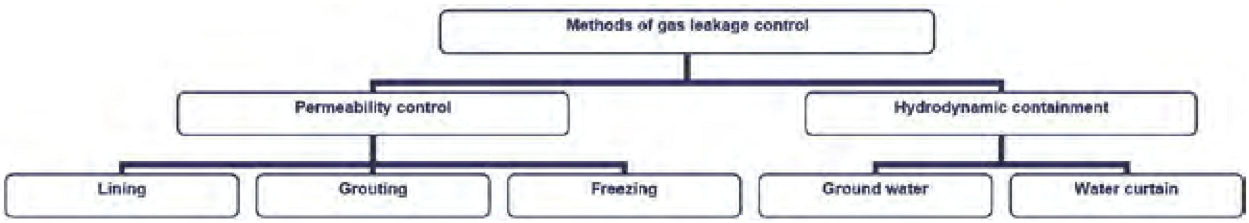


Figure 10. Methods for controlling gas leakage from a pressurised underground storage.

Project	Commis-sioned	Main rock type	Excavated volume, m^3	Cross section, m^2	Storage pressure, MPa	Head/cover*)	Experience
Compressed air buffer reservoirs							
Fosdalen	1939	Schistose greenstone	4,000		1.3		Minor leakage
Rausand	1948	Gabbro	2,500		0.8		No initial leakage
Air cushion surge chambers							
Driva	1973	Banded gneiss	6,600	111	4.2	0.5	No leakage
Jukla	1974	Granitic gneiss	6,200	129	2.4	0.7	No leakage
Oksla	1980	Granitic gneiss	18,100	235	4.4	1.0	<5Nm3/h
Sima	1980	Granitic gneiss	10,500	173	4.8	1.1	<2Nm3/h
Osa	1981	Gneissic granite	12,000	176	1.9	1.3	Extensive grouting
Kvilldal	1981	Migmatitic gneiss	120,000	260-370	4.1	0.8	Water infiltr. necessary
Tafjord	1981	Banded gneiss	2,000	130	7.8	1.8	Water infiltr. necessary
Brattset	1982	Phyllite	9,000	89	2.5	1.6	11Nm3/h
Ulset	1985	Mica gneiss	4,800	92	2.8	1.1	No leakage
Torpa	1989	Meta siltstone	14,000	95	4.4	2.0	Water infiltr. necessary

Table 3. Overview of main data for compressed air storage, including air cushion surge chambers
**) Ratio between maximum air cushion pressure expressed as head of water and minimum rock cover*

The water curtain can be used to balance the migration of the 0-isotherme by applying water with a specific temperature for water infiltration. This concept has been applied at Sture (1999).

A typical example where the two different methods are used in combination as a double barrier would be for LPG storage facilities. The product in LPG storages might hold a temperature of -42 ºC at atmospheric pressure. The surroundings of the storage consists of a saturated rock mass that is frozen to a depth reaching some 10m out from the cavern periphery, thus achieving the desired permeability requirement. A comprehensive pre-grouting scheme is required for the purpose of reducing the amount of water needed for the saturation of the rock mass and consequently reducing the energy required for the freezing process. With the double barrier used in this case the potential for a successful filling and operation of the plant is increased.

Advantages of underground storage
In the following the main advantages of underground rock storage are described. In brief these are:

- Utilising the variety of parameters of the rock mass.
- Environmentally friendly and preserving.
- Protection during war.
- Cost aspects.
- Operation and maintenance
- Protected from natural catastrophes

It has been documented that the rock mass holds a number of important parameters that are utilised in underground storage of hydrocarbon products. These capacities allow a variety of storage conditions and enable a number of diverse types of products to be stored in unlined rock caverns. With the current knowledge of the mechanical and thermodynamic behaviour of the rock mass the current use of such storage facilities can be said to take place within proven technologies. Future use of underground storages may push these technologies to its limit and thus require improved methods. This will be briefly discussed at the end of this paper.

As far as the environmental aspects are concerned the experience from Norwegian underground storage projects are unreservedly positive. So far product leakage has not been reported in any of these projects indicating clearly that the applied concept and techniques to obtain the required confinement are appropriately proven. For a subsurface solution dedicated systems for collection and handling of various types of spill can be planned thus limiting the spread of any spill. Bringing these storage tanks below the surface allows valuable surface areas to be utilized for other purposes; recreational, cultural and residential. In addition unsightly structures can be hidden away underground.

Crude oil and refined products may in a war-time situation be the subject for hostile actions. The protection against various types of bomb attacks and sabotage are indeed capabilities not widely described and published, but indeed contribute to the overall favourable application of underground storages.

Protection from natural disasters and catastrophes such as earthquakes is a beneficial advantage of underground storage. It has been acknowledged that subsurface structures have several intrinsic advantages in resisting earthquake motions. Experience and calculations show this clearly. The latest cost figures on construction costs are due in 2004. The total construction cost is in the range of 150 – 310 USD per m3 storage, out of which 50-70% is associated with mechanical and electrical installations. Shallow locations are indeed a feature that improves the cost advantage of these storages. In the figure below a cost comparison of steel lined surface tanks are compared to underground storage caverns, unlined.

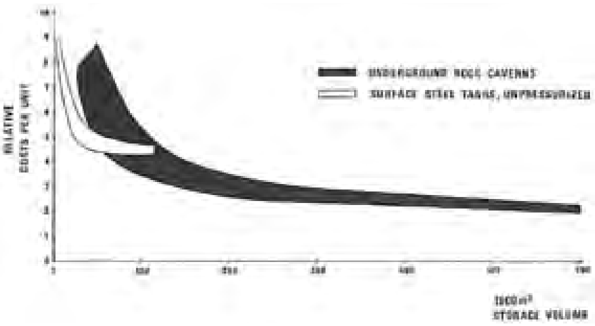


Figure 13. Relative rock cavern/steel tank costs

It has been out of our reach to obtain figures on operation and maintenance costs from Norwegian oil and gas storages facilities. The physical isolation of underground structures from the external environmental reduces the detoriation of building components and may result in low maintenance costs for underground structures.

5.3 ALTERNATIVE USE OF ROCK CAVERNS

SPORT, SWIMMING AND OTHER RECREATIONAL FACILITIES IN ROCK IN NORWAY

In Norway there are probably around 300 + caverns for underground hydropower plants and caverns for a variety of other civil purposes. Thus, telling a story of high activity and engineering expertise in this particular field.

Power houses, in some cases, with spans measuring 20 metres and more. Involved engineers and architects transformed some of these into spectacular rooms in colour, light and architectural design.

This variety led to the idea of using the same technology for sport halls and swimming pools needed around in the country. Over the years many such facilities have been finished. Other typical caverns are caverns for railway stations, oil & gas storage, car parking, water treatment plants, sewage treatment and so on. A wide variety of applications. Cavers could be from 18-20 meters span to 62 meters as is the Gjøvik hall. In the following a round trip throughout Norway will be presented, from south to north, to give some ideas of the variation in the use of rock caverns for such purposes.

Dual purpose caverns used as civil defence shelters in war time and for entertainment and recreation like sports halls and swimming pools in peace time were built, which means that they in case of war fear are quickly converted to air-raid shelters. Thus part of the construction costs is covered by the National Civil Defence Authorities as this to a large took place during the cold war period. Many underground facilities were made also for the sole purpose of serving defence and strategic purposes, these of course are not covered within this article.

Hydro power plants showed the way

The picture shows the Tonstad Hydro power Plant in Sira, a municipality in the south western part of Norway. This is one of many, showing engineering and internal architecture of high class despite being underground.
It is easy to understand that this gave the idea of putting sport halls and other recreational facilities, in rock caverns. The technique concerning geology, a variety of engineering, contracting and architectural design, was easily transferred to this new area.

The Oslo region
Oslo is the capital of Norway, situated in the south east part of the country. In this region there are seven sport facilities in rock. Three are rather large for international hand ball play, two of them includes also swimming pools. Four are smaller with activities including tennis, bowling, shooting, cinema, wrestling and others.
Pictures on the next pages illustrate these ideas.

Vassøyholtet

Vassøyholtet is a facility inside a small hill situated in the municipality Skedsmo east of Oslo. Besides the bowling alley and shooting range there is also a cafeteria and gymnasium hall.
Finished in 1979. The facility has a volume of 5.500 cub.m.

Holmenhallen

Cinema/theatrer

Sport hall

Holmenhallen is situated in Asker west of Oslo. This facility includes the following:
main hall, 25x50 m, which is standard for hand ball, rooms for cinema/theatre, clubrooms and locker rooms and showers. It was finished in 1981 and has a large volume of
35.000 cub m.

Holmlia Sport Hall and Swimming Pool in rock
This facility is situated in the south part of the capital Oslo. It is a large installation including a sport hall for different kinds of ball games, swimming pool, 20 x 37 m. both with all necessary facilities for locker rooms, showers and saunas, fitness centre, clubrooms, etc. This large facility serves the part of the City, the Holmlia region, with 10.000 inhabitants. It has been in successful use since 1984 and has a usable volume of 53.000 cub.m.
The cost in 1984 was 54 mill NOK with a split of cost as follows:

Excavating, drill and blast	13,0%
Rock support and mucking out	5,5%
Civil Works	34,8%
Electrical installations	7,2%
Lifts	0,3%
Emergency power	1,0%
Ventilation and cooling	5,0%
Sanitary and heating	4,5%
Water and cleaning system	0,7%
Sum	72,0%

Mapping and Geological investigation	0,5%
Engineering/arch.	7,5%
Taxes	17,8%
Grand sum	100%

Civil defence requirements counts for aprox 20% of the total cost, all paid by the government which was economically favourable for the city/municipality

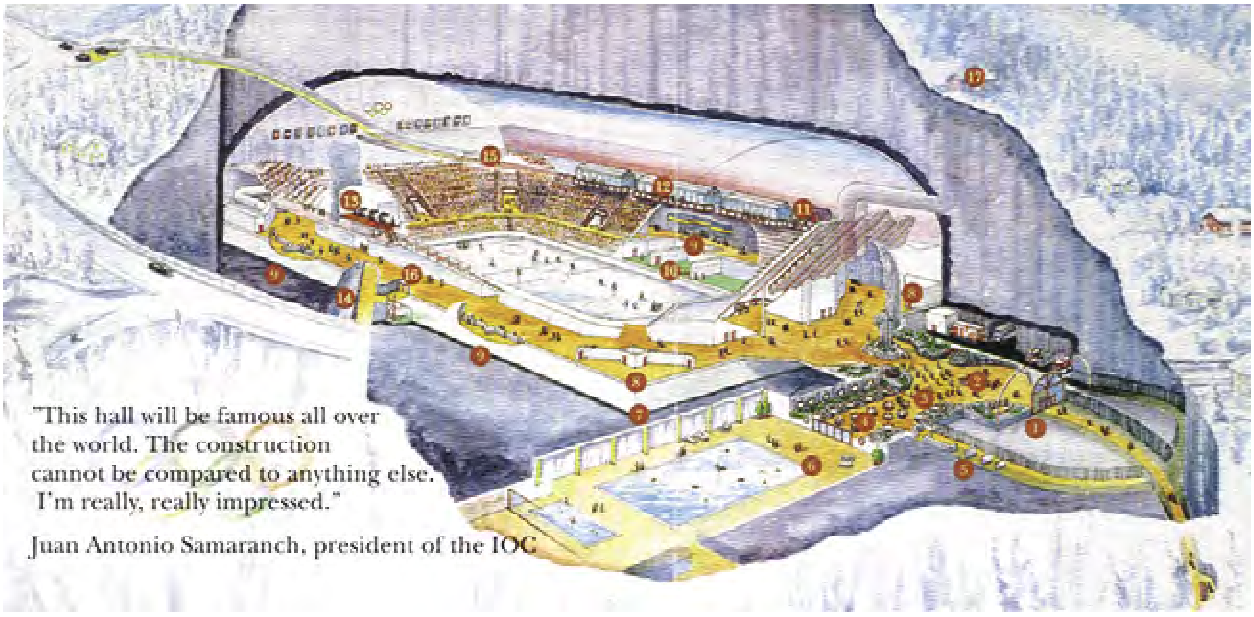
Holmlia Sport Hall. In active use since 1984. Foto:Trond Joelsen, Byggeindustrien October 2013

Holmlia in Oslo. Swimming Pool, 6 lanes, 25 m.
Foto:Trond Joelsen, Byggeindustrien October 2013

Gjøvik Olympic Mountain Hall
The town of Gjøvik is situated at the west bank of Norway's largest lake, Mjøsa in eastern Norway. The use of underground facilities began at an early stage in 1975 by building a swimming pool in rock, in the middle of the town. The experience in use, operation and maintenance, were main factors when it was decided to build a huge ice-hockey arena in the same rock for the Winter Olympic Games in 1994.
The main hall measures W: 61 m, L: 91 m, H: 25 m. Volume: 141.000 cub.m.

Gjøvik Olympic Mountain Hall is the largest facility in rock for public use so far in the world.
The construction work was finished 3 month ahead of schedule. Volume: 141.000 cub.m.

From an ecological perspective, this hall was the most innovative of all the 1994 Olympic Games Arenas. The naturally stable year-round temperature inside the rock has reduced the halls heating/cooling cost by half, compared with conventional halls. Since the hall occupies no surface area- located as it is "under" the centre of Gjøvik- no trees had to be felled and no new infrastructure had to be built.



The drawing shows the entrance tunnel to the right entering the lobby/restaurant area with the old swimming pool to the left and the huge new ice hockey arena in front

Gjøvik Swimming Pool in rock. 6 lanes of 25 m. W: 20 m Was commended the Eurostructpress Award for Architecture in 1980. In active use since 1975. Foto: Ådne Homleid, Byggeindustrien,Oct. 2013

Picture above showing the great moment 6th of May 1993: The opening ceremony with the King and Queen of Norway with 5.800 invited guests participating.

Since the Olympic Games in 1994 it has been in extensive use all year around with excellent result, both for sport activities and exhibitions as well as several other purposes.

Oddahallen
We are moving north and west in the country, to the industrial town Odda
The town is situated in the south part of Sørfjorden (South fjord) in Hardanger.
Steep mountains surround the town. Lack of suitable building ground for their new planned sport hall brought up the idea of a new facility in rock
W: 25m, L: 50m, H: 13m. Finished in 1972. Volume: 27.000 cub.m

The entrance building to the left, is situated close to the outdoor sport field.
A tunnel to the right leads to a 110 m sprint track and shooting range and club rooms.
A tunnel to the left leads through locker rooms and showers to the main hall.
After more than 40 years extensive use, the owner reports that Odda sport hall has been the "flagship" facility for the town. Activities like biathlon (shooting), fitness center, running, jumping, gymnastics, handball, football, volleyball and ski shooting are still going on around the clock.
Running and maintenance cost has been relatively small.

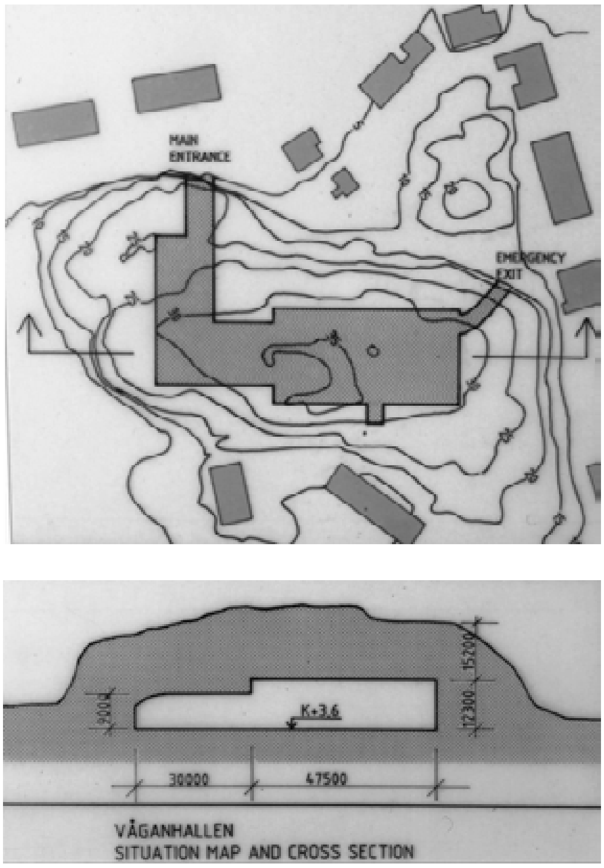
Oasen Bathing and Recreational Facility in rock
We have reached Mid-Norway and entering Namsos, a town situated in the north part of Trondheimsfjorden, which has a spectacular facility all inside rock. The main pool has 8 lanes 50 m long and there is a 5 m high diving tower, a pool, saunas, a 25 m long water "roll-ercoaster", a restaurant and other facilities. The facility was finished in 1988.

"Oasen" bathing and recreational facility in rock, Namsos municipality.Volume: 27.000 cub.m.

"Oasen": Bading, swimming, diving and dining in rock

Våganhallen
Now we are moving far north to the fishing town Svolvær in Nordland.
A rather small hill in the middle of the town was chosen as site for a standard hall for handball. (25 x 40 m.) The facility includes also lockers and showers, a shooting range and clubrooms. The most interesting thing is the optimal use of the hill as shown on the drawing below. Finished 1983. Volume: 21.100 cub.m.

Svolvær



Prestvegen Sport Hall in rock, Kirkenes
Far north in northern Norway, in the town Kirkenes, ends our journey.
The picture below speaks for its self.
The town is very proud of its rock facility!

Kirkenes : Sportshall in rock

Svalbard Global Seed Vault
Deep inside a mountain on a remote island in the Svalbard archipelago, halfway between mainland Norway and the North Pole, lies the Global Seed Vault. It was the recognition of the vulnerability of the world's genebanks that sparked the idea of establishing a global seed vault to serve as a backup storage facility. The purpose of the Vault is to store duplicates (backups) of seed samples from the world's crop collections.

Figure: Layout of the underground facility

Permafrost and thick rock ensure that the seed samples will remain frozen even without power. The Vault is the ultimate insurance policy for the world's food supply, offering options for future generations to overcome the challenges of climate change and population growth. It will secure, for centuries, millions of seeds representing every important crop variety available in the world today. It is the final back up.

The Vault is in an ideal location for long-term seed storage, for several reasons, but the main reasons are that the area is geologically stable, the temperature inside the caverns are stable around the year and humidity levels are low.

The Seed Vault has the capacity to store 4.5 million varieties of crops. Each variety will contain on average

500 seeds, so a maximum of 2.5 billion seeds may be stored in the Vault. Currently, the Vault holds more than 880,000 samples, originating from almost every country in the world. Ranging from unique varieties of major African and Asian food staples such as maize, rice, wheat, cowpea, and sorghum to European and South American varieties of eggplant, lettuce, barley, and potato. In fact, the Vault already holds the most diverse collection of food crop seeds in the world.

Figure. A monumental entrance to the seed vault

Final remarks
The aim of this journey, visiting some of our sport facilities in rock, is to give you some ideas in how utilisation of rock caverns may be applied.

The long term experience for these facilities is very good.

An inquiry to the owners of ten facilities states the following:
- The users are well satisfied with their facility and very often proud.
- Both operational as well as maintenance cost are much lower in rock compared to an aboveground solution. Thus, the lifetime cost is much lower.
- All users are satisfied with the combination as civil defence shelter.

If one decides to build a rock facility, one should have extra focus on:
- Employees desire for daylight
- Control of water leakage and humidity

In spite of the clear advantages with the rock cavern solution, the first choice for a new sport facility in Norway today, (2013) will be the aboveground solution. The main reason is that the government no longer pays for the civil defence shelter. The rock cavern solution will then normally be 10 to 20 % more costly for the municipality.

Since the rock cavern alternative also is a much better solution environmentally, a proposal has been submitted to the government to subsidise the rock alternative.

5.4.1 OREA – EXTENSION OF AN EXISTING SEWER TREATMENT PLANT IN STRØMMEN, NORWAY

OREA is the extension of a large underground sewer treatment plant (Figure 1), owned by four municipalities outside Oslo. The purpose of OREA is treatment of excess flood water that today runs via a spillway directly out to the Nitelva river. The main rock cavern of OREA is 60m long, 20m wide and 20-25m deep, rock quality variable and rock overburden shallow. The location is in an urban area, and the cavern is lying straight underneath the Strømmen College. The contract for rock excavation started in August 2016 and was completed by the end of April 2017. The rock excavation method was conventional drill and blast with great environmental challenges concerning noise and vibrations from drilling and blasting. Besides environmental issues, safe performance of tunnel excavation, stability and rock support has been a great challenge. Another great challenge during the excavation, both technical and with respect to health and safety aspects, has been the breakthrough from the rock cavern to the sewer tunnels, which were in full operation. During planning, contract preparation and construction of the caverns, the philosophy and principles of Norwegian Method of Tunnelling has been used. The paper describes ground investigations, rock modelling and stability analysis, environmental impact and mitigation measures as well as methods and procedures used for safe performance of rock excavation.

Figure 1.The existing Underground Sewer Treatment Plant underneath the Strømmen township, Norway. Location of the coming OREA extension is to the lower right in the figure.

1 INTRODUCTION
Underground sewer treatment plants are good examples on sustainable use of the underground space. In Norway, there are several, like VEAS and Bekkelaget in Oslo, IVAR in Stavanger, Kverneviken, Flesland renseanlegg, Sandviken og Holen in Bergen, Ladehammeren og Høvringen in Trondheim and in

several other cities. OREA is an extension of an existing sewer treatment plant in Strømmen township, a suburban area just outside of Oslo City in Norway. As shown in Figures 1 and 2, the plant occupies an extensive part of the underground space underneath the township. The existing sewer treatment plant is operated by NRA (Nedre Romerike Avløpsselskap IKS), owned by the four municipalities Lørenskog, Nittedal (member from 2015), Rælingen and Skedsmo. The oldest part of the existing plant was constructed late 1960 to early 1970, and opened in 1972. The plant was extended to install an extra process for removal of nitrogen in 2001 - 2003. Today, the sewer treatment plant serves 130.000 inhabitants.

Figure 2. Area of the existing underground sewer treatment plant, the coming OREA located in the upper part.

The purpose of the OREA extension is to clean the excess flood water which today runs uncleaned out into Nitelva river. In addition, the OREA extension will triple the capacity of the existing plant. The OREA extension is located in the old part of the sewer treatment plant (to the right in Figure 1, the upper part of Figure 2).

A 3D model of the OREA extension is presented in Figure 3. OREA consists of the main rock cavern (blue), intake chamber (green), connecting tunnel (orange) and cavern for technical facilities (purple). The main rock cavern has a length of 60m, span width 20,6m and height of 25m.

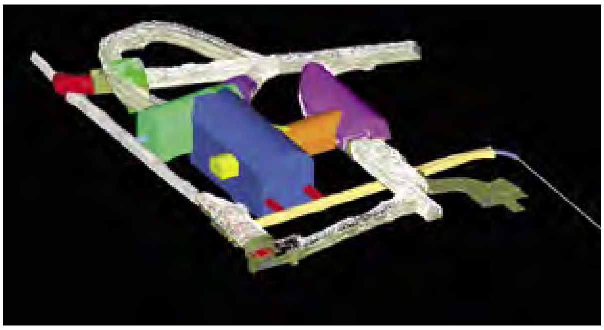


Figure 3. A 3D model of the OREA sewer extension.

The bedrock in the area is Precambrian gneiss, covered by marine deposits (clay) and various fill material for buildings and foundations. The total overburden (soil and rock) is approximately 20m, rock cover between 10m and 15m, and the Strømmen college is located right on top of the rock caverns.

With large span width of the caverns, shallow rock cover, short distance to the Strømmen college and surrounding buildings, the OREA extension represents a great challenge with respect to rock excavation, rock stability and the environmental impacts, in particular structural noise from drilling and vibrations due to blasting.

The contractor for the rock excavation and rock support was Skanska Norge AS, and the Consultant for this contract (rock excavation) is Sweco Norge AS. Due to its complexibility, the project was classified as CC/RC=3 and GC3 according to the Eurocode 7, wich requires independent control. The consulting company Multiconsult AS was selected for this duty.

During planning, contract preparation and the construction stages, the philosophy of the Norwegian Tunneling Method with ground investigations, risk shearing, contract philosophy and standard specifications has been used.

2 PLANNING STAGE

During the planning stage, an extensive program was started to identify the challenges and potential risks related to the environmental impacts such as tunnel excavation in urban area and inside the existing sewer treatment plant in full operation, as well as rock stability of large rock caverns with shallow rock overburden. The program included Environmental Impact Assessment (EIA), preparation of Environmental Monitoring Program (EMP), geological mapping, ground investigations as well as rock modelling and stability analysis. Risk assessment was used as an active tool during the planning and design process.

2.1 Environmental Impact Assessment (EIA) and Environmental Monitoring Program (EMP) for the Construction Stage
The Environmental Impact Assessment (EIA) covering the construction stage, had focus on potential damage risk due to drainage and ground settlements, vibrations from blasting, structural noise/vibrations from drilling and scaling, noise from ventilation fans, and impacts from logistics and transport of tunnel muck.

Ground settlement risk
Potential risk for settlements due to drainage of the marine clay overburden was assessed by drilling ground water measuring wells. No ground water level was found, but occasional and seasonal variations depending on rains and snow melting was recorded. The conclusion is that the marine clay deposits are drained (maybe from construction of the existing sewer treatment plant), and the ground water table is below the clay deposits. The existing tunnels and caverns are dry.

Vibrations from blasting
The damage risk from blasting was considered to be high due to the close vicinity to buildings and structures. This problem also includes the instruments and sensitive installations in the existing sewer treatment plant which is in full operation during the construction period. A program for inspection of surrounding buildings and a vibration monitoring program was prepared (Figure 4).

Figure 4. Assessment of area influenced by vibrations from blasting.

Vibration limits due to blasting was determined from the Norwegian standard NS8141. Restrictions on blasting operations was prepared and implemented in the tender documents.
The most sensitive installations/structures were identified to be the instruments and pumps in the intake chamber of the existing sewer treatment plant plus the server rack in the basement of the Strømmen college.
The monitoring program to measure vibrations in buildings due to blasting is shown in Figure 5.

Figure 5. Location of instruments for measuring vibration in buildings during the construction stage.

Structural noise
The potential problems connected to structural noise in buildings has been considered seriously, mostly related to the college with only 20m vertical distance from the top of OREA to the foundations of the building. During the planning stage, test drillings from the existing tunnels around OREA were performed to check the noise level in the college. Test results are presented in Figure 6. The measured values were below the accepted levels according to the requirements from the Norwegian guidelines on noise T1442/2012.

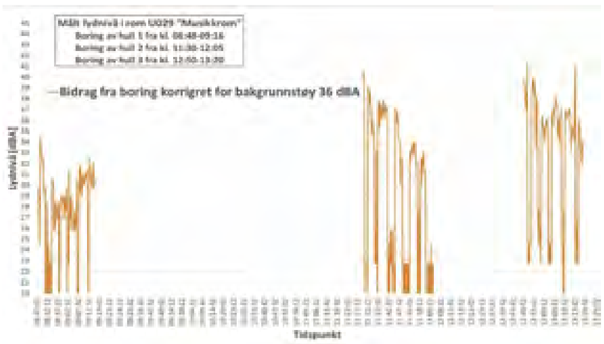


Figure 6. Test results and noise measurements from drilling underneath the Strømmen College.

Noise from ventilation fans
To avoid environmental impact (noise) from the ventilation fans, these were located inside the opening of the access tunnel to the existing sewer treatment plant, with intake of fresh air outside the tunnel opening.

Impact from transport of tunnel muck
To avoid traffic accidents due to logistics and transport (mainly tunnel muck and concrete to rock support), the transport was led in separate routes inside the NRA operating yard. On public roads, transport activities were in accordance to current regulations

2.2 Ground Investigations and Laboratory tests
The ground investigation program included sounding to bedrock, seismic refraction profiling and core drillings. Besides, detailed geological mapping was performed in the tunnel system and rock caverns of the existing sewer treatment plant. The core drillings were performed from inside of the sewer treatment plant in the rock masses of the coming OREA extension (location shown in Figure 8).

