NORWEGIAN TUNNELLING TECHNOLOGY

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DISCLAIMER

FOREWORD

The present publication, No. 23 in the English language series from the Norwegian Tunnelling Society NFF, has – as always – the intention of sharing with our colleagues and friends internationally the latest news and experience gained in the use of the underground; this time with focus on tunnelling technology in general.

Publication No 1 - "Norwegian Hard Rock Tunnelling" issued 32 years ago may have given an impression of a mountainous country with entirely solid, competent rock. The picture is more complicated, several incidents underscore variety.

Developers set new benchmarks for safety and quality, more pre-investigation is necessary, plans and specifications are more detailed as are the contract documents.

The basic situation, however, is unchanged. Cooperation, contribution and flexibility are still common approach, modifications take place when appropriate and in the context of durability and maintenance costs as seen from the owners; safety and reliability as seen from the public; methods, techniques and materials as seen from scientists, advisers, suppliers and contractors.

Publication 23 is prepared by NFF members. Some details on the authors are included in Annex I.

On behalf of NFF it is a privilege to express our sincere thanks to the Editorial Committees and the Authors. Without their efforts the preparing of this publication would not have been possible.

Oslo, May 2014

Norwegian Tunnelling Society
International Committee

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Svalbard Global Seed Vault
Three underground caverns and an entrance tunnel constructed in permafrost. Natural temperature -3°C permanently cooled to -18°C. Huge number of samples from all continents safely stored. Courtesy: LNS AS

01. INTRODUCTION

GRØV, Eivind

BACKGROUND

Worldwide there is a quest for urban space driven by the increasing urbanisation. According to the ITACUS white paper (2010) more than half of the world's population lives in urban areas. With regards to industrialised countries the figure is closer to 80%. This increased urbanisation and steadily growing number of "Mega cities" and other congested areas is a consequence of the increasing global population and migration to city areas which may offer jobs, income and improved lifestyle.

This paper introduces the readers to the history and background of Norwegian tunnelling and utilisation of the underground in Norway, both in urban as well as rural areas. There are many reasons why tunnelling has become a vital industry in Norway and the underground use being instrumental and one of the corner stones in developing the society of Norway .

The challenges are numerous and the availability of space for necessary infrastructure ends up being the key to appropriate solutions. The underground is at present only marginally utilised world wide. The potential for extended and improved utilisation is enormous.

The vision of the Norwegian Tunnelling Society (NFF), "Surface problems – Underground Solutions" is more than a slogan; for the international tunnelling industry and its members it is a challenge and commitment to contribute to sustainable development. At the time of issuing this publication Norway presents its candidature for hosting the World Tunnel Conference in 2017. Some vital aspects that should be addressed more vigorously in the future and which constitute important aspects in this application are:

(i) Legal aspects, for whom and for which purposes should the underground be available.
(ii) Overall urban planning to include planning of underground space use.
(iii) Develop better procedures and tools for such planning
(iv) For the ITA-AITES community to contribute with ideas, proposals and expertise.

The tunnelling industry has the means to make any "hole in the ground" almost in any size and shape required. The main challenge is to integrate such "holes" in the long-term development and urban planning (on Master plan level), be cost-effective, be safe and felt to be safe by the users. Underground space has stable temperature year round, confinement and protection providing new possibilities for infrastructure, services and utilities which do not claim valuable surface space.

The change of a common attitude from considering the underground the last recourse to become the long term preferred solution for new projects is a long term duty of the ITA-AITES community. This can only be reached by increasing the knowledge and competence. Smart and innovative planning of the underground space would require such as:

- New design thinking (paradigms) to fit future underground application and facilities efficiently,
- New underground design requiring interacting facilities utilizing new excavation methods,
- New facilities requiring suitable/cost-environmental efficiency for underground placement, and
- Multiple level underground facilities which require clear ownership of ground and stakeholders

Under the theme of the conference, "Surface Problems – Underground Solutions", the WTC in Bergen will focus on the wide scope and the variety of applications of underground utilization. Going underground is the future solution to preserve sustainability. Utilizing the 4th dimension is becoming a necessity in the modern planning process. Norway has a long tradition of using the underground for a wide range of applications. Through this proud tradition Norwegian tunnelling industry has provided break-through solutions and technologies that in many cases constitute state-of-the-art today.

DEVELOPMENT OF TUNNELING IN NORWAY

Norway has been a front runner in underground excavation and tunnelling for many years, even for many decades. Despite the fact that the population is very small, about 4,5 mill inhabitants at the turn of the millennium and without any major city reaching the 1 mill treshold the tunnelling industry has been able to be innovative. Its development copes with a great demand of being innovative, still cost effective and able to see opportunities in utilizing a natural resource, namely the rock mass, strong or weak, and sometimes even soil condition. This is sometimes referred to as the Norwegian tunnelling technology. New markets have been explored for the application of the Norwegian technology, as on the other hand, Norway has been able to obtain solution through inspiration from participation in overseas projects.

During the post war era in Norway utilisation of the 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 figure 1 below 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. Today, tunnelling for infrastructure projects, such as road and railway tunnels is dominating whilst other usages are fading gradually out.

Following up the development in the hydropower era in the post war years was associated high industrialization and the use of the underground found new applications in Norway. Road and railway tunnels were already in use, but car parking lots and metro stations were excavated in underground caverns. New applications were found in transport and storage of potable water as well as sewage water, underground storage of hydrocarbon products and many more utilisations.

Urbanisation required to a larger extent more suitable solutions for urban areas, new requirements and improved technological solutions. The underground industry evolved and matured to the current state-of-the-art. Still many principles remain to constitute a base for this evolvement.

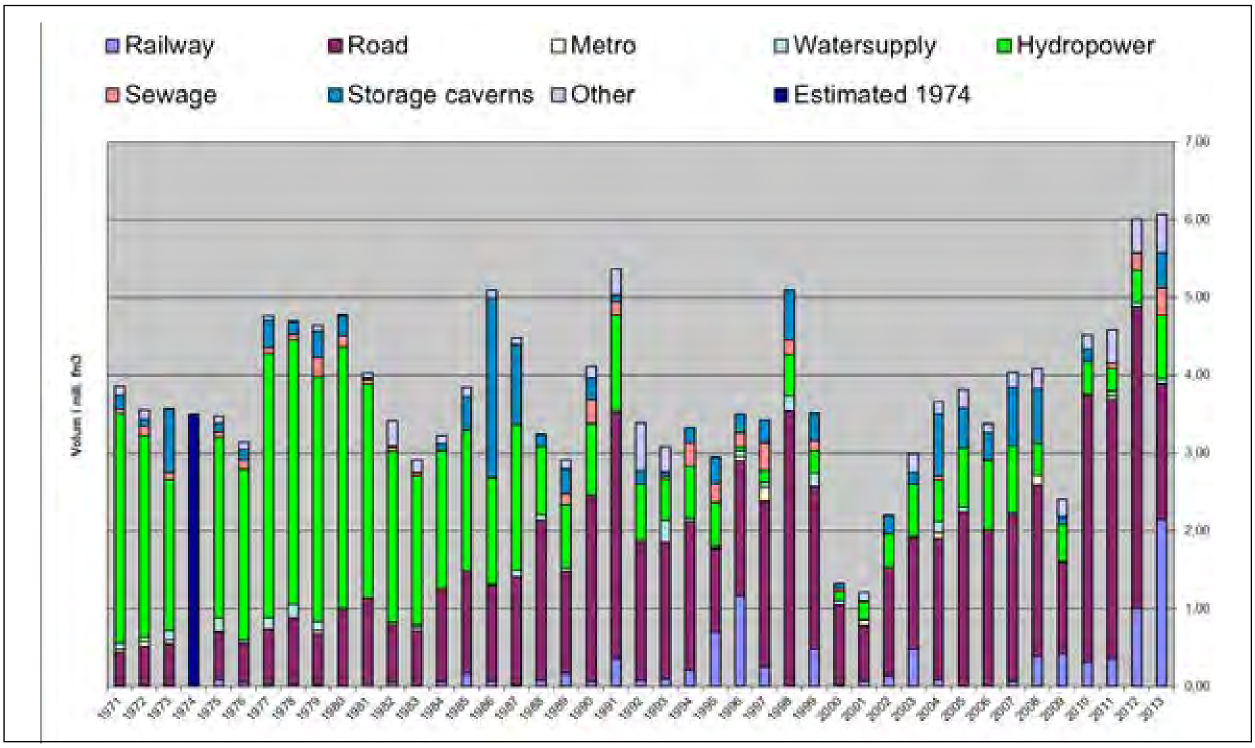


Figure 1. Development of Norwegian underground works (fm3=solid state)

A rough estimate suggests that every Norwegian has about 2 meter of tunnel.

The Norwegian tunnelling industry so dominantly has preferred to apply the conventional drill- and blast method. This may be related to the changing ground conditions that would be expected for almost any tunnel project that has been undertaken during this period of underground development in Norway. Conventional drill- and blast excavation has a great advantage over TBM in terms of dealing with and handling changing ground conditions and the need of rock support or rock mass grouting to secure safe tunnelling conditions.

At present a rough statistics of Norwegian tunnelling suggests the following:

- 750 railway tunnels
- 1000 road and highway tunnels, including 35 subsea road tunnels
- The infrastructure tunnels total almost 3000km in length
- World`s longest road tunnel in Lærdal, 24.5 km long
- Of the world’s 500 - 600 underground powerhouses 200 are located in Norway
- More than 4000 km of hydropower tunnels
- Some 60 caverns for oil and gas storage
- Clean water conveyance&storage and sewage water transport and cleaning in tunnels and caverns in all the major Norwegian cities
- Numerous civil defense and sports caverns, culminating with the Gjøvik hall
- And many more signal projects

Some of the technologies developed during the development of Norwegian tunneling industry remain state-of the-art internationally also. This publication will describe many of these and put them into their perspective.

IN THE HALL OF THE MOUNTAIN KING

In the following some main challenges for the tunnelling industry will be described in brief. A lot of organisations and institutions are looking carefully at introducing new applications of the underground, such as ITA, the International Tunnelling and Underground Space Association through its working groups and other activities. Its campaign on “Why go Underground” should be a call for Owners and investors for seeking solutions underground. We would recommend the readers to visit their web-site, particularly the section on “Why go underground”. In this drive to go underground the industry needs to be able to produce sustainable structures with predictable duration of the service life of the product, the latter being the hole in the ground itself and installations therein.

The Norwegian fairy tales tell numerous stories of trolls that lived their lives in beautiful caverns caved into the mountains in Norway. One of the most bought souvenir at departure from Norwegian airports is the Troll, in various size and shape, but still the trolls. The trolls constitute a part of the Norwegian history and culture, and the tunneling industry is following up with numerous applications of underground utilization.

In general it is just the fantasy that puts forward limitations on applications underground. Taking benefit from utilising the underground for various purposes may become the key to success in countries with mega cities, congested surface, rapid development of infrastructure, environmental problems, flooding and storm water control, storage and magazines for clean water and so on. The list may be endless. As mentioned above, a lot is done to promote an increased use of the underground, and this article will basically leave this matter here.

The future objectives of underground structures would be to take the solutions to reach levels where sustainability and long term durability are the key areas of focus.

Figure 2. Norwegian Troll, painted by Theodor Kittelsen

NORWAY, A NATION OF NOTORIOUS RISK TAKERS? AND WHY?

As described above the geological conditions in Norway must be considered as favourable for tunnelling works. However, many nations around the globe have similar and better conditions, but do they develop these in the same way as the Norwegians have. The answer is no. On a number occasions around the world the author has been confronted with the statement that the geology is so favourable in Norway. Even professionals ask what’s then the big deal in promoting “Single Shell Shotcrete lining” or ground water control by pre-grouting? The fact is that a majority of these professionals who forward that specific question fails to understand that these approaches may have been applicable in their home country, too, if they dared to try.

The Norwegians have been risk takers for many centuries and through generations. It is just over the last few decades that nature has been generous with Norwegians, when we faced the great adventure of being a major producer of the “Black Gold”. Prior to that era, the only generosity we received from mother earth was plenty of rain. This resource was in an extremely intellectual way converted to the basic need in the post war era, namely electrical power to supply a growing industry at that time.

Through the development of the hydroelectric power Norwegian tunnellers learned to utilise the capacities of the rock mass. Break through developments came successively as the technology developed. Which country can match Norway when it comes to the utilisation of the rock mass in hydropower development projects? To mention some merits:

- unlined headrace tunnels up to 1046 m static water head
- unlined air cushion chambers with rock head at a safety factor of 1
- several hundred power stations and thousands of supported by rock bolts and sprayed concrete
- lake taps at up to 160 meters water depth
- sub sea tunnels at 287m below the sea level
- unit rate contracts with flexibility to meet varying ground conditions
- wet mix sprayed concrete developed during testing at sites 30 years ago
- highly time and cost effective tunneling methods

And the list of merits may probably be even longer.

We owe the pioneers from the hydro power development a great honour, they paved the way for the tunnelling industry, and this was long before tools as numerical modelling and risk analysis became tools available in the industry. This followed a mining industry that was vital in Norway going back 2-3 centuries in time. Mining underground follows the simple principle of exploiting as much as possible of the resources to as cheap price as possible, still maintaining a safe working condition. This principle learned the mining industry to provide large underground caverns with a minimum of support, thus utilizing and trusting the self -standing capacity of the rock mass. This is a basal element of the current unlined tunneling concept. But how could this development materialise taking into account the risk associated with many of these developments? We have to go back in time and see what did constitute the basis for the Norwegians.

Figure 3. Oseberg ship used by vikings on exploration of new land, true risk takers (Dagbladet)

The basis for Norwegians has been a wild topography, cold climate, harsh environment, a deep sea close to its shores, and great difficulty in growing anything else than potatoes. No surprise that the Norwegian national liquor is made of potato, called aquavit. The fisheries have always been a major source of food and work. However, the fact that sometimes the catch was good, whilst other times came with “black sea” and no catch at all as well as the fact that many a son, husband and father never came back home alive from their fishing expeditions caused a breeding ground for high risk takers.

Then, followed the years of the Vikings who were well known for their exploring attitude and not only for their warlike spirit.

Conclusively, the message is that geology is one thing, something which we cannot change, another aspect is related to an inherited willingness to take risk and utilise its capacities.

CONCLUDING REMARK

In short, the development of tunneling in Norway is a combination of many aspects such as technology needs and development, culture heritage, experience transfer, geological understanding, but also the fact that a demand on cost and time solutions existed to bring the Norwegian society forward from its basis as a society depending on a rather unilateral economic basis being fishery and farming.

This publication will provide in details the many aspects of Norwegian tunneling, its development and status and also its particularities. This provides today a well proven and documented tunneling technology, unique in its structure and variety. Today going underground taking into the advantage of these technological developments that exists provide a basis for sustainable development.

Hardangerbrua

”Harangerbrua” spanning the Hardangerfjord was opened for traffic in 2013. The bridge has a 1310 metres long main span and 202,5 metres high towers. On both sides several kilometer of complicated tunnelis are excavated as access to the bridge. The picture is taken through one of the tunnel portals. The tunnel system was cconstructed by NFF-member AF-Gruppen and the towers by AS Veidekke, also a NFF-member. Courtesy: AF-Gruppen.

02. GROUND INVESTIGATIONS FOR NORWEGIAN TUNNELLING

NILSEN, Bjørn

1 INTRODUCTION

For any type of underground project, pre-construction investigations of high quality and 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 unnessesary high cost often will be the result for the completed project.

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

- Gives input 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 cost and schedule.
- 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 of this, and in worst case the project may end up in court with the extra cost that this will imply. High emphasis on 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 concrte. 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 on 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 on pre-construction 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.

This paper will discuss the basic aspects of engineering geological conditions, and the ivestigation approach which is used for Norwegian tunnelling today. Particular emphasis will be placed on describing the different investigation stages, the applicability of the various investigation methods and the extent of investigation.

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. Principle layout of the stepwise procedure used for ground investigations of tunnels and underground excavations in Norway.

For planning and design of road tunnels, a pre-investigation procedure based on four stages is used (NPRA, 2010):

1. Feasibility stage, for providing the geological basis for evaluating the feasibility of the project.
2. Overview plan ("oversiktsplan"), for providing the geological basis for selection of alignment alternative. Cost to be evaluated within an accuracy of ± 25 %.
3. Zoning plan ("reguleringsplan"), for providing 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 after completion of the project:

- Pre-construction phase investigations, or pre-investigations.
- Underground excavation has not 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.

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 is very valuable here.

At this early stage, desk studies of available geological information, such as reports, geological and topographical maps (scale 1:5,000) and aerial photos (scale 1:15,000-1:30,000) 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.

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 operations 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 1:5,000-1:15,000 and maps to scale 1:1,000 or 1:5,000 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.

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 tunnelling 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.

Investigation method	Bedrock location	Rockmass quality	Weakness zones	Rock stresses	Groundwater condition
-Desk study	-	(x)	(x)	(x)	(x)
-Inspection nearby excavations	(x)	x	x	(x)	x
-Site mapping	(x)	x	x	-	(x)
-Refraction seismics	x	(x)	x	-	(x)
-Core drilling	(x)	x	x	-	x
-Geotechnical drilling	x	-	-	-	-
-Seismic tomography	(x)	x	x	-	-
-Geoelectric methods	x	(x)	x	-	x
-Rock stress measurement	-	-	-	x	-
-Lab.testing	-	x	(x)	-	-

x well suited
(x) may be useful
- rarely suited/non-suited

Table 2. Overview of the most common methods for pre-construction investigation and their suitability

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.

There are many different investigation methods that may be relevant for planning of underground excavations. The most common for pre-construction investigation and their normal suitabilities are indicated in Table 2.

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

3.1 Desk study

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 1:50,000 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-sterographic aerial photo like shown in Figure 1 and satelite photos may provide very useful information for evaluation of the geology and early planning of the project.

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.

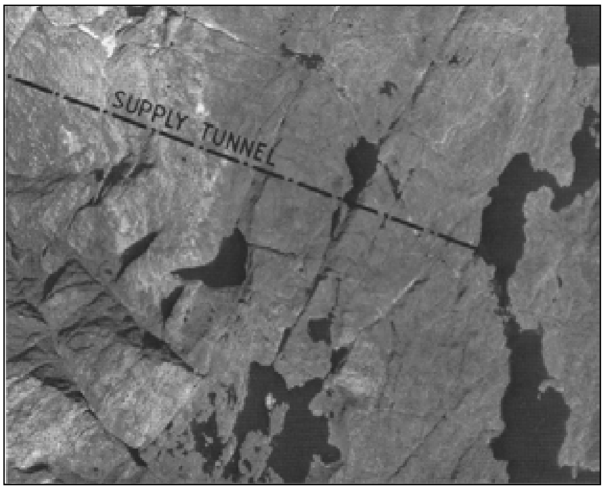


Figure 1. Aerial photo illustrating the locations of several large faults in the area of a planned tunnel (scale approx. 1:15,000).

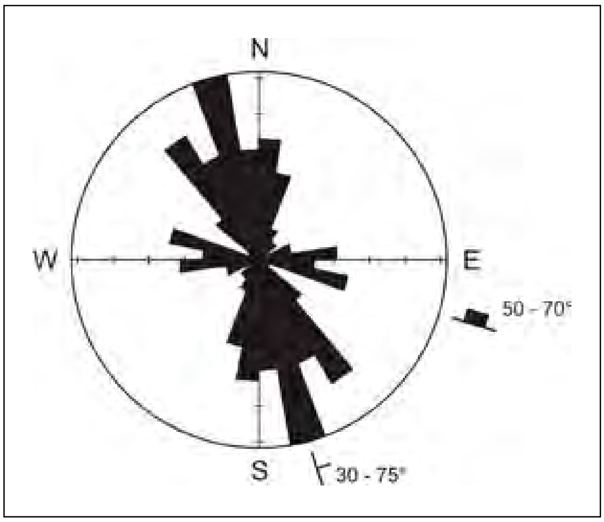


Figure 2. Mapping of joint orientation (strike and dip) and presentation of data in joint rosette.

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 in the 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.

Regarding rocks, emphasis is always placed more on character and mechanical properties than on sophisticated mineralogic and petrographic description. As part of the fieldwork, sampling is important for testing of properties that may greatly influence on the degree of difficulty and economy of the planned project, such as quartz content, rock strength and boreability/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 stereoplot gives much more detailed information on variations in dip than the joint rosette, and is also a better basis for identifying discontinuities with unfavourable orientation relatively to the actual tunnel or cavern.

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.