2.2.1 Sounding to bedrock and seismic refraction profiling
Exposures of bedrock is very scarce in the area around OREA. Soundings to bedrock were performed along profiles around the Strømmen college and in the schoolyard. The seismic profiles were performed along the same lines to obtain continuity in the bedrock level/soil overburden. Besides, the soundings were used to calibrate the seismic profiles. The boreholes were drilled 3m into solid bedrock to prove the rock surface. The seismic profiling was performed using acoustic sounding by sledgehammer on a steel plate.

Due to the existing buildings, it was limited access to perform the soundings. Acoustic sounding was tried inside/under the basement of the college, but due to concrete walls, there was too much disturbance to interpret the results.

Thus, there were a certain uncertainty related to the bedrock surface and rock overburden when the underground construction works and rock excavation started. Implementation of soundings during construction in the contract was required (Section 3.1).
Interpretation of the results from soundings and seismic refraction profiling, are shown for the main cavern in Figure 7. Red is soil overburden, yellow is difference between soundings and seismic profiles, green is assumed weathered or crushed rock, and blue is assumed solid bedrock.

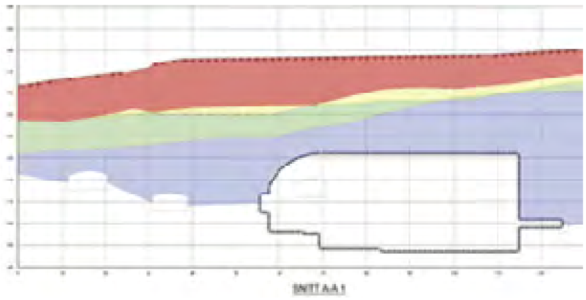


Figure 7. Interpretation of results from soundings and seismic profiles along the main cavern.

2.2.2 Core drillings
During March and April 2016, Entreprenørservice AS performed several core drillings from the existing tunnels in the sewer treatment plant. The core holes were drilled in different directions, lengths and with varying gradients in order to cover most of the area where the future rock caverns were to be excavated. Figure 8 gives an overview of the core drilling program carried out at OREA.

The rock cores were logged to identify jointing and potential weakness zones and hence to draw inferences on the likely behavior and character of the actual rock mass. The core logs indicated a fairly massive rock mass of medium to good quality with a few zones of jointed rock. All core holes were dry and Lugeon-tests were not performed.

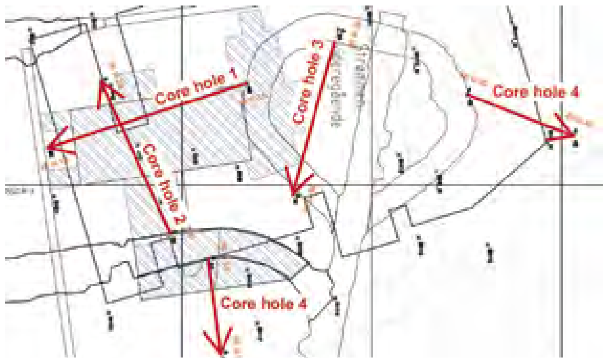


Figure 8. An overview of the core holes drilled at OREA.

2.2.3 Laboratory tests
In order to determine the rock mechanical properties of the Precambrian gneiss, core samples from two different locations at the OREA site were sent to the SINTEF Engineering Geology Laboratory in Trondheim, Norway in April 2016.

Comprehensive laboratory testing was carried out in accordance with the procedures of the International Society of Rock Mechanics (ISRM). The following tests were conducted at the SINTEF Engineering Geology

Laboratory in Trondheim:
- Density (ρ)
- Axial Young's modulus (E)
- Poisson's ratio (ν)
- Uniaxial Compressive Strength (σc)
- Basic friction angle (φb)

Table 1 presents the results from the laboratory tests on the Precambrian gneiss samples collected at OREA.

Rock Mechanical Property	Unit	Value
Density	kg/m3	2800
Uniaxial Compressive Strength	MPa	80
Axial Young's modulus (intact rock)	GPa	52.4
Poisson's ratio	-	0.18
Basic friction angle (φb)	°	27.5

Table 1. Average rock mechanical properties of the Precambrian gneiss at OREA, Strømmen. The results presented in the table are deducted from laboratory tests performed by the SINTEF Rock Engineering Laboratory, Trondheim.

2.3 Rock Modelling and Stability Analysis

To study and assess the stress distribution and stability issues governing the cavern design and rock support, extensive FEM analyses were performed. The FEM analyses were carried out in the comprehensive 2D finite element program Phase2 8.0.

Based on the lack of stress measurements in the planning stage, a sensitivity analysis was performed in order to examine how different principal stresses (stress ratios) influenced the stability of the rock caverns. Cross sections considered to be critical in terms of large spans and limited rock overburden were modelled for varying horizontal stress ratios; k = 0.33, k = 1.0 and k = 2.0, respectively. K = 0.33 was considered a conservative assumption.

According to Hoek et al. (1998) measurements of horizontal stresses worldwide show that the ratio k tends to be high at shallow depths and that it decreases at depth. Figure 9 illustrates how the ratio of horizontal to vertical stress varies with depth below surface. This theory is supported by Myrvang (2001), who stated that the stress ratio k often is >1, and even >>1 in Scandinavia. In the sensitivity analysis performed, k = 0.33 was considered a conservative assumption.

The results from the FEM analysis indicated that we with a conservative stress ratio (k = 0.33) with low horizontal stresses experience failure and stability issues with conventional rock support. The overall stability of the rock caverns was, however, ensured with a stress ratio of k = 1. The assumption of k = 1 was in accordance with practical experiences and the theories of Hoek and Myrvang and deemed a more credible assumption.
In order to verify the actual in-situ rock stresses, stress measurements were carried out during the construction stage.

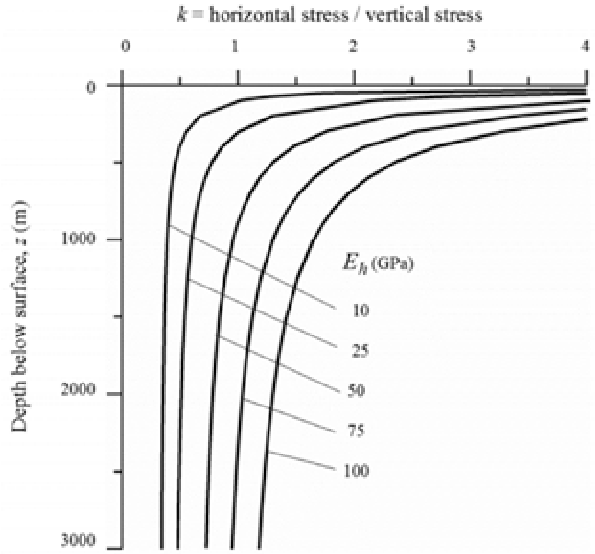


Figure 9. Ratio of horizontal to vertical stress for different deformation moduli (Hoek et al., 1998).

Figures 10 and 11 illustrates how the yielded zone around the rock caverns change with varying stress ratios in the FEM analysis.

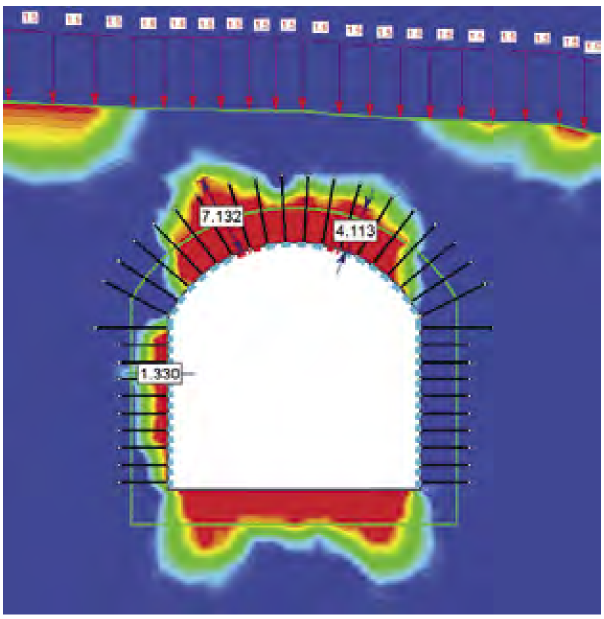


Figure 10. The yielded zone around the contour of the rock cavern with a stress ratio of k = 0.33.

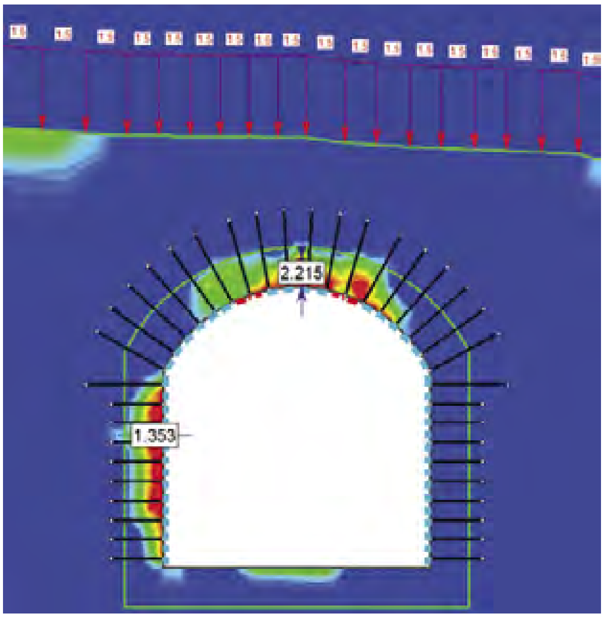


Figure 11. The yielded zone around the contour of the rock cavern with a stress ratio of k = 1.

3 CONSTRUCTION STAGE

All results and conclusions from testing and investigations in the planning stage, were implemented as guidelines, recommendations and requirements to the Contractor during the construction stage. Restrictions were set on noise and vibration levels, and also on the transport activities during, and related to the construction works (tunnel muck, concrete and equipment).
Program for sounding from the tunnel face, specifications and procedures for blasting with full and short rounds, instructions for pilot tunnel in the top heading, instructions for sequential excavation and support etc. were prepared and included in the tender documents. An investigation program was prepared and implemented to close the uncertainties from the planning stage (section 2.2.1).

3.1 Rock overburden and stress measurements

To verify the real rock overburden, extensive probing from the face of the caverns was performed each 10m during excavation. During sounding, drilling data and other vital parameters were collected electronically by using Measure While Drilling (MWD). The data are interpreted continuously, and an example of data output is illustrated in Figure 12.

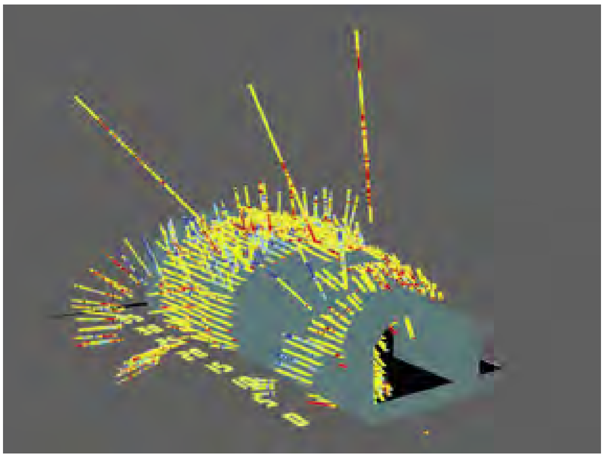


Figure 12. MWD-data from sounding and drilling at OREA.

In order to determine the minimum principal stress in the rock mass, in-situ stress measurements were implemented during excavation of the rock caverns at OREA. A program for hydraulic tests on pre-existing fractures (HTPF)/hydraulic jacking were developed in selected boreholes. The tests were conducted in boreholes drilled from the walls of excavated tunnels, Figure 13. The boreholes were drilled approximately horizontal and the principal rock type in the borehole was Precambrian gneiss with a foliation striking N-S.

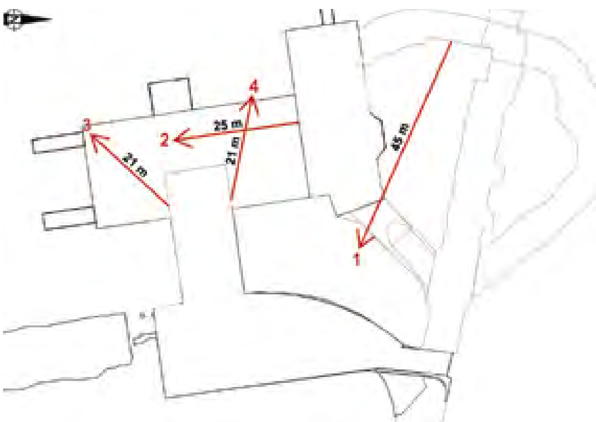


Figure 13. An overview of the drilled boreholes where the hydraulic jacking tests are conducted.

The hydraulic jacking test was essentially a cyclical Lugeon test with a single packer. Pressure was increased in steps until the flow rate rapidly changed due to jacking of existing joints in the rock mass. The cycle was repeated a minimum of 3 times.
Preliminary test results indicated significantly higher in-situ stresses than anticipated, which substantiate the assumption of a higher stress ratio than k = 0,33.

3.2 Tunnel excavation and rock support
The rock excavation is by conventional drill and blast (D&B- figure 14).

Figure 14. The rock excavation at OREA is by conventional Drill and Blast (D&B).

Due to the shallow rock overburden and great span width of the main cavern, the Contractor was instructed to divide the top heading in pilot tunnel and side benching. The top heading was divided in two halves. The drill pattern and charging plan for the pilot tunnel are illustrated in Figure 15.

Figure 15. Drill pattern and charging plan for the pilot tunnel of the top heading in the main rock cavern.

To reduce the vibrations, the Contractor used blasting with one hole per interval and short rounds when required. With few exceptions, the vibrations have been below the required levels.

Figure 16. Charging the next round in the access to the intake chamber.

Rock support is depending on rock quality and span width of the rock cavern. As minimum, systematic rock bolts 2x2m and 0,1m sprayed concrete has been installed in the roof, and rock bolts as a minimum in the walls. So far, there has been used 6m anchored and grouted rock bolts, c-c 1,5x1,5m in the roof and 4m similar bolts with pattern 2x2m in the walls. At each opening of the caverns, 6m Ø32mm spiling bolts were used around the arch of the openings. In addition, it was also prepared for reinforced sprayed concrete arches if the rock quality was poor and/or the rock overburden was shallow. For design of rock support, the method of observation and the Q system was adopted.

3.3 Breakthrough of the intake and outlet tunnels
Breakthrough to the existing sewer tunnels was another great challenge. These were in full operation. The tunnels were excavated by conventional drill and blast, until 2m of the tunnel remained. The remaining plugs were removed by rock sawing around the periphery, and the plugs then pulled out (Figure 17 and 18).

Figure 17. Preparing for breakthrough to the intake sewer tunnel – in full operation.

Figure 18. Breakthrough to existing intake tunnel with wire sawing.

4 SUMMARY AND CONCLUSIONS

Great environmental as well as technical challenges related to shallow overburden and great span width were expected during planning of the OREA sewer extension. From the Environmental Impact Assessment, the following items were addressed:

- Vibrations due to blasting
- Structural noise from drilling
- Traffic safety due to transport activities
- Existing plant in full operation

During the planning stage, rock modelling and stability analysis were performed. In situ rock stresses were found to be a critical parameter. Sensitive analysis was performed, and k=0,33 was found to be critical. In situ measurements proved that the horizontal stresses are higher, and the rock support with systematic rock bolts and sprayed concrete are sufficient. It is also prepared for reinforced sprayed concrete arches if necessary. The philosophy of the Norwegian Tunneling Method has been used.

So far, the vibration and noise levels have been below the requirements, and the stability of the excavated caverns and tunnels is good. Deformation measurements so far, indicate no deformations.

Thanks to an excellent cooperation, professional and solution-oriented attitude between the Client, Contractor and the Consultant, the rock excavation of OREA has been a great success for the involved parties.

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5.4.2 NEW OSET WATER TREATMENT PLANT FACILITIES SITUATED UNDERGROUND

Establishing water treatment installations in excavated rock caverns underground has become a common practice in Norway. Provided that a suitable location with adequate rock quality for excavation of larger tunnel spans exists, an underground location of such installations has many advantages compared to an aboveground location. Besides the contribution to cost reduction and net positive environmental impact, the underground location has a huge security advantage considering outside threats and possible terrorist attacks as well as protection against contamination. By utilising the underground, the facilities may be located near to the raw water reservoir, near or under city centres or urban areas. Underground facilities can offer steady conditions to the treatment processes and construction work without major foundation problems and independence of surface structures.

These types of facilities are widely used in the Nordic countries of Norway, Sweden and Finland. At the urban border of Oslo City, a water treatment facility was constructed in rock in the near vicinity of the existing main treatment plant for Oslo Municipality, also situated underground. The new plant consists of two main caverns with a length of 150 m, and several access tunnels. The span of each cavern is 27 m and the height varying between 16 m and 20 m. The pillar between the caverns is 17 m wide. The blasted solid rock volume is 140.000 m3. Combination of optimisation of large spans, compacting of the underground complex, minimal rock cover and the process plant design in the early pre-construction phases has been one of the major challenges for the project. The new plant will have a capacity of 390 000 m3 of potable water per day. The commissioning date for the new plant is scheduled in May 2008.

INTRODUCTION

Situating water treatment installations underground in excavated rock caverns has become common practice in Norway for water works serving large municipalities or inter-municipal collaboration projects.

Provided that a suitable location and adequate rock quality for excavation of larger tunnel spans exists, an underground location of such installations has many advantages compared to an aboveground location. Besides the contribution to cost reduction and net positive environmental impact, the underground location has a huge security advantage considering outside threats and possible terrorist attacks as well as protection against contamination.

By utilising the underground, the facilities may be located near the raw water reservoir, near or under city centres or urban areas. Underground facilities offer steady conditions to the treatment processes and construction work without major foundation problems and independence of surface structures.
Oslo Municipality has four water treatment plants in operation today. Oslo Municipality, Water and Sewage Works are responsible for the operation. The two major plants, Oset and Skullerud, are both situated underground in excavated rock caverns.

In 2003 it was finally decided to renew and enlarge the water treatment plant at Oset. The consortium AFS – Krüger (consisting of the Norwegian construction company AF Spesialprosjekt AS and the Danish company Krüger A/S) built the New Oset Water Treatment Plant on behalf of Oslo Municipality. The plant was designed and constructed as an EPC contract (turn-key), where the Norwegian consulting company Norconsult AS was the responsible civil and structural engineering consultant.

The New Oset Water Treatment Plant was at the time of completion Scandinavia's largest water treatment plant and Europe's largest located in excavated caverns. It is designed for a production of 390.000 m3 potable water per day; equivalent to 4,5 m3/sec and will deliver water to 85 % of Oslo's inhabitants.

THE ORIGINAL PLANT

The original Oset water treatment plant is located in several rock caverns east of the main drinking water reservoir for Oslo Municipality, Lake Maridalsvannet. The construction works started in August 1966 and commissioning of the plant took place in September 1971. The plant covers a total floor area of 28.800 m2. It consisted of 5 parallel caverns with spans up to 13.2 m and a pillar width between the caverns of 14 m (Figure 1 and 2). The longest caverns are 92 m long with a height of 16 m. The caverns were excavated by drill and blast technique with a top gallery of height 8 m and two subsequent benches of 4 m each. The drilling rigs had 4 drills, which were handled by a working crew of 3 men each shift. The excavation rate was 500 m3 of solid rock each working day. Totally excavated volume was 344,000 m^3.

The caverns are lined with concrete arches spanning from abutment to abutment. The concrete arches are dimensioned for minor rock falls.

THE NEW PLANT

Background

The decision to enlarge and renew the water treatment facility at Oset was made by the approval of Oslo Municipality's Water Supply Master Plan in 2003. The New Oset Water Treatment Plant should comply with the new Master Plan's safety requirements by dividing the new plant into two completely separated treatment facilities, which together replaces the old one, but uses some of the old caverns for new treatment purposes.

Oset Water Treatment Plant, which today produces approximately 85% of the potable water consumed in Oslo, has its raw water supply from the nearby Lake Maridalsvannet. The plant shall be upgraded to meet the new and stricter requirement set by European and Norwegian drinking water regulations. That includes the running of the treatment process through two hygienic barriers.

WATER TREATMENT AND USE OF ALTERNATIVE SOURCES FOR RAWWATER SUPPLY

The new plant treats the water by use of the Actiflo™ process, which in combination with UV-disinfection gives good hygienic assurance and maximum flexibility with a view to optional raw water sources.

The New Oset Water Treatment Plant was completed and ready for commissioning in 2008. The completion of the project will result in two independent water treatment plants, which have an optimum close co-localisation in rock with a total treatment capacity of 390,000 m3 water per day. Two minor existing treatment plants in the Oslo area will render superfluous with the commissioning of the new plant.

The two new independent plants will, from a process technically view, be built in a way that allow for replacement of vital components while the plant is running without reducing the production capacity. Both plants shall be able to handle and treat water from both the Maridalsvannet Lake and another possible water source concurrently, and with optional proportion of mixture between the sources.

GEOLOGICAL CONSIDERATIONS AND ROCK ENGINEERING ASPECTS

Both the existing and the new water treatment are situated in the same geological formation. The predominant rock type is a local variation of an alkali-syenite commonly found in the Oslo area. Weakness zones and joint systems are typically steep and have strike in the North-South direction. Other observed joint systems are sub-horizontal. Intrusions of diabase and syenite-porphyry appear at regular intervals as lenses in the rock mass often in parallel with main joint system direction.

Dimensions and spans of the underground excavations were governed by the overall desire to make the new treatment plant compact. Requirements given in the contract were that the stability of the existing plant should not deteriorate, and as a guideline it was suggested that the pillar width should not be less than 2/3 of the cavern span and as a minimum requirement not less than 10 m. Large cleaning filters that were to be mounted across the cavern cross-sections were dimensioning the minimum spans. This gave in the first place spans in the magnitude of 30 m. Due to several crosscuts between the caverns it would be cost improving to minimise the distance between the caverns.

The exact location and geometrical layout of the plant was governed by the requirement of best possible cooperation with the existing treatment plant. In order to ensure good access and straight connection axes to the existing plant, the new plant was located just North of the existing and as close as possible towards the hillside surface coming up from the Maridalsvannet raw water reservoir (Figure 3).

Final layout and location was considered based on rock cover, rock stress conditions and intersecting weakness zones (Figure 4). The chosen location was considered favourable with regards to predicted water ingress and possible negative environmental impact on the two small lakes, Småvannene, situated to the East at the top of the hill.

Contractual upper limit for tolerable water ingress to the entire new underground complex was set to 100 litres/min.

CAVERN LAYOUT AND PILLAR WIDTH

In order to minimise the length of crosscut tunnels both numerical and analytical analyses has been utilised to optimise the design. For the numerical analyses the 2-dimensional program Phase2 ™ (Finite Element Method) has been utilised. The model is shown in Figure 5. Analyses have been run with three different rock mass quality parameters and three different pillar widths.

Both the analytical and the numerical analyses showed that a pillar width of 15 m (Figure 6) gives satisfactory global stability for both the existing and new underground facility. These findings applied also for abnormally unfavourable rock stress conditions and rock mass quality poorer than what were expected from the pre-investigations. In the final cavern layout the pillar width has been set to 17 m in order to allow for overbreak in the contour.

The analyses also showed that rock bolt lengths of 4 m in the pillar and 6 m in the crown were sufficient to maintain the stability of the underground opening.

GEOLOGICAL FOLLOW-UP DURING CONSTRUCTION

The contractor's experienced engineering geologists performed continuous mapping of geological structures and rock mass conditions in parallel with the excavation progress. As a requirement from the Client, the mapping was conducted in accordance with the Q-system and the result of the geological mapping is shown in Figure 7.

Dependent upon the actual observed Q-value, rock support was applied in accordance with 5 pre-determined rock support classes (Figure 8).

The registered rock mass quality had the following distribution:
Rock Support Class SK I Q > 40 smaller portions of the transport and access tunnels

Rock Support Class SK II Q = 10 – 40 ca 70 % of the caverns

Rock Support Class SK III Q = 1.0 – 10 ca 30 % of the caverns close to weakness zones

Rock Support Class SK IV Q = 0.1 – 1.0 only occasionally

Rock Support Class SK V Q < 0.1 not registered

A total of 3,500 m3 of steel fibre reinforced sprayed concrete and 6,000 CT-bolts has been applied as permanent support at the face in crown and walls of the underground facility.

ROCK EXCAVATION

Excavation Methodology

The two caverns have been excavated by drill & blast technique with an approximately 9 m high top gallery. The gallery was sub-divided at the middle of the 27 m wide cross-section with a pilot face one blast round (5 m) ahead of the slash face in a one-way excavation direction from the access tunnel side. The remaining bench was excavated in the same direction as the gallery by means of horizontal drilling and blasting. The caverns were excavated in parallel with the transport tunnel. This methodology gave in fact 5 tunnel faces, which gave sufficient flexibility for the pre-grouting works to go on independently of the rock excavation, as there were always 3 faces to choose between for drilling, mucking out or rock support works.

Control of Blast Induced Vibrations

Strict blasting vibration requirements applied for the performance of blasting operations. The limits set in the contract was a PPV of 25 mm/s for the concrete structures and a PPV of 20 mm/s for the electrical switchboards in the existing treatment plant. The typical rock type in the neighbouring area towards the existing plant was relatively stiff and transmitted the wave propagation induced by blasting well. The existence of unfavourably orientated weakness zones in the pillar rock mass resulted in relative displacement and aggregation of vibrations at certain locations.

The main challenges related to vibration control were therefore blasting close to the existing plant. All blast rounds less than 30 m away from the structures inside the existing plant were performed with a high degree of scattering in detonator numbers in order to reduce the charge per delay interval and use of connector blocks to subdivide the delays further. Explosives in tube cartridges were commonly used at the most critical locations.

PROJECT IMPLEMENTATION

Process Plant Design and Rock Excavation in Parallel - Experiences And Challenges

Rock excavation for the underground facilities started up prior to the completion of the process plant design. This situation was especially challenging during the rock engineering design phase when optimisation of the project implied aiming for a compact underground facility with minimised rock excavation volumes at the same time as the process plant design was in the early stages of basic conceptual design.

The experience gained is that the underground excavations could have been slightly larger in order to facilitate more design freedom for the process plant and other disciplines with regards to optimised technical and economical solutions.

Rock Excavation close to Plant in Operation - Experiences and Challenges

The new plant is situated adjacent to the existing plant with a constant distance of 25 m to the closest cavern. The portal area for the access to the new plant was situated less than 10 m above the main existing raw water supply tunnel in full operation. In addition two major weakness zones were intersecting in the same area. At three locations crosscut tunnels were to be excavated into the existing plant in full operation.

Maintaining the operation of the existing plant was one of the Clients main concerns and a stop in, or contamination of the water production causing supply cut-off for the consumers was therefore subject to a hefty fine of 5 MNOK per undesired event. So far the ongoing construction works have not caused any interruption in the water production.

Water Control by Rock Mass Pre-grouting - Experiences

The rock cover above the caverns varies between 20 m near the entrance to 70 m in the inner part of the plant. Systematic rock mass pre-grouting has been performed throughout the entire plant. Only cement-based grouting material has been used.

In the tunnels a standard procedure for pre-grouting involving drilling of grouting fans consisting of 21 m long holes densely spaced around the whole profile with a look-out of 5 m for every third blast (distance 15 m) has been followed.

In the caverns all pre-grouting has been carried out from the top gallery. This caused a large look-out angle in the bottom of the fan in order to reach and cover the rock mass below the lower bench. Due to the large cross-section the fans were also vertically divided into two grouting rig positions enabling the excavation work to start at the first grouted side of the face while grouting was completed at the other.

The contractor AFS in close dialog with the Client's representative elaborated all grouting mix design. Due to a rock mass containing a high portion of very fine fissures all grouting rounds were started with micro-cement and if pressure build-up was achieved the round was completed with micro-cement. In the case of no pressure build-up after a pre-determined grout take amount was reached, the round was completed with rapid cement and possible use of controlled grout setting by means of accelerator added at the nozzle.