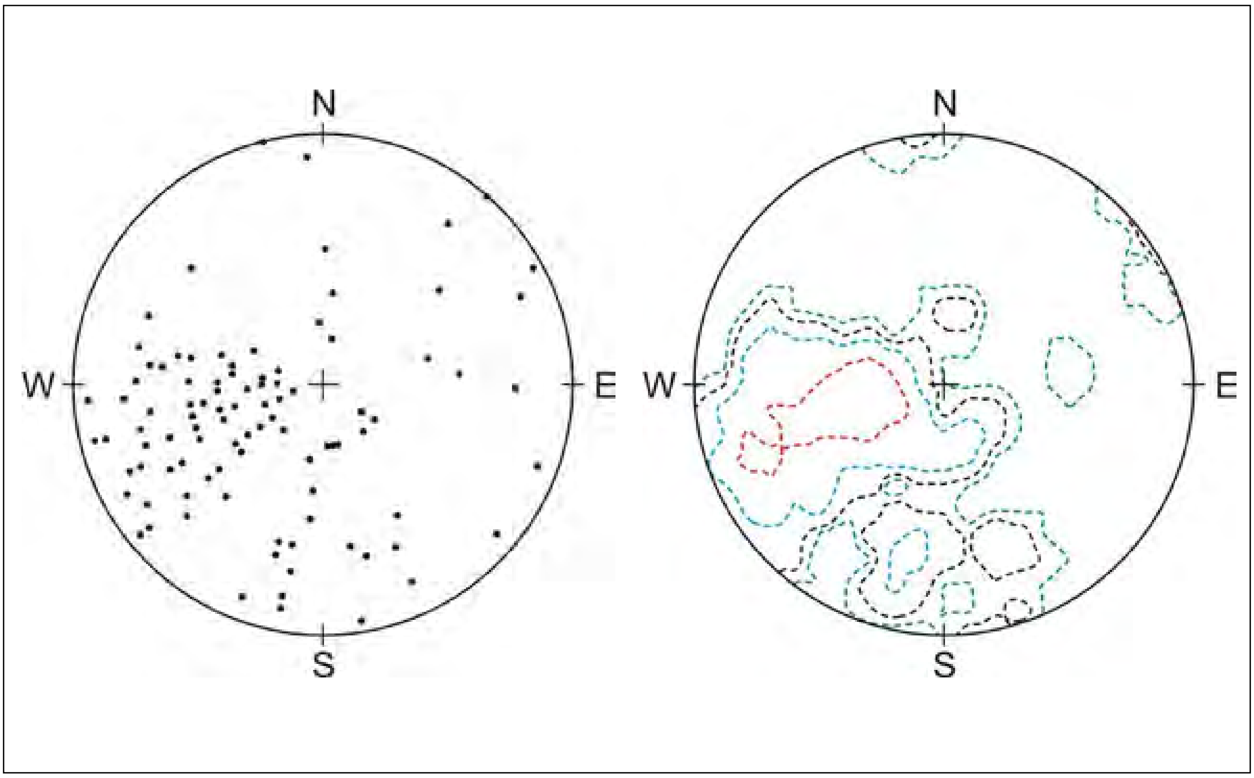


Figure 3. Presentation of the same joint data as for the rosette in Figure 2 in stereographic projection (equal area projection, lower hemisphere, with pole plot to the left and contored plot with contours representing 1, 2, 4 and 8% densities to the right).

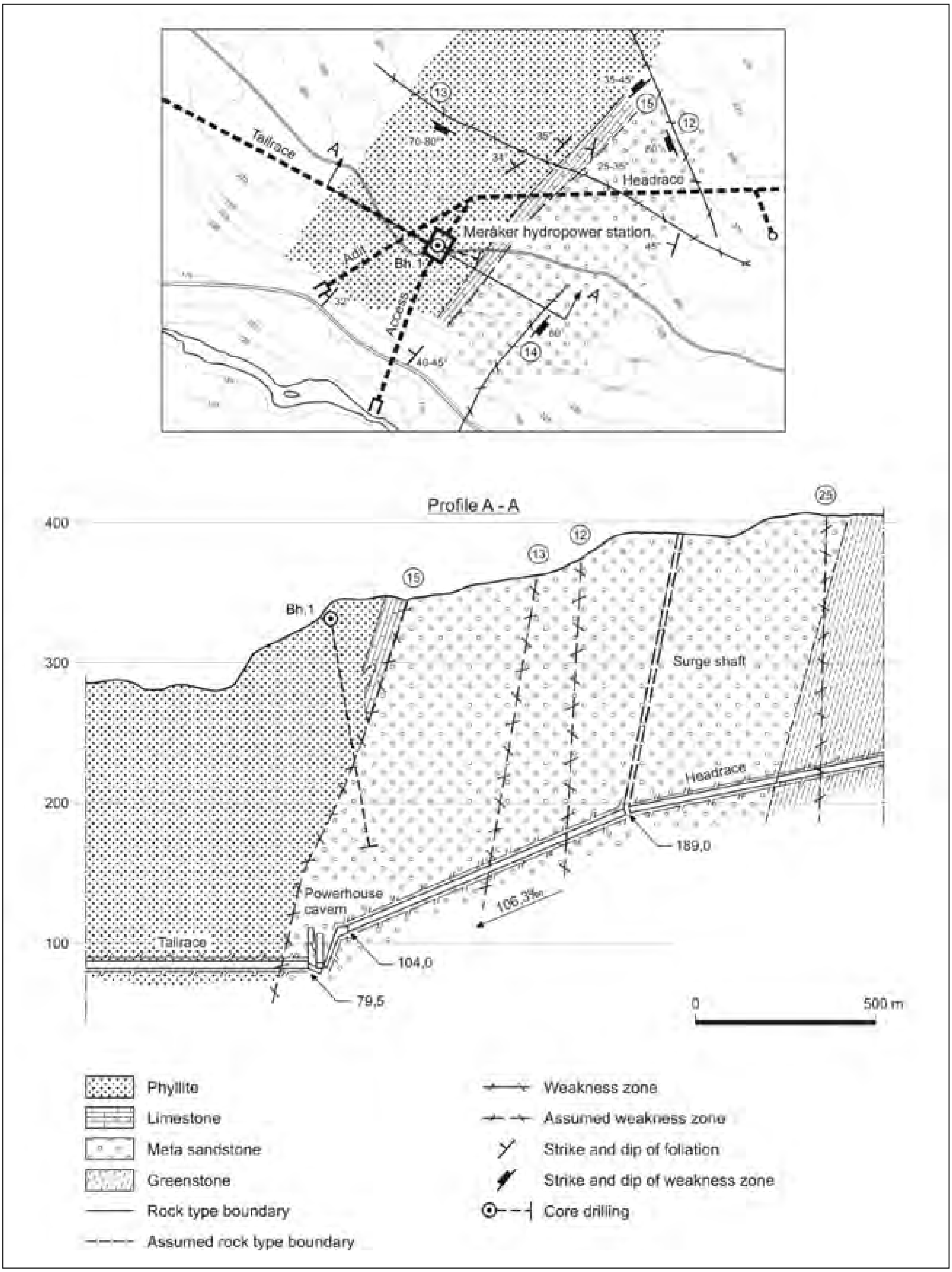


Figure 4. Engineering geological map and profile for Meråker Hydropower Plant.

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 1:10,000, is shown in Figure 4.

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 reprentative 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.

3.3 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 for logging the thickness of soil cover and for evaluating rock mass quality. An example of the use of refraction seismic combined with shallow reflection seismics (and core drilling) for planning of a subsea tunnel is shown in Figure 5.

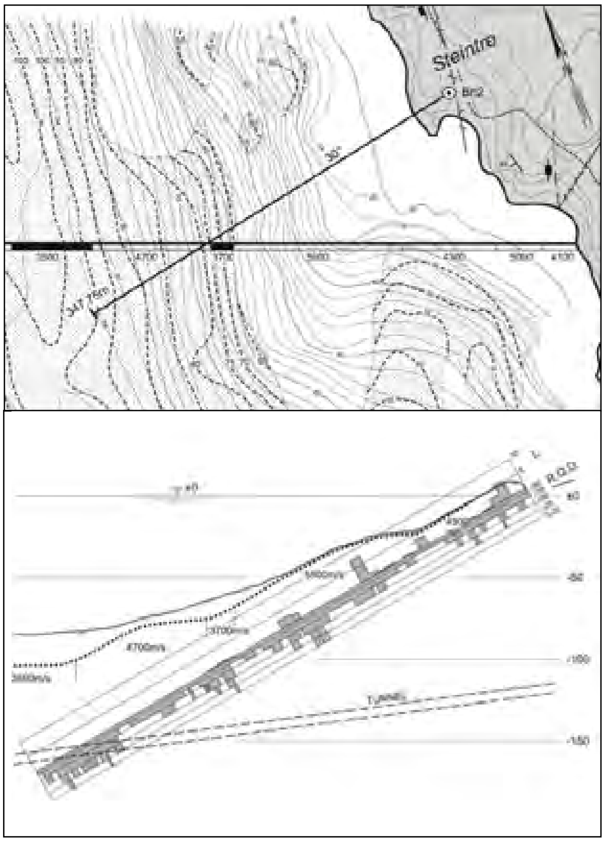


Figure 5. Investigation results for part of the Karmsund subsea tunnel.

The contour lines of the plan view in Figure 5 are based on shallow reflection seismics ("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 seisming 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 seismics, 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/sec 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 difficult to evaluate 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. Thus, seismic methods therefore do not automatically give high quality results in all geological environments.

Seismic tomography is used relatively rarely, but may be very 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 geoelectrical, 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.

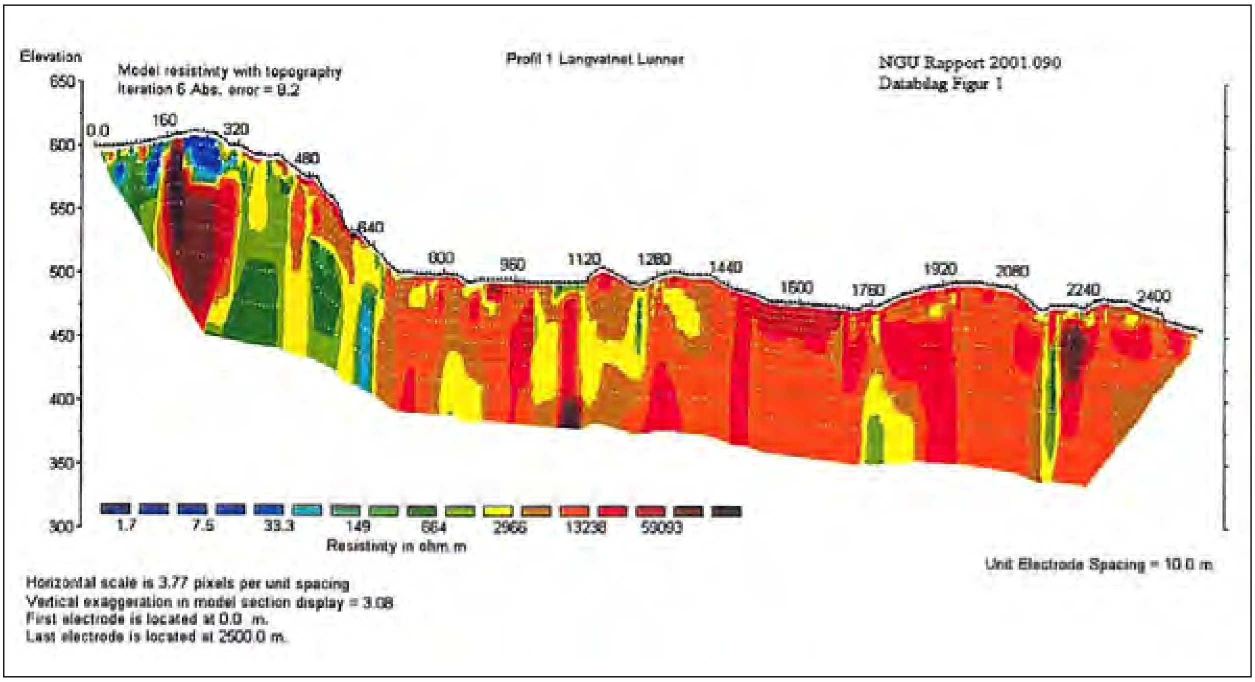


Figure 6. Results from resistivity measurement, with potential weakness zones (low resistivity) shown in blue color (from Rønning, 2002).

The geoelectrical 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 for investigating rock mass conditions for underground excavatuions. This is a method which in some cases has been useful for regional mapping of deep weathering.

3.5 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.

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/ limestone, considered to be unsuitable for location of the powerhouse cavern, and much better quality metasandstone.

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 for estimating RQD-values, for permeability (Lugeon) testing and for providing 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 possiblity of observing the walls of the borehole and making joint rosettes or stereoplots based on registration of joints, and may be particularly useful.

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

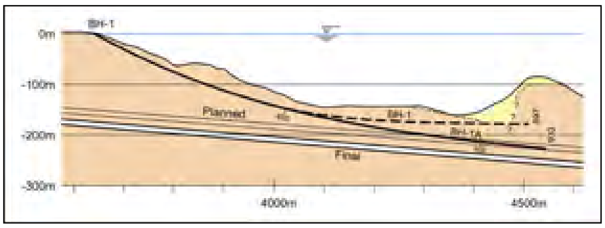


Figure 7. Directional drilling at the Bømlafjord subsea tunnel (from Palmstrøm et al).

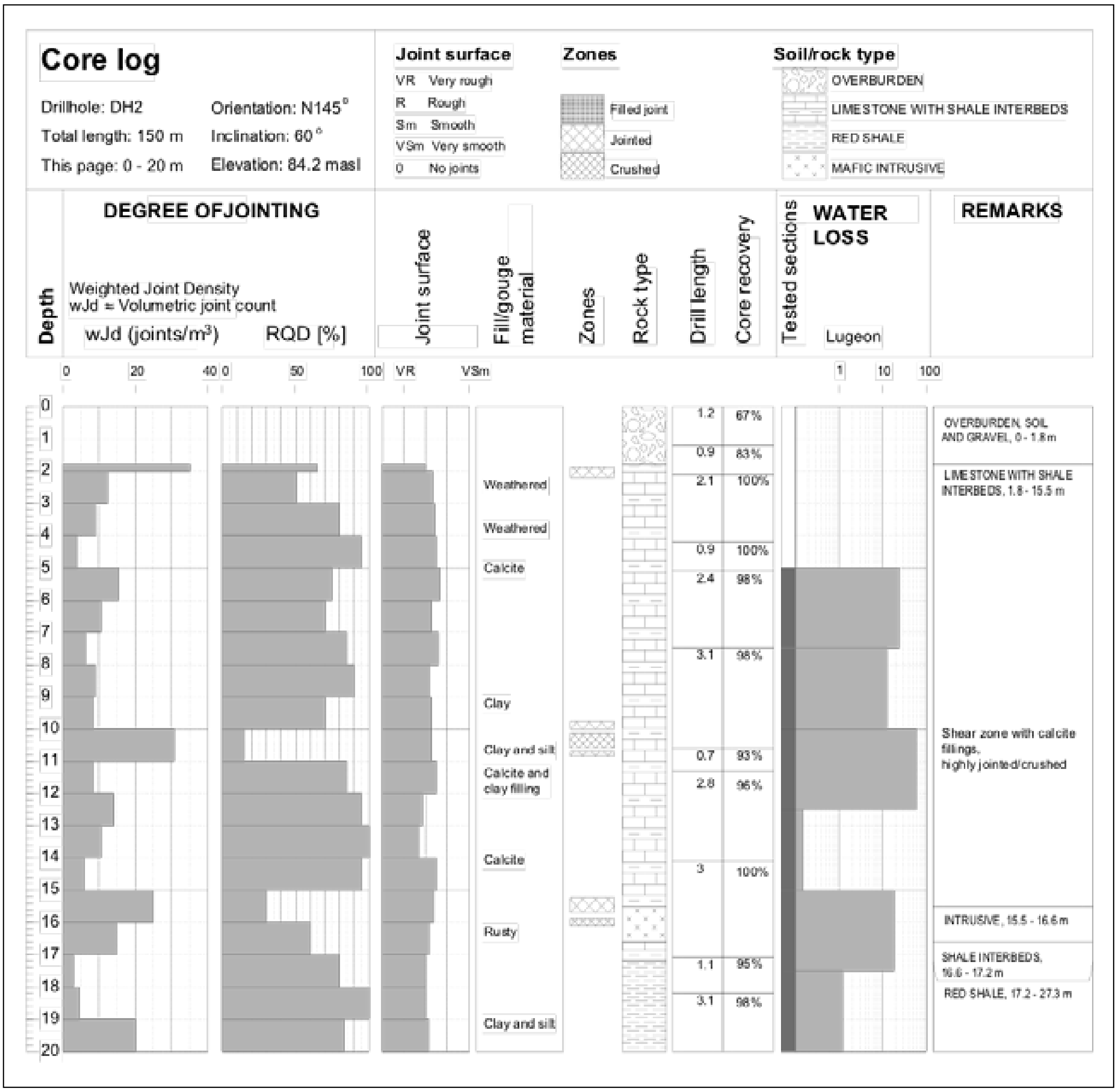


Figure 8. Example of core logging (from Nilsen & Palmstrøm, 2000).

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. An example of detailed core logging is shown in Figure 8.

3.5 Exploratory adits and shafts, geotechnical drilling
Exploratory adits and shafts are extreme alternatives for collecting geological information, which are used only in rare cases, i.e. for investigating the site of a large dam. Shafts for checking the thickness of soil cover is also rarely used. More commonly, geotechnical drilling is used for this purpose.

3.6 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 in Norwegian tunnelling practice (based on Nilsen & Palmstrøm, 2000).

3.7 Rock stress measurements

Information about magnitudes and directions of the principal stresses is particularly important for planning and design of deep seated underground excavations. Surface indicators such as exfoliation and core disc-ing (intense parallel fracturing perpendicularly to the core axis) may give a warning of high rock stresses. However, full information on stress magnitudes and directions 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.

The drill hole overcoring technique has the longest tradition. Figure 9 illustrates the principles of the most commonly used version of this method, which involves the following three steps:

1. A diamond drill hole is drilled to the desired depth. A concentric hole with a smaller diameter is drilled approximately 30 cm further.
2. A measuring cell containing three strain rosettes is inserted, and the rosettes are glued to the walls of the small hole.
3. The small hole is stress released based on overcoring by the larger diameter bit, and the resulting strains are recorded by the rosettes. After this, when the elastic constants are known, the triaxial state of stress can be computed.

The overcoring technique is normally used in underground openings, although triaxial overcoring has in a few cases also been carried out from the surface in 20-30 m deep drillholes.

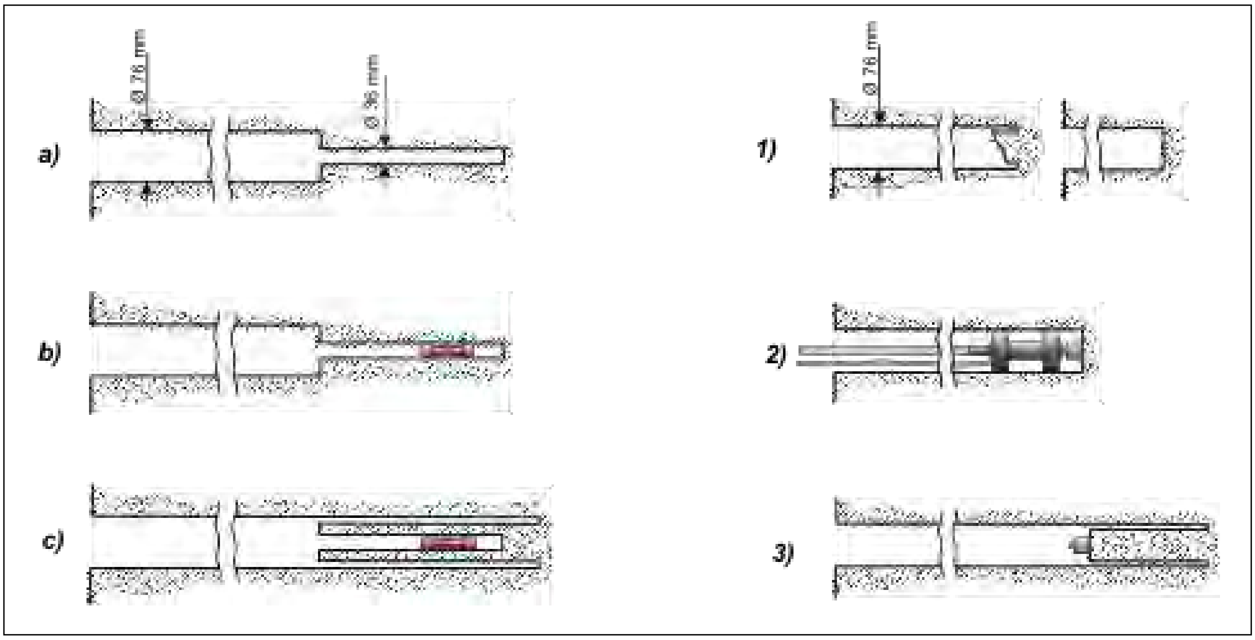


Figure 9. The principle of three-dimensional (to the left) and two dimensional (to the right) rock stress measurements by overcoring (after Myrvang, 1983).

The basic principle of hydraulic fracturing is to isolate a section of a drill hole and, by gradually increasing the pressure of water which is pumped into the hole, to obtain fracturing of the surrounding rock. By recording water pressure and flow, the principal stress situation may be evaluated. Hydraulic fracturing has in several cases been successfully applied in more than 100 m deep drill holes.

An idealised hydraulic fracturing record is shown in Figure 10. The drill hole fluid pressure at the moment of fracturing is termed the "fracture initiation pressure"

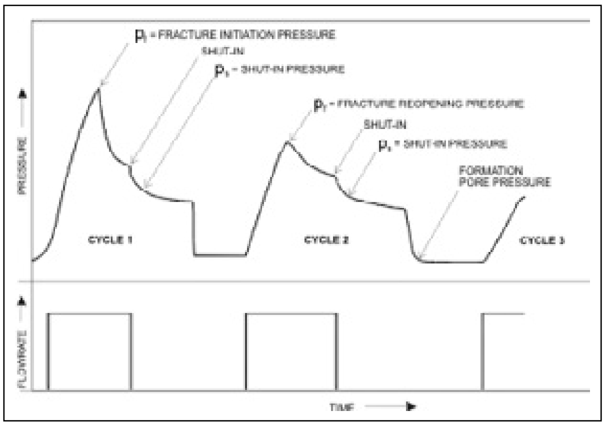


Figure 10. Idealised hydraulic fracturing pressure record (from ISRM, 1987).

(pf) or breakdown pressure. After injecting a volume sufficient to propagate a fracture length about three times the drill hole diameter, injection is stopped and the hydraulic system is sealed or "shut-in", yielding the "instantaneous shut-in pressure" (ps). Additional re-pressurisation cycles are used to determine the "fracture reopening pressure" (pr) and additional measurements of the shut-in pressure (ps).