The stop criteria utilised through out the project was a modified GIN-principle criterion with maximum pressures of 40 bars in areas with little rock cover or in the vicinity of the existing plant and 65 bars elsewhere.

In certain zones, often related to lenses of diabase (black zones in Figure 7 and zones marked VIII and IX of Figure 4), water leakage from the face, prior to grouting, was reported as high as 100 litres/min. Control measurement of water inflow to the whole new underground complex after completed excavation shows water ingress of in total 27 litres/min.

Compared to the pre-set requirement of maximum 100 litres/min, the result of the grouting operations is considered to be more than satisfactory. Incidentally, the rock mass grouting works were the only part of the contract that was quantity and unit-price regulated. The total grouting figures amounted to 2,300 tons of cement, which correspond to a grout take of 16 kg per m3 of excavated rock.

Rock Mass Deformation Monitoring - Experiences

In order to document “stable-state” conditions in the caverns, displacements in the crown were measured by means of installed measuring bolts in monitoring stations every 30 m. The exact position of the measuring bolts was measured with the surveying total station.

Now, 11 months after the excavation work was completed and some 16 months after the monitoring stations were installed, “stable-state” conditions has been observed over a 15 months long period. The displacements measured are small and within the range of inaccuracy for this optical monitoring system.

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5.5.1 TUNNELS FOR NORWEGIAN RAILWAY PROJECTS

General introduction

The bedrock in Norway has been subjected to erosion through a series of ice ages. Areas with many fracture, fault or crushed zones and pockets of less resistant rock types, intrusives or diabase dikes have undergone more erosion than areas with homogenous hard rock types. As a result, the rock surface has been shaped into high peaks and deep valleys. After the last ice age, the ocean flooded areas that lie above sea level today. Marine sediments, mostly clay, were deposited over several thousand years in the valleys that were under water, and in some places the thickness of these clay sediments is substantial. Since this sedimentary deposition, land uplift has occurred and a number of these valleys or troughs that are filled with thick layers of clay now lie above sea level. This is a condition we find at a number of places along the Oslo Fjord and the Trondheim Fjord.

If these fractured zones cross a tunnel or other underground facility, they may act as drainage channels (Kalager, 2016) and cause a reduction in the pore pressures in the clay sediments above and to the side of the tunnel, resulting in settlement of buildings and structures built with their foundations on these soil-like masses.

Construction of the Holmenkollen Line in the centre of Oslo

The first time this problem was encountered on any significant scale was during the construction of the Holmenkollen Line, a metro or underground rail tunnel through the centre of Oslo. (Karlsrud, 1981). Construction work started in 1912, and the tunnel was excavated by blasting. Several water-bearing weakness zones were encountered and the project had to cope with substantial leakages in a number of areas. The leakages resulted in relatively large settlement damage not only to the buildings that were above and immediately to the side of the tunnel, but also to buildings located within the influence area of the clay-filled troughs in question. This area could in some cases extend as far as 500 metres to the side of the tunnel. After excavating about 870 metres, the work was stopped in 1914.

Attempts were made to seal the tunnel by installing a concrete lining, but the leakages and settlement at the surface continued. At the time there were much debate and technical disagreement among the experts as to whether the leakages really were the direct cause of the settlement and damage to buildings. Evidence of a direct connection was documented in 1919 through the first dilatometer tests on clay that are described in geotechnical literature. (Statens Järnväger, 1922).

Work to excavate the rest of the 1.4km-long tunnel was resumed in 1926 and completed in 1927. This stretch of the tunnel also crossed beneath several clay-filled troughs, resulting leakages through fractured zones and substantial settlement damages to buildings above and to the side of the tunnel.

The Bergen Line

However, the first long regional link built was between Oslo and Bergen. The Bergen Line was originally the 485km-long railway between Oslo and Bergen via Drammen. The line opened in 1909 and cost in the order of one annual state budget at that time. In connection with the works, a 120km-long construction site access road, the Rallarvegen, was built over the roadless mountains between Flåm and Haugastøl. The construction of the mountain section of the line was a huge challenge. It had to be laid in inhospitable terrain high above sea level, in places with no roads and where in the winter there were several metres of snow. The construction of the 5,311-metre-long Gravhals Tunnel was particularly difficult. This was the longest tunnel in northern Europe at that time, and three shifts worked every week for six years in order to complete it. To provide power for this part of the project, two power stations were built, one at Gangdalsfossen west of the tunnel, and one, somewhat smaller, at Kjosfossen east of the tunnel where the Flåm Line runs now. The whole line between Oslo and Bergen had 182 tunnels, with a total length of 73km. The longest tunnel on the Bergen Line today is the Finse Tunnel, with a length of 10,300 metres.

Construction of the Oslo Tunnel – a railway tunnel crossing the centre of Oslo from east to west

The Oslo Tunnel, a double-track railway tunnel, which also includes the National Theatre underground station, was built in the period 1973 – 1980. The tunnel runs essentially through the same geological formations as the Holmenkollen Line. Based on the experience of the problems caused by the construction of the Holmenkollen Line and later experience with deep excavation in the centre of the city, those responsible for building the tunnel were prepared to deal with leakage problems when crossing troughs with weakness zones and intersecting water-bearing diabase dikes (Karlsrud, 1981).

The tunnel was excavated by blasting and then had a concrete lining installed along its full length. To minimise leakage during the construction work, the first step taken was sporadic pre-grouting, based on probe holes and water loss measurements ahead of the working face. Grout was injected at relatively low pressure. However, this approach did not give satisfactory results, so a change was gradually made to systematic pre-grouting with overlapping screens. Post-excavation grouting was found to be difficult and gave little effect.

Despite an extensive grouting programme, it was not found possible to seal the tunnel in a satisfactory manner until an attempt was made to grout at a higher pressure, 30 – 40kg/cm2. The leakages were then reduced. But it was also found that some of the grouting material penetrated to the surface.

After the concrete lining had been installed, there were still large leakages at some points in the tunnel. The leakages were primarily concentrated around the joints. Contact grouting with cement was performed, and all voids between the rock and the concrete structure were filled before a second round of high-pressure grouting was executed. This gave good results, and the tunnel today is as good as dry along these sections where large leakages were encountered during the driving of the tunnel.

In this project, to compensate for the reduction in pore pressure as a result of leakages into the tunnel, a number of attempts were made to infiltrate water into the ground. Infiltration from the surface caused least disturbance to the tunnel excavation work. However, infiltration directly into the soil-like masses or in the transition between such masses and rock was found to result in washout and, over time, local collapse in the soil-like masses. The rock was also relatively impermeable, so infiltration directly into it had no effect either. Cutting off water-bearing diabase dikes and crushed zones with connection to the troughs crossed by the tunnel gave good results in a number of areas. Pore pressure stopped falling and started to rise, whilst further subsidence of the surrounding area ceased.

In the case of this project too, the leakages resulted in a reduction in pore pressure in the troughs where water-bearing zones or crushed zones crossed the tunnel alignment. Buildings here suffered settlement, but since control was gained over both leakages and reduction of pore pressure through trying different methods for grouting and sealing the concrete and by using water infiltration as a temporary remedial measure, the scope of the damage was somewhat reduced compared to what had been the case during the construction of the tunnel for the Holmenkollen Line in 1912 – 1914 and 1926 – 1927.

Extension of the National Theatre station

From 1996 to 1999 work was done to extend the National Theatre station. See Figure 1.

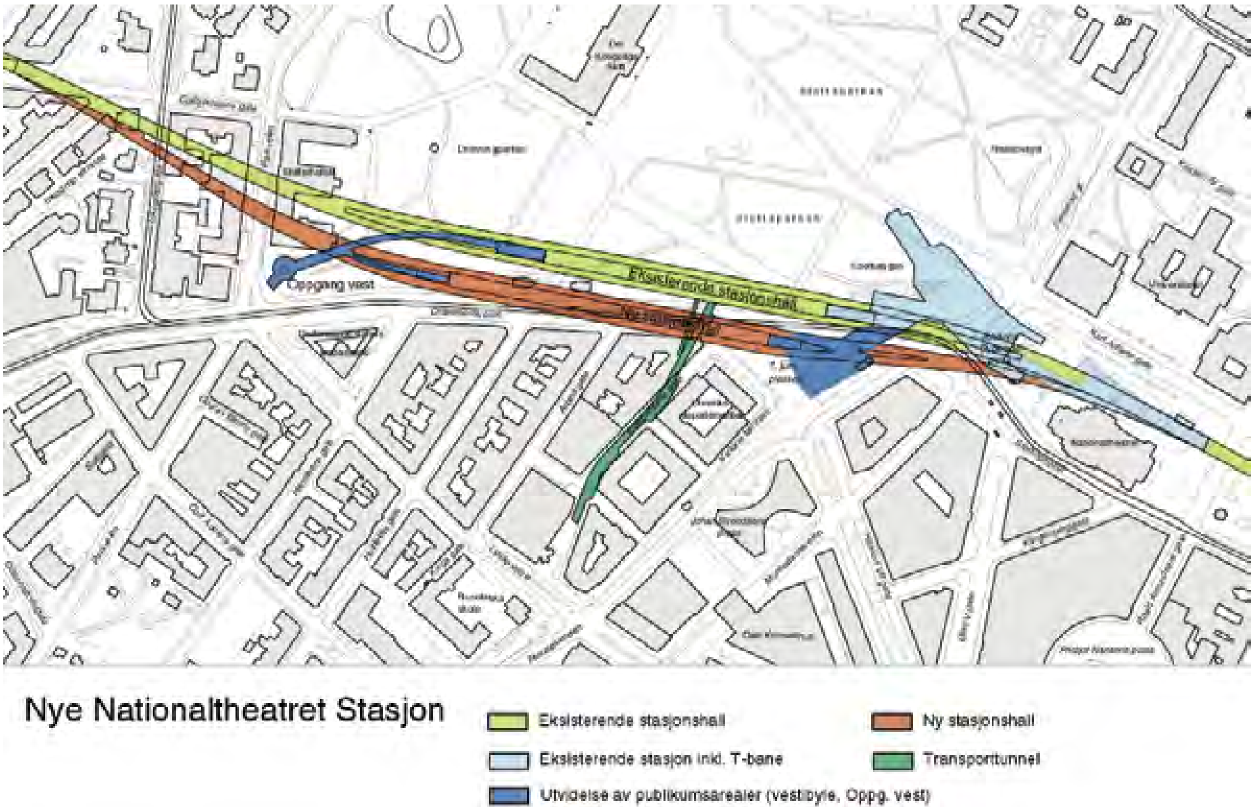

Figure 1. The new National Theatre station (red indicates the new station in a rock cavern, green and blue are existing caverns, whilst dark green and dark blue are extension and service facilities)

This was done by branching off from the existing Oslo Tunnel in the concrete tunnel east of the existing station, building a 250-metre-long new station rock cavern with two tracks and a platform between them parallel to the old station cavern and a link to the rock tunnel west of the station. Maximum width of the station cavern is 22 metres and the distance between the old and the new station caverns is about 10 metres. In the east, the works consisted of extending the lower level in the existing concrete culvert, the upper level of which houses the metro. This work took place close to the National Theatre building. The existing concrete tunnel was located in soft clay but constructed on a foundation of piles driven into rock.

The whole station cavern and connecting part in the west was located in rock, but with an overburden of as little as about 3 metres. The new tunnel, including the station cavern, has a total length of 1km.
The tunnel itself crosses a couple of the troughs that were known from the construction of both the Holmenkollen Line and the Oslo Tunnel, where there was a potential for leakage. To prevent damage to the surrounding buildings, a comprehensive monitoring system was planned ahead of the project in combination with systematic pre-grouting, installation of a watertight concrete lining and water infiltration contingency plans during the construction process.

A comprehensive pre-construction condition survey of all buildings within a possible influence area of the tunnel was conducted. See Figure 2.

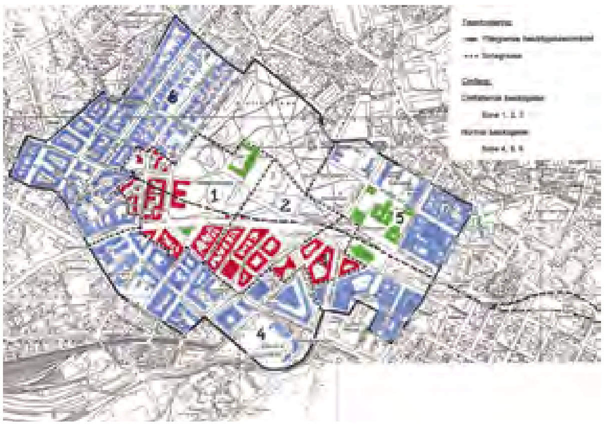

Figure 2. Area in which pre-construction condition surveys were conducted

These surveys included levelling all the buildings ahead of construction start. Follow-up of leakages, pore pres-

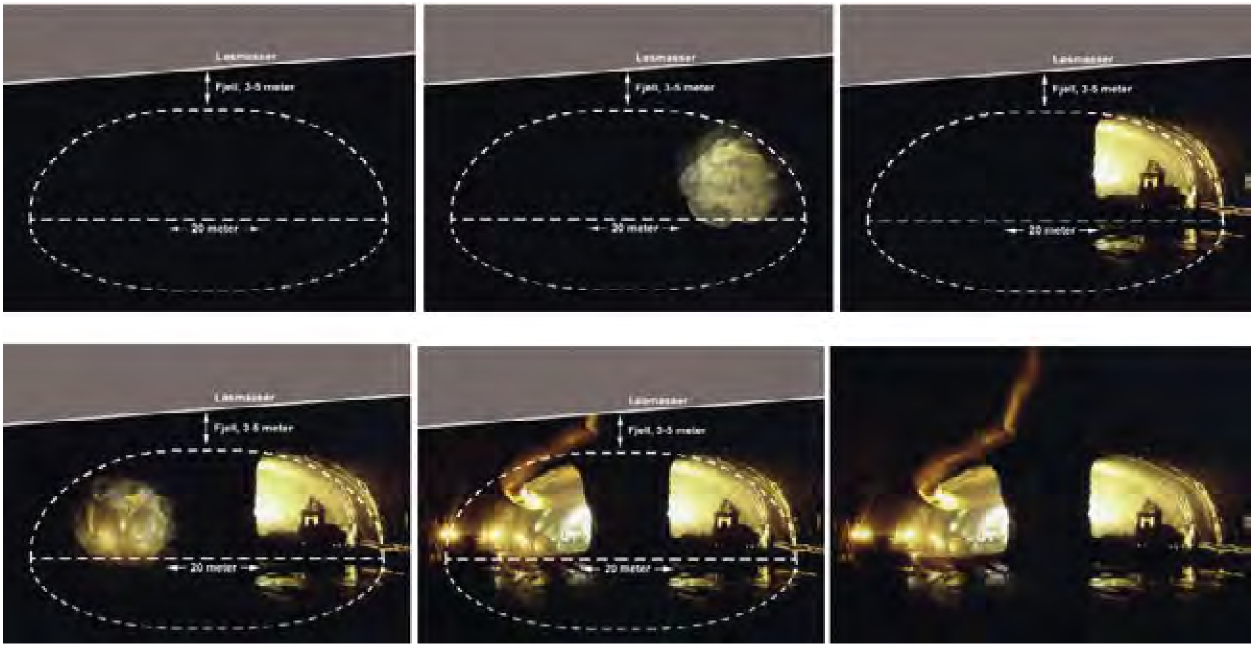

Figure 3. Staged excavation of the new cavern for the station

sure and settlement was done on a continuous basis throughout the construction period. Where leakages occurred, water infiltration was used as a temporary measure to compensate for ingress and prevent a drop in pore pressure in the soil-like masses.
The station cavern was blasted using reduced rounds and in sections without closing the old station cavern. See Figure 3.

The connection to the rock tunnel in the west was demanding, and it was for the most part carried out whilst there was traffic in the existing tunnel. A steel shield was established in the tunnel between the concrete cladding and the track. Excavation work towards this concrete vault was then done partly by blasting and partly by back ripping from the rear side. In the transition, support was installed in both ceiling and floor on both sides of the concrete structure before an opening towards the existing tunnel was made. See figure 4.

The project was carried out without any significant damage to either buildings or surrounding structures. The positive experience with the execution and follow-up methods used in the extension work on the National Theatre station was repeated later with good results in the construction of three new double-track tunnels west of Oslo.

The Follo Line – A new double track under construction between Oslo and Ski
The Follo Line consists of a 20km-long tunnel. Owing to the length of the tunnel, and in consideration of safety, future operation and maintenance of the railway and robust train operations, this tunnel is being constructed with two separate tubes.

At an early stage, it was decided that the innermost section towards Oslo Central station, about 1.5km, where there is no room to operate a tunnel boring machine (TBM) and there is also close proximate routing to other sensitive infrastructure, should be excavated partly by careful drill and blast and partly by drill and split. (Kalager, 2016) See Figure 5.

In this part of the tunnel section, the works will involve building of two tubes for the Follo Line and a single tube for the realignment of one of the two tracks of the existing Østfold Line.

The distance between the existing E6 highway tunnels and the ceiling of the future railway tunnels is as little as about 4 metres. A rivertunnel located between these tunnels is required to be reinforced before the excavation of the railway tunnels take place. This excavation is done by careful drill and blast in combination with drill and split methodology.

Both the single tube of the realigned Østfold Line and the two Follo Line tubes also run close to several large unlined rock caverns for storage of petroleum products. Water curtains prevent leakage of hydrocarbons into the surrounding rock. To avoid vibrations and disturbances of this facility, excavation work in this area is also performed by the use of drill and split methodology.

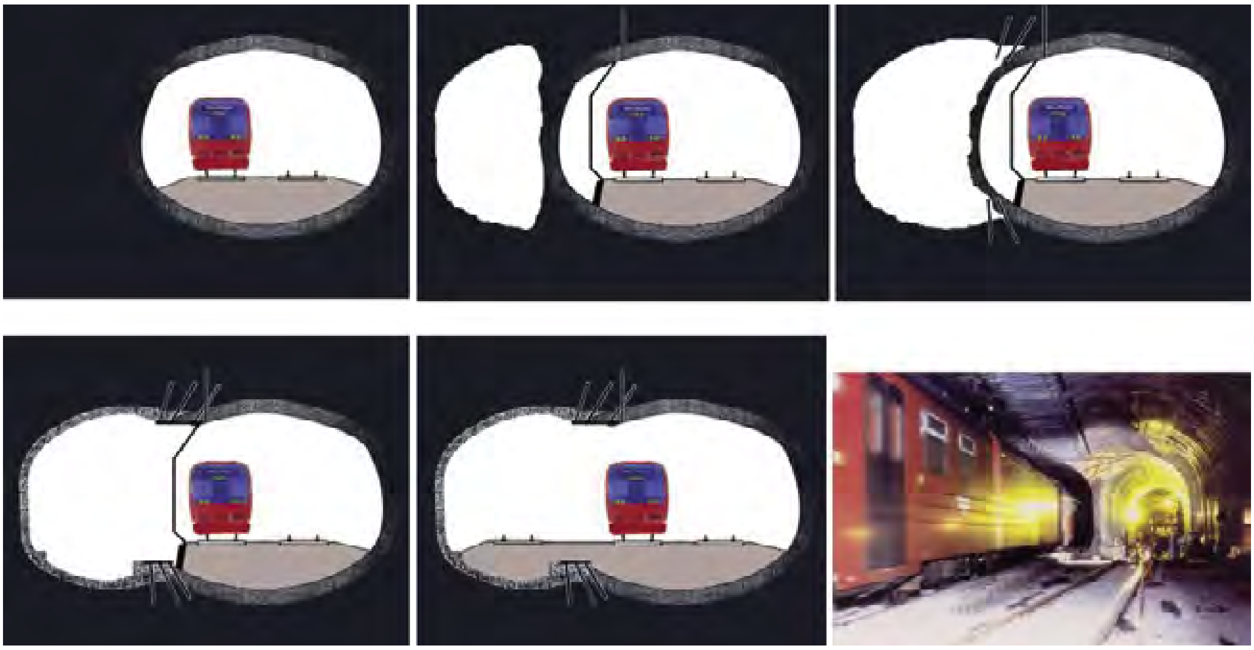

Figure 4. Connection between the new and the existing tunnels

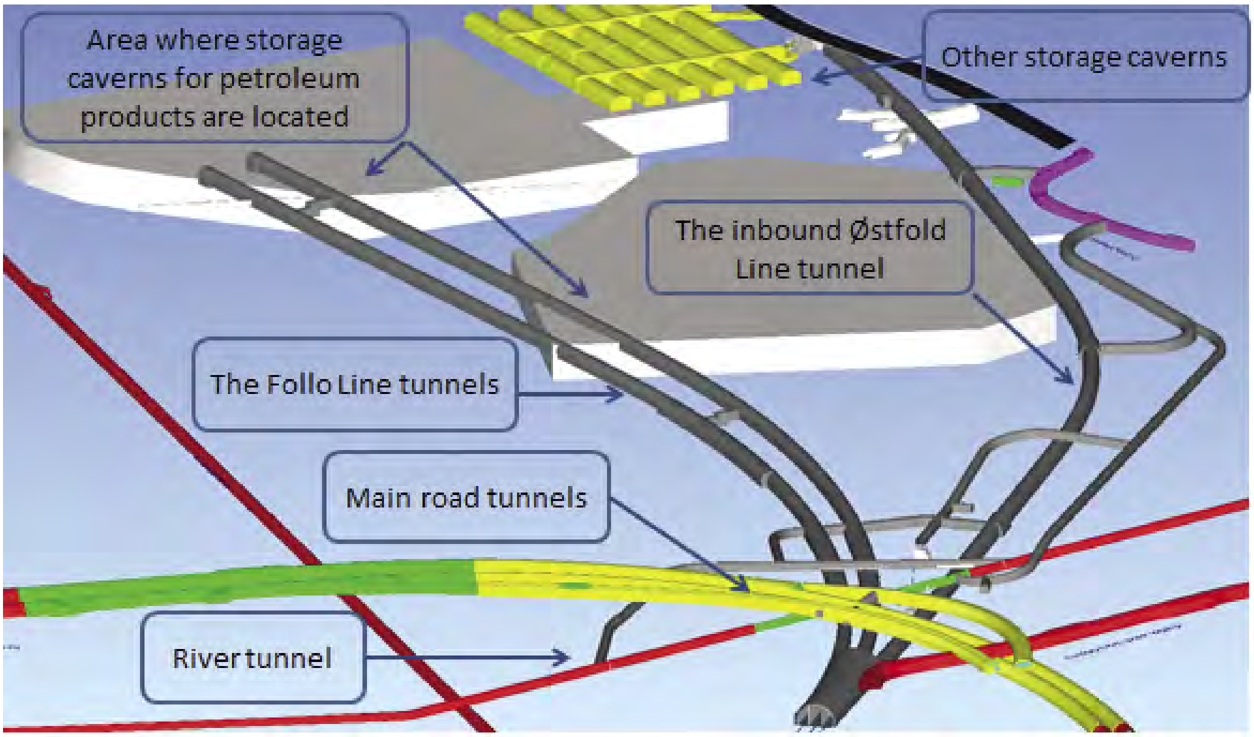

Figure 5. A spaghetti of tunnels in the Ekeberg hill

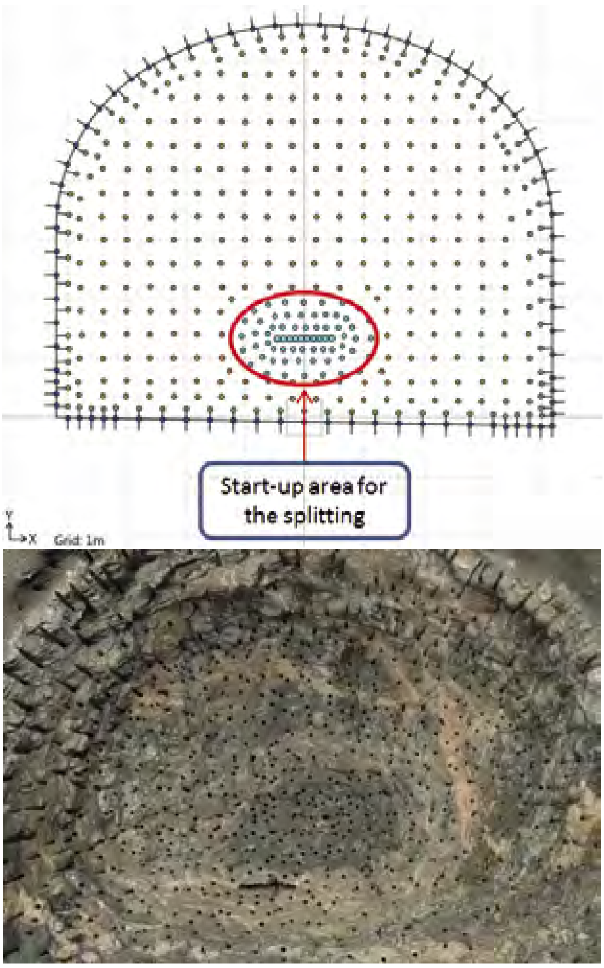

Figure 6. Drill and split

To execute the drill and split method, a dense network of holes, about 2 metres long, is drilled. About 500 holes are drilled within a cross-section of about 70m2, after which the rock is split by hydraulic wedges. See Figure 6.

The rest of the tunnel, 18.5km, is to be excavated using four tunnel boring machines (TBM) from one compact and centrally located rig area, with direct access to the main road. This rig area is large enough for many of the activities related to the tunnel production to take place "in house" within this area. This also includes a large area for deposit of the excavated material.

The tunnel is being clad internally with a 40cm-thick concrete lining. This construction constitutes a closed ring consisting of seven elements with water-tight gaskets between them. The concrete elements are installed continuously as the four double-shield machines advance. Two machines are working their way towards Oslo whilst the other two are tunnelling in the direction of Ski.

The TBMs were assembled in two huge assembly caverns that were blasted in the rock in advance. Two access tunnels of about 1km in length and a number of logistics tunnels for use during the construction process have also been excavated. This part of the tunnel system will later function as an emergency area when the railway is put into operation. See Figure 7.

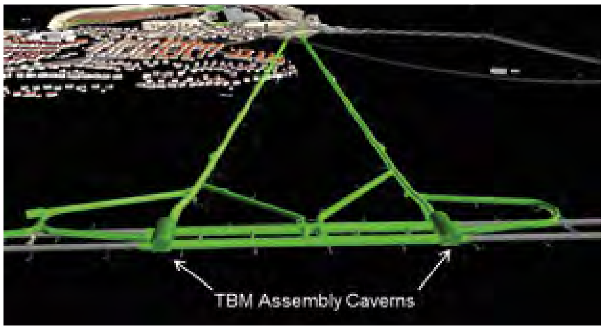

Figure 7. Assembly caverns and access tunnels to the TBM start-up points in the Follo Line Tunnel

All the concrete elements are produced in the rig area and taken directly from the storage area outside the factory and into the tunnel for installation.

The extracted masses are being used for the filling and landscaping of a future housing development site. Both the owner and the contractor have their offices in the rig area, and at any given time about 450 workers also have their living quarters there. By gathering all these functions in one place, it has been possible to eliminate a great deal of transport to and from the construction area. See Figure 8.

Figure 8. Rock dump area

The successful experience with pre-construction surveys of both buildings and the environment and close follow-up of any leakages, changes in pore pressure and subsidence during tunnelling, has also been repeated in this project, even though here it is basically TBMs that are being used to excavate the tunnel instead of conventional blasting. The TBMs are specially designed to be able to drill into the hard Precambrian gneisses that are found along this stretch. Like the bedrock in the centre of Oslo, these are in places penetrated by water-bearing crushed zones and intrusives. Leakages may occur wherever the tunnel tubes cross these zones. Pre-grouting can be done from the TBMs, which will minimise leakages, but there is a major difference in the handling of leakage water in the execution method chosen for the Follo Line and in the execution method developed for conventional tunnel excavation.

As the Follo Line tunnels are being drilled with double-shield machines with the installation of a watertight concrete lining immediately behind the shield, the tunnels will only be exposed to the various leakage points/areas for a relatively short time. As soon as the concrete elements have been installed and the space between them and the rock has been backfilled with cement, the tunnel is sealed and the construction forms an undrained solution. In conventional excavated and drained tunnel solutions, it is necessary to operate with more strict sealing requirements in connection with the pre-grouting work, which often requires considerably more extensive injection.

In the Norwegian tunnelling society there has always been a focus on sharing experience and by trial and error further developing methodology for safe and efficient excavation of tunnels and caverns adapted to the conditions on site.

Other projects in progress or completed

Other major projects in which tunnels form a large part include the stretch Farriseidet - Porsgrunn on the Vestfold Line, which has just been completed. This project involved the construction of seven railway tunnels with a total length of 15 kilometres. A joint road/rail project (new double track and four-lane motorway) along Lake Mjøsa has seen the excavation of three railway tunnels, totalling 4730 metres. Another project has been the stretch Holm - Nykirke where a 12km-long tunnel has been driven as well as a large rock cavern that is to house the new Holmestrand station, (see below).

An underground railway station in a large span cavern

The Norwegian National Rail Administration has just recently commenced construction work on a new high-speed railway on the west coast of the Oslo Fjord connecting the towns south and south-west of Drammen with a new link to Oslo, the Vestfold Line. The town of Holmestrand is located along this particular rail link. Holmestrand is a town that has the sea to the east and a mountainous hill to the west. Due to the fixed curve radius of the high-speed rail link it was decided that the railway should be placed in a tunnel to pass Holmestrand. This means that a railway station in open air was abandoned. However, a railway station was considered as an important part of this railway link, and therefore the station was replaced by an underground facility located in the mountainous ridge west of Holmestrand. The speed for this railway link is 250km/h. The rock cavern, which is designed to house the station area at Holmestrand, is built with a cross-section of theoretically 350m2 and a length of almost 900m. The width of the cavern is 36m, and the height 15m (Figure 9). This is sufficient to accommodate a total of four parallel tracks and platforms. The cavern is located with a horizontal distance to the surface of 100m.

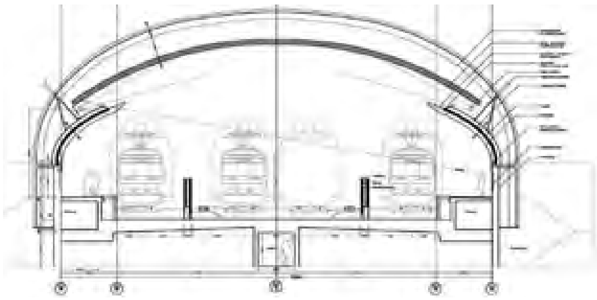

Fig. 9. Cross-section of the underground station (Ill. Norwegian National Rail Administration)

The site location has been subject to a typical pre-investigation scheme based on Norwegian standard, including surface geological mapping, core drilling, structural geological analysis etc. The rock mass consists of basalt which has a typical UCS of approx. 240MPa. The area is cut by several joint sets that have been identified at the surface, and also by one significant weakness zone. More important though, and the key to such an excavation are the stress measurements being conducted. These consist both of hydraulic fracturing and 3-D overcoring. It was found that the major stress component acts perpendicular to the cavern axis and the following magnitudes were found:

- 3D-overcoring gave 4-11MPa
- Hydraulic fracturing gave 2-6MPa

As a comparison, the rock cover is about 70m, which would produce a vertical stress component gravitationally of approx. 2MPa and a calculated horizontal component in the range of 0.7MPa using a Poisson ratio of 0.25. This example demonstrates the need to perform stress measurements and utilize these measurements to optimize the cavern design.

Arna-Bergen, a stretch of railway under construction today

The line between Arna and Bergen is very heavily trafficked. The capacity of today's single track is inadequate, and the construction of a double track along this stretch will improve the situation for both goods and passenger traffic. Most of the stretch runs in a tunnel through Mount Ulriken. The Norwegian National Rail Administration is now building a new, parallel tunnel to increase capacity. The double track will also permit higher speed and more flexible traffic handling. The project consists of a number of important elements for efficient train operations in the area around Bergen.

The new tunnel is being driven from the Arna side of Mount Ulriken. Both conventional blasting and tunnel boring machine are being used. This is the first time that a tunnel boring machine is being used for a railway tunnel in Norway.

The first 765 metres of the 7.8-kilometre-long tunnel have been blasted in the conventional way. The reason for this is that the tunnel is to accommodate an extra passing track, and must therefore be much wider here. The cross-section of this part of the tunnel varies from 144 to 300 square metres, whilst in the rest of the tunnel is 68 square metres.

In addition, two diagonal tunnels have been blasted between the old and the new tunnel so as to allow the trains to cross between tunnels. Each of these is 150 metres in length. Sixteen smaller cross-passages will also be blasted between the tunnels to provide evacuation routes and to house technical installations. The remaining 7 kilometres of the new tunnel are being driven using a tunnel boring machine.

The Ringerike Line, under planning

A final project that is in the planning stage at present is the Ringerike Line, which will have a 23 kilometre double track in the same tunnel. Bane NOR, the Norwegian railway infrastructure company, has decided on the tunnel concept for this line. The tunnel will be constructed with two tubes, but both railway tracks will be laid in one double-track tunnel. The other tube will serve as a parallel service and evacuation tunnel and will have access at both Sundvollen in the municipality of Hole at one end and at Jong in the municipality of Bærum at the other.

Two alternative tunnel concepts were considered for the Ringerike Line between Sandvika and Sundvollen. Both variants comprised two tunnel tubes because the tunnel will lie so deep beneath the Krokskogen forested plateau that there will few possibilities for evacuation to the surface. In one alternative, the railway tracks are laid in two singe-track tunnels, whilst in the other concept, which is the one that has been chosen, a double-track tunnel and a parallel service tunnel will be built. The construction methods that are under consideration are conventional blasting and use of a tunnel boring machine. This will be decided at a later time in collaboration with the executing contractor.

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5.5.2 NORWEGIAN ROAD TUNNELS - A HISTORICAL PERSPECTIVE

Background

Tunnels have played a major role as a part of the Norwegian road network for well over 100 years. The first road tunnels were built as early as the end of the 19th century, and today there are more than 1100 road tunnels with a total length of about 1100km on the public road network. Most of them are built in solid rock, and only a few have steel or concrete as load-bearing element. These steel or concrete tunnels pass through soil-like or weak rock masses, and two tunnels are located on the seabed. The last-mentioned tunnels are the Opera Tunnel in Bjørvika, Oslo, and the Skansen Tunnel under the Trondheim Canal.

Compared with other countries, Norway has a very large number of road tunnels, most of which have been built in the last 40 years. Each year some 20 to 30km of new road tunnel is added to the public road network, and there are currently 50 to 60 under construction. However, tunnels are an expensive part of the road network, with construction costs today ranging from NOK 150 000 to 300 000 per running metre of completed tunnel. During a period of 20 years, these costs have risen tenfold. This is due to the inclusion of more equipment and safety measures in new tunnels, and to a sharp rise in prices in the period.

The Stetind Tunnel in the wild countryside. Photo Lars Arne Bakken

When the first road tunnels were built there was considerable railway development taking place in Norway, and the road tunnels were built using the same technology as that used on the railways. Later there was a period of hydroelectric power development and transport of water in long tunnel systems. This tunnel technology had an impact on the way in which Norwegian road tunnels were built until the 1970s. Since then continuous development work has been done in relation to the actual construction of tunnels, with the focus on efficiency, safety and the working environment. This evolution can be summed up in a few words: transition from handheld tools to electro-hydraulic drilling rigs, rock reinforcement with new bolt types, eventually in combination with shotcrete, increased emphasis on and improvement of water and frost protection methods, transition from packaged explosives to explosives in the form of a powder or slurry, use of non-electric initiators, grouting materials and equipment, flexible shift schedules and better methods of ventilating the tunnel during driving.

In 1969 there was a near-disaster in the 0.7km-long Rørvikskaret Tunnel in Lofoten. During driving, the construction crew entered a crushed zone containing swelling clay (a clay mineral in old rocks). There was a major collapse and it was decided to transport the fallen material out of the tunnel. Cave-in and removal of the material from the tunnel continued. Eventually, there was a cave-in shaft extending about 200 metres up to the pass above the tunnel, and the existing road running westwards in Lofoten almost collapsed into the hole. After five years' work, the hole in the tunnel was concreted up, and the tunnel was opened in 1974. Despite many incidents, there were no work-related accidents. What happened in the Rørvikskaret Tunnel led to a major focus on support methods ahead of the work face where drilling and blasting take place, and on the importance and significance of engineering geology competence both during the planning and the building of tunnels.

Advances in tunnelling technology were especially significant in the 70s, 80s and first half of the 90s. This development work resulted in a favourable price trend for unlined tunnels until about the year 2000. The concept was known internationally as "Low cost road tunnels". This method can be briefly described as follows:

1. Pre-investigations
- Geological mapping supplemented with acoustic measurements and seismic profiles
- Limited core drilling

2. Construction
- Conventional drilling and blasting
- Bolting and shotcrete as the main rock support
- Grouting of cement and chemicals to prevent ingress of water

3. Equipment
- In all tunnels through rock there is an inflow of water. Different types of water and frost protection solutions have been used
- Limited volume of technical installations

As a result of more stringent road traffic and fire safety requirements, the "low cost" concept was gradually abandoned. What has been kept, although with a little less emphasis, is that the tunnel standards include requirements as regards design, equipment and traffic management adapted to the actual traffic volume.

Full-face boring (TBM) has been used in only three road tunnels. The Svartisen Tunnel, 7.6km in length, in the county of Nordland was the first. There, about 5km was bored using a TBM. At almost the same time the Fløifjell Tunnel in Bergen was also excavated using a TBM in two tubes of respectively 3.2km and 3.6km in length. The third tunnel, which was excavated after the Fløifjell Tunnel, was one of the tubes of the Eidsvåg Tunnel, some 950 metres in length. In these tunnels, the sides had to be blasted out. This entailed in extra costs, which made the tunnels expensive. These tunnels were built more than 30 years ago and since then TBMs have not been used for road tunnels in Norway.

Until 2003 all tunnels were built partly by the project owner and partly on contract. There are no statistics for this division, but roughly speaking the breakdown was about 50/50. This combination was found to be an excellent solution for developing both technology and competence.

Traffic, safety and aesthetics

With the exception of the city tunnels, there is, in general, very little traffic in most tunnels in Norway. On average, about 4000 to 5000 vehicles a day (AADT) pass through a Norwegian tunnel, whilst the average in Europe is about ten times higher. Surveys show that fewer accidents happen in tunnels than on roads in the open. Most accidents occur in the tunnel opening. It is also in the opening that the biggest problems arise with the change in light conditions, condensation on vehicle windows, and sometimes water and ice on the carriageway.

A good tunnel portal according to requirements in the 1990s. Today, requirements as regards traffic safety in tunnel openings are more rigorous, especially when it comes to guardrails, funnel design and adaptation to the terrain. Photo Eirik Øvstedal

To increase both safety and the aesthetics of a tunnel, a great deal of work has been put into designing the tunnel portals. The road standards today describe requirements for portal design; such requirements did not exist 30 years ago. The lighting in the tunnel is also of major importance for safety and driving comfort. The best known is the lighting in the Lærdal Tunnel, but several other tunnels also have special lighting today.

The worst scenario for tunnel accidents is vehicle fire resulting in a large tunnel fire with a lot of smoke. In the last few years there have been five major fires in three Norwegian tunnels. These fires have aroused great interest in tunnelling circles in Norway and elsewhere in the world. The Norwegian Directorate of Public Roads has produced an overview over the cause of the fires, the consequences they have had and the step taken to minimise the risk of similar fires in the future, cf. Report 240 issued by the Norwegian Public Roads Administration. The three tunnels involved are:
- the Oslofjord Tunnel, 29 March 2011
- the Oslofjord Tunnel, 23 June 2011
- the Gudvanga Tunnel, 5 August 2013
- the Gudvanga Tunnel, 11 August 2015
- the Skatestraum Tunnel, 15 July 2015

No one has lost their life in these tunnel fires, but a number of people suffered smoke-related injuries, some serious. There was major damage to the tunnel structures in some cases, and the tunnels were closed for a long time as a consequence.

The accidents in the Mt Blanc, St Gotthard and Tauern Tunnels in around the year 2000 led to a much more stringent safety directive in the EU. This directive, which tightened the requirements as regards design and fire safety, was implemented in Norway in 2006. The fire safety requirements (including evacuation), in particular, became much stricter. The introduction of the directive has also made it necessary to carry out extremely extensive rehabilitation work in many existing road tunnels. Once this work has been done, both new and old tunnels will be efficient, attractive elements in the road network, and safety will be further enhanced.

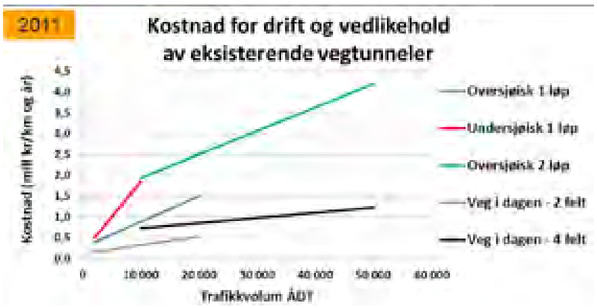
It has also been found that frost protection consisting of polyethylene (PE) foam insulation material was highly flammable when unprotected. Fire protection comprising shotcrete with added polypropylene fibres and the use of mesh reinforcements has, however, been found to be highly effective and is used extensively in tunnels today.

Shotcrete on insulation material. Photo Harald Buvik

Shotcrete gradually came into use as rock support in road tunnels in the 1960s to 1970s. During the construction of the Spiralen Tunnel in Drammen, major problems with spalling rock (large stresses in the rock) were encountered. Concrete was sprayed on and the problems were greatly reduced. Extensive use was made of sprayed concrete to support weak rock in the Skarvberg Tunnel in the county of Finnmark in the 1960s. This 3km-long tunnel was completed in 1968. The first road tunnel in which shotcrete was used systematically against spalling rock was the Tafjord Tunnel in the county of Møre and Romsdal. Shotcrete has later proven to be a highly successful form of rock support.

Concrete pavement has been used in some tunnels, mostly laid as roller--compacted concrete. It has been found difficult to obtain an even pavement that does not cause great deal of rumbling from cars, but concrete has shown more resistance to studded tyre wear than asphalt. There is however a problem with the dust from concrete pavement wear, as it is very fine and produces suspended particles in the tunnels, which in turn cause visibility problems. The concrete pavement has also the disadvantage that it becomes polished and slippery, and is thus hazardous to traffic.

Operation and maintenance of tunnels is costly. Experience has shown that over time road tunnels are five to ten times as expensive to maintain as a similar stretch of road in the open. The main reason for this is that there is a great deal of electrical equipment in tunnels, which has a limited useful life. A damp and dusty environment causes rapid degradation of all the components of a tunnel.



Operation and maintenance costs for existing tunnels and for a road in the open.
Source: Johnny Johansen, ViaNova.

Ventilation

Fresh air is essential in all road tunnels. In many tunnels, local climatic conditions will mean there is a natural draught. Increasing traffic and periods with poor natural draught create a need for a ventilation system. In Norway the majority of tunnels have longitudinal ventilation, just a few have venting shafts, and none at all have so-called transverse ventilation. Transverse ventilation, which involves supply of fresh air in a duct and extraction of used air in another duct along the entire length of the tunnel, has been used in many tunnels in the Alps.

When the Kalvik Tunnel on the E6 highway in Nordland was to be built at the beginning of the 1960s, there was a great deal of doubt about the need for ventilation. The tunnel was 2.7km long and one of Norway's longest at the time. Some of those involved in the road project went on a study tour to Switzerland, and they concluded that steps had to be taken to install transverse ventilation. The tunnel, which was partly excavated as a railway tunnel by the Germans during World War II, was built extra wide so that it would later be possible to build a wall on each side of the carriageways. Air was to be pumped in behind one of the walls, whilst the other side was to be used for extraction. However, the concrete walls were never built and it was not until ten years after the opening that the first fans for longitudinal ventilation were installed.

At the start of the 1970s there was little expertise on ventilation of road tunnels in Norway. At the Department of Road and Railway Engineering, Norwegian Institute of Technology (NTH), a ventilation project was started in 1972. Jon Kvåle, siv. ing., was appointed as project leader and created the design basis. Kvåle has later been project manager for a number of road and power station projects in the county of Sogn and Fjordane, including the world's longest road tunnel, the Lærdal Tunnel, 24.5km in length. The ventilation project was run by Professor Rasmus Nordal. Earlier, senior lecturer T. B. Riise in the same department had carried out some research on airflows in railway tunnels, so there was some basis for placing this project in the hands of NTH. Jon Kvåle also performed a number of laboratory experiments. The experiments using cigar smoke in a plexiglass tube are well remembered, but probably were not of groundbreaking importance

During the construction of the Lærdal Tunnel, an access tunnel was established from a side valley. Today this access is used as a part of the ventilation system. Fresh air is drawn in from both tunnel openings and the polluted air is pumped out into the side valley.

In some tunnels there has been a problem with too much cold air coming into the tunnels in the winter. This has resulted in water freezing, which has also produced icicles in the tunnels. Several attempts have therefore been made to establish doors in the tunnels. One of the first tunnels to have doors installed was the Steinfjellet Tunnel between Namdalen and Røyrvik in North Trøndelag. This tunnel was built in the 1960s and goes up to a height of about 700 metres. A substantial problem with ice in the tunnel led to a single door being installed. North Trøndelag engineer Willy Pettersen, siv. ing., came across the door in 1970, and he says there were a lot of problems with it. It was frequently out of service, and motorists sometimes drove into it in despair when it would not open. In the 1980s robust garage doors were installed in many tunnels in several counties. These doors were controlled by temperature, traffic and hazardous gases. However, it was difficult to get the doors to work properly and many of them have been removed today.

Door in the Steinfjellet Tunnel in 2003. Photo Karl Melby

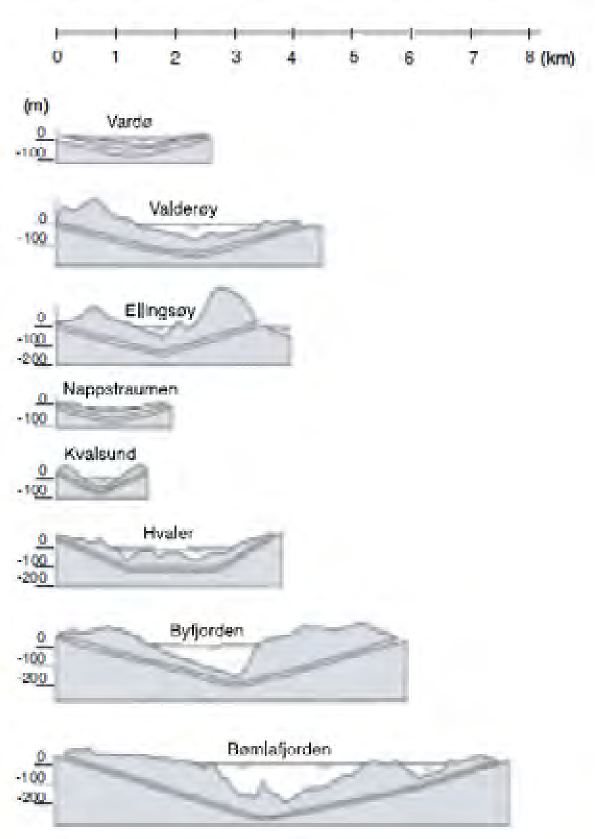
Subsea tunnels

The first subsea road tunnel was built in Vardø from 1979 to 1982. Today, we have well over 30 subsea tunnels either under construction or completed and opened to traffic. The subsea tunnels lie along the whole of the Norwegian coastline, and have been of huge importance for settlement and economic activity throughout coastal Norway. What would the "salmon adventure" on Hitra and Frøya have been without the subsea tunnels to the islands?

With the exception of one tunnel, all of them are a part of the public road network. The private tunnel is 2.3km long and goes from Hammerfest to Melkøya, where there is a large landing facility for gas from the Snow White field. The longest tunnel is the Bømlafjord Tunnel, which is 7.9km long and goes down to a depth of 260 metres below sea level. The deepest is the Eiksund Tunnel, which goes down to a world-record depth of 287 metres below sea level. When the subsea tunnel at Ryfast close to Stavanger opens, with a length of 14.3km in two tubes, it will be Norway's longest subsea tunnel. And when the Rogaland Fixed Link or so-called Rogfast subsea tunnel of almost 30km in length is completed some time after 2020, it will break all records for length and depth.

Earlier, a number of tunnels were built with a gradient of 10% and an extra lane. For reasons of traffic safety and in accordance with EU regulations, this is no longer allowed.

No other country has so many subsea tunnels, and therefore there was a great interest in them about 10 to 15 years ago. Our neighbours in Iceland and on the Faeroe Isles have even built tunnels based on the Norwegian model.



Length and depth of some subsea tunnels.

Ground freezing in the Oslofjord Tunnel. Photo Eirik Øvstedal

Ground freezing in tunnels
During tunnel driving, extremely poor conditions may be encountered, which require very special support methods. One such method is ground freezing. The execution of this method is highly demanding technologically, and it has only been used once in Norwegian rock tunnels. During the excavation of the subsea Oslofjord Tunnel a pocket of moraine material was encountered. These deposits were frozen to minus 30 degrees at a minimum thickness of 3 metres around the tunnel. Brine cooled with liquid ammonia was used to freeze the material. The freezing time was about three months and it took about three months to drive through the zone. Short blast rounds were used, followed by the installation of a double-reinforced concrete lining of a good metre's thickness. Such an approach is both costly and time-consuming. It goes without saying, therefore, that it is a method that can only be used in exceptional cases and is explained in more details in a separate chapter of this publication.

Costs involved in Norwegian tunnel operation
Norwegian tunnels are about four to six times more expensive to build than roads on the surface. It can be estimated that tunnels have six to ten times higher costs for operation and maintenance than roads in the open. For example, the operating costs for the tunnels in Oslo and Akershus take about one-third of the total operating budget (2014 figures). Converted into running metres, this is just over NOK 4,000 per metre in a four-lane tunnel, excluding the share of Road Traffic Control Centre costs and depreciation. If these costs are included, operating costs will amount to about NOK 12,000 per running metre for heavily trafficked four-lane tunnels. In today's situation, funding for operation and maintenance must be taken from other road initiatives. Little allowance has been made for this in the long-term planning and future operation costs for the road network.

A comparison with European tunnel construction
In the last few decades Norwegian tunnelling practice has gone through a number of development stages as regards methods, machines and equipment. The common denominator for this development is increased mechanisation, digitalisation and higher tunnel driving rate.

In Norway the bedrock is basically of such quality that the rock can be used as the main construction material. Norwegian tunnelling practice entails work and risk sharing between the contractor and the owners, with the owner taking the main responsibility for the geological conditions.

Another characteristic of Norwegian tunnelling practice is that the support method and extent is determined continuously during construction and is adapted to the local conditions. The practice that the contractor decides on the extent of temporary support, and the owner the permanent support, is well established. In recent years developments have been such that most of the permanent support has been installed at or behind the work face in close connection with the temporary support.

Norwegian practice differs from the tunnelling methods of other countries (especially compared with countries on the Continent) in that the permanent support methods and extent are primarily decided by those who follow the tunnel driving at the work face on a daily basis, the tunnel foreman and the chief engineer.

A common feature of many tunnels in Europe is that the ground conditions are often poorer than in Norway, making it necessary to have robust rock support structures to take up the forces from the overlying rock or soil-like masses. The support means consist typically of bolts, shotcrete, steel arches, mesh reinforcement, and non-reinforced and reinforced concrete linings. Rock bolts are typically not regarded as part of the permanent support. It is also typical that the tunnels are fully lined when completed.

As a rule, the bolts are temporary, but they help to stabilise the ground in the period before the permanent concrete lining is installed. Shotcrete is the main support method in combination with steel arches, mesh reinforcement and full lining. In addition, a membrane is usually used to prevent water leakage into the traffic space. The membrane is held in place by the concrete lining, which gives an internal tunnel surface of constructional concrete. The solution is durable, has predictable static conditions and its lifetime can more easily be documented.

Another area where there is a major difference between European and Norwegian tunnelling practice is the design of the water and frost protection methods. Whilst in Europe this is largely provided by the permanent concrete lining, in Norway we have used independent water and frost protection claddings to guide the leakage water down into the ground and into the drainage system. This has been a challenge in terms of durability as so many solutions of this kind have consisted of light-duty structures. Today lessons have been learned from this, and heavy-duty solutions are now used which meet far stricter requirements for long-term durability.

Compared with European tunnel construction, it is especially the thoroughness characteristic of the planning, construction, follow-up and documentation that stands out. It is typical for European tunnel construction that, for instance, support methods and extent are determined in great detail in advance by the consulting engineer and are based on conducted ground surveys. This is monitored during construction by measurements in the tunnel.

An important factor, which may explain many differences between Norwegian and European tunnel construction is the economic conditions on the basis of which road projects are decided and built. Many countries in Europe have rules for depreciation and requirements for cost effectiveness that are different from the corresponding rules and requirements in Norway. For most European countries a period of depreciation of 40-60 years is normal, whilst in Norway it is currently 25 years. Similarly, the requirements for cost effectiveness are less stringent in Norway. This means that in Europe today it is economically correct to choose solutions that have a higher quality and that are more durable.

Tunnel training

Traditionally, there has been no special training in tunnelling at the Norwegian Public Roads Administration. Planners, managers and controllers have had their basic education at universities, technical colleges and vocational schools, whilst the workers have had their training through practical work and short courses. This has worked well until recently. After 2008 a special tunnel school, commonly called "Tunnel academy", has been arranged under the direction of Norwegian University of Science and Technology (NTNU) and the Norwegian Public Roads Administration, Directorate of Public Roads in cooperation with the Norwegian National Rail Administration. In this course the most important is not blasting technique and rock support, but rather the focus on an overall transfer of competence across the disciplines of planning, construction, operation and maintenance. The course was developed for employees of the Norwegian Public Roads Administration and the Norwegian National Rail Administration, but is now open to everyone in the tunnelling industry. The aim of the course is to contribute to a greater interaction between the different technical environments and thereby achieve better quality in all phases of a tunnel project. The course consists of five three-day sessions in one calendar year and takes place at different locations in the country. At the end of the course there is a compulsory examination at NTNU.

The Coastal Highway Route E39 and other future major projects

When looking at the total cost/benefit of infrastructure projects, we see that building tunnels to shorten journeys and establish fixed links to replace ferries is good economy for the country as a whole. On the west coast of Norway in particular, tunnels allow population centres to be more closely linked, thus facilitating the establishment of a larger work market and a better exchange of goods and knowledge without the interference of ferries or mountains.

The topography of Norway is challenging, especially on the west coast with all the deep fjords and the high mountains in between. Half of all road tunnels in Norway are already situated in this area.

Future road tunnels in Norway will include many city tunnels, which will remove traffic from the city centres, but the majority will still be built to shorten distances and prevent the impact of rockslides/avalanches, especially on the west coast.

We will also have some challenging subsea tunnels in the near future:

- Ryfast Tunnels – ongoing 14km, 290m below sea level, will be finished in 2019
- Rogfast Tunnel – 27km long and 390m below sea level, will be built 2018-2026
- New tube of the Oslofjord Tunnel is to be decided shortly

All the focus on the impact of roads in West Norway, where 60% of Norway's traditional export comes from (oil and gas not included) and one-third of Norway's population lives, has made NPRA (Norwegian Public Roads Administration) start a programme called the Coastal Highway Route E39. The programme looks at the main road from Kristiansand in the south to Trondheim in the middle of Norway. It examines social, economic, environmental and technical solutions to remove the remaining seven ferries and establish nine fixed links with bridges, subsea tunnels or even floating submerged tunnels.