Based on certain assumptions, the principal rock stresses may be calculated based on hydraulic fracturing. The method may be used applied in deep boreholes from the surface. In Norway this method is however most commonly used as "pilot test" for defining the safe location of high pressure hydropower tunnels. The method is then referred to as hydraulic jacking, and is often applied in the access tunnel during the early stage of construction of a hydropower project. In such cases the final decision regarding design of the high pressure tunnel system is actually taken during construction (as construction stage investigation, see Chapter 2.3).

4 CONSTRUCTION STAGE INVESTIGATION METHODS

Even when very comprehensive pre-construction investigations have been carried out, there will still be som degree of uncertainty connected to the ground conditions. As discussed in Chapter 2.3 it is therefore very important that investigations are continued and supplemented during excavation.

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 mappinghas 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 on 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.

An example of presentation of data based on tunnel mapping is shown in Figure 11. For additional documentation, photos are also included. When the tunnel mapping and all investgations are completed, a final report is made. Tunnel logs like shown in Figure 11 are included in this report.

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 sufficiently far from the face for appropriate measures to be made for a safe tunnelling through them. Sometimes, probe drilling is also used to check the rock cover.

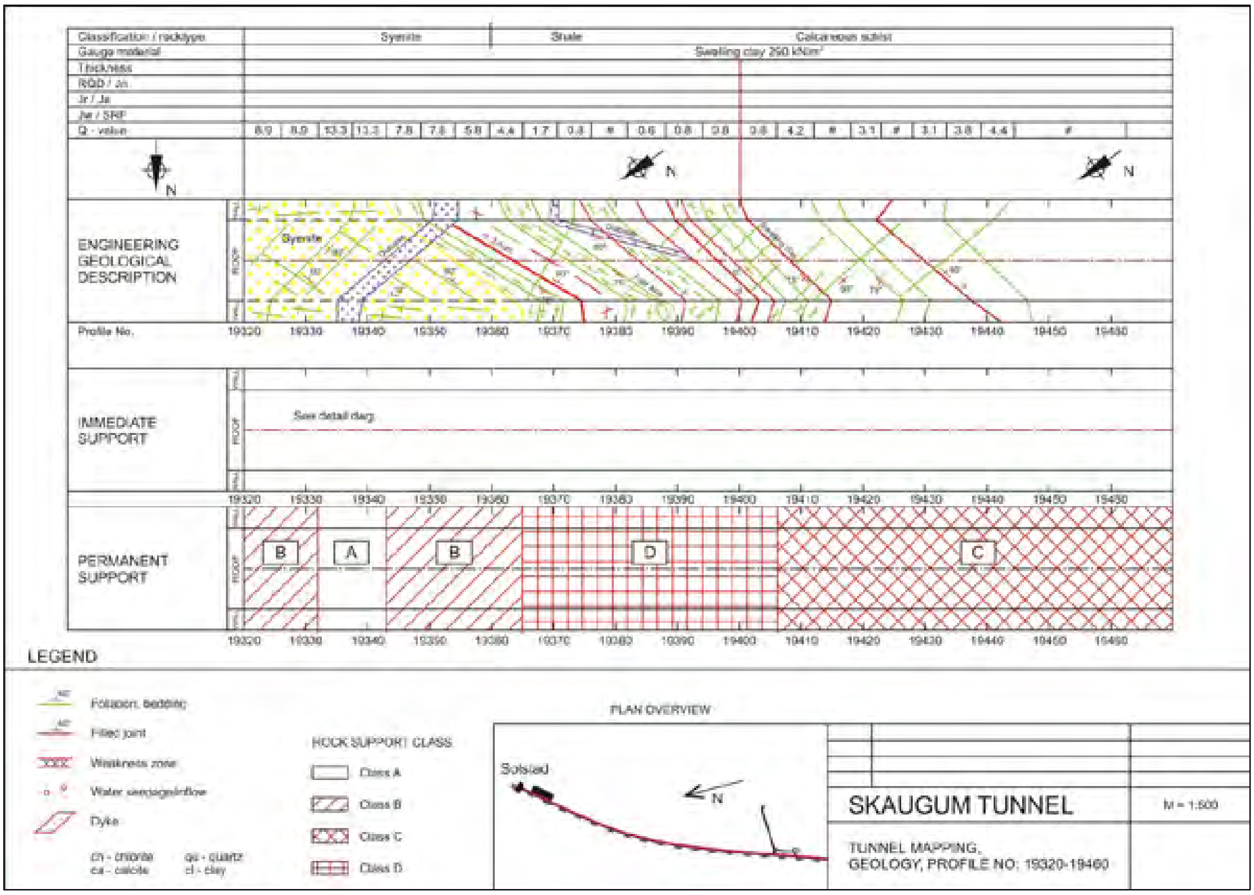


Figure 11. Example of log from detailed tunnel mapping, with geology and rock support class indicated (revised from NBG, 2008).

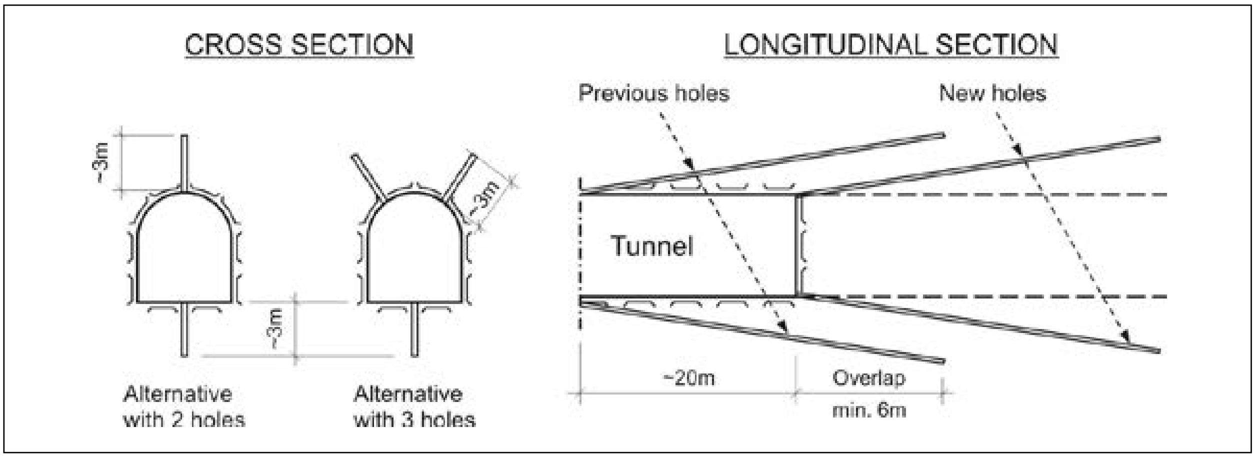


Figure 12. Examples of common layouts of percussive probe drilling.

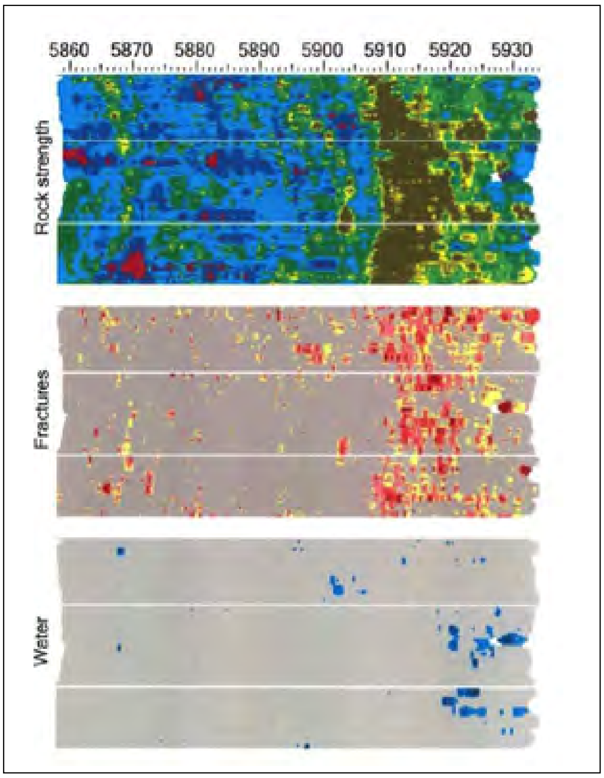
Normally, the tunnel jumbo with percussive drill is used for the probe drilling. Core drilling is used where particularly difficult rock conditions are expected. The extent probe drilling is determined based on expected rock mass conditions, rock cover and previous experience on the project. The number 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. In stead of Lugeon testing as basis for evaluation, the decision today is normally based simply on measuring the volume of leakage water from the holes, and grouting is carried out if the leakage 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.

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 rock mass parameters, prediction of factors such as rock strength, degree of fracturing and water inflow may be made. An example on the use of this method is shown in Figure 13.

Figure 13. Use of MWD for predicting rock mass conditions ahead of the tunnel face in the Karmsund subsea tunnel. In the upper part, red and blue represent hard, competent rock while yellow and brown represent weak rock, in the middle red and yellow represent high and medium degree of fracturing, respectively, and in the lower part blue represents water inflow (from Moen, 2011).



5 INVESTIGATIONS DURING OPERATION

For stability control of underground projects after completion, monitoring instruments such as extensomenters 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 outv at certain time intervals, and based on the results from this control, maintenance is carried out when required.

6 EXTENT OF INVESTIGATION

The basic philosophy regarding extent of ground investigation for underground excavations 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 simle geology, for instance, the requirement for ground investigation is less than for a 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 the 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 4. 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, and 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 reserch 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 are described in Palmstrøm et.al (2003). A water supply tunnel in simle geology typically will end up in class A, while a road or subway tunnel most commonly will end up in class C.

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).

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 preinvestigation, 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 (althoungh 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.

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: 20,000 NOK/m
Excavation cost: 20,000 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.

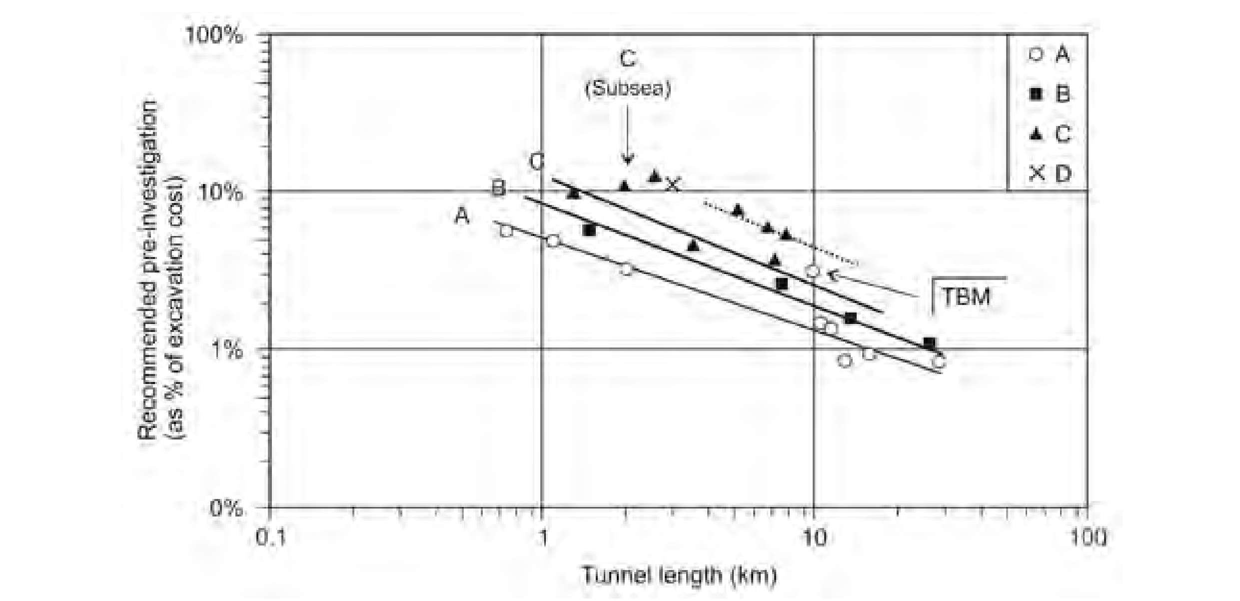


Figure 14. Recommended extent of pre-investigation as function of tunnel length for the respective investigation classes in Table 5. The excavation cost includes blasting, loading/hauling and 25 % rig costs. After Palmstrøm et.al (2003).

7 CONCLUDING 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 have 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 tunnelling practice some of the detailed design is postponed so that results from contruction 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.

REFERENCES

Myrvang A (1983): "Practical use of rock stress and deformation measurements". Norwegian Soil and Rock Engineering Association". Publ. No.2, p. 9-14.

NFF (2008): "Heavy rock support in underground excavations". NFF Handbook No.5 (in Norwegian), 72 p.

NPRA (2010): "Handbook 021: Road tunnels". Norwegian Public Roads Administration, Directorate of Public Roads (in Norwegian, 2004-edition in English), 133 p.

Nilsen B & Palmstrøm A (2000): "Engineering Geology and Rock Engineering". NBG (Norwegian Group for Rock Mechanics), Handbook No.2, 249 p.

Palmstrøm A, Nilsen B, Pedersen KB, Grundt L (2003): "Proper extent of site investigations for underground excavations". Report R&D project "Tunnels for the Citizens". Publication no. 101, Directorate of Public Roads (in Norwegian), 116 p.

Rønning, JS (2003): "Newer pre-investiagtions methods for tunnels". Report No. 12, R&D project "Tunnels for the Citizens". Directorate of Public Roads (in Norwegian), 12 p.

Standard NO (2008): "Eurocode 7: Geotechnical design – Part 1: General rules". Norwegian Standard NS-EN 1997-1-1:2004+NA2008.

03. EXCAVATION AND SUPPORT METHODS

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1 INTRODUCTION

Almost every tunnel in Norway has been excavated in formations we can characterise as hard rock. Traditionally the tunnels were supported by scaling and a few rock bolts.

A modern industrialised nation needs a modern infrastructure for transportation, and communities with a sustainable environment. This has forced us into geological formations with partly poor to extremely poor rock mass quality for new tunnels and underground constructions. National and international regulations require a wide scale of actions to take care of sensitive environment during construction and operating new tunnels. These are one group of moments of importance when contractors or owners are taking decisions about excavation method for a new tunnel. Today one of the most important elements in the discussion regarding excavation method is maximum allowed amounts of water seepage into the tunnel concerning the environment above the tunnel and the operation of the tunnel.

2 EXCAVATION METHODS

Conventional drill and blast (D&B) tunnelling is by far the most common tunnel excavation method in Norway. Traditional D&B with tunnel jumbos has also been the preferred excavation method in Norway for the last decades. Under hard rock conditions with large cross-sections this gives a great flexibility in handling the water seepage by continuous probe drilling and pre grouting. This flexibility also gives contractors and owners an opportunity to do an "on site" variation in use of reinforcement methods and strategies, according to the contract and preferred documentation system. In this matter the Q-system is most common in use in Norwegian tunnel projects.

Nevertheless, full face tunnel boring with tunnel boring machines (TBM) manifested itself as the selected excavation method for around 50 projects in Norway between 1972 and 1992. TBMs were mainly selected for hydropower tunnels due to its relative low construction time,

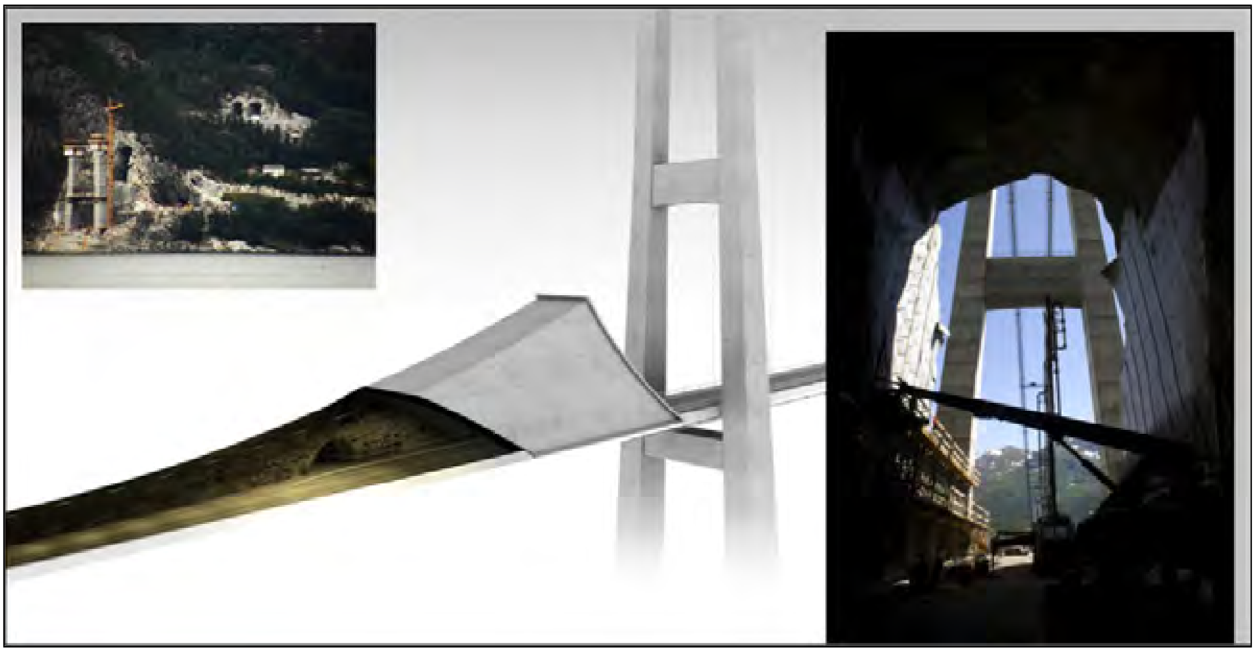


Figure 1: Hardanger Bridge Tunnels (NPRA 2013)

Figure 2: Follo Line Project approaching Oslo Central Station (Norwegian National Rail Administration - NNRA 2013)

smooth tunnel contour which is beneficial for reduced head loss, and small tunnel cross sections with limited adit possibilities. For Norwegian road tunnels, there are some examples of TBM tunnelling from the 1980's. The TBM tunnels in Norway were all excavated with open gripper TBMs (also referred to as main beam TBMs).

The rock support consisted mainly of rock bolts and shotcrete, without any continuous concrete lining as used in single and double shield TBMs. From around 2010 however, Norway faces a small TBM "renaissance" with the planning of two railway lines and one hydropower scheme. The Follo Line (southbound tracks from Oslo) is likely to be excavated with 4 double shield TBMs with continuous concrete lining, which will involve a new and rapid rock support technique in Norway.

3 WATER CONTROL

Norway is a blessed country with regards to water and the ground water level is usually just a few meters below the terrain surface. Every tunnel represent as such a draining element in the ground as all Norwegian tunnel projects up to this time with few exceptions have been constructed as globally drained structures. This is an important factor in relation to the flexibility in varying the permanent rock support.

4 CONTRACTS

The tender documents and contract usually reflect the results from the pre-investigations and the planning process including local authorities and neighbours. The documents will describe which prime methods of support the Owner want to use on the project. This is based up on construction regulations given in manuals and the results from the pre-investigations. The contracts which have traditionally been unit price contracts, will usually also describe a variety of alternative methods. More advanced methods are usually planned to be used in foreseen or suspected weakness zones and sections with poor rock mass quality. The variation in use of the different methods will be decided in close cooperation between the contractor and the Owner usually represented by the always present site engineering geologist. The decision will be taken with all available information from pre-investigation, continuous mapping, probe drilling and general observations.

The contracts put the responsibility for the "working safety" on the contractors. This leads to the fact that the contractor, alone makes the most decisions about the primary support. Usually this includes scaling, rock bolting and fibre reinforced sprayed concrete. It is of great importance that the primary support is being seen as a first step of the permanent stabilisation and rock support. This has been a commonly used principle in Norwegian tunnelling, and presents the tunnel surface to the users as a rough surface of sprayed concrete in most of low traffic tunnels and railway tunnels. Tunnels constructed for more traffic and new railway tunnels can be covered with fire protected membranes/ insulation to reduce water drips on road or rails.

Road and rail tunnels constructed for high traffic density are now given an inner lining of segmental precast concrete elements or a cast-in-place concrete lining.

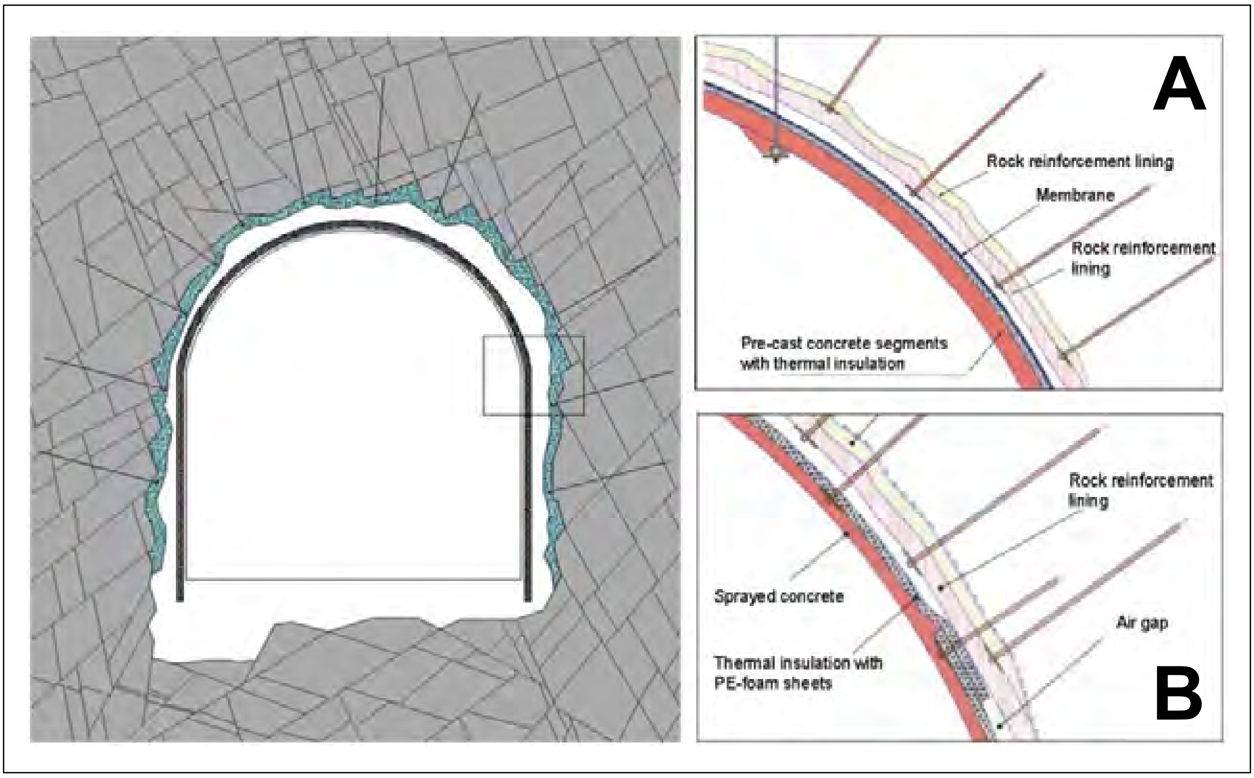


Figure 3: Layout of traditional Norwegian tunnel lining system with two options.
A: Shield system with thermally insulating pre-cast concrete elements. B: Shield system with PE sheets (NPRA 2012).

5 ROCK SUPPORT PHILOSOPHY AND PRACTICES

Incidents with rock falls in tunnels, subsequent inspections of both the rock mass and the installed rock support in Norwegian road tunnels revealed a need to introduce a more systematic determination and execution of permanent rock support compared to previous practice in Norwegian road and railway tunnels.

Norwegian Public Roads Administration (NPRA) Handbook No. 021, Design Manual for Road Tunnels provide the requirements of the permanent rock support for long term stability, and this includes feasibility studies, competent site follow-up, documentation during excavation and standardised permanent rock support in accordance with rock mass quality classes. The NPRA is continuing the Norwegian Tunnelling Technology concept, which is based on the principle of taking advantage of the rock mass as a competent building material. The rock support system that is described in Handbook No. 021 aims to ensure the stability and a service life of 100 years or longer based on the NPRA's longstanding experiences that are summarized in the NPRA Research and Development Program, "Modern Road Tunnels".

6 GEOLOGICAL PRE-INVESTIGATIONS AND FEASIBILITY STUDIES

Pre-Investigations comprise geological field surveys, different geophysical methods and probe/exploratory drilling ahead of tunnel face. Within all of these themes, there have been major improvements and new developments. Laser scans of the terrain gives a realistic 3D-map model which is a prerequisite today for safe project execution, good geological feasibility studies as well as documentation. Such a map model is important no matter the discipline with the opportunities that exist to make discipline related layers, as for instance the rock surface with a soil cover on top of it.

The Geological Survey of Norway (NGU) and NPRA have over several years worked together, in order to improve existing investigation methods and develop new methods for feasibility evaluation of tunnel projects. Methods in question are:

- Awareness map for deep weathering during tunnel planning

- 2D resistivity with modelling of resistivity responses
- Evaluation of tomographic inversion of refraction seismic data (work in progress)
- Preparation of guidelines for resistivity measurements with a compilation of the resistivity and seismic velocities for Norwegian rocks

7 INVESTIGATIONS DURING TUNNEL EXCAVATION

When drilling is performed with automatice "drill and blast" tunnelling jumbos, it is possible to "see" ahead of the face by use of "Measurement While Drilling" (MWD) and "Borehole Parameter Interpretation" (BPI). Further development of the method has been obtained by thorough evaluation and comparison of the results from the geological mapping at the face from several projects. The lessons learned are that the method gives reliable records of structures such as fractures or weakness zones, intrusive veins or the transitions between the rock types. The method does not render geological expertise at the face superfluous, but is a supplement and gives support to the systematic manual mapping. Active use of BPI-software makes it possible to identify unstable fracture zones in front of the face utilising the MWD-data from the long-hole drilling or blast round drilling. The method is in most cases an alternative to core drilling at the face.

8 EXPERTISE AT THE TUNNEL FACE DURING SCALING AND GEOLOGICAL MAPPING

As an important part of the Norwegian rock support strategy, requirements have been set that all blast rounds/faces shall be mapped geologically by an engineering geologist or other experienced rock engineering personnel before the rock surface is covered by sprayed concrete. In order to achieve this, the Owner pays for the stoppage time necessary for conducting proper mapping as soon as scaling of the tunnel face is completed. Scaling operations are usually carried out in to steps as mechanical scaling first with a pneumatic / hydraulic hammer and secondly as manual bar scaling from a platform. The engineering geologist is commonly taking

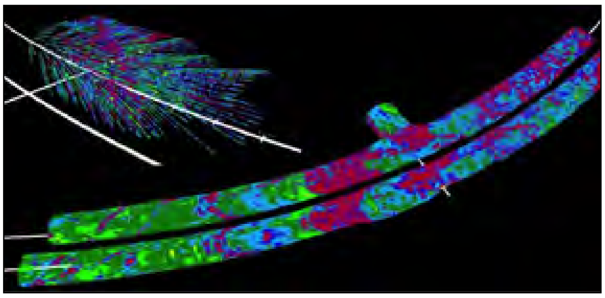


Figure 4: BPI image of MWD-recorded rock mass hardness in the Løren Tunnel in Oslo (NPRA 2011)

Figure 5: Illustration of geological mapping performed from a scaling platform (NPRA 2011)

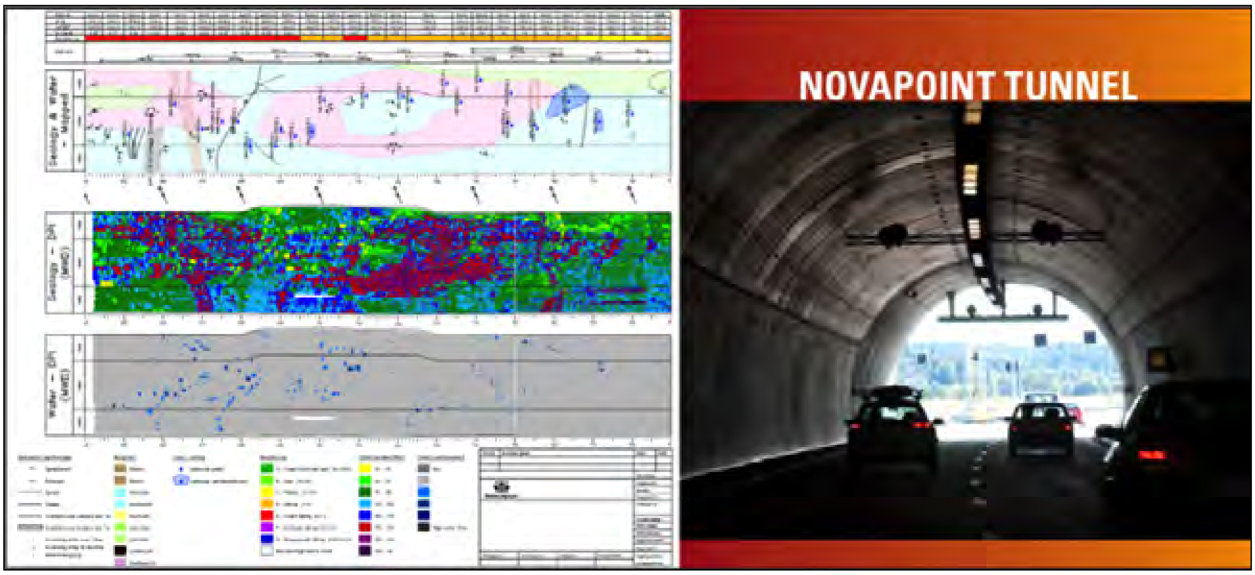


Figure 6: Novapoint Tunnel documentation with geological mapping and BPI in the Løren Tunnel in Oslo (NPRA 2011)

part in the manual bar scaling crew. Sufficient manning from the Owner side of all shifts is then a prerequisite. All rock surfaces are to be mapped at close range from the manual bar scaling platform usually mounted on the wheel loader bucket. The required staffing and competence level is dependent on the project's complexity.

In the project Modern Road Tunnels a complete system for follow up, registration and documentation of the geology and rock support - Novapoint Tunnel: Geology and Rock Support, has been developed.

9 PERMANENT ROCK SUPPORT STRATEGY

Manual 021 Road Tunnels provide requirements for follow-up under excavation of tunnels and to the documentation of the geology and rock support. The Rock Support Class table in the manual then defines the different classes for permanent rock support that is attached to the rock mass classification using the Rock Mass Condition Classes. Several methods can be used for determination of the Rock Mass Condition Classes in which classification by use of the Q-value is the most

Rock Mass Class	Rock Mass Conditions (Q-value[1])	Rock Support Class (Permanent Rock Support)
A/B	Low-jointed rock mass Average joint spacing > 1 m Q = 10-100	**Rock Support Class I** - Spot bolting - Reinforced sprayed concrete B35 E700, thickness 80 mm in crown and walls down to max 2 m above tunnel invert
C	Moderately jointed rock mass Average joint spacing 0.3 – 1 m Q = 4 – 10	**Rock Support Class II** - Systematic bolting (c/c 2 m), end-anchored, pre-tensioned, fully grouted rock bolts - Reinforced sprayed concrete B35 E700, thickness 80 mm, in crown and walls down to tunnel invert
D	Heavily jointed rock mass or stratified schistose rock mass Average joint spacing < 0.3 m Q = 1 – 4	**Rock Support Class III** - Reinforced sprayed concrete B35 E1000, thickness 100 mm or more - Systematic bolting (c/c 1.5 m), end-anchored fully grouted rock bolts as permanent support
E	Very poor rock mass quality Q = 0.1 – 1	**Rock Support Class IV** - Spiling bolts when Q < 0.2, Ø25 mm, max c/c 300 mm - Reinforced sprayed concrete B35 E1000, thickness 150 mm - Systematic bolting, c/c 1.5 m, fully grouted rock bolts - Reinforced ribs of sprayed concrete, Q < 0.2, rib dimension E30/6x Ø20 mm, c/c 2 – 3 m between ribs, bolted systematically c/c 1.5 m, 3 – 4 m long rock bolts[2] - Decide if cast-in-place invert slab is necessary
F	Extremely poor rock mass quality Q = 0.01 – 0.1	**Rock Support Class V** - Forepoling, c/c 200 – 300 mm, ø32 mm or self-boring stud bolts - Reinforced sprayed concrete B35 E1000, thickness 150 – 250 mm - Systematic bolting, c/c 1,5 m, fully grouted rock bolts, c/c 1,0 – 1,5 m - Reinforced ribs of sprayed concrete, rib dimension D60/6+4 x rebar Ø20 mm, c/c 1.5 - 2 m between ribs, bolted systematically c/c 1.0 m, 3 – 6 m long rock bolts[2] - Decide if cast-in-place invert slab is necessary - Cast-in-place reinforced invert slab with rise-to-span ratio min. 10 % of the span of the tunnel
G	Exceptionally poor rock mass Q < 0.01	**Rock Support Class VI** - Special evaluation of excavation and rock support

[1] Q-values are given for uniaxial compressive strength, UCS = 100 MPa

[2] Requirements to materials, methods, and solutions are described in «Teknologirapport nr. 2538: Arbeider foran stuff og stabilitetssikring i vegtunneler» (Technology report No. 2538: Works ahead of the tunnel face and rock support in road tunnels), in Norwegian only

Figure 7: Table 7.1 from Manual 021 Road Tunnels (NPRA 2010)

commonly used. The Rock Support Classes are designed based on many years of experience with installed rock support. Rock mass classification contributes to both a more objective assessment of conditions and more effective communication between professionals in the selection of permanent support levels.

For Norwegian road tunnels the NPRA Technology Report No. 2538 "Works ahead of the face and rock support in road tunnels", draws up standardised rock support with focus on longevity and stability.

In order to increase the life time of sprayed concrete and bolts, sprayed concrete thickness has increased to 80 mm minimum average thickness in terms of service life. Fibre content is also increased as a result of the new requirements for energy absorption classes in the Norwegian Concrete Association publication No. 7.

Sprayed concrete is introduced in all tunnels and in all Rock Mass Classes, also in the best classes A and B. The background for this requirement is the experience with the fact that the rock mass is affected by blasting, and have micro cracks and fissures that after a period of moisture and freeze/ thawing cycles can give fallouts of pebbles and in the worst case, large blocks of loose rock. Existing tunnels with no sprayed concrete in the walls and crown are to be treated with manual bar scaling on a regular basis.

In order to improve and maximise the life time of rock bolts for stability support it is recommended to use fully grouted bolts in order to get the best possible corrosion protection where there is no rock pressure problems. In areas with high rock stress it is advised to use resin (polyester) end-anchored bolts.

Execution of reinforced ribs of shotcrete is detailed in NPRA Technology Report No. 2538. Arches can be built with curved pre-deformed Ø20 mm rebar with the arch foot founded and anchored to the rock by casting and bolting.

This is a change from previous execution where Ø16 mm rebar was attached to the rock surface and bent to follow the blasted surface. The use of curved pre-deformed Ø20 mm rebar makes it possible to do stability calculations on an arch form. With the new requirements the reinforcement is now placed correctly on the tension side of the arch construction instead of attached to the rock surface at the pressure side as was carried out in the past.

Figure 8: Reinforced Ribs of Shotcrete (RRS) in the Grønlia Tunnel in Oslo (NPRA 2008)

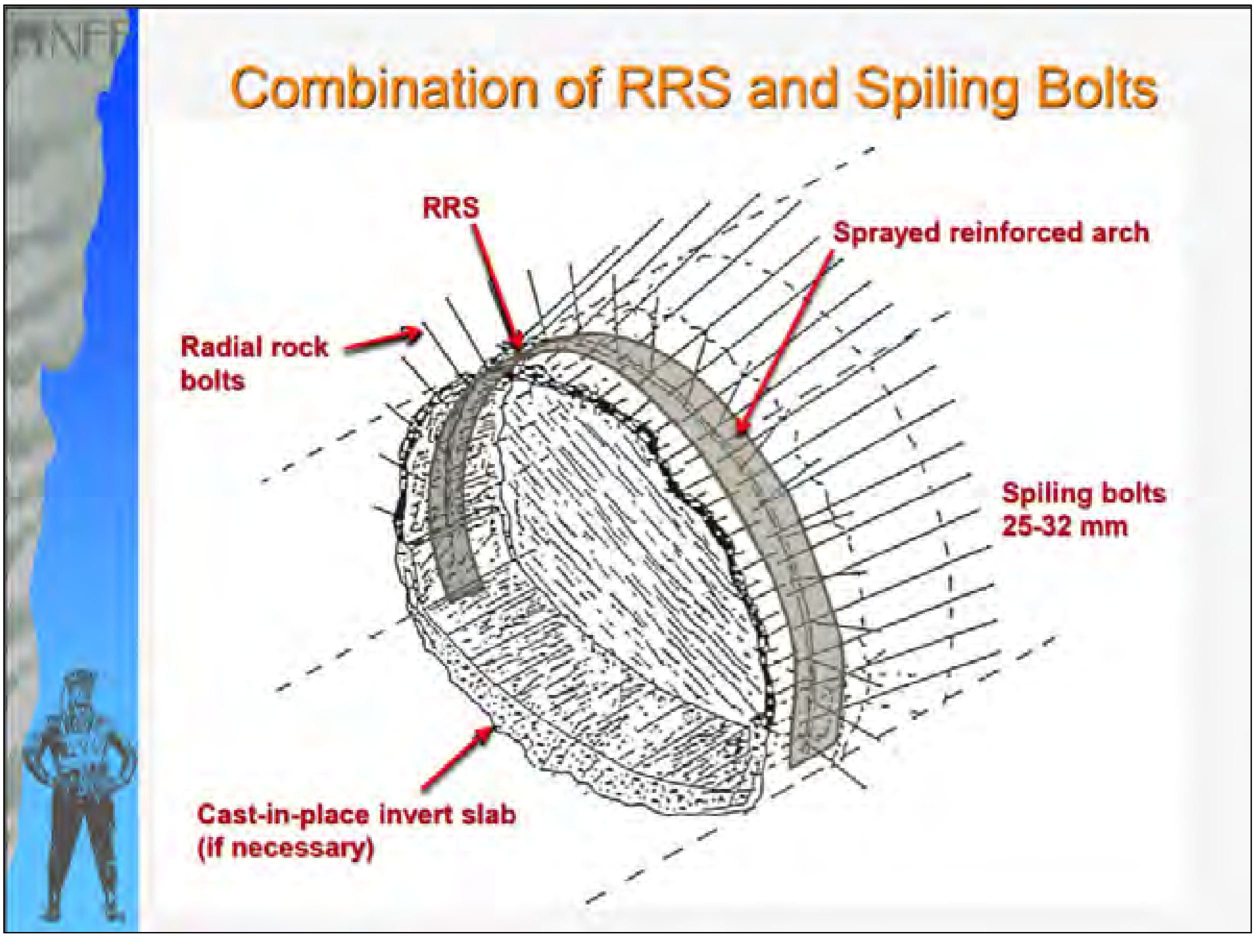


Figure 9: Principle of combined RRS and spiling bolts (NFF 2012)

The Technology Report also describes the execution of spiling bolts and forepoling in combination with reinforced ribs of shotcrete. The combined method ensures that spiling bolts will have anchoring both inside the rock mass ahead of the blast round and in the concrete arch at the face. Cast-in-place invert slabs are described for Rock Mass Classes with adverse rock mass quality.

In Manual 021 Road Tunnels an awareness zone of minimum 15 m in front of any predicted weakness zone for the conduct of pre-investigations and rock mass grouting has been introduced. This requirement ensures that each zone can be passed with a higher level of safety by allowing the stability support with arches and forepoling to be implemented at the appropriate time. Final decision on the permanent rock support is being made after the passage and after observation of the behaviour of the installed rock support.

In the project Modern Road Tunnels the focus is also put on smooth contour and reduced damage zone from blast induced cracking, which are important parameters to ensure long term stability and lifetime. This is an area where there is a significant potential for improved quality. Measures, both contractually and on the equipment/ explosive development side will be introduced in order to achieve more optimal contour.

10 GROUND FREEZING

In Rock Mass Class G and the corresponding Rock Support Class VI no standardised rock support is given. When performing the evaluation of excavation method and rock support, one likely result may be the ground freezing option.

The usage of the ground freezing method is based upon the facts that the strength of the ground increases when the ground freezes, and that the frozen ground becomes watertight when saturated. Ground freezing is mainly used to create a temporary supporting structure or a watertight barrier, - but commonly a combination.

The freezing is done by installing freezing pipes into drilled holes. Temperature gradients are set up by a cold

Figure 10: Pilot tunnel through 90 m of frozen ground at Hallandsås twin railway tunnels (GEOFROST 2012)

medium, and heat is transported away by the medium flowing through the freezing pipes.

The frost cylinders form around each freezing pipe as the isotherms spreads from the pipe. After a period of time the frozen cylinders are merging into a frozen structure.

Frost structures can be made in all geological formations, but the different materials may differ a lot in the frozen properties. These can be tested in laboratory.

There are basically two freezing methods; indirect (brine freezing) and direct (nitrogen freezing). In indirect freezing with brine, the brine is circulating in a closed system, delivering the heat from the ground to a freezing plant run by electric power. Heat is removed from the freezing plant by cooling water. In direct freezing, heat from the ground boils the liquid nitrogen which is let into an open pipe system. The vapour (the nitrogen gas) is blown back to the atmosphere and constantly refilling is required.

Both methods are environmentally friendly, leaving no chemicals in the ground.

When the water in the ground freezes, the binding of the grain structure gets stronger than in unfrozen condition and the strength continues to increase as the temperature decreases.

Designing a frozen structure means calculating necessary thickness and temperature based on purpose of the structure, external loading, and properties of the frozen ground as well as risk aspects concerning failure.

Which freezing method and what pipe spacing to use depends on project, location, ground conditions, heat sources, thermal properties of the ground, available time and of course the required temperature and thicknes of the frozen structure.

The achieved frozen structure is controlled by temperature measurements. Results and costs are predictable and controllable.

Ground freezing in tunnelling can be used as temporary ground support, water cut-off or a combination of both. Usually, grouting, de-watering, compressed air and forepoling/pipe umbrella etc. would be first choices when tunnels enter into poor ground conditions. However compressed air, dewatering and grouting are only effective in a limited range of soil particle sizes. Such constraints are not usually imposed on the ground freezing process. Freezing can very well be combination with the drill and blast method, being a useful back up in shifting ground conditions. For TBM excavation ground freezing can be used for the cross passages and near start and entering shafts.

Ground freeing may very well be used for soil tunnels being too short or having too complex geometry for TBM excavation.