5.6 STAD SHIP TUNNEL – TECHNICAL AND COST-EFFECTIVE CONSIDERATIONS

STATUS AND FURTHER WORK

The idea of a tunnel through Stad was first launched back in the year 1874. This was at a time when smaller tunnels were quite frequently used for ships, ships that travelled the canals in the UK and France to mention a few. Stad is a peninsula on the western coast of Norway at approximately latitude 62°N. It blocks the otherwise sheltered shipping corridor along the coast. The local weather is fierce, and the seas hazardous 90 to 110 days a year. Mariners have long feared Stad, as around it currents are strong, waves high, and shipwrecks frequent.

Norwegian transport minister Ketil Solvik-Olsen has given the go-ahead to the Stad Tunnel, the world's first ship tunnel. The announcement came after the release of the country's National Transport Plan 2018-2029. Funding has been earmarked for the project in the first half of this period. The earliest the project could start is 2019.

The tunnel will make passage through a hazardous shipping lane much safer. A combination of currents and topography makes navigation very unpredictable and high waves make the exposed region dangerous. The Norwegian press has remarked that "even the Vikings feared these waters". There are no outlying islands protecting the peninsula from rough seas, meaning Stad is one of the most dangerous stretches of the Norwegian coast. There have been dozens of deaths in the area and it is now some 140 years since proposals for a tunnel first appeared in the Nordre Bergenhus Amtstidende newspaper.

SUMMARY

The Stadhavet Sea is the most exposed and dangerous stretch of sea along the Norwegian coast. The aim of the tunnel project is to ensure safer navigation along Stad.
In its National Transport Plan 2018-2029, the Norwegian government has allocated NOK 1.5 billion to the project in the first period, and NOK 1.2 billion in the second period, and has indicated that construction work should start in the next parliamentary term (2018-21).
The tunnel will be excavated using conventional drill and blast techniques, with top heading and benching. The excavated masses will be used in land reclamation projects in the local area. Socio-economic analyses show that net benefit will be minus NOK 1.3 billion, with the project's total costs estimated to be NOK 2.7 billion.

Figure 1. Tunnel portal at Kjødepollen

INTRODUCTION

The Stadhavet Sea is the most exposed and dangerous stretch of sea along the Norwegian coast. The area is subject to difficult waves conditions and has by far the most and strongest winds. Kråkenes lighthouse, which lies just to the south of Stad, is the meteorological weather station that records most storms, with anything from 45 to 106 days of gale a year. The combination of wind, currents and waves around Stad make it a particularly demanding part of the Norwegian coast.

Owing to the combined effect of currents and underwater topography in the area, the wave conditions are especially complex and unpredictable. Very high waves come from different directions at the same time, and can create critical situations. Moreover, the conditions here may cause heavy waves to persist for several days after the wind has died down. This means that navigation conditions can be challenging, even on days with little wind

The object of this tunnel project is to improve navigation and safety for shipping past Stad.

Figure 2. Geographical location of the tunnel

Studies conducted in 2000-2001 and 2007-2008 analysed a number of alternative cross-sections and alignments for the tunnel. The final alignment was chosen because the Stad peninsula is at its narrowest here, and also because the waters are sufficiently sheltered to allow shipping traffic to use the tunnel in most weather conditions. Three alternative cross-sections were considered: a reference alternative, a small tunnel alternative and a large tunnel alternative. The small tunnel alternative was designed to accommodate vessels such as purse seine trawlers 13m in width and with a draught of 8m and/or freighters (freezer ships) 18m wide and with a 6m draught.

The basis for the design of the large tunnel alternative was that ships such as the "Hurtigruten" coastal cruise ships should also be able to pass through the tunnel.

Studies providing the basis for the choice of alignment and cross-section were conducted in connection with the concept choice study (CCS) and subsequent external quality assurance process QA1. During the debate on the National Transport Plan 2014-2023 (Report to the Storting No. 26), the Norwegian parliament voted to proceed with the large tunnel alternative, and this is taken into account in what follows.

TECHNICAL CONSIDERATIONS

In the studies, the Stad Ship Tunnel is viewed as comparable to an extra long rock cavern or to a road tunnel of an extra large cross-section. Many rock caverns of this size have been constructed in Norway and the complexity of the project is regarded as manageable.

Some key figures for the tunnel:
- length: 1700 metres
- height between tunnel floor and ceiling: 50 metres
- width between the tunnel walls: 36 metres
- cross-sectional area: 1661 m2
- volume of solid rock extracted: approx. 3 million m3, corresponding to about 7.5 million m3 of blasted rock
- total costs: approx. NOK 2.7 billion
- construction time: 3-4 years

Excavation method

The tunnel will be excavated using conventional drill and blast techniques, with top heading and benching. The upper part of the tunnel, the roof/ceiling, will be excavated using a tunnel boring machine. Other blasting will be carried out during benching.

To ensure dry excavation of the cutting and tunnel, it is planned to establish a dam in front of the tunnel openings by retaining a threshold of existing rock at both ends of the tunnel.

See figure 5 showing a sectional view from the area development plan.

There are plans to use the tunnel masses in land reclamation projects in the local area.

Support method

Geological surface mapping, refraction seismics and core drilling have been carried out to map out the

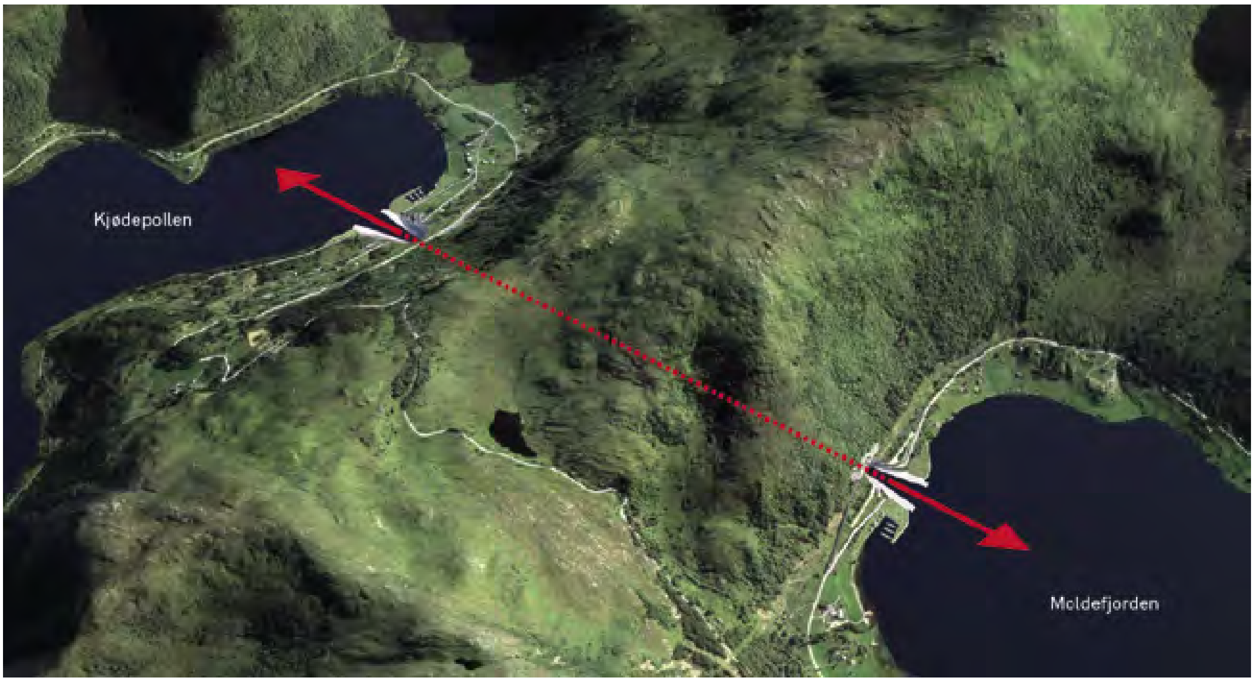


Figure 3. The alignment area between Kjødepollen and the Molde Fjord

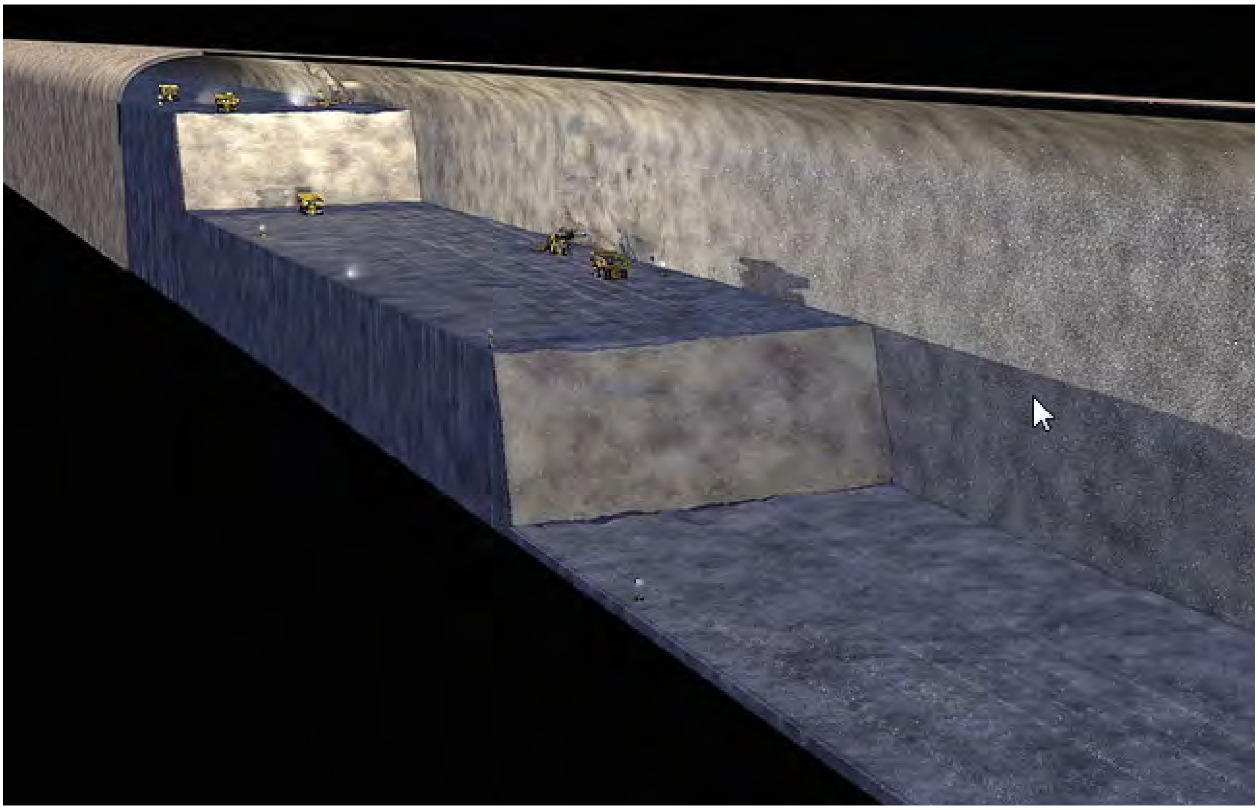


Figure 4. Tunnel excavation at multiple levels

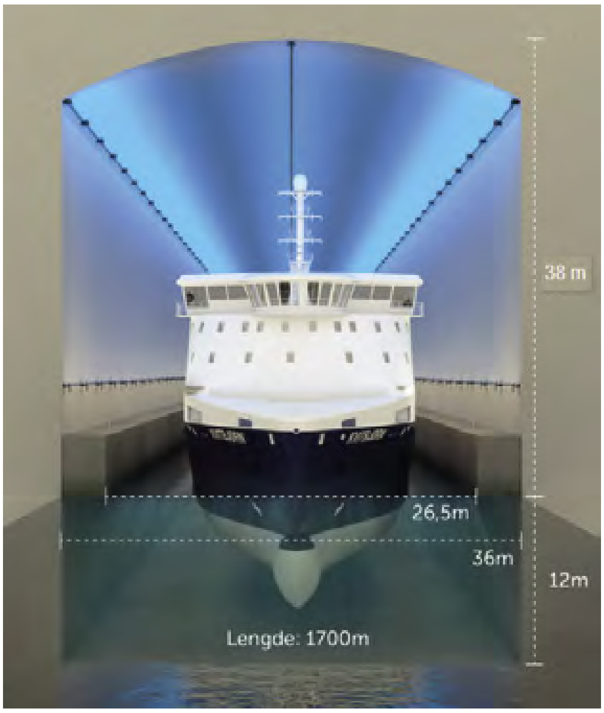


Figure 5 Illustration of tunnel cross-section and a ship

geology, and rock tension measurements have been made. In addition, numerical models have been used to evaluate what types of support means should be used.

Peliminary estimates of support quanitites are given in the table below.

Item/process	Unit	Quantity
33.1 Scaling/rock removal	m2	5 300.00
33.2 Anchor 10m	pc	45.00
33.2 Anchor 15m	pc	5.00
33.221 Rock bolts, fully grouted, length 3.0m	pc	285.00
33.222 Rock bolts, fully grouted, length 4.0m	pc	185.00
33.2291 Rock bolts, fully grouted, length 2.4m	pc	110.00
33.2292 Rock bolts, fully grouted, length 6.0m	pc	120.00
33.2293 Rock bolts, fully grouted, length 8.0m	pc	20.00
33. 412 Shotcrete with added fibres, 8-2cm thick	m3	270.00
330.322 Rockfall netting, bolts and ties	m2	900.00

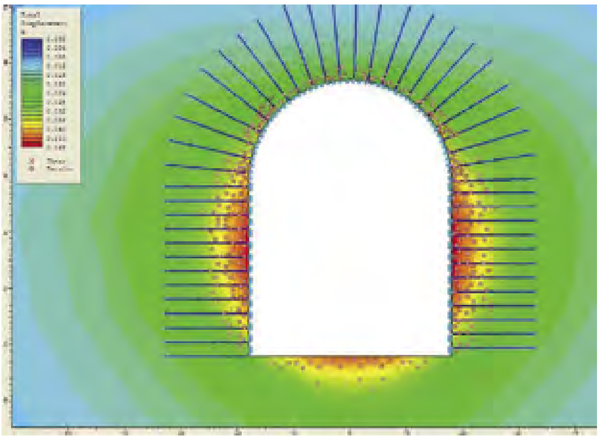


Figure 6. Numerical analysis of the tunnel stability

COST-EFFECTIVE CONSIDERATIONS

Here, it seems natural to look at the possible socio-economic effects of this project. A socio-economic analysis of the project has been carried out to consider these effects, and it has been concluded that the anticipated socio-economic costs will be NOK 2.7 billion. The quantified, anticipated socio-economic revenues (benefit effects) are expected to be in the order of NOK 1.4 billion. These estimates give a quantified net benefit of minus NOK 1.3 billion.

STATUS AND FURTHER WORK

The Norwegian Coastal Administration has prepared a concept choice study (CCS 2010) that discusses the reference alternative (with no new measures), the small tunnel alternative and the large tunnel alternative (coastal cruise ships). An external quality assurance QA1 report, commissioned by the Ministry of Fisheries and the Ministry of Finance has been produced. This document quality assures CCS 2010 and makes recommendations for a possible continuation of the project.

Over the years a number of studies and analyses have been made with respect to the project, and the level of investigation lies well above that normally expected of a QA1 phase. The report specifically requests that, should the project go ahead, the following, among other matters, be looked into:
- operational concept
- additional geological surveys
- identification of project-specific goals
- establishment of a benefits realisation plan

In addition, the following must be carried out:
- revision of the current area development plan, adapted to the large tunnel alternative
- requirements according to the central steering documents

User benefits – persons and traffic	
Saved waiting time	1 257 514 000
Reduced sailing time	-2 177 000
Reduced fuel consumption	151 690 000
Budget impact	
Investment costs fairway	-2 631 645 000
Investment costs navigation devices	-33 426 000
Operation, maintenance and renewal costs	-17 469 000
Accidents	
Reduced risk of groundings	6 214 000
Reduced risk of collisions	8 270 000
Reduced risk of fire and explosions	4 220 000
Reduced risk of sinkings and structural defects	199 455 000
Reduced risk of loss of life	109 879 000
Reduced risk of personal injuries	72 074 000
Environment - greenhouse gas emissions	
Reduced greenhouse gas emissions	105 801 000
Tax costs	
Tax costs	-536 508 000
Net benefit	-1 308 906 000
Net benefit per budget krone	-049

Table 7-16 Priced consequences, NOK (2016)

The Norwegian government has stated in the National Transport Plan 2014-2023 that it wishes to proceed with a preliminary project for QA2, and that this is to be based on the large tunnel alternative as it is considered to have a greater benefit potential. NOK 1 billion has been allocated in the second half of the planning period with a possible project start after 2018.

The Ministry of Transport commissioned a feasibility study from the Norwegian Coastal Administration on 18 February 2015. The Coastal Administration has now prepared a preliminary project, including approved area development plans, which will be submitted to the Ministry of Transport in the middle of May 2017. The project will then be ready for external quality assurance, QA2.

Phases in government projects

Figure 7: The phases of government projects

REFERENCES

The information in this document has been found primarily in the Area Development Plan and associated impact study, the technical preliminary project and the strategic steering document.

5.7 STENDAFJELLET ROCK QUARRY AND UNDERGROUND WASTE DISPOSAL SITE

Abstract

In Stendafjellet, 8 km south of Bergen center, and 5 km from Bergen Airport, Fana Rock and Recycling A/S (Fana Stein og Gjennvinning A/S, FSG) is operating an underground quarry and crusher plant. Excavated rock volumes are later used for depositing sites for contaminated soil and masses with low organic content. 350 000 tons of rock material for construction purposes is produced yearly in up to 13 different grain size fractions. A considerable quantity of this material, including fines are tarmac additives. Previous experience of excavating caverns 25 m wide and 50 m tall, in the same type of geological conditions, help in planning and design.

Figure 1. Location of Stendafjellet, and Fana Rock and Recycling A/S. (http://kart.gulesider.no/)

A qualitative cost comparison between excavating the top header and a 35 m tall bench is examined. The results from this cost comparison describes some characteristic differences in cost elements between underground and above ground excavation.

Favourable rock quality enables construction of tall caverns with high degree of stability with relatively little rock support. Together with a normal level of preconstruction investigation, the Q-system gives acceptable basis for design and construction, if continually supervised by an engineering geologist on site.

The permits for depositing polluted materials in underground caverns depends on identified waste categories according to waste regulations.

It is recommended to map rock material quality fit for quarrying and possibilities for underground depositing. And including these in urban planning. Supplying good quality rock material for construction purposes, near the consumer market, result in low transportation costs for clients nearby.

Introduction
Fana Rock and Recycling (Fana Stein og Gjenvinning, FSG) was established in 1954. The company has two separate businesses: quarry/chrusher/sorting plant and waste disposal for contaminated substance/mass. They also work with entrepreneurship and tunnel excavation.

The quarry and crushing plant mainly produce gravel. They also sell natural products as foundry sand, scouring sand, shell sand and bark.

The waste disposal is a business idea, exploiting the volume of the excavated rock caverns. The purpose is to safely handle contaminated masses and prevent pollution of the environment.

The company has 20 employees. Daily operation is mainly automated. Subcontractor perform excavation and transportation of rock to the main crusher.

Today the plant comprises eight rock caverns for rock mining/waste disposal and an underground plant for

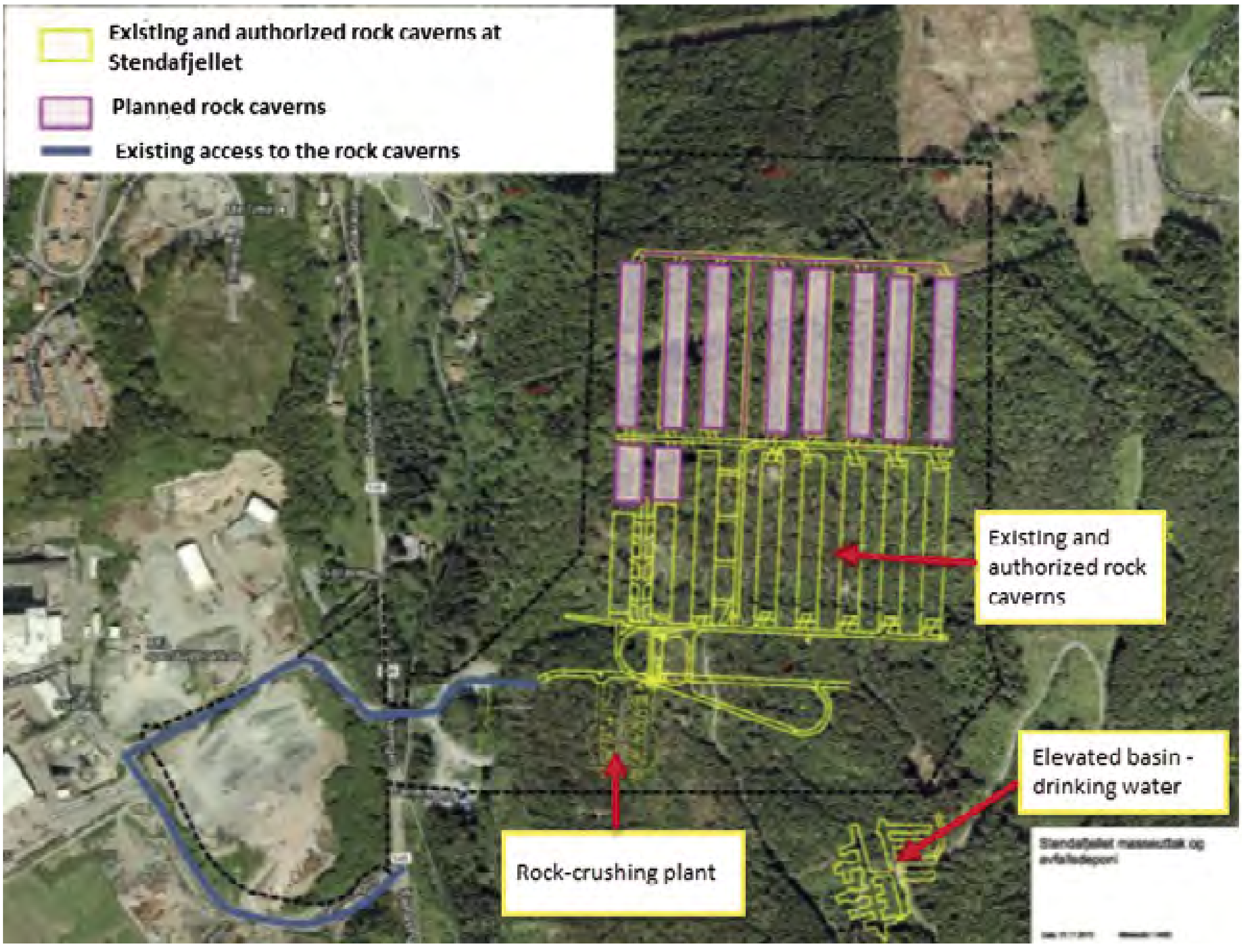


Figure 2. An illustration of the existing and planned rock caverns and plant at Stendafjellet.

crushing, fractioning, storing and loading of produced materials. There are a system of tunnels connecting the ends of the caverns at two to three levels, allowing rational access for both the excavating rock operation, rock support work, and infilling of material.

In 2003 FSG was given permission to deposit waste. Later they received extended permissions due to changes in the legal framework and state of the marked. The authorization applies to deposition of waste in the existing rock caverns. According to the authorization given by the county administrator, 27th of July 2012, FSG can accept waste in category 2 (ordinary waste) and category 3 (inert waste). The authorization applies to a yearly waste quantity of 350 000 tons

Geology

Stendafjellet is a hill of medium to fine-grained granitic, syenitic and mangeritic rocks of Precambrian age, generally altered and deformed to gneisses by Caledonian processes. Except for a few distinct shear zones, the rock mass quality in the area of the underground quarries generally seem to be of good quality. The brittleness and flakiness classifications are within category 1, which is the best category according to testing methods in Norwegian Road construction codes.

Favorable rock quality help in construction of tall caverns. This makes a high degree of stability with relatively little rock support possible. Together with a normal level of pre-construction investigation, the Q-system gives acceptable basis for design and construction. In addition, the site needs continual supervision by an engineering geologist.

Joint systems

The structure and degree of fracturing of the rock mass varies in the area. The jointing system are mainly dry, but there have been registered some areas with water from gouges in the system.

The existing rock caverns are in an area with few weakness zones. During excavation of the rock caverns, a small number of weakness zones was encountered. None of these contained any large amount of water. Heavy jointing, as well as infillings with altered swelling clay materials have challenged the rock support operations as well as the productivity of the last caverns.

Hydrology and water treatment

Stendafjellets highest peak is 232 meters above sea level. The water drains naturally northwest towards Råtjørna (40 meters above sea level), northeast towards Apeltunvatnet (32 meters above sea level) and west towards Grønnestølen, to a creek and bog that follows the bottom of Rådalen valley and further to Råtjørna from the highest point of Stendafjellet. Apeltunvatnet run-off to Nordåsvatnet (sea), and Stendavatnet drains towards the Fanafjord.

A meteorological station at Stend show normal yearly precipitation at 2 041 mm. The Flesland meteorological station show 1 815 mm of normal precipitation during one year. It is anticipated a higher precipitation at Stendafjellet due to topography, compared to Flesland.

NVE (The Norwegian Water Resources and Energy Directorate) has measured the water run-off at two locations at the foot of Stendafjellet. They found yearly water run-off to be 1986 mm/year and 1936 mm/year in 2013. The yearly water run-off, calculated by NVE, is 60-65 liters per second per km2. From the entire Stendafjellet area it is presumed that the water run-off is about 36-40 m3/hour. This constitute about 16-18 m3/hour above the existing rock caverns, and 19-21 m3/hour above the planned rock caverns.

The terrain at Stendafjellet is relatively steep and water run-off therefore probably happen as rapid surface run-off. The ground conditions appears dry, and there is no ponds, lakes, rivers, creeks or bogs on the mountain.

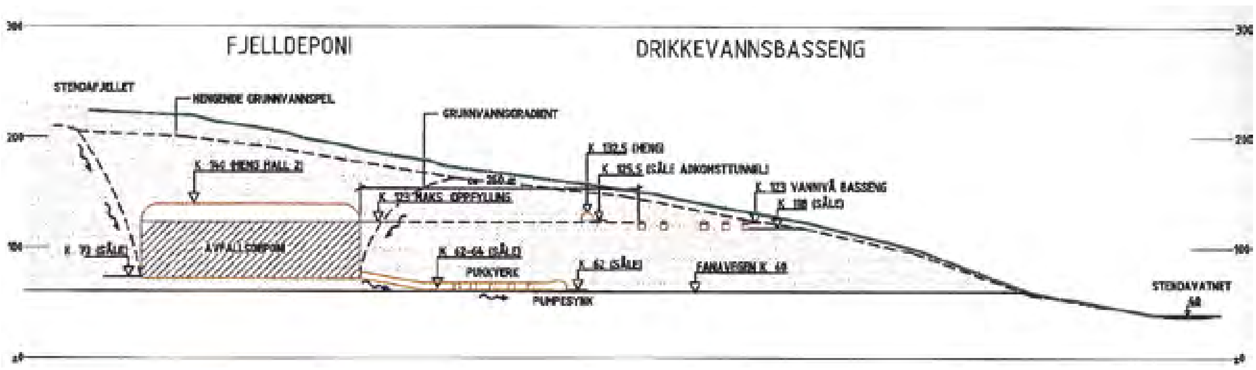


Figure 3. Cross section a long a north-south profile through the cavern.

Water flow happen within joint systems and channels in the rock mass. The permeability of the rock mass is low, apart from the joints- and weakness zones. These zones are considered "watertight". At the current climatic conditions.

As mentioned before, the rock mass at Stendafjellet consist of few joints- and weakness zones. During excavation of the first rock caverns, there were two areas with considerable water leakage. The leakage at these locations decreased rapidly and stopped after some time.

In weakness zones crossing the plant, there is only registered small amounts of water leakage. This is due to filling material, which is powdered and fine structured rock material, partly clay fractioned.

A common problem is the connection between excavation of rock caverns and the drawdown of the water table, and/or changes in water level and discharge of lakes, rivers or bog areas. Because of the great distance from the closest recipients and the absence of surface water, wetland areas and bogs, this is of no concern at Stendafjellet.