REFERENCES

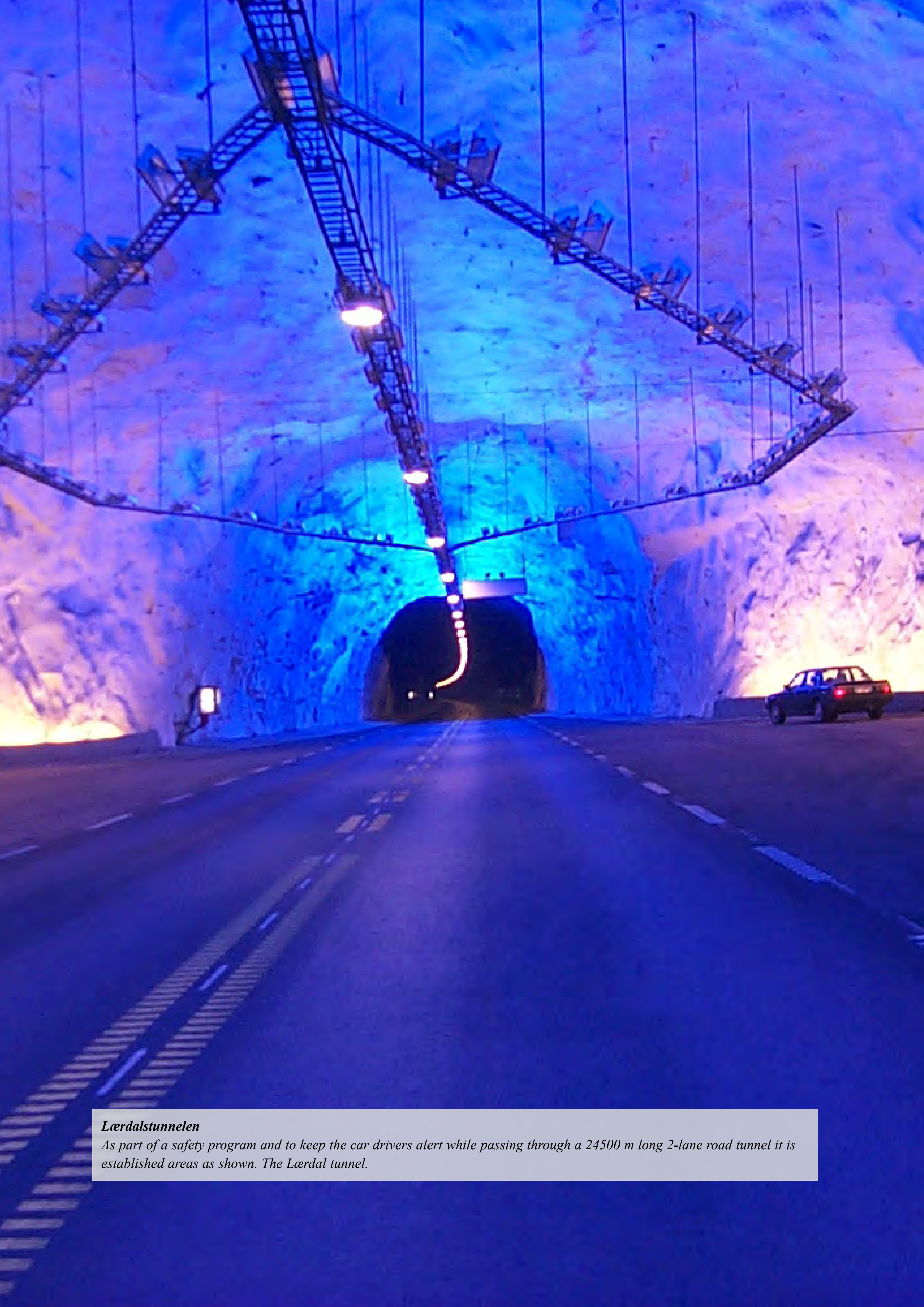
Barton, N., Lien, R., Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics, 6(4): Springer Verlag, Vienna, pp 189-236

Holter, K.G., Buvik, H., Nermoen, B., Nilsen, B. 2013. Future trends for tunnel lining design for modern rail and road tunnels in hard rock and cold climate. Proc. World Tunnel Congress 2013 Geneva, Underground - The Way to the Future! pp 1435–1442

NPRA Norwegian Public Roads Administration. 2010. Technical Report No. 2538. Works ahead of the tunnel face and rock support in road tunnels, NPRA, Oslo (Norwegian)

NPRA Norwegian Public Road Administration. 2010. Handbook 021, Norms: Road Tunnels (Norwegian)

NNRA Norwegian National Rail Administration, 2012. Design guide; Jernbaneverket; Underbygning/ Prosjektering og bygging/tunneler, fra teknisk regelverk, 6.1.2012 (Norwegian)



Lærdalstunnelen

As part of a safety program and to keep the car drivers alert while passing through a 24500 m long 2-lane road tunnel it is established areas as shown. The Lærdal tunnel.

04. TUNNEL AND CAVERN SUPPORT SELECTION IN NORWAY, BASED ON ROCK MASS CLASSIFICATION WITH THE Q-SYSTEM

BARTON, Nick
GRIMSTAD, Eystein

1 INTRODUCTION

Norway is a country with a small population, yet 3,500 km of hydro-power related tunneling, about 180 underground power houses, and some 1,500 km of road and rail tunnels. This has meant that *economic* tunnels, power-houses and also storage caverns, have always been needed, especially prior to the development of North Sea petroleum production. The Q-system development in 1973 always reflected this, and single-shell tunnel support and reinforcement, meaning shotcrete and rock bolts as final support has been the norm, both before and since Q-system development. The first 200-plus case records from which Q was developed were mostly from Scandinavia, and already represented *fifty different rock types*, which is perhaps surprisingly for those who may focus on the quite frequent pre-Cambrian granites and gneisses. Norwegian and Swedish hydro power projects dominated these early cases, giving a wide range of excavation sizes and uses (i.e. access tunnels, headrace tunnels, powerhouses). An update of the Q-system support methods, presented by Grimstad and Barton, 1993, was based on 1,050 new case records collected between 1986 and 1993. These were deliberately chosen to be independent of Q-system application. They were mostly developed from road tunnel projects, where higher levels of support were generally used. This update specifically replaced S(mr) with S(fr), meaning the replacement of steel mesh with steel fibre-reinforced shotcrete. In 2002, approximately 800 more case records were added, giving further independent measures of S(fr) thickness and bolt spacing. The Q-value had been logged, but was not used in many cases. Some inconsistent results can be noted, including three collapses where Q-recommendations were not used. (Appendix A4).

2 HOW AND WHY Q WAS DEVELOPED

A question from the Norwegian State Power Board (Statkraft) which was passed to the first author at NGI, was the following: 'Why are Norwegian powerhouses showing such a wide range of deformations'? A lack of quantitative methods for describing rock quality in 1973, besides Deere's RQD from 1964, and the need to consider excavation dimensions, depth and possible stress levels, together with the different support measures used at that time, meant that a new and integrated method was needed. After 6 months of extensive case record study, using a successively updated list of rock mass parameters and constantly updated ratings, the Statkraft question could be finally be answered. This 6 months delay saw the development of the Q-system (Barton, Lien and Lunde, 1974), which has eventually become one of the main rock mass classification methods used throughout the world of mining and civil engineering. It is often used alongside RQD and RMR (Bieniawski, 1989), both of which were developed before Q. Both RMR and Q have made use of RQD, and in the case of Q, the RQD % is used directly, unless it is < 10%. (The minimum used is 10%).

3 CLASSIFICATION METHOD BRIEFLY DESCRIBED

Trial and error using two, three, four and finally six parameters, with successive adjustment of ratings to get the best fit between rock quality, excavation dimensions, and support quantities, resulted in one of the simplest equations regularly used in rock engineering.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (1)$$

Due to the need for the rock mass classification to fit the case records, the ratings and format of the Q-equation resulted in something resembling a log-scale, with Q ranging from approximately 0.001 to 1000, from the worst (faulted, squeezing, water-bearing) conditions, to the best (massive dry) conditions. The formal definitions and ratings of the six parameters are tabulated in the Appendix. It should be noted that the three pairs of parameters *RQD/number of joint sets, joint roughness/joint alteration-filling, water/stress-strength,* resemble, in very approximate terms: block size, inter-block shear strength and active stress.

Q-parameter definitions	Q case-record back-ground
RQD is the % of *competent* drill-core sticks > 100 mm in length *in a selected domain*. (In tunnel mapping imagine cores or scan-lines).	The initial data base in 1973 was 212 cases of single-shell tunnels and caverns, for hydropower, road, rail, storage, sewage.
Jn = the rating for the number of joint sets (9 for 3 sets, 4 for 2 sets etc.) *in the same domain*.	About 60% of the initial cases were from Scandinavia and about 40% were from Europe, USA, etc.
Jr = the rating for the roughness of the *least favourable* of these joint sets or filled discontinuities, *in the same domain*.	About 50% of the initial cases were from hydropower projects in Norway and Sweden. Fifty rock types were initially represented. The majority were igneous and metamorphic rocks, with a smaller number of weak sedimentary rocks.
Ja = the rating for the degree of alteration or clay filling of the *least favourable* of these joint sets or filled discontinuities, *in the same domain*.	Numerous shear zones and faults containing clay, and numerous cases with clay-coated and clay-filled joints were included.
Jw = the rating for the water inflow and pressure effects, which may cause outwash of discontinuity infillings, *in the same domain*.	Numerous cases of weathered conditions were also included, with all Q-parameters adversely affected.
SRF = the rating for faulting, for strength/stress ratios in hard massive rocks, for *squeezing* or for swelling *in soft rock – in the same domain*.	In 1993 another 1050 case records were added, mostly from road tunnels. S(mr) was replaced by S(fr) – fiber reinforced shotcrete. S(mr) for tunnel support was totally replaced by S(fr) by 1983.
(Note: in the 1993 update, three new high-SRF classes related to the observed effects of high stress and extreme support needs were added, specifically for the case of 'spalling' and 'bursting' in initially *massive* rock). See Appendix A1, Table 6b, L, M and N. Stress-induced fracturing: $\sigma_{\theta(max)} > 0.4\sigma_c$ (However in deep mines with significant numbers of joint sets one should use the original lower SRF values from 1974).	These updates provided case records in which the Q-recommendations were not used, ensuring 'independence'. In 2002, approximately 800 more cases of S(fr), RRS and bolt spacing for permanent support were added. The scatter seen in non-Q practice is sometimes wide and includes cases of cave in. See Appendix A4. There are now approximately 2,060 tunnelling/cavern cases in total, which lie behind the Q-support-and-reinforcement recommendations for tunnelling.

Table 1 A short summary of the Q-system parameters and the case record back-ground.

The Q-system is designed to assist in feasibility studies, and to be actively used in detailed site characterization when mapping exposures, interpreting seismic velocities and logging drill-core. It is also used systematically once tunneling begins, since the mapped *rock class* following each blast can be a basis for selecting tunnel reinforcement (bolting) and support (fiber-reinforced shotcrete). In the following sections we will give photographic examples of core logging, surface-exposure logging, and tunnel logging, so that potential users, or those just interested in the Norwegian way of doing things, can get some feel for the method.

The Q-system needs to be used by engineering geologists with some reliable training and experience. The initial assessment naturally involves an evaluation of the degree of jointing, the number of joint sets: i.e. the general degree of fracturing and block-size, followed by

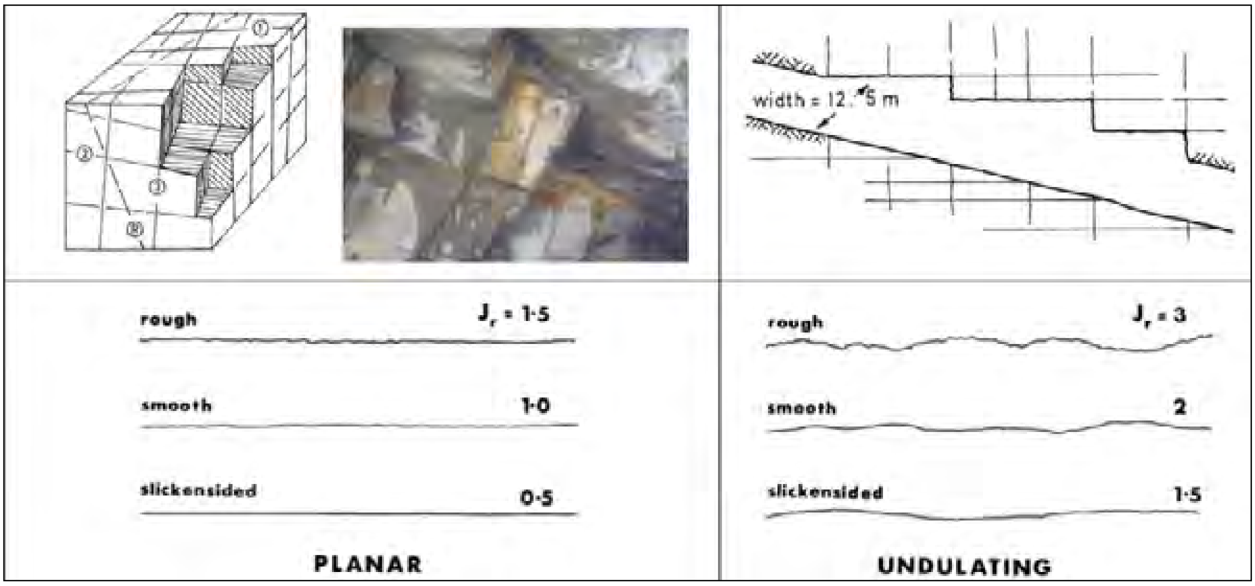


Figure 1. Some graphic illustrations of the workings of the Q-parameters, using number of joint sets (Jn) and roughness (Jr). Sufficient numbers of joint sets may or may not cause over-break. When Jn/Jr ≥ 6, over-break becomes extremely likely, even with careful blasting. High Ja obviously assists here.

Figure 2. Contrasting worst (Q ≈ 0.001) and best (Q ≈ 1000) rock mass qualities. The logarithmic appearance of the Q-value scale, stretching over six orders of magnitude, has proved to be a great advantage, and results in simple empirical equations for relating to velocity, modulus, and deformation. The large numerical range appears realistic when considering shear strength and modulus variation.

an assessment of the most adverse Jr/Ja combination, taking into account favourable and unfavourable orientations. What is causing most over-break (e.g. Figure 1), and what would happen with no reinforcement or support? Experience is also essential in the determination of the necessary SRF category. Evaluation of his parameter involves knowing the depth or likely stress level in relation to the probable strength of the rock. The degree of stress-induced fracturing, if already occurring, or the amount of shearing and clay that is present in the case of fault zones, will each give clues to the appropriate value of SRF. Water inflow is also assessed, with or without the availability of Lugeon or permeability test results in the early stages of logging, and when local measurements are not available.

4 EXAMPLES OF CORE-LOGGING WITH Q

In projects where there are poor exposures due to weathering, the first sight of the rock may be via drill-core. It is strongly advised that a significant number, perhaps most of the bore holes, should be deviated from vertical, because of the frequency of sub-vertical structure which is poorly sampled by vertical holes. In a recent rail tunnel in Norway, all five boreholes were strongly deviated, thereby sampling folded and steeply tilted inter-bedding much more effectively. Vertical holes may give false higher quality, and unrealistically low permeabilities.

Deeply weathered granite. Few pieces of this core box qualify for RQD assessment: i.e. 0, 20, 0, & 0 %. Change RQD of <10% to 10% when calculating Q.

Shale inter-bedded with limestone. Note recording of Jr = 1 and 1.5, therefore low roughness JRC. Q_{range} = (30-50)/9 x (0.5-1)/(3-4)x0.66/1≈ 0.8-0.9.

Good quality granite sampled in a deviated hole. Q_{range} = (65-100)/(6-9) x (1-1.5)/1 x 0.66/1 ≈ 5 – 25 Note that RQD continues its function when core pieces are > 10cm in length. RQD also indicates anisotropy.

Partly altered joints in tuff. When logging core close to tunnel depth (175m), degree of joint connectivity gives clue to Jw prior to Lugeon test. Q_{range} = (10-25)/(12-15)x1.5/(2-4)x(0.5-0.66)/2.5≈0.1-0.3.

A weakness zone with swelling clay from the Finnfast sub-sea tunnel site investigation. Recovery from bore hole drilled in tunnel face. Q_{range} ≈ 10/(15-20) x 1/(13-16)x(1.0-)/2.5 ≈ -0.01- 0.02.

Crushed zone in black shale from the Rogfast sub-sea tunnel site investigation. Seabed-to-borehole seismic tomography borehole. Q_{range} ≈ (10-30)/(15-20)x(1-2)/(6-8)x(1.0-0.5)/2.5≈0.01-0.3.

Figure 3. Six contrasting core boxes from road and rail tunnel projects in Norway and Hong Kong. In both countries use of the Q-system for core logging and tunnel logging is required by the authorities. The weakness zone in the Finnfast tunnel was quite dry both during core drilling and after excavation. There was little water in the drill hole at Rogfast.

5 ADDITIONAL ADVICE CONCERNING CORE LOGGING

Since drill cores are often missing where the rock quality is very poor due to poor recovery (e.g. see the plastic containers in Figure 3.1), the rock mass lack-of-quality has to be assessed by other methods, such as seismic velocity or resistivity. Where cores do exist, and there is good recovery, the first four of the Q-parameters may be evaluated with a relatively high degree of accuracy. However, special attention should be addressed to the following:

- Evaluation of the large and medium scale roughness parameter Jr may be difficult when joints are intersecting the borehole at an obtuse angle, due to short samples.
- As water is generally used during drilling, mineral fillings like softer clay minerals may be washed out, making it difficult to evaluate Ja in some cases.
- Joints sub-parallel to the borehole will be under-represented, and will give too high RQD-values and too low Jn-values. So both Q and permeability will be affected.
- RQD is often calculated for every meter. However Jn must usually be estimated for sections of several metres, by observing the core boxes from above and below.

- Water loss or Lugeon tests are often carried out during core drilling, and can form the basis for evaluation of the Jw-value. Since grouting often reduces the permeability, and tends to improve many Q-parameters, there will be an increase in the estimated Q-value in case of logging grouted sections of the rock mass.
- An estimation of SRF in massive rock can be made based on the height of overburden, or the height/steepness of an eventual mountain side. If stress measurements are carried out in boreholes, or experiences from nearby construction sites are available, these should be used so that the probable stress magnitude can be compared with an estimate or measurement of the uniaxial strength of the rock. (Core-disking and subsequent stress-induced fracturing each give clues to stress levels).

6 CHARACTERIZING SURFACE EXPOSURES WITH EXAMPLES

In Nordic countries in particular, where glaciation has exposed a lot of rock, it is possible to gain a good assessment of the higher end of the rock quality scale and likely best tunneling conditions, by observing and mapping surface exposures. When road cuttings are also available, the rock conditions in these better rock-quality terrains can also be readily mapped. However, seismic refraction measurements (next section) and dedicated deviated drilling of low velocity weakness zones will be needed where exposures are absent in flatter and lower areas. These low-relief areas may nevertheless be tunneled under, such as in the case of future high speed railway tunnels. One of these projects is about to start near Oslo.

More than 300 rock cuttings were Q-logged to obtain rock mass quality input for Q_{TBM} prognoses for upcoming rail tunnels near Oslo. However this exposure logging gave data of relevance only to the top five rock classes, and seismic results and core logging of weakness zones was needed to provide approximate information on the lowest rock classes. In Figure 4, some examples of exposure logging using the Q-method are illustrated, using a deliberately wide variation in rock mass quality from the Oslo region.

Arguments are sometimes heard in conferences that Norway just has pre-Cambrian granites and gneisses, and therefore excellent tunneling conditions. In fact Norway has some (lower percentage of) extreme tunneling conditions, with quite frequent swelling clay, occasional sand inrushes, rock bursting where high cover, and some extensively sheared and clay-bearing rock masses, requiring heavy support, and the actual need of local concrete lining. There are at least ten named collapsed caldera in the geologic history of today's Oslo region.

7 ADDITIONAL ADVICE CONCERNING SURFACE EXPOSURE LOGGING

- The near surface rocks will often be more jointed than the unweathered rock masses at a greater depth. This may especially be the case in schistose rocks, which often have a tendency to disintegrate near the surface. Frequently only the better quality rock masses are exposed at the surface.

- Exposures in the terrain are often well rounded by the ice in Nordic countries and weathered in other countries, reducing the possibility to see joints undisturbed, therefore making reliable description of roughness Jr and joint filling Ja, rather difficult. The parameter RQD will usually be underestimated from natural outcrops, due to weathering or frost damage, while Jn will tend to be over-estimated. However in competent hard rock which has been rounded by ice, RQD will be over-estimated and Jn will be under-estimated, due to erosion of the more jointed materials.

- In weathered rock, the joints may be hidden at the surface. Hence the Q-values relevant to tunnel depth could in some cases be over-estimated. However, depending on rock type, the quality at depth may often be seriously underestimated using surface exposures, and experience is needed to make relevant adjustments for this.

- In high road cuttings or other excavated slopes, the joint surfaces are normally well exposed after blasting, giving a more reliable basis for estimating RQD, J_n, J_r and J_a.

- Rock cuttings excavated in different directions, if sufficiently high, give approximately the same Q-values as in a tunnel, but ignore small cuttings in partly weathered rock.

- The water leakage in a tunnel, J_w, will be difficult to predict from field mapping only. Water loss tests in boreholes and/or empirical data from projects in similar rock masses are necessary to obtain good predictions of the water conditions.

- A prediction of the SRF-value may be made based on the topographic features and knowledge of the stress situation in nearby underground openings in the region. High and steep mountain sides often give an anisotropic stress field.

The massive nature of this motorway rock cutting can be judged by the 2m high 'elg fence'. Class 1 granites. $Q_{range} \approx$ (90-100)/(6-9)x(1.5-2)/(1-2)x(0.66-1)/1≈ 5 - 66. Massive, abrasive, hard to bore with TBM.

Drammen granite near Lier Tunnels. Classic three joint-set rock mass. Joints in full sun-light have least favourable Jr/Ja combination. $Q_{range} \approx$ 100/9 x 1.5/(1-2) x 0.66/1 = 6-11.

Well-jointed shale close to Oslo tunnel portals. Note closely-spaced half-barrels. The shale was interbedded with nodular limestones of higher quality. $Q_{range} \approx$ (10-20)/9 x 1/(1-2) x 0.66/1 ≈ 0.4 – 1.5.

Sheared and clay-bearing hornfels next to granite batholith near Asker along E18 motorway. Examples of three Jr/Ja 'contact' categories. Q_{range} ≈(10-30)/(9-12)x(1-1.5)/(4-6)x(0.33-0.66)/5 ≈ 0.01-0.2.

Figure 4. Rock exposures selected from the Oslo area, mostly connected to tunnels built or planned. Note that in the case of clay-bearing rock, permeability (and water pressure) may be partitioned. High pressures can occur on just one side of a fault zone, until penetrated.

- Geological structures, such as fractures parallel to the mountain side, and sickle shaped exfoliation, are indications of high, anisotropic stresses. The limit for exfoliation in high mountain sides or spalling in a tunnel is dependent on the relation between induced stress and the compressive strength of the rock. In hard rock this limit normally occurs between 400 and 1100 m rock cover above the tunnel. This depends on the compressive strength of the intact rock and the gradient of the mountain side.

- Stress measurement within drill holes may be carried out before tunnel excavation in some of the larger tunnelling or hydropower projects, and this makes SRF estimation more reliable. Note that stress-induced fracturing referred to above starts when the ratio of the maximum estimated tangential stress compared to uniaxial strength (σ_θ/σ_c) exceeds about 0.4. This signifies the starting point for considerably increased SRF values. This experience is confirmed in mining and in deep road tunnels.

- Mapping for subsea tunnels is limited to the outcrops which are visible on both sides of a strait or a fjord under which the tunnel is planned. For subsea tunnels use of seismic techniques is even more important. Core drilling from the shoreline or islands are carried out. Deviated and steered drill holes up to 1000m long may be used for seabed to borehole seismic tomography. More seldom, because of the expense, core drilling from a ship may be carried out. This will be done for the 27km long planned Rogfast subsea (-390 m) road tunnel, which will be the world's longest road tunnel.

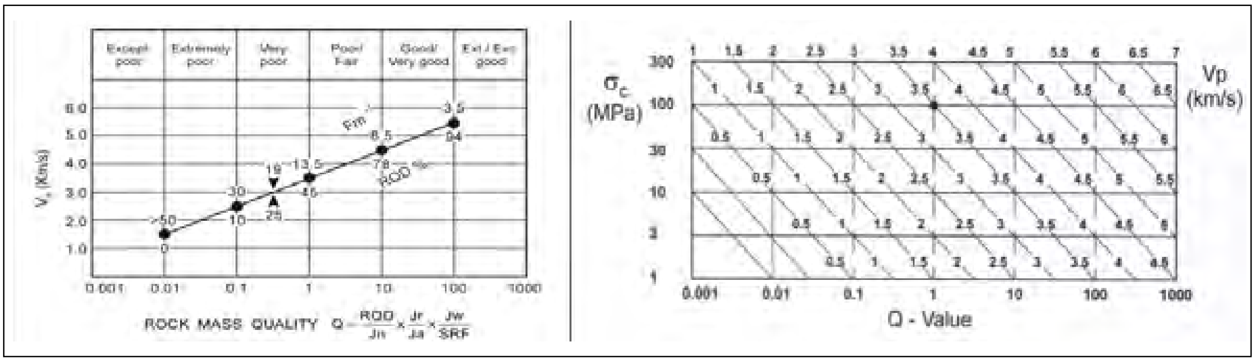


Figure 5 Left: Hard rock, shallow seismic refraction mean trends from Sjøgren et al. (1979). The Q-scale was added by Barton (1995), using the hard rock correlation Vp ≈ 3.5 + log Q. By remembering Q = 1: Vp ≈ 3.5 km/s, the Q-Vp approximation to a wide range of qualities is at one's fingertips (e.g. for hard, massive rock: Q = 100: Vp ≈ 5.5 km/s). Right: Generalization to include rock with different σc values.The results still apply to shallow seismic.The source of this figure is explained in Barton, 2006.

8 USING SEISMIC VELOCITY AND Q TO INTERPOLATE BETWEEN BOREHOLES

An empirically-based correlation between the Q-value and the P-wave velocity derived from shallow refraction seismic measurements was developed by Barton, 1995 from trial-and-error lasting several years (Figures 5 and 6). The velocities were based on a large body of experimental data from hard rock sites in Norway and Sweden, thanks to extensive documentation by Sjøgren et al., 1979, using seismic profiles (totaling 113 km) and local profile-oriented core logging results (totaling 2.85 km of core). The initial V_P-Q correlation had the following simple form, and was relevant for *hard rocks with low porosity*, and specifically applied to shallow refraction seismic, i.e. 20 to 30m depth, as suggested by Sjøgren.

$V_P \approx 3.5 + \log Q$ (units km/s) (2)

A more general form of the relation between the Q-value and P-wave velocity is obtained by normalizing the Q-value with the multiplier UCS/100 or $\sigma_c/100$, where the uniaxial compressive strength is expressed in MPa ($Q_c = Q \times \sigma_c/100$). The Q_c form has more general application, as weaker and weathered rock can be included, with a (-ve) correction for porosity.

$V_P \approx 3.5 + \log Q_c$ (units km/s) (3)

The derivation of the empirical equations for support pressure (originally in Barton et al., 1974) and for the static deformation modulus (in Barton, 1995, 2002) suggest an approximately inverse relationship between *support pressure needs* and rock mass *deformation moduli*. This surprising simplicity is not illogical. It specifically applies with the mid-range Jr = 2 joint roughness.

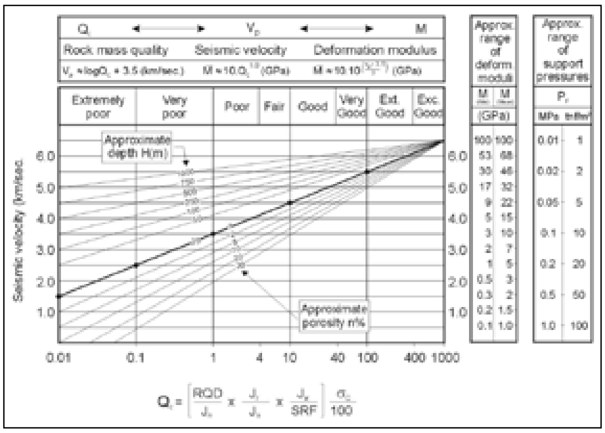


Figure 6. The thick 'central diagonal' line is the same as the sloping line given in Figure 5, and this applies to nominal 25-30m depth shallow seismic refraction results. In practice the nominal 1% (typical hard rock) porosity would be replaced by increased porosity if rock was deeply weathered, and the more steeply sloping lines (below the 'central diagonal') would then suggest the approximate (-ve) correction to VP. Note that very jointed rock with open joints may have lower velocity than saturated soil. The less inclined lines above the 'central diagonal' represent greater depth (50, 100, 250m etc), and these lines correct VP for documented stress or depth effects (+ve). These depth-lines were derived from several sets of deep cross-hole seismic tomography, with Q-logging of the respective cores (Barton, 2002). Note the inverse nature of (static) deformation moduli and support pressure shown in the right-hand columns. These derivations are described in Barton, 1995 and Barton, 2002. For a more detailed treatment of seismic, for example the effects of anisotropy which are accentuated when the rock is dry or above the water table, refer to the numerous references in the text book of Barton, 2006.

Rock mass classification has to be done close to the crown, using a hydraulically raised cage, mounted on the drill jumbo. It is easy to overlook altered rock and clay when too far below the arch in large tunnels.

Inspection of initial shotcrete (Sfr) support must also be carried out from high in the tunnel arch, using a crowbar to check for 'drumminess'. Poor areas due to insufficient jet-washing need to be repaired.

Three joint sets (Jn = 9), planar and rough-surfaced (Jr = 1,5). Sandstone in the Bremanger tunnel in Western Norway. Note that the location with over-break with three well-developed joint sets attracts Q-loggers attention, but may be more jointed than else-where. $Q_{range} \approx (30\text{-}80)/(6\text{-}9) \times 1.5/1 \times 1/1 \approx 5\text{-}20$..

Shallow cavern, with weathering or clay coatings on several of the joint sets. The granite has high RQD (90-100%) and large block sizes. This emphasizes need for additional Q-parameters to reflect the lower quality. Serious over-break due to critical ratio of $Jn/Jr \geq 6$. $Q_{range} \approx (90\text{-}100)/9 \times 1.5/(2\text{-}4) \times 0.66/2.5 \approx 1\text{-}2$.

Stress-induced fracturing in marble in the walls of the Jinping I headrace tunnels, where two large-diameter TBM were eventually removed, due to > 2 km cover. Completion by drill-and-blast: total of four // tunnels. $Q_{range} \approx (90\text{-}100)/(2\text{-}3) \times (2\text{-}4)/1 \times (0.5\text{-}1)/(50\text{-}200) \approx 1$.

Tunnel face in (pre-injected) shales. Note first layer of S(fr) and permanent (CT) bolts close to previous face. A conservative 3m advance. In fact the prior quality of the shales has been improved by 10 MPa pre-injection. $Q_{effective}$ has improved from ≈ 1 to 30.

Figure 7. Some figures to illustrate tunnel inspection needs following blasting and prior to final support and reinforcement decisions. Some diverse tunnel Q-logging examples are also given.

9 CHARACTERIZING THE ROCK MASS IN TUNNELS BY INSPECTING EACH TUNNEL ADVANCE

The final role of the Q-system is to document the rock mass quality of each advance of the tunnel, and thereby assist in the selection of the final support (Sfr) and reinforcement (B) class. Steel-fiber (or polypropylene) reinforced shotcrete and systematic, corrosion-protected rock bolts (B + Sfr) form the usual Norwegian single-shell tunnel (or cavern) 'lining'. There is infrequent use of rib-reinforced shotcrete (RRS), and occasional cast concrete (CCA) in short sections of bad rock. All of these measures are selected with the help of the Q-support chart (see next section). However, special conditions may demand special measures, so general Q-based methods can be modified where needed. This will be discussed in the next section.

In Norway, following occasional adverse experiences in the past, what has become known as 'the Owner's half-hour' is allotted to thorough rock mass inspection and characterization, generally before temporary or permanent support and reinforcement operations are commenced. This 'half hour' is reserved (and fully costed) so that the engineering geologist representing the contractor, and the engineering geologist representing the owner, can jointly try to come to agreement about the quality (or lack of quality) of the newly exposed rock mass. Having a standard method like the Q-system, and time for discussion, adds to the reliability, and both engineering geologists learn from each other. Shift work for the project engineering geologists is obviously needed when tunneling is progressing during two or more shifts each 24 hours.

The structural-geological (rock-type and joint-set recordings) and Q-logging is of course done following blast-gas displacement, and following scaling ('barring-down') by the contractor. The fact that wet-process shotcreting is used, as opposed to dry-process methods, plus the relatively small number of operatives and vehicles in Norwegian tunnels, means that air quality is generally superior to what is experienced in many other countries. This makes rock mass inspection easier and it is therefore more likely to be correct. Single-shell tunneling demands this reliability. Some examples of Q-logging in tunnels are illustrated in Figure 7.

Because tunnel cross-sections can be quite large, and because full-face blasting is common where rock quality allows this, the height of the tunnel arch usually means that rock mass inspection from a hydraulically-lifted cage is imperative. The rock mass quality (especially the lack of quality) is much more likely to be seen when close to the rock surface. Features such as clay-filled discontinuities are less likely to be missed. Geological hammers and a readily available scaling-bar to extend reach and avoid too-frequent moving of the cage, are obvious features of this inspection and decision-making. While some consulting companies may have performed numerical modelling of representative rock mass and tunnel support classes, now is the time to decide which support class and not wait for external decisions. This is important when 40 to 80m per week per face is the typical range of advance rates of single-shell NMT excavations.

There have been two deservedly much-publicised road tunnel rock-falls in the last 20 years in Norway, fortunately with no injuries or fatalities involved. Both have involved incorrect application of the Q-system, with an error in one case of assuming Q =70, while independent engineering geologists recorded Q=0.07 after the event, obviously with the benefit of hind-sight, including post-failure observation. In other words there was a 1000:1 error in the Q-estimate, due to failure to recognise a clay-infested section of a sub-sea tunnel, due to inadequate arch inspection and Q-logging routines.

10 EFFECT OF ORIENTATION OF GEOLOGICAL STRUCTURES ON Q-VALUE

Over the years many have commented on the *apparent* lack of discontinuity orientation in the derivation and application of the Q-value. Unlike the case in RMR, there is no specific term for an 'orientation rating'. Nevertheless there is the instruction to try to consider the least favourable joint set or discontinuity from the point of view of over-break potential or instability, when selecting the appropriate Jr/Ja ratio. This aspect of Q-logging sometimes requires significantly more experience than required when using RMR, because one needs to vizualise the consequences of continuing tunnel advance in case of not providing specific 'feature' support.

Figure 8a shows two cases which can be used for illustration. On the left is a small detail from one of Norway's numerous headrace tunnels. A graphite-coated minor fault strikes sub-parallel to the tunnel axis, while perpendicular to the axis is a set of chlorite coated joints. If considered individually the respective Jr/Ja ratios would be 1.5/3 and 2/4 respectively. The graphite-coated feature follows the tunnel axis for many meters, and is the chief cause of over-break and significant potential instability. Even if the Jr/Ja ratios had not been equal, the Jr/Ja = 1.5/3 combination applies in this case, while the more stable perpendicularly oriented Jr/Ja = 2/4 feature merely contributes directly to a lower RQD.

Figure 8 a and b Some specifically oriented details in a headrace tunnel in granite and in an old road tunnel along the west coast of Norway in massive schist.The discontinuity or joint set most adverse-for-stability gives the appropriate Jr/Ja ratio.

In the case of the apparently unsupported portal of the old coastal road tunnel shown in Figure 8b, the smooth sub-vertical joint set with strike sub-parallel to the tunnel axis should have supplied the appropriate Jr/Ja ratio of 1/2. With a local portal rock mass quality of 100/(9x2) x 1/2 x 1/2.5 ≈ 1 (note: 2 x Jn and SRF = 2.5 for portals) one would expect B of 1.7 m c/c and 7 cm of S(fr) (see next section), if the tunnel had come under today's Q-support decision-making. However, the tunnel has existed for a long time without support, so the Q-system is seen to be conservative, if correctly applied.

11 TUNNEL SUPPORT RECOMMENDATIONS BASED ON Q – SOME HISTORY

The Q-system was originally developed from more than 200 case records, which were mostly Scandinavian or of international origin. The single-shell support methods in the early seventies were B + S(mr), i.e. systematic bolting and mesh reinforced shotcrete. The table of support recommendations was based on the location of the case record in 'span-versus-Q' space, as illustrated in Figure 9a, from Barton et al., 1974. With the gradual addition of 1050 more case records by Grimstad, the support and reinforcement recommendations were simplified to the graphic method shown in Figure 9b, from Grimstad and Barton, 1993.

In the original Barton et al., 1974 version of Q, rock support and rock reinforcement recommendations were 'separated' by the conditional factors RQD/Jn (i.e. relative block size) and Jr/Ja (i.e. inter-block shear strength). In other words, smaller block sizes (and lower cohesive strength) apparently (and logically) required more S(mr), while lower internal friction apparently (and logically) required closer rock bolt spacing. Later it was discovered (Barton, 2002) that Q, or more specifically Q_c closely resembled the multiplication of 'c' and 'tan φ'. This 'semi-empirical' (*a posteriori*) derivation of the two strength components of a rock mass differs greatly from the *a priori* complex algebra of the Hoek-Brown GSI-based rock mass 'strength criterion' (Barton, 2011) which so many young people use with continuum finite element modelling of the assumed tunnel behaviour.

As discussed later, it is wise to combine empirical methods with numerical methods if one wishes to 'design' tunnel support based on numerical modelling. The predicted 'plastic' zones seen in numerical models, and the numerically modelled deformation need to be compared, and sometimes corrected, by empirical Q-deformation data. A very simple method will be illustrated later, which is important for verification, since continuum modelling may underestimate deformation, and exaggerated joint-modelling may over-estimate it.

12 THE COMPONENTS OF Q-SYSTEM BASED SUPPORT: S(FR), CT BOLTS, AND RRS

This section consists of illustrations of some key items of the Q-support recommendations shown in Figure 9: including yesterday's S(mr) and the last thirty years S(fr). Photographs and figures are used for reasons of brevity and stand-alone completeness. Today's essential support and reinforcement methods for single-shell tunnels are dealt with in detail by other authors of NFF-23. (Please refer to the table-of-contents).

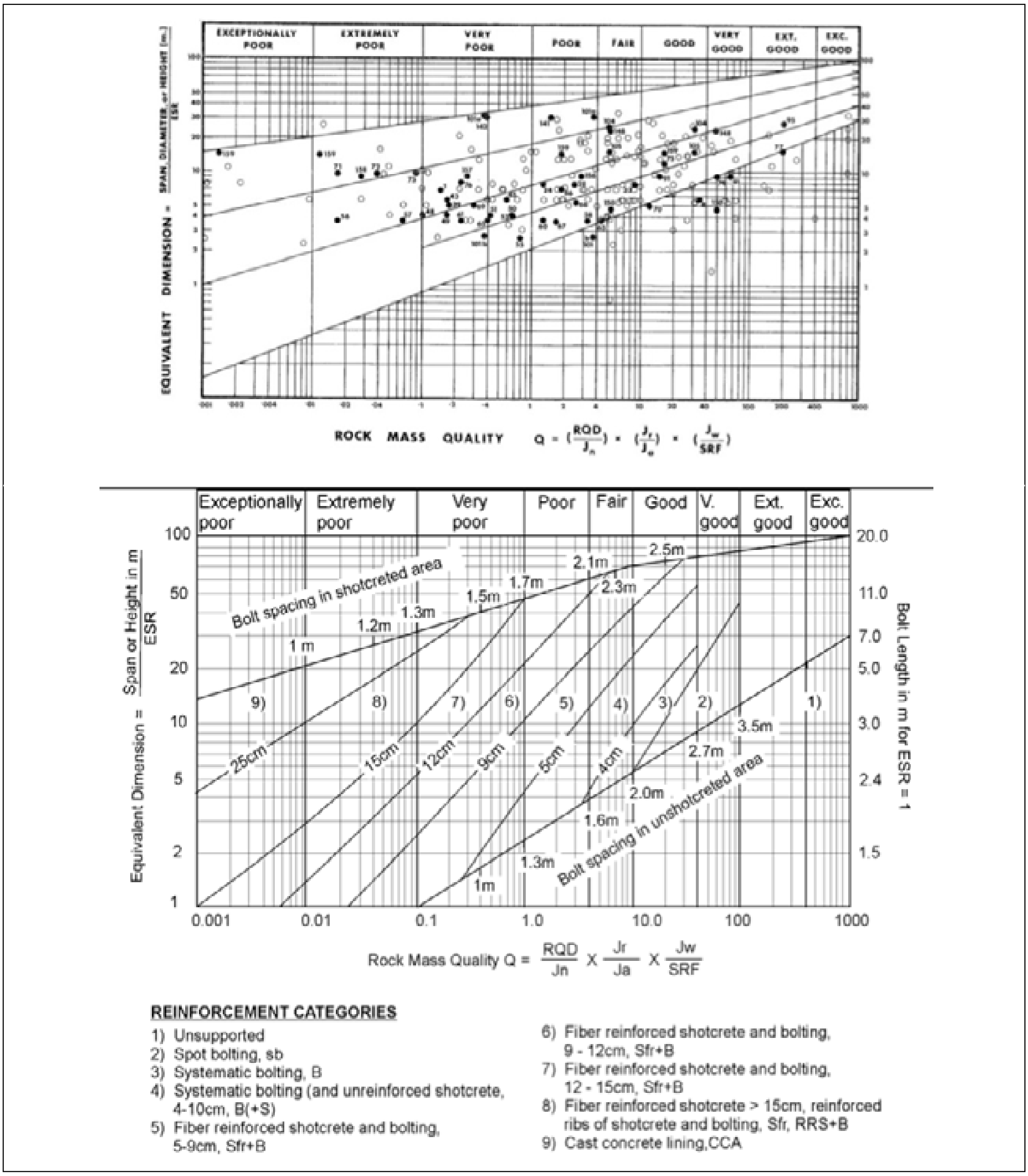
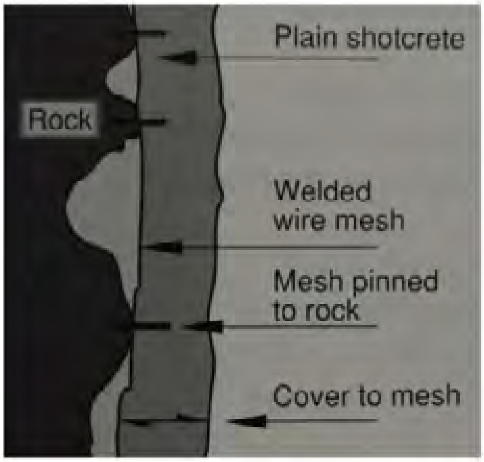
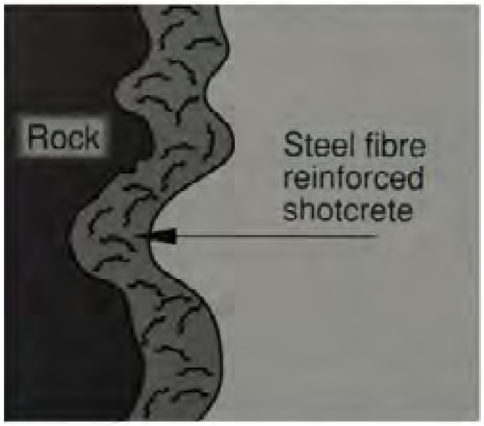


Figure 9 Top: The Barton et al., 1974 Q-based support chart, with each of the (38) boxes having a separate support and reinforcement recommendation. There were 212 case records, and at this time the standard single-shell method was B+S(mr) – i.e. bolting and steel mesh reinforced shotcrete. By about 1983 S(mr) had gone out of use as a tunnel support measure in Norway, after the development of robotically-applied wet process S(fr) in 1978/1979. Note that the SPAN (the width of the tunnel or cavern) is divided by a 'tunnel-use' safety requirement number ESR, shown later. Bottom: the Grimstad and Barton, 1993 tunnel support and reinforcement chart, which was based on some 1,050 new case records. This chart gave permanent single-shell support and reinforcement, also for large caverns. As will be noted later in Figure 13, some small adjustments to minimum shotcrete thickness were made in 2006 (described in Grimstad, 2007), based on experiences from 800 new cases assembled in 2002/2003.. In addition in Figure 13, the RRS (rib reinforced shotcrete) listed under category #8, is given specific dimensions for a wide range of tunnel sizes and Q-values.