Measurements around the existing rock caverns show ground water levels lie deep in the rock mass at Stendafjellet. The measurements also show that the water level varies in correlation with precipitation and seasons. The ground water level has not been affected by previous excavation.

Water enters into the rock caverns with a very low flow rate. The gravitational water from the rock caverns are sampled, analyzed and categorized as contaminated according to the law on pollution, but not dangerous.
Extensive monitor programs for registering the ground water and chemical quality in and around the rock caverns have been established. Registration started before development of the waste disposal site, and therefore this is a good basis for evaluation.

Initially, the ground water level was measured at eight locations. Today, the monitoring program consist of sampling from seven wells for inspection and one reference point. Two wells are inside the rock caverns to survey the area between the waste disposal site and drinking water basin. Five wells are at potential effluent water areas around Stendafjellet. Two wells are located between the waste disposal site and water wells at Rådalen.

The water level in the wells outside the rock caverns have been continuously supervised over the last couple of years. Measurements show that water level vary rapidly with strong correlation to precipitation.

Sampling tests 4-6 times a year analyze for about 200 different environmental toxins. In addition, quarterly samples of the seepage water is tested before it is transmitted to the municipal water grid. As mentioned before, there is no surface water near the plant, and the county administrator in Hordaland has given exemption from the instruction of The Norwegian Environment Agency (1st of July 2013) concerning the collecting of samples from the surface water.

Analyzes show that the ground water is not influenced by seepage water from the waste disposal site. Water level measurements confirm that the ground water gradient has direction towards the rock caverns.

The existing rock caverns make an artificial barrier. The water can not flow uncontrolled from the caverns to the surface. The water is collected and lead to a suction pit at the lowest point of the plant. This water is lead to the municipal treatment plant.

Location and design

The first generation plant consisted of crushing- and sorting plant, access tunnel and storage chamber for rock. The rock cavern is excavated by top heading, and deep-hole drilled benches. The bench heights are more than 25 m.

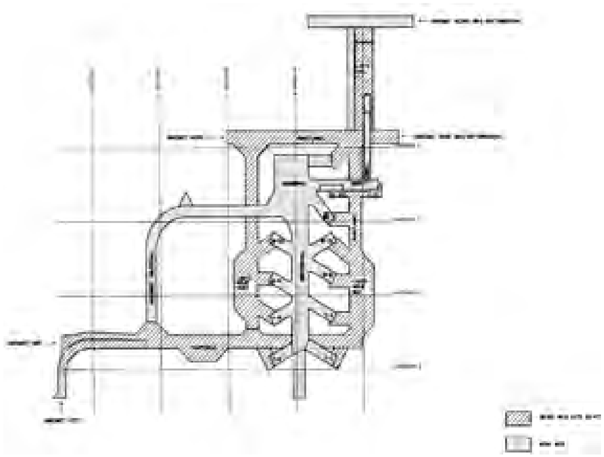


Figure 4. The mining area and the crushing plant are connected through the loading area at the lowest level where the primary crusher is located. The secondary crusher is located at the upper level and connected to the primary crusher and storage shafts and niches at the lowest level by conveyor belts. The lowest level are shaped as a ring tunnel, connected to the south and north access and transport tunnels.

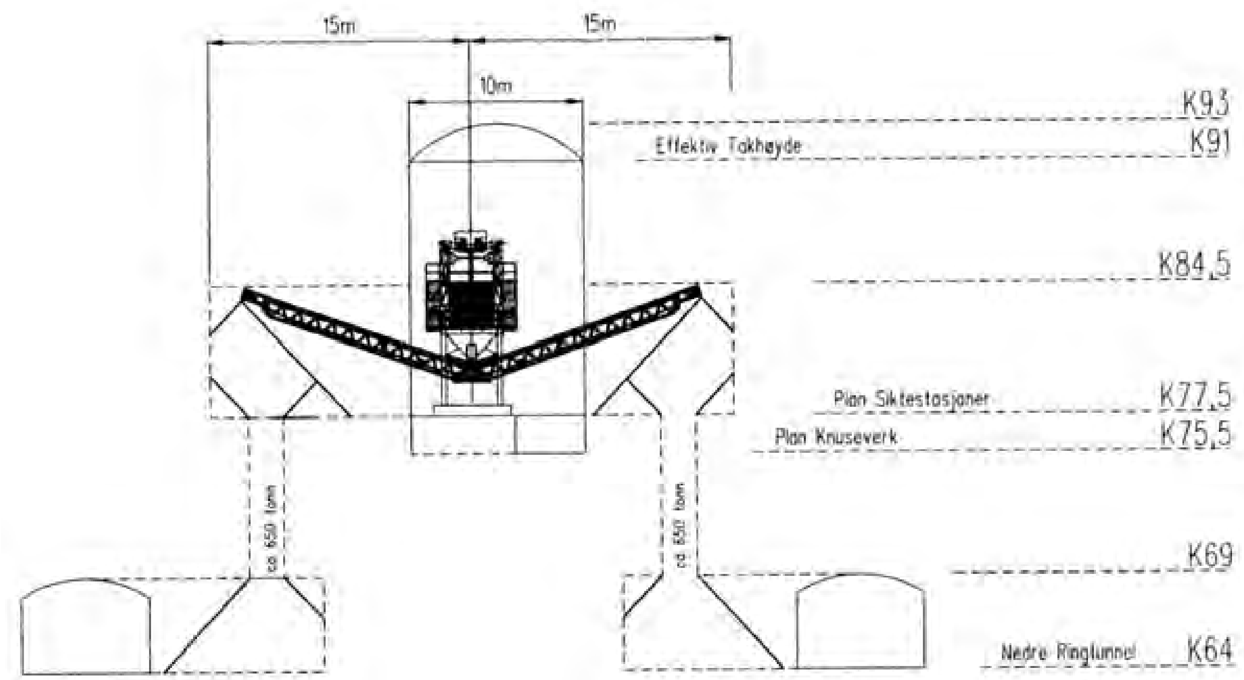


Figure 5. Cross section of the secondary crusher/sorting cavern with conveyors to storage niches.

The lowest level is a ring-shaped tunnel, connected to the south and north access and transport tunnels. Operation of the quarry have high level of automation, requiring one person for running the operation, and two persons driving a wheel loader and dump truck delivering stone to the crusher.

Caverns for mining and storage

Today, the caverns are excavated with top heading, access to the middle level and access from the lower level in one part of the cavern. This is due to the loading and transportation logistics. The bench heights are normally 12 m. The top heading rock support consist of system bolting and fiber reinforced shotcrete. The benches are supported by end-anchored bolts, and shotcrete if found necessary. The access for transport vehicles at the lowest level is restricted with safety zones between transport roads and the cavern walls.

The caverns are orientated north south. The width of the caverns is 25 m, the height is approximately 50 m. The lengths varies between 120 and 200 m, limited by existing shear-zones. The distance between the caverns is 25 m, and the volumes vary from 120.000 m3 to 200.000 m3.

It is applied for excavation of eight similar and parallel caverns from the west side to the east side of Stendafjellet. The length of the remaining, and planned caverns, might increase up to 200 m and 400 m. It is also possible to increase the height of the caverns up to 100 m.

Photo 1. Rock cavern, 200 m long and 25 m high (http://www.fsg.no/index.php/bilder/bilder-deponi)

Photo 2. Lorry-mounted inspection platform for mapping and installation of rock support. Photo by FSA, Multiconsult.

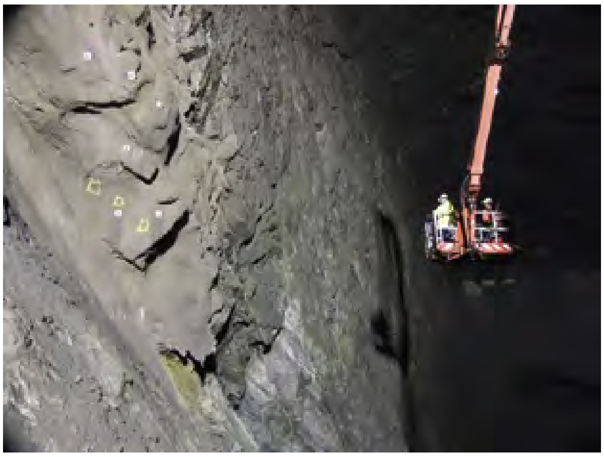


Photo 3. Inspection of weakness zone in east wall of cavern no. 7. Photo by FSA, Multiconsult.

Photo 4. Close up photo of zone with infilling of swelling clay (light gray). Photo by FSA, Multiconsult.

Photo 5. Installation of rock support in wall by drilling with long stick excavator with hydraulic hammer. Bolt installation is carried out by operator in a separate lifting platform. Photo by FSA, Multiconsult.

Photo 6. Photo showing access tunnels at different levels at the south end of the cavern. Photo by FSA, Multiconsult.

Operation of the waste disposal plant

All incoming material is controlled and checked. The caverns are filled with remote controlled equipment from the upper level access tunnel. Only a small amount of manpower is required for operations. It is possible to do filling from separate tunnels from above or the side of the caverns.

The existing permit for depositing polluted materials in the caverns are summarized by a list of identified waste categories.

Regular control of the drainage water is carried out in order to check for contamination and for analyzes purposes. The low permeability of the rock result in very small leakages (0,1-0,2 m3/min) for the whole cavern system.

Photo 7. Inside the rock cavern, upper side, for waste disposal of contaminated masses. (http://www.fsg.no/index.php/bilder/bilder-deponi)

Environmental benefits and main conditions for successful quarry

The excavation increase the surface altitude, and reduce air pollution, noise and vermin in the adjacent areas with respect to both the crushing plant and the waste disposal site. It minimizes the risk of ground water contamination. The underground openings are designed to allow water from the production areas and the disposal areas to drain naturally towards an effluent control pit with pump instalations. This pit is located in the south access tunnel, at the lowest point in the entire system of underground excavations. After passing through an oil and mud separator, the drainage water connects to existing pipeline system for collection of drainage water from old waste disposal sites at Rådalen, and subsequently to existing sewage plant and approved sea discharge.

All urban societies are consumers of crushed rock and produces waste and polluted materials. At Stendafjellet, short distance to the consumer compensates partly for the more expensive method of quarrying for construction quality stone material by underground methods. In filling of polluted materials, which otherwise would have to be transported long distances to other approved deposit sites, contributes to a profitable operation. Looking at the environmental side of the operation, no agricultural or forest area have been altered, the noise and dust from quarrying is negligible. CO2 – emissions from the transport distances for materials in and out are also considerable when comparing to existing alternatives.

Further development

Whether the quarry/deposit is to be extended further, will depend on the market for both aggregates and depositing. And on the taxation and permits for competing quarries and depositories in the area.

The operation also shows that it is possible to excavate caverns with large vertical dimensions at a low cost in an area with a high level of urban development. Such compact low-cost volume storages can replace storage methods which normally require larger areas, e.g. container storage and unmanned robotic storage installations.

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5.8 SPECIAL TECHNOLOGIES AND METHODOLOGIES

5.8.1 GEOTECHNICAL INVESTIGATION WITH DIRECTIONAL CORE DRILLING

1. Introduction

Traditionally, geotechnical pre-investigation has focused on coring vertical or inclined boreholes along the planned tunnel alignment, aiming to intersect uncertain geological zones located by other exploration methods. With such boreholes it can be difficult to confirm whether the zone actually has been intersected, as well as knowing the extent of it and whether it will look the same at the depth of the tunnel alignment.

To overcome this uncertainty, directional core drilling may be utilized. A specially designed steerable core barrel is used to either drill from ground level at an inclined angle turning to horizontal or directly horizontal, depending on topographic conditions. The borehole is directed to drill along a pre-defined trajectory and collect core samples over the full hole length. The core gives firsthand information about the rock quality, water inflow and geological structural information near the tunnel alignment.

The use of directional drilling is likely to result in improved support estimates, reduced contractor claims and minimized problems during construction. Additionally, the increased certainty in the geological formation may reduce conservatism in the tunnel design and increase construction rates.

The aim of this paper is to present the technology used for directional core drilling, how it is used and the factors to consider when deciding if the technology is suitable for a specific project.

2. Equipment specifications
The directional core drilling technology developed by Norwegian company Devico AS, consists of a wireline operated core barrel that corrects and adjusts the trajectory of the borehole. This core barrel, DeviDrill, is used only in the sections where directional change is required, while a standard wireline core barrel is used in the reminder parts.

The DeviDrill is an N-sized (76 mm diameter) core barrel designed to fit directly on the drill string similar to the standard core barrels. As the borehole reaches the depth where directional drilling is necessary, the drill string and standard core barrel is retrieved from the borehole, and the standard core barrel replaced with the directional. The directional barrel is designed to operate with the same parameters and with the same equipment as the standard core barrel.

The DeviDrill consists of a packer system holding a fixed orientation during drilling, and an offset drill bit steering the borehole in the direction of the set orientation. During drilling a 3 meter core sample is collected in an inner tube assembly that is retrieved from the borehole with a wire and overshot system.

Figure 1: Directional core barrel

Once retrieved, an empty inner tube may be inserted in the drill string and pumped down to the core barrel with water pressure. The core sample has a diameter of 31 mm during directional drilling, while 50 mm during the standard N-sized drilling.

3. Data collected
Directional core drilling provides the opportunity to guide a borehole along a predefined trajectory and intersect specified zones of interest. During the drilling, a rock sample, core, is continuously collected from the full borehole length.

When the borehole is steered to follow the planned tunnel alignment, the properties of the collected core are representative of the ground properties that will be encountered during construction. The location of geological features (such as fractures or fault zones) can be determined with high accuracy, while mechanical properties can be tested directly on the core. Since the direction of the hole and the tunnel is nearly identical, the magnitude and the properties of the various ground structures seen in the core will be the same as what will be encountered during tunnel excavation.

Figure 2: Position and extent of fault zones are mapped (4)

For additional information of the structural data, orientation equipment is used during the drilling process. This equipment is fitted to the inner tube assembly and uses accelerometer technology to log the orientation of the core sample as it was positioned in the borehole. When combined with directional survey data the core orientation enables calculation of strike and dip of the structural data.

As the borehole progresses the ground water conditions and permeability may be measured with packer testing. Both absorption (Leugon) and inflow may be tested to get the most realistic understanding of the conditions.
A few geophysical tests may also be performed upon completion of the borehole, for instance borehole-surface or borehole-borehole tomography.

4. Area of use
Directional coring may be useful in any tunneling project, but is most common when tunnels are planned in areas with complex geology or underneath urban areas, water or areas with restricted access.

When the geology is fluctuating or consists of zones that are less desirable for tunneling, drilling single holes at set intervals may not be sufficient to get a complete understanding of the geology. Using directional coring to get a continuous core samples from the tunnel alignment will give a more complete picture of the geology and help determining the optimal tunnel design and location of the tunnel alignment. In figure 3 an example from Detroit is shown. Due to the results of other investigation techniques the engineers designed a complex vertical split tunnel alignment, but were able to adjust this to a single tunnel alignment after completing a directional coring program.

Figure 3: Original design to the left and final design after

directional coring to the right (5)

The ability to steer the borehole trajectory is of additional value when the drill rig cannot be aligned with or positioned near the planned tunnel. This is the reason why directional coring has been part of the pre-investigation program for most subsea tunnels in Norway and multiple tunnels in densely populated Hong Kong. Using this technology the borehole may be started at an angle from the tunnel alignment before it is adjusted to run parallel to the alignment, as illustrated in Figure 4.

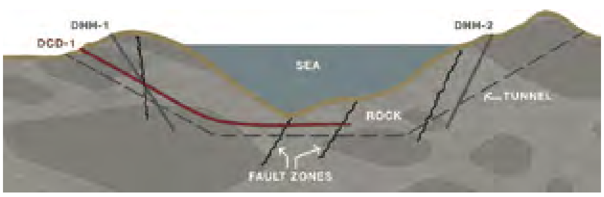

Figure 4: Directional borehole (red) turns horizontal and reaches fault zones in relevant area.

When a standard and/or directionally drilled borehole is completed, a new branch hole may be drilled from the existing hole. Directional coring is used to deviate the branch hole away from the existing hole, without the need of moving the drilling rig. This method is commonly used for drilling programs where the first hole encounters low quality rock zones. The tunnel alignment may then be adjusted and a sidetrack borehole used to explore the new path, as shown in Figure 5 from the Bømlafjord tunnel.

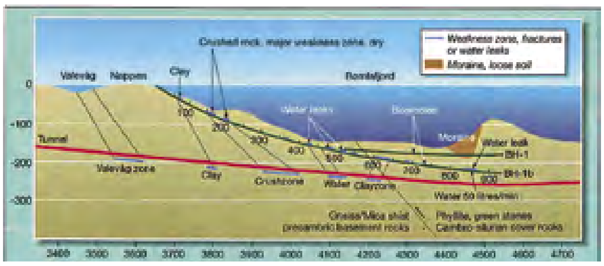

Figure 5: The borehole initially hit moraine (BH-1) and a branch hole(BH-1b) was drilled underneath (2)

5. Technical considerations and limitations
Before directional coring is initiated several factors should be considered in order to design a feasible and efficient borehole that achieves the target of the exploration program. A borehole that is not properly planned before start-up may not be able to follow the designed path or intersect specific areas of interest.

Borehole path
The borehole may be started parallel with or at an angle to the planned tunnel alignment. This may depend on topography, infrastructure and general access to the area. The borehole should be drilled through soil and overburden with standard core drilling, and directional drilling first initiated when rock formation is reached.

When parallel with the tunnel placing the borehole within the planned tunnel profile should be avoided. This to prevent creating a passage for water to enter into the tunnel, and to reduce the risk of drilling equipment potentially left in the borehole causing issues during construction.

In most cases it is beneficial to keep the borehole a few meters above the tunnel crown, as this is the area that generally needs the most stabilization work and where the rock formation is of the lowest quality. In a few situations coring at the bottom or sides of the tunnel may be more valuable, for instance in changing geology where specific formations should be avoided during construction.

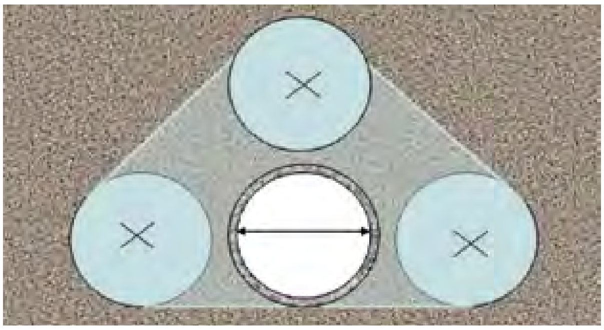

Figure 6: Potential borehole locations relative to tunnel alignment (3)

Tolerance envelope

As the borehole is drilled parallel with the tunnel alignment natural deviation will make it drift and deviate from the planned path. A tolerance envelope may then be set, allowing the borehole to deviate a certain amount from the planned centerline. Most commonly it is acceptable for the borehole to stay within a 5 meter radius from the plan, while smaller and larger tolerances may be used.

The tolerance envelope should be selected based on what is necessary for collecting sufficient geological data. Selecting a very small tolerance will require more frequent surveys and directional corrections for the borehole to stay within the tolerance envelope, increasing the production time of the borehole.

Curve radius

Deflecting a borehole is a gradual process, performed with a curvature radius of approximately 200 m. The radius may be increased, but should generally not be reduced. A larger radius will give less friction between the drill string and borehole wall, and that way ease the drilling process and increase the potential borehole length. A small radius will bring the borehole quicker onto the planned trajectory, but a radius that is too small will cause wear and potential damage to the drill string.

The effect of the directional core barrel will vary depending on the geology encountered, and the curve radius achieved will always fluctuate over and under the planned. It is important to survey frequently to quickly detect if the radius turns significantly larger or smaller than planned. In these cases the directional core barrel must be pulled out to adjust the intensity of the deflection angle. If the radius is considerably smaller than planned, the borehole must be reamed or cut to avoid potential damage of the drill string.

Start angle

In most cases the borehole will be started at an angle to the planned tunnel alignment, and directional coring used early to guide the borehole onto the desired path. A steeper angle will bring the borehole quicker to the tunnel alignment, however it will also require the directional drilling to be starter earlier and last for longer.

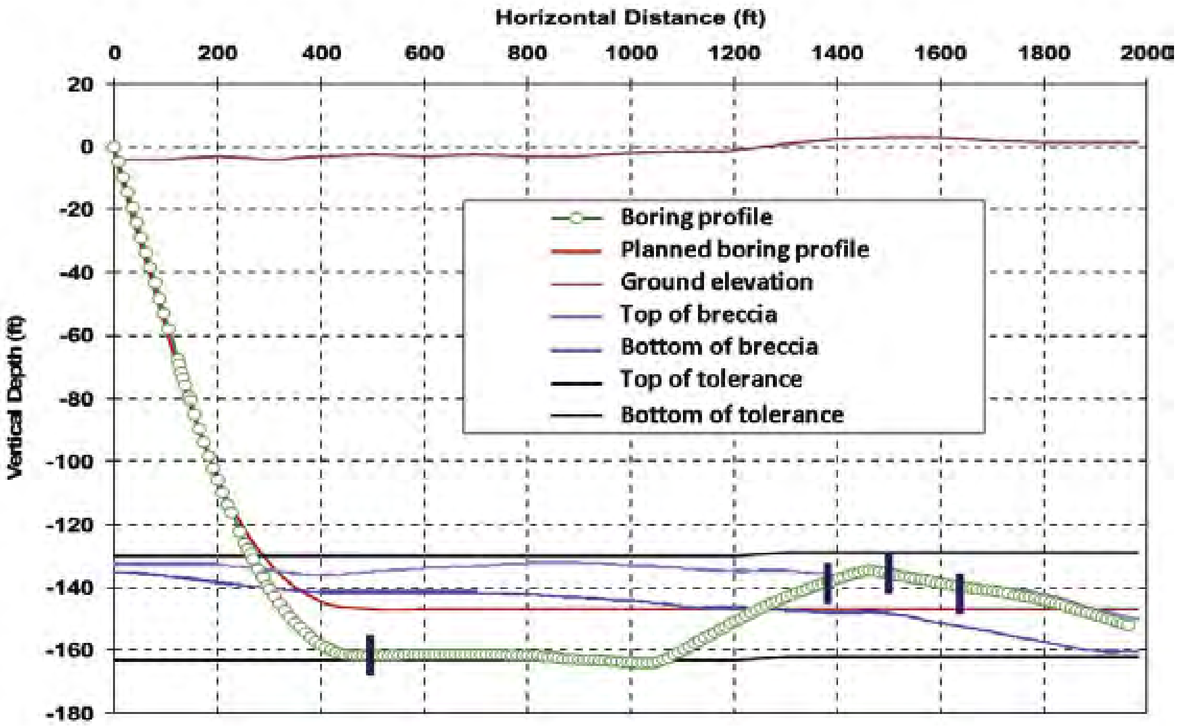

Figure 7: Directional coring used to stay within specified tolerance envelope (5)

The borehole is rarely started more than 45 degrees off the direction of tunnel alignment, but flatter if the start position is close to the planned alignment. If the angle is too steep or the correction started too late it will not be possible to correct the borehole in time. The correlation between start angle, distance to alignment and correction length is presented in the figure below. As an example, if the borehole is started with an inclination of -20 degrees it will take 75 meters directional coring to correct the borehole to horizontal. During the directional section the vertical displacement of the borehole will be about 12 meters.

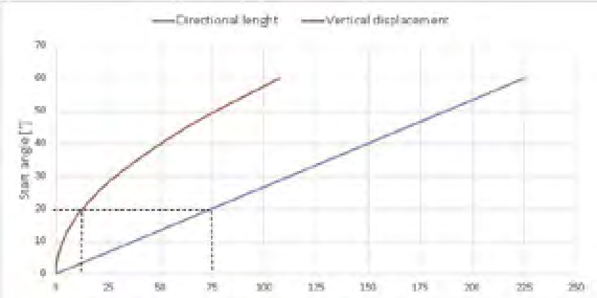

Figure 8: Correlation between start angle, vertical displacement and directional coring length (4)

Borehole length
The most common length of directionally controlled core holes for tunnel pre-investigation is about 7-800 meters, but holes with lengths down to 150 meter and up to about 1500 m have been done. Theoretically even longer boreholes can be completed with a sufficiently powerful drill rig and moderate steering, however the time perspective must be considered.

In horizontal boreholes the inner tube assembly and the overshot used to retrieve the inner tube assembly must be pumped into the drill string using water pressure. The longer the borehole is the more time is spent pumping equipment, leading to reduced production rates. Maneuvering through incompetent ground conditions will also be more challenging and time consuming as the depth increases. Instead of drilling one very long borehole it may therefore in some cases be more efficient to drill two shorter holes, covering the same area.

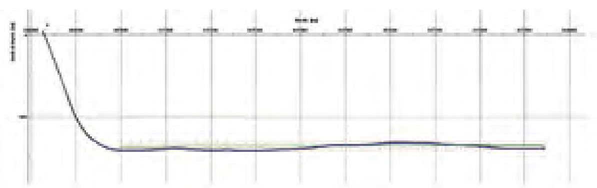

Figure 9: 1250 m long borehole drilled in Hong Kong with -45 degree start inclination (1)

Borehole diameter
The directional core barrel is available in N-size (76 mm diameter), but may be combined with other sizes if necessary. The drill strings are designed in such way that smaller sizes may be used inside larger, making it possible to telescope as the borehole progresses. The hole may be started in a larger size with larger core diameter, downsized to N for the directional sections, and later downsized further to maximize the potential reach of the drill rig.

If larger core size is required the borehole may also be downsized just for the directional sections, and then enlarged when returning to standard drilling after the correction. It must then be noted that larger drill strings are less flexible a require a larger curve radius than the N-size. For the next size up (H-size - 96 mm) the radius should be limited to 350 meter, compared to 200 for N.

Geology
The directional core barrel was designed for use in hard rock formations, and fault zones or soil formations should be avoided. In these formations it may be challenging to control the direction, while the production rate will be low. When such formations are encountered, standard core drilling should be performed until a more competent formation is reached.

Fault zones may also affect borehole stability, and should always be injected and re-drilled to prevent collapse or circulation problems.

Torque
Horizontal holes, and especially directional holes, subject the drill string to more friction than vertical or inclined holes. This is due to the drill string resting on the bottom of the borehole creating more friction.

To ensure the borehole reaches its planned length a drill rig with sufficient torque must be selected. It is important to note that the drill rig depth capacity is most often specified according to the pullback force and amount of drill rods it can hold. In horizontal holes this is not relevant, and the torque is the most important parameter.

If depth capacity is an issue steps may be taken to reduce the friction. Planning the borehole path properly, applying correct curve radius and avoiding unnecessary directional changes is important. A drilling fluid program aimed at lubricating the drill string and properly removing cuttings should be standard in all horizontal boreholes. In severe cases grease may also be applied to the drill rods.

6. Summary
Directional coring is an exploration technique that provides the design engineers with a relevant impression of the geology along the tunnel alignment. The collected core as well as the borehole can be studied and tested, providing information about the ground conditions difficult to obtain with other exploration techniques.
There are several factors affecting the success and efficiency of a directional coring program, for instance the borehole design, requirements, and geological conditions. The most vital part of a directional coring project is the initial planning. Mistakes in the drill rig set-up or start depth of directional drilling can lead to difficulties in the drilling operation and occasionally failure in completing the borehole plan.
Directional coring has been utilized for pre-investigation of tunnel alignments in multiple parts of the world, and is considered for all major and high risk tunneling projects in large markets like Norway and Hong Kong.

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5.8.2 GROUND FREEZING OF GLACIFLUVIAL SUBSEA ZONE AT 120 MBSL, IN THE OSLOFJORD TUNNEL.

The longest tunnel on the Oslofjord Crossing is the 7.2 km long undersea crossing itself. The 78 m2 highway tunnel comprises three lanes - one lane going downhill and two lanes on the uphill slope – and has an inclination of 7% to minimize tunnel length. It was the 19th subsea road tunnel to be built in Norway by The Norwegian Public Roads Administration (NPRA). SRG was the main contractor.

Ground freezing was required to get through a rock depression filled with permeable glaciofluvial material. Sea level was 120 above the tunnel crown. GEOFROST had the full responsibility for all the activities from design, procurement, construction, commissioning and hand over of the frozen structure.

GEOLOGY
The Oslofjord is situated in a major regional rift belt, the Oslo grabend, in which the total vertical displacement is about 2000 m. The bedrock consists of deep eruptive rocks and dykes of Permian age, dominated by coarse-grained granite, traditionally termed Drammens-granite. The rift belt crosses the Oslofjord tunnel.

The rift belt here consists of several faults and weakness zones, some of the major ones following the fjord over long distances. The seismic survey showed three wide channels in the threshold of the fjord, eroded by glaciations. In each of these channels, major weakness zones were confirmed. The "Hurum weakness zone" having the lowest seismic velocity of 2600 m/s was further investigated, among other methods by two penetrating core holes. One of the holes was made by directional core drilling along the assumed tunnel alignment. The other core hole was made from the other side. Core material consisted of familiar crushed rock and clay. They both missed the depression filled with frictional soil.

The 15 m wide zone of loose glacifluvial deposits at the bottom of a deep channel above the "Hurum weakness zone", was found by probe drilling from the tunnel head. It is believed that the channel was cut by a glacial melting river, leaving behind a permeable soil deposit containing sand and gravel in addition to the larger rounded blocks of rock; a glacifluvial deposit. Without any fines the zone was highly permeable with constant hydrostatic water pressure of 12 bars at the crown.

TUNNEL EXCAVATION

The Oslofjord tunnel contract was won by the Scandinavian Rock Group AS (SRG). There were 3 headings: one from the east side of the fjord and one in each direction from a 730 m long adit at the sea front on the west side of the fjord. The full face of the tunnel was excavated by conventional drill and blast methods, with an average advance of 30 to 40 m a week at each tunnel face.

It was from the eastbound heading from the adit the glacial zone was first encountered by a probe hole. As a routine, three 30 m long probes were drilled in the crown, having a 15 m overlap. It was soon after the overlap with the previous probe, the new probe struck water at enormous pressure, and it was clear that the probe had intersected with the full hydrostatic pressure of the 120 m head of the fjord above. Further investigations of the area showed that almost half of the cross section would run into the loose glacifluvial soil. The lower half contained fractured rock of the crushing zone.

It was decided to excavate a lower bypass tunnel, spiraling away from the main tunnel alignment, passing some 20 m below the glacial channel, and rising up to the main tunnel alignment on the other side of the zone. From there the tunnel proceeded eastward under the fjord as well as backwards to the problem zone. The bypass tunnel later became the drainage sump, replacing the one originally designed.

Grouting was the preplanned solution for progressing through this zone. Continued drilling problems indicated that the soil zone would be difficult to stabilize by grouting. The attempts to inject grout and seal the zone had no effect. In fact, more than 700 tons of cement based grout was pumped into the zone without the slightest effect in cutting off the water ingress or reducing the water pressure. Large volumes of grout/concrete would be needed and it would be difficult to control the result, if continued grouting were to be the final solution. It was therefore, after 5 months of grouting trial, decided that freezing was necessary to stabilize the zone before excavation. The decision was based primarily on the assumed higher and more controllable safety of the freezing method. The bypass tunnel had taken the problem zone out of critical time schedule.

GEOFROST, as a subcontractor, was responsible for both design and execution of the artificial ground freezing works, including drilling. 46 m was left between the two tunnel faces. It included the approximately 15 m wide zone of permeable glacifluvial and morainic deposits at the bottom of the deep channel, and some good quality rock on each side.

DESIGN OF THE FROZEN STRUCTURE

GEOFROST designed the frozen structure based on the Berggren creep model for frozen soil and design loads from GeoVita. Third party design verification was done by SINTEF.

Ordinary core drilling failed to give undisturbed samples suitable for laboratory testing, except for two small pieces. As a result, the frictional material flushing out of the drill holes were gathered, compacted and saturated with sea water before frozen and tested. All tests were performed at The Norwegian Institute of Technology (NTNU).

Due to the relatively low strength caused by the salinity of the porewater at normal ground freezing temperatures, it was decided that the design temperature should be as low as -28 oC. This temperature was both obtainable in the laboratory and could practically be achieved in the field. The cycle of drilling, blasting, excavate and produce the permanent lining of a section was planned to take one week.

Before excavation could start, three different requirements had to be fulfilled for the frozen structure:

1) The temperature of the main frozen structure should be -28 oC or lower.
   The length of each drill and blast round was then given as a function of the temperature and thickness of the frost structure. As an example, a thickness of 3 meters at -28 oC resulted in an allowed unsupported length of 2.7 m.
2) The soil part of the face should all be frozen to ensure safe working area and support of the stabilizing frozen structure.
3) The complete circumference around the tunnel should be impermeable to avoid water seepage through the invert and thereby thermal erosion of the frozen structure.

FIELD EXPERIENCES

A drilling chamber was established at one side of the zone. In the main frozen structure through the soil area there were two rows of freezing pipes, elsewhere only one. Drilling was performed by the companies Brødrene Myhre and Båsum Boring. They used down the hole hammers for drilling, and drilled through safety valves. Due to the high water pressure the normal air driven hammers were exchanged by water driven hammers with good results. It is believed to be the first time in the world water driven hammers were used in soil.

Drilling was extremely difficult. The soil consisted of very good quality boulders in a matrix of loose sand and gravel. Earlier grouting had no positive effect on permeability and hole stability. Drill rods and grouting equipment left in place from these works, as well as rock bolts, gave the drillers a hard time.

Deviation measurements were carried out for all holes, by Devico. Despite all the complications, only 12 of 115 holes were abandoned.

Coaxial freezing pipes were installed inside the casing to circulate
brine which was cooled by an ammonia based freezing plant, placed in the tunnel.

To ensure the design requirements were fulfilled, both before starting and during excavation, GEOFROST carried out a thorough temperature measurement program.

EXCAVATION SCHEME
The full face of 130 m2 was excavated by means of short rounds of drill and blast and then the full concrete lining, before advancing to the next section.

The excavation started from the opposite end of the installations. Because the freezing pipe pattern was coned, the thickness of the frozen structure decreased as tunneling proceeded. For this reason, excavation sections were planned to decrease from 3.0 to 1.5 m in length.

Excavation and lining were undertaken by the main contractor, SRG. Drilling pattern and loading of the holes were worked out in cooperation with Dyno. Special precautions were taken as the temperature of the contour were approximately -30 oC. Detonating fuse was used in the two outermost rows. Igniter was electronic. Otherwise anolit was used for the rest of the face of 130 m2. There were no problems with drilling in the frozen material and work proceeded as planned, with good results.

At the most, 40 % of the face consisted of soil. The bottom layer was a morainic material, well rounded, containing all fractions up to boulders of 3 to 4 m3. Above there was a layered glaciofluvial material, and stones from most parts of southern Norway was recognized in the zone.

The vault and face were shotcreted, with layers up to 20 cm, to avoid stones falling from the surface when heat from the working machinery rises and thaw the surface. Rescon Mapei delivered additives to the shotcrete works, preventing any problems with shotcreeting the very cold surface.

Concrete lining was 1.2 m in the bottom and 1.0 m in the vault. It was designed to take full water pressure. Strength requirements of the concrete in the lining before next blast, was 40 MPa. This was reached after approximately 20 hours as prescribed.

CONCLUSIONS
It was possible to plan, design and control all operations necessary for the project fulfilment. Drilling for the freezing pipes was the most challenging task. There were neither stability problems nor any water leakages. Shotcreting and concreting against the exceptionally cold ground surface worked very well without any problems. A very difficult situation was solved safely in a controlled manner.

Fig. 1: Tunnel face with freezing pipes installed.

Fig. 2: Frozen glacifluvial material after blasting.

Fig. 3: Frozen contour with freezing pipes appearing as cone is decreasing.

5.8.3 CT-BOLT A NORWEGIAN ROCK BOLT CONCEPT

1 THE HISTORY OF ROCK BOLTING

Rock bolting is one of the most efficient supporting devices ever developed. The earliest case of using bolts as a rock reinforcement method was in a slate quarry in North Wales in 1872 (Schach et al., 1979).

In 1918 a coalmine in Germany introduced bolts as a means of ground reinforcement (Lang et al. 1979). Mechanical rock bolts were applied in a metal mine in the United States in 1927 (Bolstad and Hill, 1983).

The U.S Bureau of Mines (USBM) attempted to carry out early studies on the fundamental mechanism of bolting techniques and applies the roof bolting system to maintain roof strata stability in 1947.

Image 2 - Typical old rock tunnel

NORWEGIAN TUNNELING

Norway is a typical hard rock area (see Image 3), where use of rock bolts and min. 80 mm shotcrete generally makes the permanent support in normal rock quality.

In area with weaker rock, spiling bolts in combination with steel bands are used, or steel arched anchored to the rock bolts and shotcrete

Bolt length is normally 2,4 to 4,0 m. The main type is a rebar bolt M20 anchored with a single resin capsule (used in area with rock burst) or fully grouted.

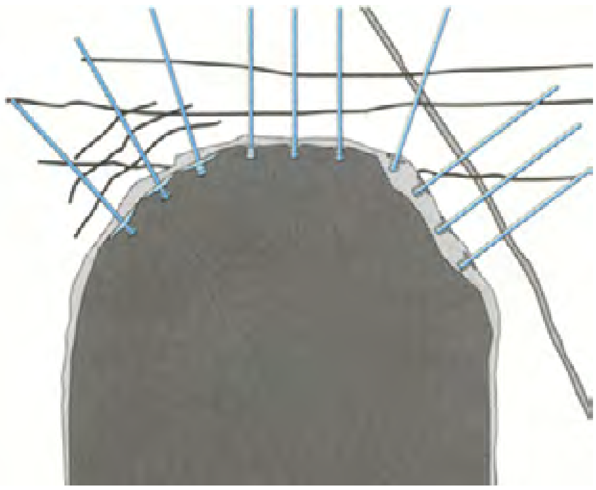

Image 3 – Rock bolts and shotcrete

A rock bolt type which has seen national and international application the last 10 year is the CT-Bolt ®. This bolt was innovated by Vik Orsta AS in 1994. The CT-Bolt® can be effectively installed in borehole diameter Ø43-Ø48 during the drill and blast process (see Image 4).

Image 5 - The CT-Bolt

Permanent rock bolts are hot dip galvanized with a zinc layer of minimum 65 micros and with an option of minimum 60 micros powder coating to a total protection of 125 micros. Especially in sub sea tunnels the rock bolts are both hot dip galvanized and powder coated. (Combi Coat®)

INNOVATION OF ROCK BOLTS
From the history of rock bolts, we can observe that it has taken 30-40 years between each innovation of a new complete rock bolt.

Until the CT-Bolt® was innovated by VikOrsta AS there have been four categories of rock bolts globally;

3.1 Mechanical bolts
Mechanical bolts or point anchor bolts where the anchor is an expansion shell. Normal bolt diameter 20-25 mm. Mechanical bolts are normally used as temporary support. (see Image 5).

Image 5 - Mechanical bolts

3.2 End anchored rebar bolts with resin
Normally Ø20 mm with or without a special spring for mixing the resin. In Norway this type of bolt is used in tunnels with rock burst (see Image 6).

Image 6 - End anchored rebar bolts 1

3.3 Fully grouted bolts.
With resin or cement grout. The bolt is based on a rebar steel Ø20-25-32 mm or a cable. Fully grouted bolts are normally used in area which require more support than a single anchor can give. Fully grouted bolts are better protected against corrosion than mechanical bolts (see Image 7).

Image 7 - Fully grouted bolts

3.4 Friction bolts.
There is mainly to types of friction bolts, the Split Set bolt that is driven into the hole by a hammer, where the hole has a smaller diameter then the bolt (see Image 8).

Image 8 - The Split Set bolt

The alternative friction bolt is the Swellex Bolt, which has a smaller diameter than the Split Set. The diameter of the Swellex is smaller than the borehole and the bolt is expanding after installation by using high-pressure water to blow it up (see Image 9).

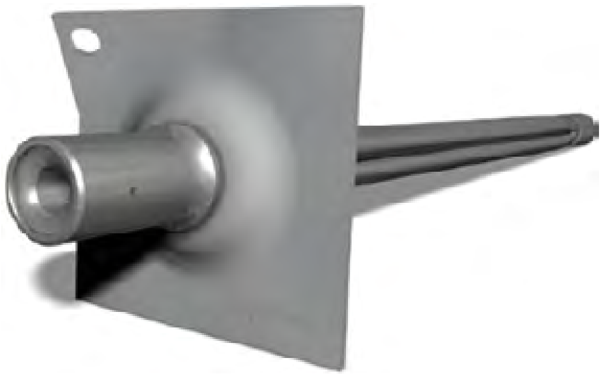

Image 9 - The Swellex Bolt

3.5 Combi rock bolts, the CT-Bolt®
Further description of this bolt on the following pages.

Image 10 - The CT-Bolt

THE CT-BOLT ® - THE OPTIMAL SOLUTION FOR LONG TERM ROCK SUPPORT

The CT-Bolt® is the latest global innovation of a complete rock bolt. The bolt was developed specially for the Norwegian sub sea market due to heavy corrosion from salty conditions.

Following a successful introduction of the technically advanced CT-Bolt® in the Norwegian and the Nordic market, the CT-Bolt® became a global product in the tunnel and mining industry.

The CT-Bolt® is installed as a temporary support with an expansion shell for different bore hole diameters. Later, the bolt can be fully grouted and become permanent. Before grouting the bolt, it can be prestressed.

The bolt is quick to install, simple to grout even with varying VC-ratio. At least the polyethylene sleeve seals the bolt from corrosion, even in holes with running water.

The bolt can be installed manually or fully mechanized from a bolting jumbo.

The CT-Bolt® has a special hemispherical dome which serves both to hold the rock load on the plate and as a grouting channel. The cement grout is pumped into the hemispherical dome and flows through the polyethylene sleeve. At the end of the bolt and near the expansion shell, the grout flows out of the sleeve and return back to the other end in the gap between the outside sleeve and the bore hole wall (see Image 11)

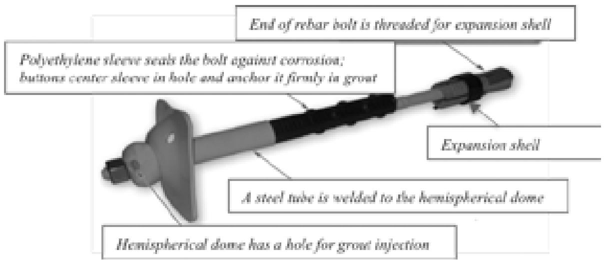

Image 11 - Functions of the CT-Bolt

When the bolt is grouted, it takes loads along its full length as well as the bearing plate. The polyethylene sleeve also seals the bolt against corrosion. The buttons on the sleeve transfer the load from the rock to the rebar bolt inside the plastic sleeve and center the bolt in the borehole. If the area must be sprayed with shotcrete before the bolt is grouted, it is possible to connect tubes to the hemispherical dome and the plate, and grout the bolt after shotcrete the process. Many projects globally have proved the efficiency and reliability of the CT-Bolt® since its first use in 1994. It has been specified in sub sea tunnels, road and railway tunnels, sewer tunnels, mines and slopes globally.

Table 1 describes the capacity and dimensions of the CT-Bolt®.

Dimension mm	Yielding load	Failure load	Lengt up to mm
M20	140 KN	170 KN	8000
M22	290 KN	290 KN	8000
M33	345 KN	410 KN	8000
M33	345 KN	410 KN	6000 +6000

Table 1 - Capacity and Dimensions CT-bolt

8. 5 DOCUMENTATION OF THE CT-BOLT®
The CT-Bolt® is documented by an external consultant Noteby AS, (Multiconsult) Norway and a doctor degree by Gisle Stjern at NTNU, Trondheim Norway.

5.1 Details from The Noteby report:
Field tests with the CT-Bolt® was done in the sub sea

tunnel project, Hitra-Frøya during the period the tunnel was driven (see Image 12).

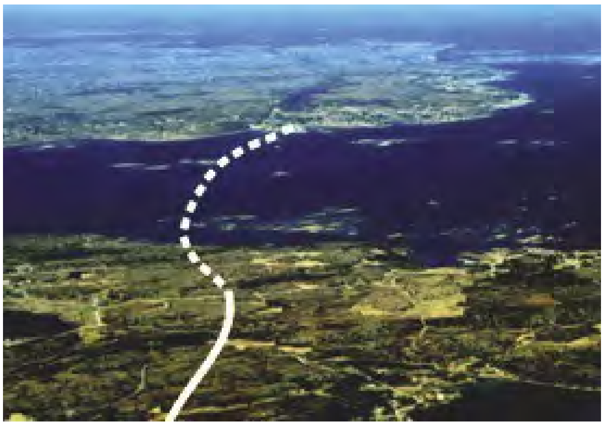

Image 12 - The Hitra-Frøya subsea tunnel

Because of highly corrosive environment, the bolts where protected with hot dip galvanizing and epoxy powder coating.

The CT-Bolts® where installed at the face for temporary support and later grouted. No major problems were observed during installation or grouting. The cement grout was mixed to a w/c-ratio of approx. 0,40-0,45, which gives a creamy consistency.

Four of the installed CT-Bolts® where later removed by large diameter core drilling (see Image 13), and transported to the laboratory for pull testing. The bolts where only 2.0-meter-long to facilitate the core drilling and achieve cores parallel to the installed bolts.

Image 13 – Core sample

In the laboratory bonding between the grout and the PVC-tube was tested through pull tests on the four bolts, which were removed from the tunnel site.

In the laboratory CT-Bolts® where installed in plexiglas tubes and grouted with different w/c-ratio. 0.38-0.42-0.46-0.50. After the grout in the plexiglas tubes had set, the tubes where stored under constant temperature, +20 gr.centigrade, and humidity 50-70%. After approx. four weeks the tubes were cut in halves with a diamond saw.

5.2 Conclusion from the Noteby report
The CT-Bolt® where grouted with a ready-mix mortar. Recommended w/c-ratio is 0.38-0.46.

Other types of mortars may require different w/c-ratio. The tests indicate that the CT-Bolt® has better than normal grouting properties.

With a bond length of 280 mm the CT-Bolt® has a capacity which complies with the yield strength of a 20 mm K500TE rebar. This implies that the introduced PVC-tube does not reduce the capacity of the rock bolt.

CT-BOLT®: INSTALLATION
The CT-Bolt® is installed in a bore hole of 44-51mm diameter, using the expansion shell. The hole is drilled to at least the length of the bolt. The bolt is prestressed so the plate give immediate support (see Image 14).

Image 14 - Installation of the CT-bolt by expansion shell

To grout the bolt, use an injection nozzle on the grout hose. Insert the nozzle in the hole in the hemispherical dome and pump the grout. It flows up inside the polypropylene sleeve and back down between the sleeve and the drill hole (see Image 15).

Image 15 – Grouting process

When the grout flows out through the hole in the plate and cure, you have solid full-length anchorage. The polypropylene sleeve seals the bolt against corrosion (see Image 16).

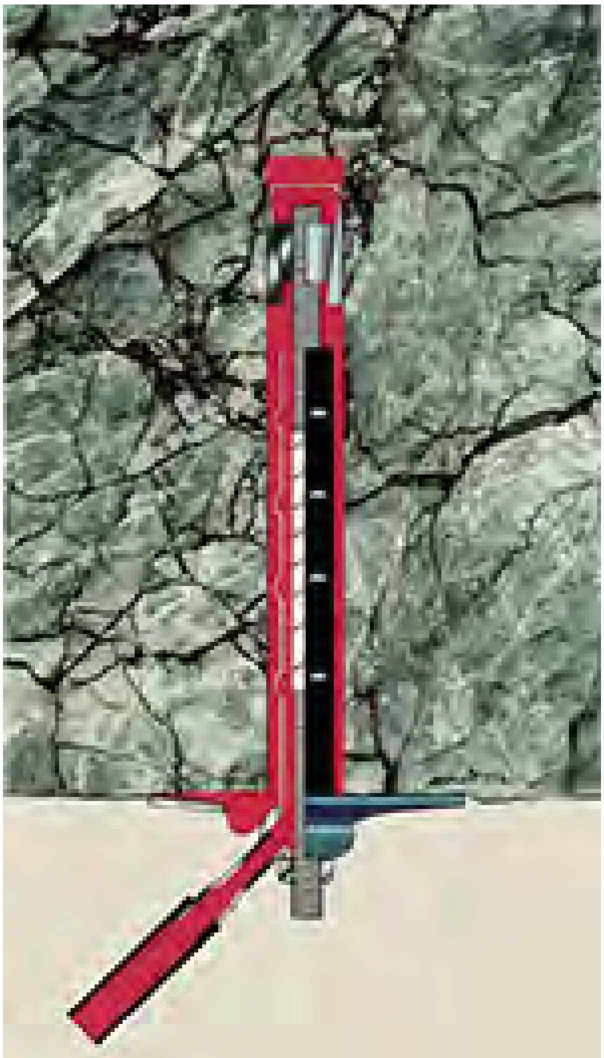

Image 16 - Grouting process

Image 20 – Installation of the CT-Bolt®

WHY A DEVELOPMENT WAS NEEDED?
For many years, various methods of fully grouted bolts have been considered as acceptable corrosion protection of rock bolts. Results from over cored rebar bolt show that the quality of grout, seldom give a 100% corrosion protection.

The first rock bolts that were used in the Norwegian sub sea tunnels were only hot-dip galvanized. The next generation we developed was a tubular bolt (1982) with hot dip galvanizing and a layer of tectyl as corrosion protection. Then we developed the process for galvanizing and powder coating of rock bolts, also called the CombiCoat® process.

The CT-Bolt was developed to meet the future demand from Public Road Authority, which required 100 years of lifetime for the rock bolts.

The CT-Bolt® is easy to grout and the plastic sleeve center the rebar and seal the bolt if there is running water in the bore hole. Therefore, no water or moisture will reach the steel since the bolt is 100% grouted inside the plastic tube.
(see Image 26).

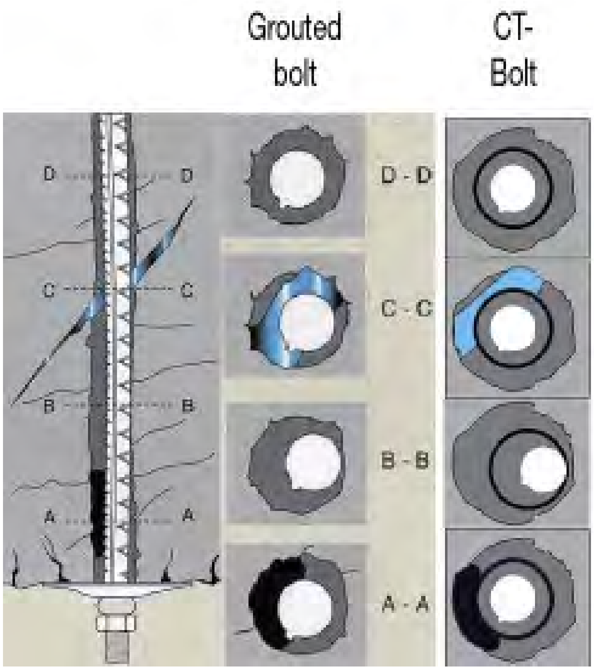

Image 17 – Illustration of boreholes with running water

In extreme environment, or projects it is not possible to enter later, for example caverns for storage of nuclear waste, the CT-Bolt® can be delivered complete in stainless steel.

REFERENCES
Schach et al., 1979, Jalalifar, 2006, Peng, 2008, Hillyer, 2012, Cao 2012 referred to in The History of Rock Bolting, URL: http://rockbolting.miningst.com/history-of-rock-bolting/.

5.8.4 INTERPRETATIONS AND POSSIBILITIES WITH SOFTWARE-CONTROLLED TUNNELLING EQUIPMENT

INTRODUCTION
The collection of Measurement While Drilling (MWD) data from drilling jumbos started in the late 1980s in Scandinavia, with the idea that it might, at some point, be useful in some way. Today, the interpretation of MWD data is widespread in Scandinavia, and for some geologists considered an absolute necessity. The Norwegian Public Roads Administration (SVV) and BaneNOR both have MWD interpretation set as a standard requirement in their tunnelling projects.

As the MWD interpretation has become more widespread, software for a better understanding of other tunnelling equipment has also emerged, especially where the potential for cost reduction has appeared to be the largest. Among these are shotcrete scanning, and grouting control. Some applications will be presented below.

MWD INTERPRETATION
Most drill rigs meant for large-diameter tunnelling in the Scandinavian market are pre-fitted with sensors for logging the different parameters of the booms of the navigated jumbo. Normally, every 0,02m of drilling, the parameters hammer pressure, rotation pressure, rotation speed, feeder pressure, water pressure, water feed and penetration rate are logged by the sensors and stored as text files on the jumbo's computer. After the end of each round, the data can either be transferred to an office computer by memory stick, or directly to a dedicated server if WLAN is set up in the tunnel and on the jumbo.

The Bever Team Online (BTO) software is an online service for registered users of the specific tunnelling project. The software collects data from the previously mentioned dedicated server every 5-10 minutes, so that information is always up-to-date for any user in the project. The web site can be reached as long as there is internet access, and can be used with any smartphone, tablet or computer.

The most desired possibility with the MWD interpretation is to discover in advance when the jumbo is going to go through weakness zones. For that purpose, MWD interpretation is used with long hole drilling, usually 4-7 holes of 20-30 meters of length, to discover both the rock quality ahead, and the possibilities of water inflow.

Figure 1 shows how the changing parameters of the long hole drilling can be identified visually as a weakness zone by the aid of a rotating 3D-PDF. This can be used for documentation purposes, either by saving the 3D-PDFs, or generating .tiff-files like in Figure 2 to be exported into for example NovaPoint Tunnel.

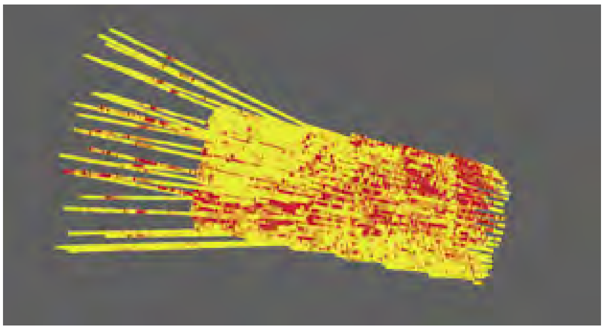

Figure 1: Long holes give a preview of the coming weakness zone, in red. From a movable 3D-PDF [2].

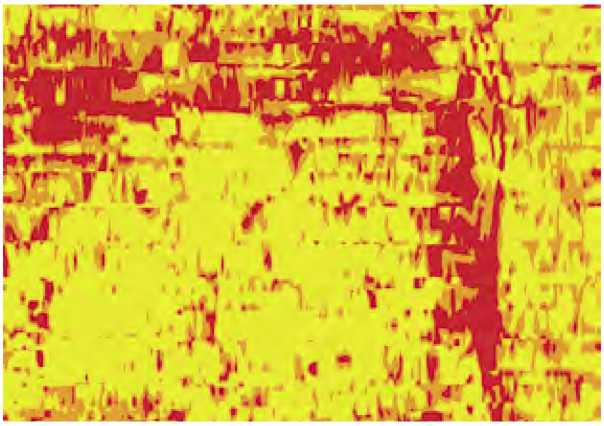

Figure 2: .tiff image file for documentation purposes. [2]

Other possibilities with navigated drill data is, among other things, to show location and length of bolt drilling, put together drill logs from several tunnel faces to show upcoming larger-scale weakness zones, and place the MWD over maps to show when and where one passes critical points in the construction, like rivers or housing areas.

SHOTCRETE SCANNING

In Scandinavia, shotcrete is normally used as final lining for permanent rock support in tunnelling. Shotcrete thickness is a critical parameter for the success of the process. Bever Control has developed a laser scanning system that can be operated from a mobile vehicle like a drill rig or shotcrete robot. The system is robust and well suited for the environment in tunnel construction. Scanning results are presented in a topographic map of the tunnel surface. The system gives the operator measurement results during spraying, as well as it will be possible to present a documentation of thickness over each blast or setup of the shotcrete operation.