Because the Q-system was developed in 1973, the single-shell case records had permanent shotcrete support and bolting reinforcement of lower quality than that available in the decades that followed. This is an example of poor-practice S(mr), with all the disadvantages well illustrated.

Vandevall, 1991 illustration of the pitfalls with mesh reinforced shotcrete: three processes, risk of 'shadow' and /or some rebound, corrosion of the mesh due to electrolytic currents, delayed installation.



Wet process steel-fiber reinforced shotcrete, applied after thorough washing, and use of corrosion-protected rock bolts (e.g. CT-type) are the most important components of the updated Q-system support/reinforcement recommendations.

Vandevall, 1991 illustration of the obvious advantages of S(fr): better bonding, no shadow, less corrosion, much lower permeability, faster, cheaper per meter.

Figure 10. The advantages of S(fr) compared to S(mr) are easily appreciated in these contrasting examples. The sketches from Vandevall, 1991 'Tunnelling the World' are not-exaggerated.

The reality of single-shell NMT-style tunneling, in comparison to double-shell NATM-style tunneling is that each component of support has to be *permanently relied upon*. There is nothing like the *neglect* of the contribution of temporary shotcrete, temporary rock bolts, and temporary steel sets, and reliance on a final concrete lining, as in NATM. Thus more care is taken in the choice and quality of the support and reinforcement components B+S(fr) + (eventual) RRS. Figure 10 (bottom) illustrates application of S(fr). Figure 11 illustrates (in the form of a shortened demo) the workings of the CT bolt. And finally Figure 12 illustrates some of the internal reinforcement details and final appearance of RRS (rib reinforced shotcrete).

It should be noted that the 1993 Q-support chart (shown earlier in Figure 9) suggested the use (at that time) of only 4-5 cm of *unreinforced* sprayed concrete in category 4. The application of *unreinforced* sprayed concrete came to an end during the 1990's, at least in

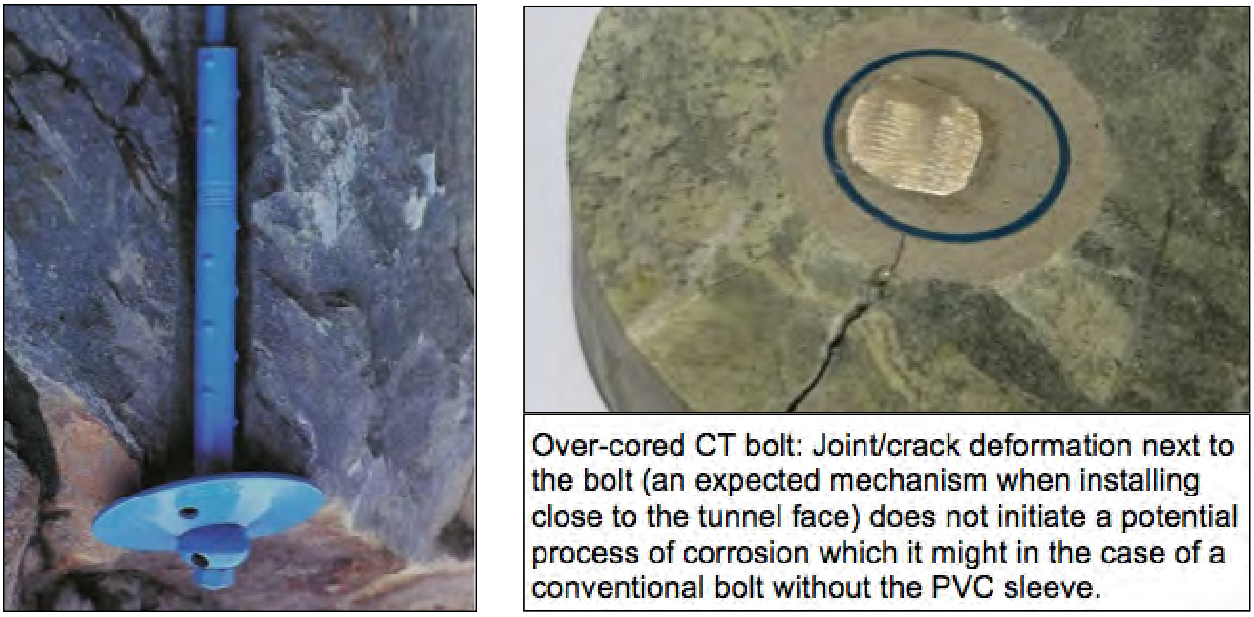


Figure 11. Because single-shell (NMT) relies on high quality S(fr) and long-life rock bolts, the multi-layer corrosion protection methods developed by Ørsta Stål in the mid-nineties, became an important part of NMT. The left photo shows a blue-coloured PVC sleeve: there are also black and white PVC.

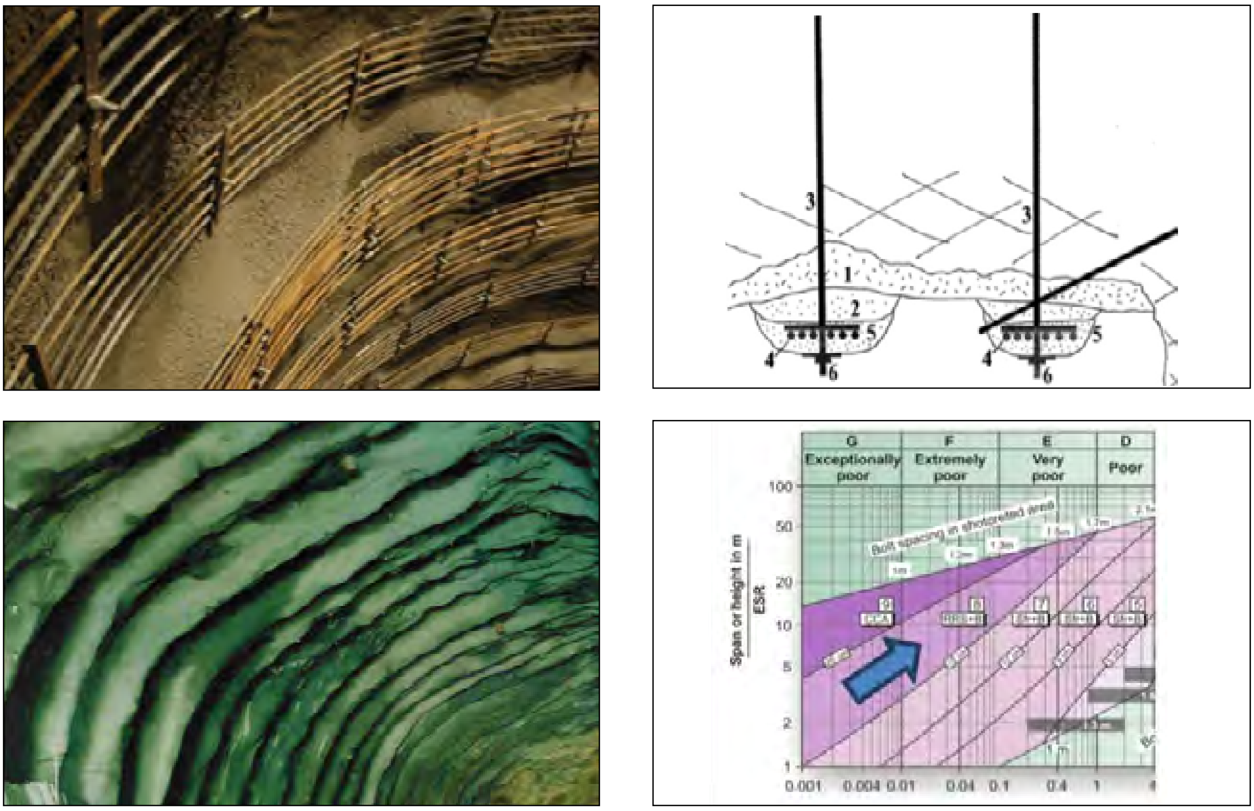
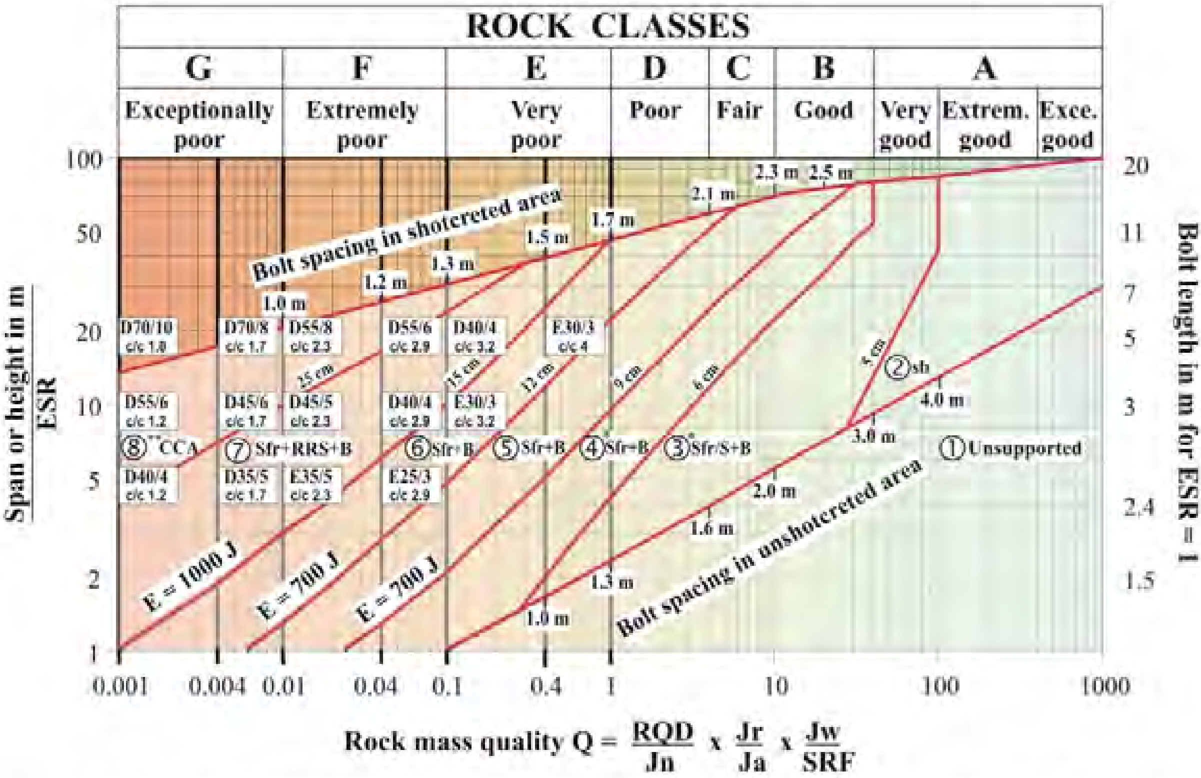


Figure 12. Some details which illustrate the principle of RRS, which is an important component of the Q-system recommendations for stabilizing very poor rock mass conditions. The top left photograph is from an LNS lecture published by NFF, the design sketch is from Barton 1996, the blue arrow shows in which part of the Q-chart the RRS special support-and-reinforcement measure is 'located' (see greater detail in Figure 13). The photograph of completed RRS is from one side of the National Theater station in downtown Oslo, prior to pillar removal beneath only 5m of rock cover and 15m of sand and clay. Final concrete lining followed the RRS for obvious architectural reasons.



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, and unreinforced or fibre reinforced shotcrete, 5-6 cm), **Sfr/B+S**
4) Fibre reinforced shotcrete and bolting, 6-9 cm, **Sfr+B**
5) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr (E700) +B**
6) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr (E700) +B**
7) Fibre reinforced shotcrete > 15 cm + reinforced ribs of shotcrete and bolting, **Sfr (E1000) +RRS+B**
8) Cast concrete lining, **CCA** or **Sfr (E1000) +RRS+B**

The bolts are 20 or 25 mm in diameter

E) Energy absorbtion in fibre reinforced shotcrete at 25 mm bending during plate testing

D45/6 c/c 1.7 = RRS with totally 6 reinforcement bars in double layer in 45 cm thick ribs with centre to centre (c/c) spacing 1.7 m. Each box corresponds to Q-values on the left hand side of the box

*) Up to 10 cm in large spans
) Or **Sfr+RRS+B

Figure 13. The updated Q-support chart first published by Grimstad, 2007.The details of RRS dimensioning given in the 'boxes' in the left-hand-side of the Q-support diagram were derived by a combination of empiricism and some specific numerical modelling by a small team at NGI. Details of this modelling are given by Grimstad et al., 2002, 2003. Note that each 'box' contains a letter 'D' (double) or a letter 'E' (single) concerning the number of layers of reinforcing bars. (Figure 12a shows both varieties). Following the 'D' or 'E' the 'boxes' show maximum (ridge) thickness in cm (range 30 to 70 cm), and the number of bars in each layer (3 up to 10). The second line in each 'box' shows the c/c spacing of each S(fr) rib (range 4m down to 1m). The 'boxes' are positioned in the Q-support diagram such that the left side corresponds to the relevant Q-value (range 0.4 down to 0.001). Note energy absorption classes E=1000 Joules (for highest tolerance of deformation),700 Joules, and 500 Joules in remainder (for when there is lower expected deformation).Note: S(fr) rib below steel bars, then S).

Norway. Furthermore, thickness down to 4 cm is not used any longer in Norway, due to the already appreciated risk of drying out too fast when it is curing. The Q-chart from 1993 (Figure 9) and also an updated 2002/2003 version, indicated a very narrow category 3 consisting of only bolts in a 10m wide tunnel when Q was as high as 10-20. This 'bolt-only' practice is not accepted any longer in Norway for the case of transport tunnels. The category 3 in 1993 and 2002/2003 has been taken away in this newest 2007 chart (Figure 13) which was fine-tuned by Grimstad at NGI in 2006. However for less important tunnels with ESR =1.6 and higher, only spot bolts are still valid. Hence we may distinguish between transport tunnels (road and rail) and head race tunnels, water supply etc.

13 NMT SINGLE-SHELL TUNNELING CONCEPT SUMMARIZED IN 1992

Shortly before the publication of the updated Q-system tunnel support recommendations by Grimstad and Barton, 1993, a multi-company, multi-author group from Norway (Barton, Grimstad, Aas, Opsahl, Bakken, Pedersen and Johansen) from the companies NGI (2), Selmer, Veidekke, Entreprenørservice, NoTeBy and Statkraft, described the main elements of the Norwegian way of tunnelling, calling it NMT, in deliberate competition to the much more expensive double-shell NATM. This two-part article in World Tunnelling (Barton et al., 1992) described Q-logging, numerical modelling, tunnel support selection, robotic application of wet process S(fr), support element properties, and the Norwegian tunnel contract system. The initials NMT are now well known after 20 years referencing and inclusion in university courses outside Norway. This is helpful for distinguishing it from the very different NATM.

Figure 14. The 'design-and-execute' tunneller's desk-of-drawers, used by Barton, 1996 to summarize key elements of NMT for an international readership. From top-left to top-right, proceeding downwards from drawer to drawer, the following is indicated, using connecting text.

14 CONCERNING BOLTING AND FIBRE TYPES IN THE Q-RECOMMENDATIONS

- The early Q-system nomenclature B_{utg} shown above refers to *untensioned grouted bolts*, which are very stiff. Their use has to be carefully considered when there is early large deformation, spalling or rock burst. Grouting of end-anchored rock bolts too early may increase the adverse effects of spalling, and bolts may also fail in tension in large numbers. It is better to grout the bolts when deformation has slowed down. A highly recommendable alternative is the use of energy/deformation absorbing D-bolts.

Rock mass characterization *using the six Q-parameters. A relationship between Q and V_P and deformation modulus M is indicated, using Q_c, σ_c, matrix porosity n% and depth H (m) or stress level.*	***Site investigation*** *using seismic refraction, radar, cross-hole V_P or E_{dyn} tomogram, or attenuation tomogram. (Note $Q_{seismic}$ =1/attenuation is numerically close to M GPa).*
Support design measures *consist of none, sb, B, B+S, B+S(fr), RRS, CCA. (Untensioned grouted B_{utg} bolts, tensioned resin end-anchored bolts, and CT bolts). Also spiling, drainage, pre-injection, and freezing.*	***Numerical verification of support designs*** *using codes like UDEC, UDEC-BB, UDEC-Sfr, FLAC, FLAC-3D and 3DEC. Relevant parameters JRC, JCS, φ_r, M, kn and ks, c + φ.*
S(fr) robot technology *using Portland cement, silica fume, plasticizer, super-plasticiser, aggregate and non-alkali (low) accelerator. Steel fiber: EE 20-25 mm (previously), Bekaert 30-35mm / 0.5mm. (Today: also PP fiber Barchip 48mm, 0.4 /1.4mm).*	***Norwegian tunnel contract system*** *uses a flexible contract, with unit prices for all possible measures in tender documents: use the motto 'expect the unexpected'.*
Rapid advance *due to wet process S(fr) shotcrete, gives low rebound and improved environment.*	***Low cost and*** *less conflicts, permanent single-shell support compared to double-shell NATM.*

- EE-fibers went out of use in Norway in the mid 1990's. Bekaert steel fibers 30-35mm long, among others, are partially being substituted by polypropylene fibers, such as Barchip Kyodo 48mm long, in some sections of the tunneling industry, but it is important that these fibres are rough-surfaced to ensure their deformation resistance.The decades-long behaviour of polypropylene fibres is not yet possible to document, but extensive use in parts of the tunneling industry, such as in subsea road and rail tunnels and when large deformation is expected, is a positive signal.

15 CONTRASTING SINGLE-SHELL NMT AND DOUBLE-SHELL NATM

The use of steel sets is avoided in the practice of single-shell NMT, due to the potential loosening of insufficiently supported rock in the periphery of the excavation. It is difficult to 'make contact' between the steel sets and the rock, especially when there is over-break. The results of experiments using different support methods are illustrated in Figure 15. The left-hand diagram shows the results of trial tunnel sections in mudstone (Ward et al., 1983). The five years of monitoring clearly demonstrate the widely different performance of the four different support and reinforcement measures.

In the right-hand diagram, from Barton and Grimstad, 1994 the contrasting stiffness of B+S(fr) and steel sets is illustrated in a 'confinement-convergence' diagram, with the implication that SRF (loosening variety: see APPENDIX A1 and A2) may occur when using steel sets. It should be clear that the early application of S(fr) by shotcrete robot, and the installation of permanent corrosion protected rock bolts from the start, as in single-shell NMT, is likely to give a quite different result from that achieved when using NATM.

In the latter, the commonly used steel sets and mesh-reinforced shotcrete and rock bolts are all considered just as temporary support, and are not 'taken credit for' in the design of the final concrete lining. These temporary support measures are assumed to eventually corrode. It is then perhaps not surprising that convergence monitoring is such an important part of NATM, as a degree of loosening seems to be likely when so often using steel sets. The standard procedures involved in NATM are illustrated in Austrian standards, and are reproduced for reference (Figure 18) since so remarkably different to single-shell NMT.

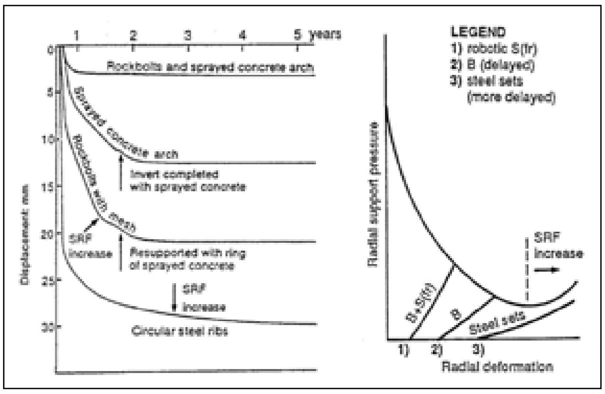


Figure 15 Left: Results of five years of monitoring test-tunnel sections in mudstone, using four different support and reinforcement measures, from Ward et al., 1983. The obvious superiority of B+S in relation to steel sets is clear. The last 35 years of B+S(fr), as practiced in Norway would presumably give an even better result. Right: Representation of the relative stiffness of different support measures, from Barton and Grimstad, 1994. SRF may increase due to loosening in the case of steel sets. See Figure 16, which illustrates the implicit difficulty of controlling deformation with steel sets/lattice girders.

Figure 16 Left: An illustration of the challenge of making contact between the excavation periphery and the steel sets, even for the case of limited over-break. In NATM the 'sprayed-in' steel sets, and S(mr) and bolting are considered temporary, and are not included in the design of the final concrete lining. Right: Steel sets are actually a very deformable type of tunnel support. However in squeezing rock as illustrated, the application of RRS might also be a challenge, unless self-boring rock bolts were used to bolt the RRS ribs in the incompetent (over-stressed) rock that is likely to surround the tunnel in such cases.