The system was developed in cooperation with the LKAB - Kiruna mines from 2012-2016. Boliden have two unites in operation, and NCC in Norway have one unit in operation in E16 Sandvika. The system has proven reliable and today it is used for one scanning every 2nd day. In the operator training programme, the scanning is an efficient tool to see the operator performance.

Thickness is measured with better than 10 mm accuracy as average per m2. This is proven with extensive drill tests and reproduction of the results are very good. LKAB Berg & Betong has reported 20-30 % savings on concrete volume due to more accurate thickness control and operator training. In one year, the concrete volume is reduced from 2.8 m3/tunnel meter to 2,0 m3/tunnel meter. Reduction was possible due to good knowledge of thickness distributions and LKAB Berg & Betong could reduce the systematic shotcrete volume. Based on an annual spray concrete work of about 200 mill SEK/year this will save the Kiruna mines an estimated 40-60 mill SEK/year.

In Figure 3, some of the views available after a round of shotcrete scanning. The desired average for this 6-meter round is 80 mm, with tolerances from 40mm to 120 mm. The light green colour displayed mostly in the roof shows that the shotcrete lining is too thick there, compared to the specifications. The walls are within range, and closer to the tunnel floor the lining is thinner than desired.

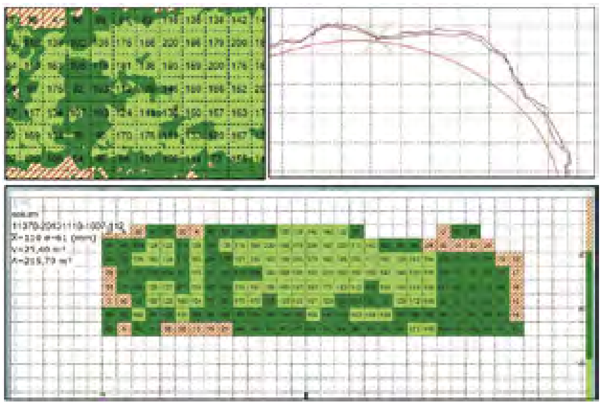

Figure 3: Shotcrete thickness maps [2].

GROUTING CONTROL

In modern tunnelling, extensive grouting work is performed to reduce the inflow of water. Sealing the tunnel is especially important in urban areas and subsea tunnelling, to avoid a lowering of the ground water table, or seawater inflow. In Norway, large volumes of materials are used for this purpose. Cost in time and workload is increasing, along with redeeming operations when the sealing has been unsuccessful. To optimize the quality and cost, more precise control criteria are needed. Real time monitoring is one way to obtain this.

Figure 4 shows an example of a grouting umbrella containing 20-meter-long holes, where the ends are designed to reach a distance of 5-6 meters from the tunnel contour. Grouting operators will normally manage 3-4 grouting lines simultaneously, where they need to consider recipe changes and trends for the controlling parameters. Stop criteria may be a pre-determined end pressure in the hole, or total litres of mass injected.

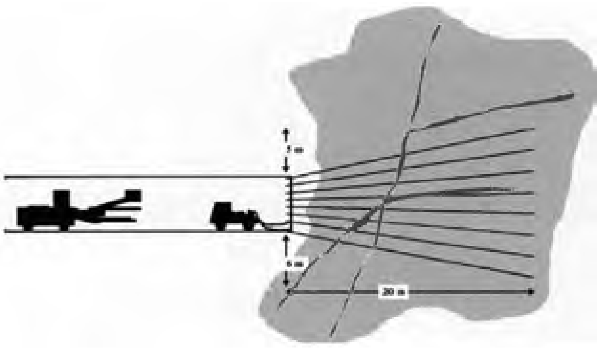

Figure 4: Grouting umbrella as applied in Nordic tunneling [1].

With real-time monitoring, sensors on the grouting rig record pressure, flow and accumulated volume for each grout hole. The information is displayed graphically to the operator, as in Figure 5. Colours indicate the amounts of each chosen recipe.

Transfer of data to the office could be via memory stick, but tunnel WLAN and internet gives the advantage of remote support. The data transfer includes project settings such as drill pattern, drill log (m/min), material and time logs, shown in Figure 6. Geologists may operate a handheld device (tablet, smart phone) to get progress status anywhere, also in the tunnel or at home if needed.

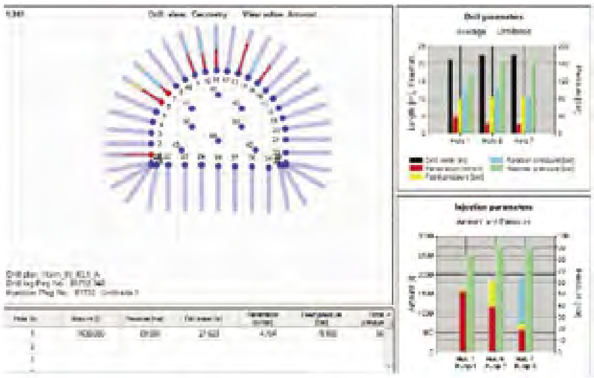

Figure 6: Drill plan for grouting in the office version. Drill parameters shown top right, lower right displays grouting parameters for the same holes [2].

Flow/pressure diagrams, like the one in Figure 7, may be used to characterise the grouting progress, and to identify incidents of uncontrolled increase in joint volume (hydraulic jacking). Having these available in real-time will help the operator determine the correct stop criteria for the relevant hole, and possibly prevent

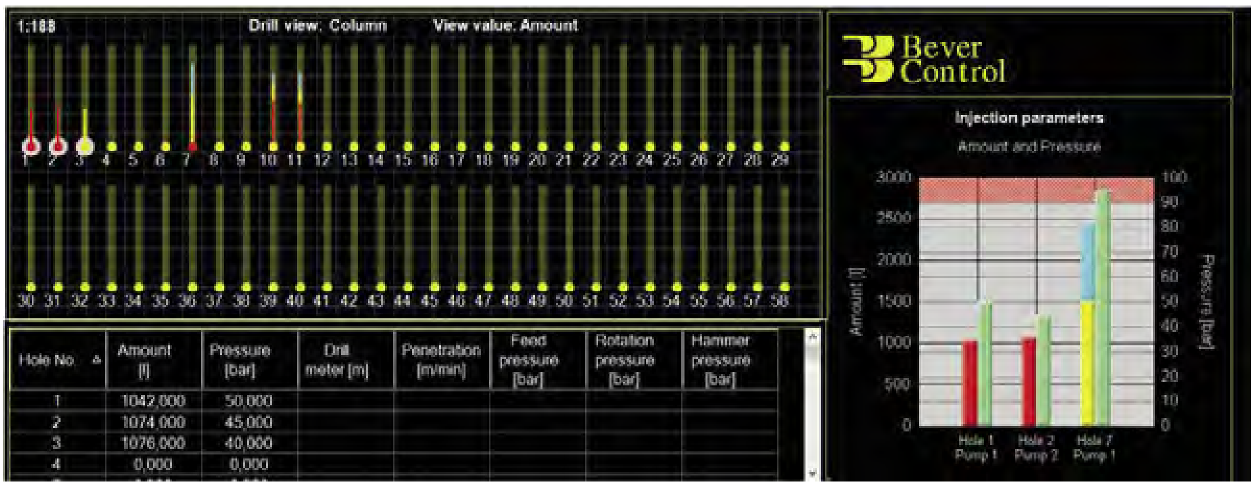

Figure 5: Columns according to hole number in grouting sequence. Active holes are monitored on the right. The colours indicate the amounts of each chosen recipe. Edited screenshot [2].

jacking and re-jacking of the holes. This will reduce total grouting time, and as is shown later, also reduces the amount of material used per grouting umbrella.

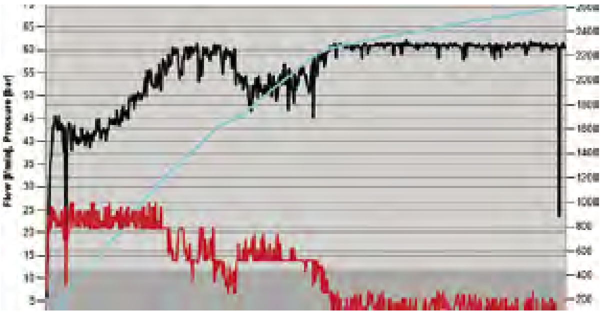

Figure 7: Flow/pressure diagram showing the real-time flow [l/min] in red, and pressure, in black, of current holes. The blue line represents accumulated volume of grout in litres. Screenshot [2].

Documentation for the client is automatically generated at the end of each grouting umbrella, and can be displayed both numerically and graphically. All data can be exported as spreadsheets or text files.

CONCLUSION AND FURTHER WORK

These are some of the options for software control and documentation currently in use in Norway and parts of Scandinavia today. Several additional applications are under development, amongst other things automatically generated drill plans. The grouting control project is currently in the TIGHT program for a better understanding of the grouting process.

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2. **Bever Control AS. (2017).** Bever Team 3 - software. Tranby: Bever Control AS.

5.8.5 THE NTNU PREDICTION MODEL FOR HARD ROCK TUNNEL BORING

INTRODUCTION

The NTNU prediction model for hard rock tunnel boring was first published in 1976 (1). The most recent version of the model was published in 2016 (2), representing version 7 of the prediction model. The NTNU model comprises four parts:

• Net penetration rate (m/h)
• Cutter ring life (m3/ring)
• Gross advance rate (m/week)
• Excavation costs (NOK/m).

Several other prediction models for TBM tunnelling exist. However, few or none comprises all four items covered by the NTNU model.

Based on field data from around 300 km of hard rock tunnels, NTNU has derived some important understanding of rock breaking under disc cutters as well as critical design principles for hard rock TBMs.

THE NTNU PENETRATION RATE MODEL

The basic concept of the net penetration rate model is to simulate or predict the progress of the penetration curve shown in Figure 1.

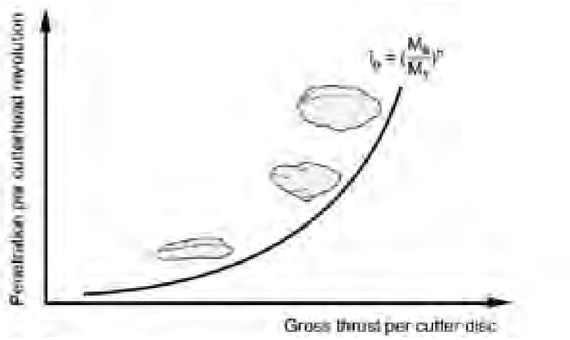

Figure 1. General progress of a penetration test curve. After (3).

The model considers rock mass boreability (M1 and b) and TBM design (MB), see equation in Figure 1. Rock mass boreability comprises one rock strength parameter (i.e. ability to resist breaking under repeated blows, Brittleness Value S20) and one rock mass parameter (i.e. degree of fracturing or average spacing between planes of weakness). TBM design comprises four parameters; the gross, average applicable thrust per cutter, the cutter diameter, the average cutter spacing on the cutterhead and the applied cutterhead RPM.

As can be seen in Figure 1, the progress of the penetration curve is exponential. The exponent b have been observed to vary from slightly larger than 1.0 in rock mass with extremely good boreability, to higher than 6 in rock mass with extremely low boreability. Hence, the stronger the rock mass is, the more significant the effect of increased cutter thrust will be. This basic learning from the penetration curve has been the impetus to develop hard rock TBMs with large cutters and high thrust for improved rock breaking efficiency.

UNDERSTANDING ROCK BREAKING UNDER A DISC CUTTER

In general, the energy or work needed to break a material into smaller fragments is proportional to the area of new surface created in the process. The larger fragments we are able to create, the less the new surface will be. Hence, efficient rock breaking under a disc cutter is characterised by creating a low portion of fines from the process.

As illustrated in Figure 2, radial fissures are propagating into the rock from the contact area between the cutter disc and the rock. The propagation is governed by the contact stress, which results from the size of the contact area and the thrust force. This understanding leads to the optimal cutter design; a small diameter ring with a narrow edge able to withstand a very high contact stress. However, materials with such properties are not yet available from a cost point of view.

It can also be derived from Figure 2 that the spacing between adjacent cutter tracks will influence the rock breaking efficiency. The ideal large chip is made when the radial fissures propagating between adjacent kerfs meet deep in the rock. Hence, in very strong rock mass, the kerf spacing should be considerably lower than in weak rock mass.

The propagation of the radial fissures also depends on the time the contact stress under the cutter is allowed to work. Seen from a given point along the cutter track, the time the contact stress is working depends on the rolling velocity of the cutter along the track, decided by the cutterhead rpm. Hence, efficient rock breaking in very strong rock mass will happen at a lower cutterhead rpm than in a more weak rock mass.

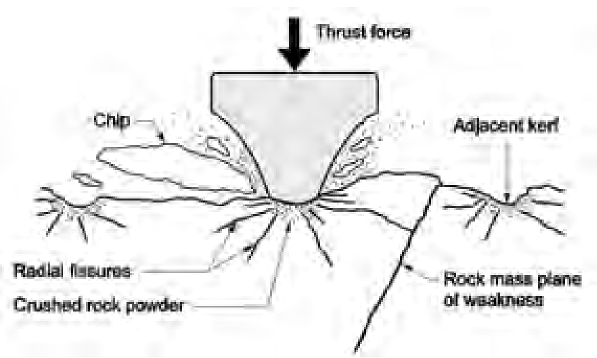

Figure 2. Rock breaking under a disc cutter in hard rock (3).

Figure 2 also illustrates that planes of weakness in the rock mass will assist the breaking of larger fragments. I.e., the plane of weakness will form part of the surface of the fragment, requiring less energy to make a fragment of a certain size. However, it is not the "pre-broken" effect of planes of weakness in the rock mass that makes the degree of fracturing the most significant factor for rock breaking efficiency of a TBM cutterhead. As illustrated in Figure 3, planes of weakness will very likely cause voids in the tunnel face. When a cutter rolls across such a void, the contact force between the cutter and the rock will disappear for a short time. The force carried by this cutter must be redistributed to other cutters until the cutter again will have contact with the rock. The variation in cutter forces due to redistribution causes a dynamic thrust situation for a given cutter and for the cutterhead as a whole. The more planes of weakness present in the tunnel face, the stronger the variation in cutter loads will be. Looking back to Figure 1, these peak forces will make the penetration rate higher.

Figure 3. Void in the tunnel face created from a plane of weakness (3).

CUTTERHEAD AND TBM DESIGN IMPACTS

For the individual disc cutter, the high average and peak cutter forces have strong impact on the design. In the light of the available materials technology, the trend has been to increase the cutter bearing size and the cutter ring diameter in order to be able to apply higher cutter thrust. Standard cutter diameter for hard rock conditions has been 483 mm since 1985, with 500 mm or more as the possible next step.

The design of the cutter ring itself relies very much on the current material (i.e. steel) technology. The constant cross section type is used, with varying cutter ring edge width (15 – 25 mm). The necessary edge with is generally larger in the outer part of the cutterhead than in the inner part. A cutter in the outer part will have higher rolling velocity and is exposed to higher peak loads than a cutter in the inner part of the cutterhead. Hence, a wider ring is needed to avoid destructive wear in these positions.

The number and layout pattern of the disc cutters on the cutterhead plays an important role in hard rock conditions. Figure 2 indicates that the spacing between adjacent cutter tracks influences the necessary thrust to break large chips, i.e. the larger the spacing is, the higher the necessary thrust is. The second fact to consider is that the rock breaking work increases with the square of the radius from the centre of the tunnel face and outwards. Hence, spacing between cutter tracks must decrease towards the gauge.

When considering rock breaking efficiency and cutter wear only, the ideal cutter layout pattern would be to place all cutters along one diameter line of the cutterhead, with cutters in tracks 1, 3, 5 etc. on e.g. the left hand radius and cutters in tracks 2, 4, 6 etc. on the right hand radius. However, such a design would generate extremely high and unbalanced forces on the cutterhead structure and the cutterhead main bearing. The seemingly best alternative is to apply the same alternating cutter placement as above along the two arms of a double spiral starting in the cutterhead centre.

The general experience is that the total number of cutters on a cutterhead intended for boring in hard rock conditions should correspond to an average cutter spacing of approximately 70 mm over the cutterhead. When extremely hard rock conditions is expected, one must consider the option to have more cutters on the cutterhead in exchange of smaller openings for muck removal.

Due to very high average and peak cutter forces, the main bearing will have to respond to a very high and unbalanced load situation. Considering the total rock breaking work of the tunnel face, half of the work will be outside 0.7 of the cutterhead radius. Also, the cutter peak loads will increase towards the gauge due to the higher rolling velocity of the cutters and the curvature of the cutterhead structure. Hence, the main bearing diameter should be in the range of 0.7 of the cutterhead diameter.

Efficient boring in hard rock conditions is associated with strong vibrations originating from the high peak loads of the rock breaking process itself. These vibrations will put large strain to the cutterhead structure in general and the cutter housings in particular. The simple solution to mitigate the destructive effect of a high vibration level is to put more steel, i.e. more structural strength and more weight, into the cutterhead.

One more item to consider is the stiffness of the cutterhead structure and the cutterhead thrust system. The stiffness of the cutterhead structure will be improved by more steel in the cutterhead structure, as discussed above. The recommendation for the thrust system is to increase the hydraulic stiffness by increasing the thrust cylinders' diameter and/or increase the number of thrust cylinders.

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5.8.6 BUILDING INFORMATION MODELLING (BIM) IN UNDERGROUND PROJECTS

Introduction

Building Information Modelling (BIM) has in recent years become more common in connection with infrastructure projects also involving tunnels and underground construction. Although BIM has been utilized extensively in engineering, the interface with the contractors has to a large extent been based on publishing 2D drawings. For the two large hydropower projects Vamma 12 and Nedre Otta for Hafslund Produksjon AS and Eidsiva AS respectively, the initiative was taken to change this practice and challenge the contractors to perform the construction directly based on BIM without delivery of 2D drawings. The reasoning behind this approach was the desire to ensure optimal flow of information between the parties and to reduce manual data handling to an absolute minimum. The aim was to ensure that all parties had full understanding of how the various elements were interacting to create the final product, to achieve optimum flow in the process, and to ensure high productivity and correct fit in all interfaces.

Before the completion of the tender documents for construction works and supply of electromechanical equipment and hydraulic steel works, a dialogue with all pre-qualified contractors was initiated to anchor the approach and identify needs in relation to construction based on BIM. After evaluation and contract negotiations AF Anlegg AS was chosen as the contractor for Vamma 12 HPP and Skanska Norge AS was chosen for Nedre Otta HPP. In addition to price, expertise and approach to construction, the contractors were also evaluated on their attitude and approach to the utilization of BIM in the construction process. In addition to the civil works contract, a number of other contracts for the supply of electromechanical equipment and hydraulic steel works, HVAC, low voltage installations etc. were negotiated. These supplies are performed as turnkey contracts (EPC) and are therefore separately responsible for providing their design directly into a joint Coordination Model (BIM).

The approach was thoroughly founded in the contract by inclusion of specific clauses as additions and deviations to the contract standards (NS8405 for civil works and KOLEMO for other supplies). Rather than establishing rigid instructions for cooperation in BIM, it was chosen to establish a joint BIM strategy, signed by all parties, where cooperation, flexibility, and joint development of "best practices" were key elements. As the approach with utilizing BIM as the direct basis for construction was new to all parties, focusing on flexibility and common goals were essential for success. This has also proven to be the correct approach. The intension of the BIM approach was to bring the parties closer together and secure that all participants had a common understanding of the totality and potential points of conflict as early as possible. The risk is greatly reduced as conflicts are resolved early instead of appearing as surprises during construction. Through a process of cooperation and common learning, the workflows were adjusted to optimize and ensure good processes and structured access to critical information. Such a process would obviously never run without challenges, but based on the strategy for BIM and focus on cooperation and common development, all parties helped improve the process.

The article first describes the overall approach to BIM and how it is applied to the projects to ensure good production. Then we go into how BIM has played and can play an essential role in the planning, design and construction of cuts, fills, tunnels and underground facilities.

Virtual design and construction (VDC)

The utilization of BIM in conjunction with engineering and construction is as much about process and communications as it is technology. The abbreviation VDC, which stands for Virtual Design and Construction, has become the abbreviation for this approach. BIM is central to the process, and as Professor Martin Fischer of Stanford University said during a seminar in Stavanger earlier in the fall of 2016; "BIM is the best tool I have found to secure that everyone is working on the same project". The phrase describes the fact that if the multidiscipline BIM (coordination model), is kept updated and shared openly in the project, one will secure that everyone can observe the evolution of the design of each discipline in concurrence with the other disciplines, enabling interdependent concurrent engineering to take place in an orderly and coordinated manner.

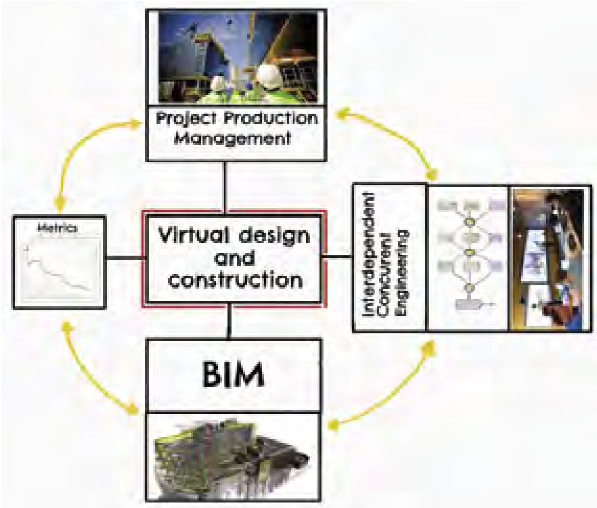

Figure 1. The elements of VDC

This contrasts with the traditional approach where each discipline, is working separated from each other. In many projects this has led to a notion that conflicts are detected too late in the design process or in many cases during construction. The traditional approach entail lengthy processes with large amounts of requests for information (RFI) going back and forth between the parties to arrive at a fully coordinated design. The flexibility of making design changes also suffer in this process as such changes late in the process causes ripple effect into other disciplines that many times lead to extensive delays and cost overruns in the design process. In many cases the coordination has not been sufficient, and end up with conflicts between different disciplines during construction. This again leads to extensive rework and bad quality of the final product. The rework causes delays and cost overruns.

VDC aim to reduce the conflict level and through "virtual construction" reveal and resolve challenges up front. The BIM is a good tool for planning and testing of constructability. When all disciplines are working into a joint coordination model, the design team can, through a structured process of Interdependent Concurrent Engineering (ICE) ensure that the interface coordination is handled continuously. To ensure an effective process, the so-called "big rooms" were utilized where all involved disciplines could meet in so called ICE-sessions and interact with BIM in the center of the process.

The coordination model, which was shared in a cloud based collaboration platform, secured that all parties had access to the latest versions of all discipline models merged into a joint BIM. During the ICE-sessions the BIM was always up on the screen and was utilized as the basis for discussions and clarifications. Involving the owner, designers, and contractors in the ICE-sessions secured clarification of issues and clear decision making. It brought the parties closer together to optimize the design from the point of view of operations, constructability, quality, cost, and schedule.

To be able to fully optimize time and cost, the BIM should be utilized directly as basis for construction (production at site). In this approach the contractor plans construction and harvests all necessary data (such as geometry and specifications) directly from the BIM. In the following the process utilized at Vamma 12 HPP and Nedre Otta HPP is further elaborated.

BIM for excavation and tunneling
For the excavation and filling works the civil works contractors fed the geometrical data from BIM into their GPS operated excavators. For the excavation of tunnels, shafts and caverns, the 3D geometry was transferred from BIM directly into the excavation planning tools to produce blasting plans that further were loaded into the Bever Control system to control the drilling jumbo. In this way data were transferred directly from design into production, reducing time spent on data handling and improving quality by deleting the re-entry of data which in many instances introduce a source of human error.

After blasting and excavation the contractor carried out scanning of the rock surface for geometrical control. The point cloud and triangulated surfaces were also forwarded to the designer for inclusion in the BIM. Based on this the designer could make necessary adjustments of structures etc. to fit the excavated surface.

Comparing the scanned surface with the theoretical (designed) contour allows analysis of overbreak caused by drilling and blasting as well as by geological conditions. With high resolution scanning it is possible also to perform an automated categorization of the different discontinuities based on strike and dip. This may be illustrated in the BIM by means of e.g. colour coding. In many cases the shape of the scanned surface may also help identify the intersection between the tunnel and weakness zones etc.

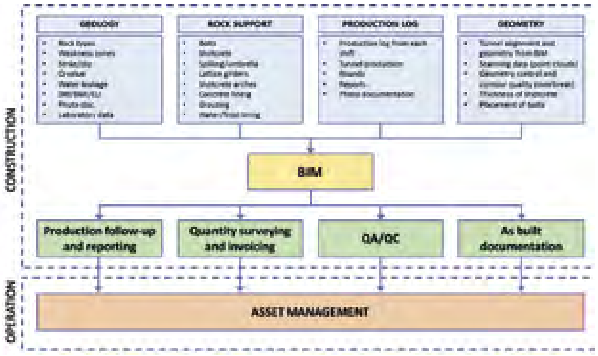

Figure 2. Data collection and analysis for tunnelling using BIM to bring all of it together.

The surface may be scanned both before and after the contractor applies shotcrete for rock support. By comparing the two scans, a map showing the thickness of the shotcrete layer may be constructed. This can be utilized as as-built documentation, quality checking (adherence to required thickness) and as basis for quantity calculations.

Rock support bolts and anchors installed by the contractor could be imported directly into the model based on xyz coordinates collected from the data logger on the main jumbo or the bolting rig. Data from the MWD log (measure while drilling) could also be interpreted and brought into the model as a colour coded 3D volume, and compared with the geological mapping as well as photo documentation from the face of the tunnel. While scanning the surface of the tunnel prior to applying shotcrete, it was also possible to collect high resolution orthophotos that could be draped on the surface of the models. If the quality of the photos are sufficient, they may act as a support in the geological interpretation and be strong as-built documentation. These data can be collected and be made available in the BIM as illustrated in figure 2 above.

An example of a system that enable projects to both do design and collect data in a good and systematic manner in BIM is Novapoint Tunnel, developed in Norway. In Norwegian road and railroad projects the clients now demand that all geological and rocks support data shall be collected and systematically entered into such a system.

In the future, this process should be automated as much as possible, but it is important to notice that skilled engineering geologists and tunnelling workers that enter the tunnel to "touch and feel" the rock mass is of key importance when interpreting the geological situation and responding appropriately.

The system only becomes valuable if the systematization of data is utilized in an active way to improve interpretation and optimize the tunnel support to the best for both safety and cost.

Conclusion

Applying BIM as the basis for planning, design and as direct basis for construction worked very well and helped to ensure good cooperation, data exchange and production on Vamma 12 as well as Nedre Otta power plants.

Although experiences were very positive, the future will show if the industry is able to utilize the full potential inherent in the approach.

BIM is a very useful tool for underground construction. Merging data from multiple sources into BIM and using the data actively to monitor and analyze the conditions and adopting the design and construction method to the challenges has a great potential.

However, it is important to note that BIM is a tool, and as with all tools it takes skills and training to use them in a good manner.

Without adaptation of processes, trust between the players and focus on common goals, any system will malfunction. BIM could contribute to secure that "all parties are working on the same project, towards common goals" and would therefore be a good foundation for cooperation.


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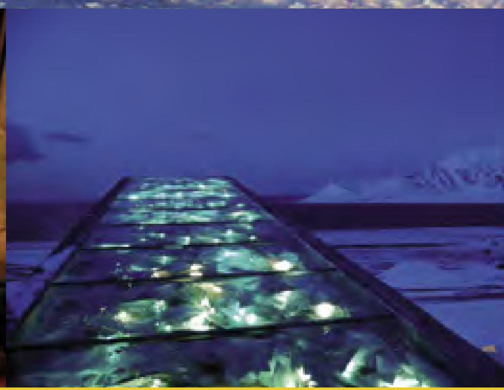

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NFF, Publication no 26
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