Figure 17 Left: Illustration of a 'missing component' of support. When the rock mass is significantly jointed and with low cohesion, bolting and mesh alone are clearly inadequate. Right: An illustration of the use of temporary steel sets and mesh reinforced shotcrete, both of which potentially invite increased deformation and possible loss of strength. A local collapse and large deformation is shown.

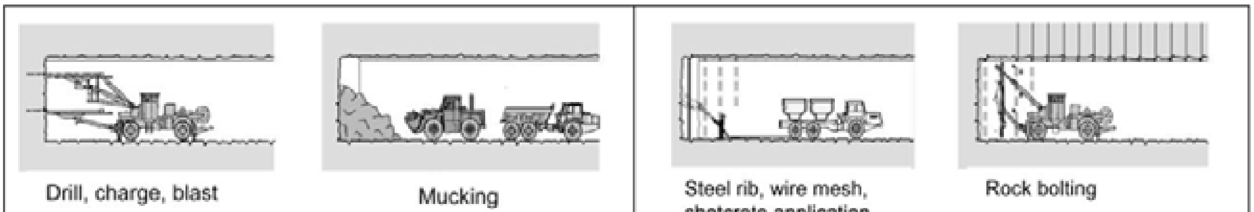


Figure 18. Schematic construction sequence of a typical NATM tunnel, apparently used in both softer and harder rock, from "Austrian Society for Geomechanics, 2010. NATM, 'The Austrian Practice of Conventional Tunnelling'. This method has been observed in many countries when Q is 'poor', 'fair', 'good' i.e. Q = 1 to 40, where NMT would be eminently suitable and much faster.

In contrast to the sequences of NATM shown in Figure 18, in NMT the excavation is usually full-face, both for speed and to avoid a very unfavourable top-heading section (as illustrated in Figure 18) which invites the initiation of invert heave if the tunnel is at significant depth and not in hard rock. Final support in NMT is usually B+S(fr), while in double-shell NATM the final support is only the final concrete lining, which also secures the drainage fleece and membrane. The concrete lining is designed to take all ultimate loads from the rock mass. The temporary steel sets, S(mr), and bolting are assumed in the long-term to have corroded and are not featured in the final concrete load-bearing structure. This design philosophy, which is surprising to many, adds to the time and cost of NATM.

Inevitably the cost difference between NATM and NMT is of the order of 1:3 to 1: 5, but this depends on rock mass quality, and hence on the type and amount of rock support. The cost difference also depends on differences in labour cost in different countries. There may be a 1:10 difference in the number of tunnel workers involved, and the speed of NATM, including the final liner, is inevitably much slower than single-shell NMT, due to all the operations, as may be visualized in Figure 18. Those using NATM will often point to poorer rock conditions where NATM tends to be applied. This can be only partly acknowledged, since double-shell NATM procedures are also specified in rock masses of comparable quality to those where NMT would be most applicable. This has been observed in many countries.

Norwegian road and railway tunnels of high standard, with all the technology installed for ventilation, lighting, drainage, safety and communication, cost about 18,000 to 27.000 US $ per meter (road) and about 25,000 to 33,000 US$ per meter (railway), depending on the dimension of the tunnel. International tunneling literature frequently documents 80,000 to 100,000 US per meter for the case of NATM double-shell tunnels.

Concerning tunnelling speed, recent Norwegian world records of 164 m and 173 m in best weeks by two different Norwegian contractors, and a 104 m/week project average for 5.8 km in coal-measure rocks, obviously requiring significant rock support and reinforcement, suggests that NMT is a more efficient process. Figure 19 shows the source of very fast NMT single-shell tunneling; namely the fast cycle time. This is usually below 10 hours for a wide range of Q-values (i.e. 1 to 100), and is as low as 5 to 6 hours at the top end of the rock mass quality scale where immediate support and reinforcement is obviously not needed.

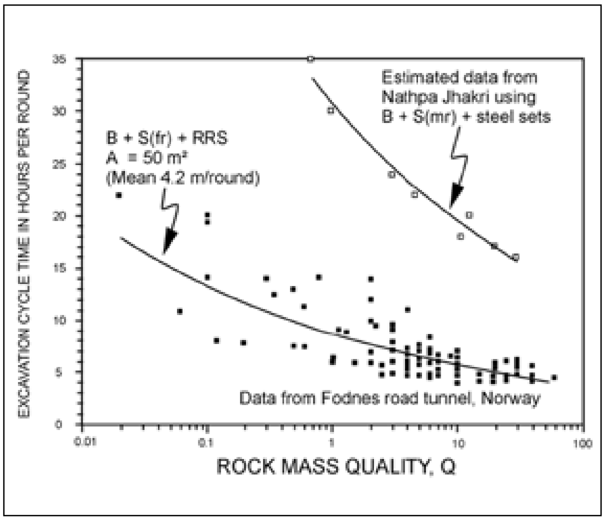
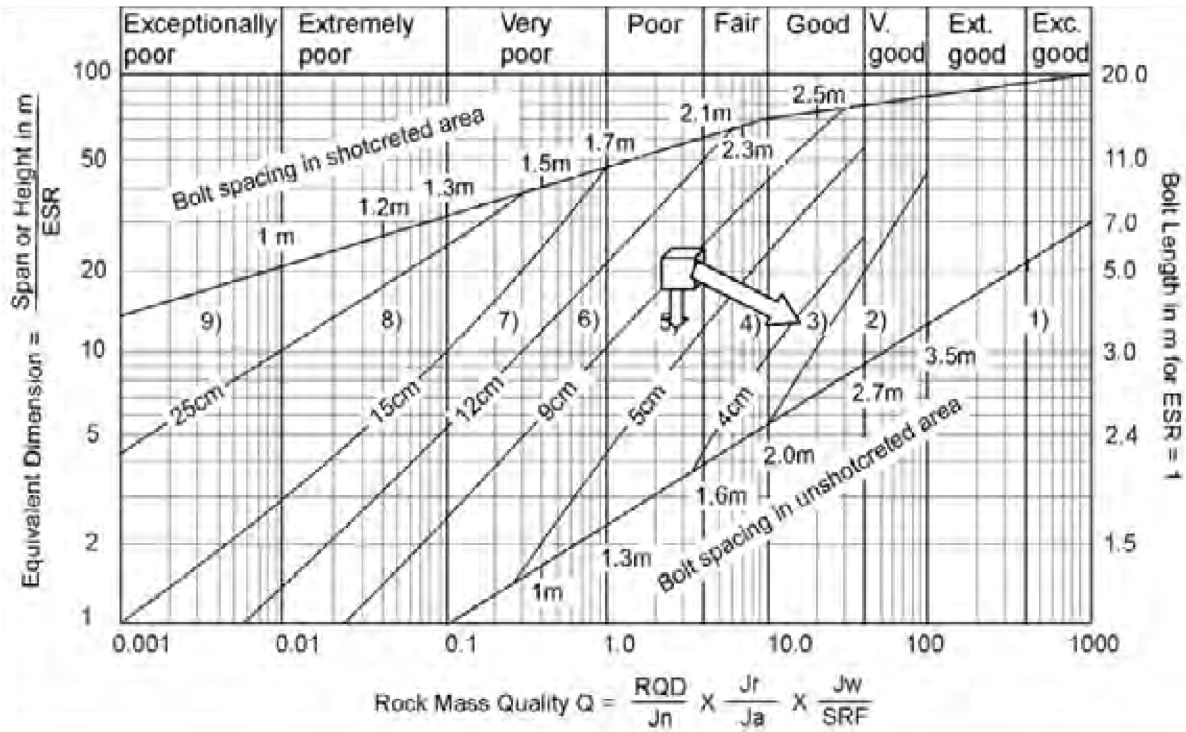


Figure 19. Cycle-time (drilling blast holes, loading with explosives, blasting, waiting for gasses to clear, scaling, geological inspection, mucking, reinforcement and support) as observed by Grimstad in the Fodnes road tunnel, which has a cross-section of 50-55 m2. For comparison, the cycle-time for labour intensive temporary support methods in a hydroelectric project in India is also shown in relation to the logged Q-values.

16 USING THE Q-SYSTEM FOR TEMPORARY SUPPORT SELECTION

When the Q-system was first published in 1974, it was designed to provide guidance on suitable permanent support for a variety of tunnel and cavern sizes. Almost by way of a footnote, it was suggested that the Q-system could also be used for guiding *temporary support* selection. The suggested rule-of-thumb was '5Q and 1.5 ESR. This means a diagonal shift, downwards and to the right, on a Q-support chart, as illustrated by one example at the top of Figure 20. This method has been used systematically by Hong Kong road, rail and metro authorities for at least 20 years, as the preliminary stage of NATM-style tunneling and station cavern development. The table (left-side of Figure 20) shows the ESR values recommended in 1993 for various types of excavation. With the world-wide demand for increased safety in the last two decades, the recommended updated areas of the ESR table are shown on the right, with an explanation below the tables. The final two diagrams of Figure 20 illustrate the workings of ESR in relation to the Q-value and the SPAN.



Type of Excavation		ESR
A	Temporary mine openings, *etc.*	*ca* 2-5
B	Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers	1.6-2.0
C	Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels	1.2-1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	0.9-1.1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels	0.5-0.8

ESR recommended
ca. 2 to 5
1.6 to 2.0
0.9 to 1.1 Storage caverns 1.2-1.3
Major road and rail tunnels 0.5 to 0.8
0.5 to 0.8

The ESR table published in 1993 was correct in the 1970's and in the 1980's. However the demand for safety has increased world-wide and also in Norway, particularly in the case of transport tunnels where small rock falls were accepted in minor road tunnels in the1970's. Now there is no tolerance for any rock falls, even in minor transport tunnels. Minor road and railway tunnels should now have ESR = 1. Water treatment plants with a lot of expensive installment and representing a daily working place should have ESR = 0.9-1.1, and are increasingly more important than storage caverns. Major road and railway tunnels may need ESR = 0.5 - 0.8. These suggested updates are tabulated on the right-hand side of 1993 values.

Figure 20 The workings of ESR, the tunnel use-and-safety number, for modifying SPAN to equivalent span. Note the rule-of-thumb '5Q + 1.5 ESR' for estimating temporary support needs for NATM, as used in road, rail and metro tunnels in Hong Kong for the last 25 years. B + S(fr) is used as temporary support, supplemented by steel sets when Q is 'very poor'. The way ESR modifies the equivalent span is shown by the sloping lines, assuming ESR = 1.6 marks the unsupported boundary (for hydro-power). Note that 'unsupported span' in the Q-system refers to the width of excavation. In Bieniawski, 1989 RMR, the 'unsupported span' is the longitudinal distance from the face to the nearest support or reinforcement. These two 'spans' are sometimes confused by international consulting companies.

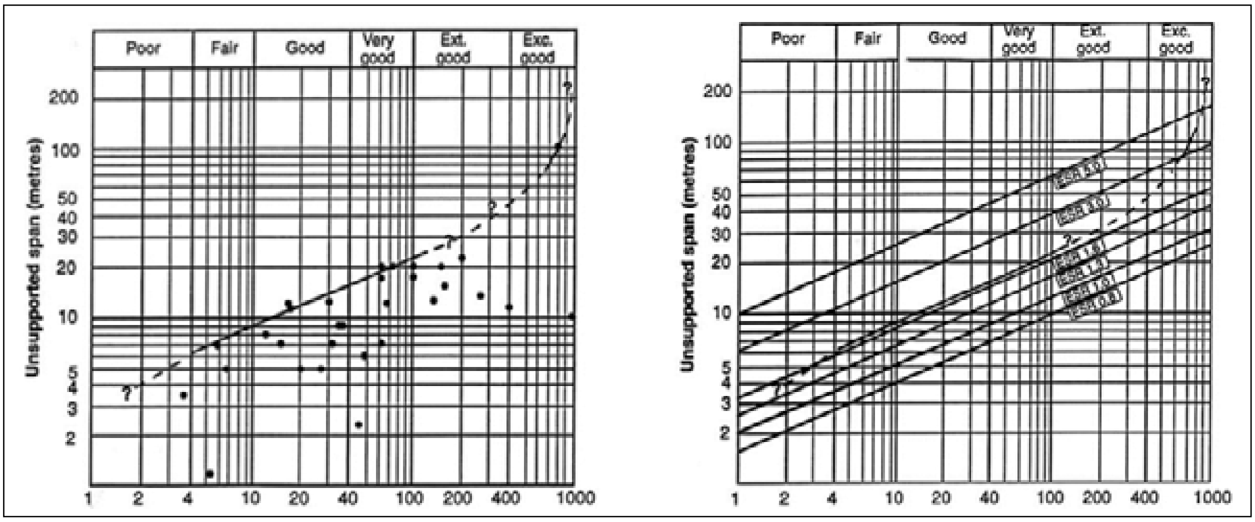


Figure 20.

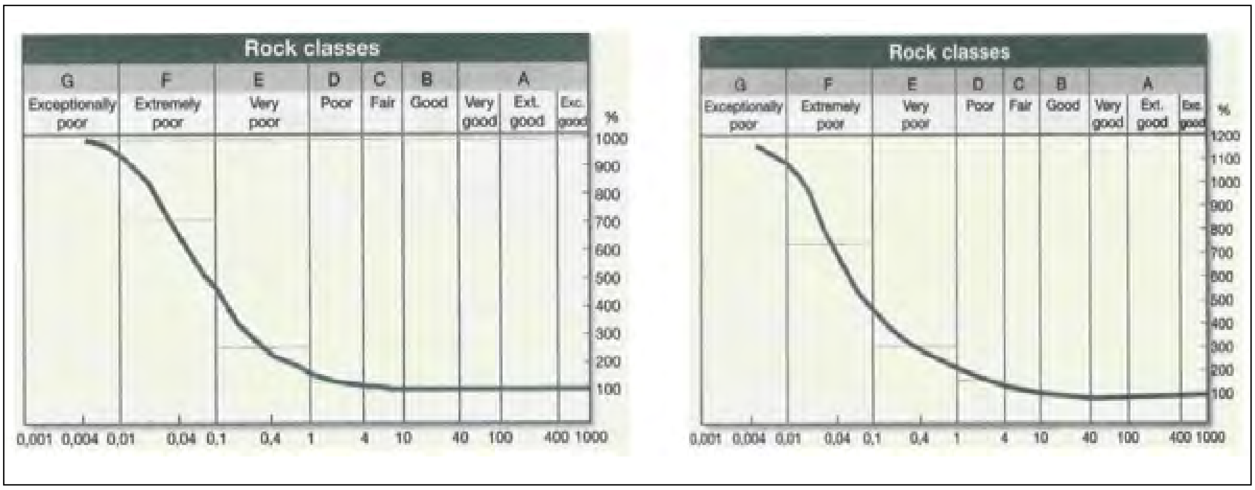


Figure 21 Top: Relative time (top-left) and cost (top-right) of tunnel construction in relation to Q-value, according to a 50 km survey of tunnels carried out by Roald, and published in Tunnels and Tunnelling as Barton et al., 2001.

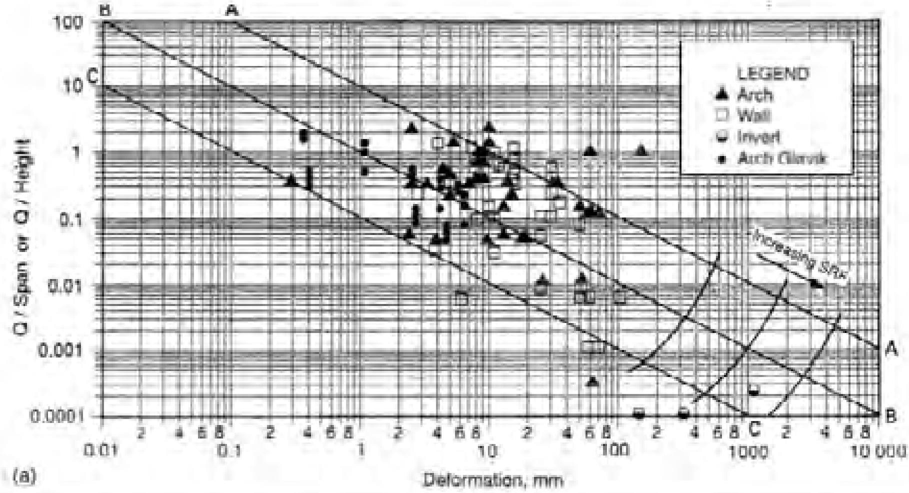
17 RELATIVE TIME AND COST IN RELATION TO THE Q-VALUE

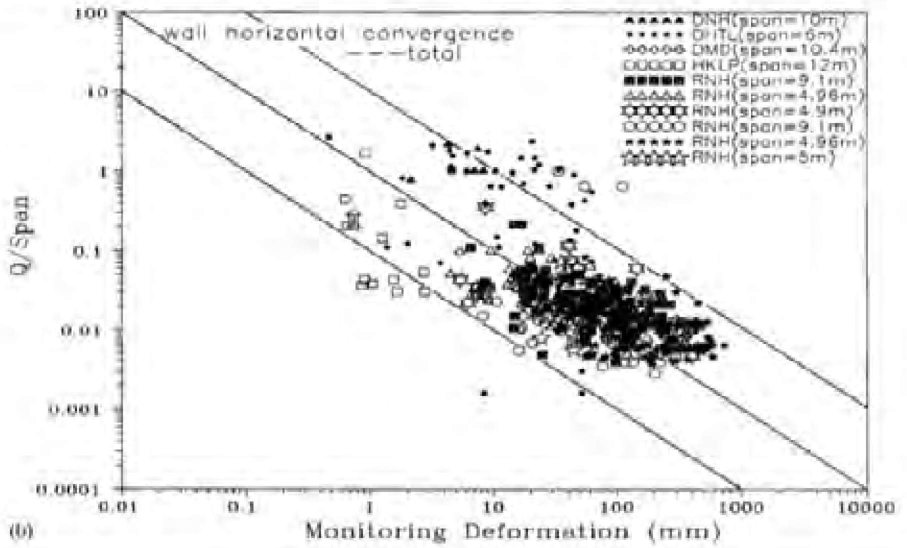
As a result of a survey of some 50 km of tunneling mostly in Norway but also in Sweden, Roald produced the two figures of relative time and cost of tunneling in relation to the Q-value shown in Figure 21. These important trends were subsequently published in Barton, Roald and Buen, 2001, in which the main topic was rock mass improvement by pre-injection. In fact this was the first exploration of possible improvements in some of the Q-parameters as a result of high pressure pre-injection. A brief discussion of this topic is given near the end of this chapter on the Q-system.

Figure 21 demonstrates the strong influence of the Q-value on tunneling time and cost. This is independently confirmed by cycle-time changes with Q, as recorded by Grimstad in a Western Norway road tunnel. This was shown in Figure 19. The general trends shown in Figure 21 concerning relative cost can also be independently derived by a rigid application of the Q-system recommendations for arch and wall support over the whole range of Q-values, and for a wide range of tunnel spans. With knowledge of Q-values, costs can be derived.

18 ESTIMATING TUNNEL OR CAVERN DEFORMATION IN RELATION TO Q

It appears that the large numerical range of Q (0.001 to 1000 approx.) referred to in the introduction, helps to allow very simple formulæ for relating the Q-value to parameters of interest to rock engineering performance. Refer to data and equations shown in Figure 22.





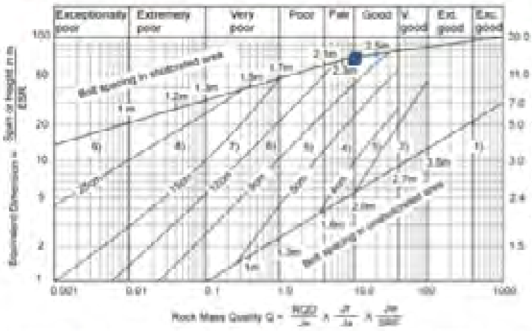
$\Delta = \dfrac{SPAN}{Q}$	$\Delta_v = \dfrac{SPAN}{100Q}\sqrt{\dfrac{\sigma_v}{\sigma_c}}$	$\Delta_h = \dfrac{HEIGHT}{100Q}\sqrt{\dfrac{\sigma_h}{\sigma_c}}$	$k_o = \left(\dfrac{SPAN}{HEIGHT}\right)^2\left(\dfrac{\Delta_h}{\Delta_v}\right)^2$

Nathpa Jakri power station cavern	**Gjøvik Olympic cavern**
$\Delta_v = \dfrac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28$ mm $\Delta_h = \dfrac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56$ mm (SPAN = 20m, HEIGHT = 50m, Q = 3, σ_v=4 MPa, σ_h = 6 MPa, σ_c = 35 MPa). (In the middle of the range of MPBX measurements for the arch and walls)	$\Delta_v = \dfrac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9$ mm (SPAN = 60m, Q_{mean} = 10, σ_v = 1 MPa at 40 m depth, σ_c = 75 MPa) (Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).

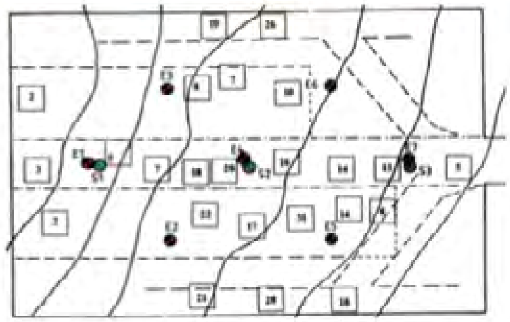
Figure 22 Top: The log-log plotting of Q/span versus deformation was published in Barton et al., 1994, with fresh data from the MPBX instrumentation of the top-heading and full 60 m span of the Gjøvik Olympic cavern. Shen and Guo (priv. comm.) later provided similar data for numerous tunnels from Taiwan. When investigated, the central trend of data was simply Δ(mm) ≈ SPAN(m)/Q. An empirical improvement is shown, by employing the 'competence factor' format of SRF (i.e. stress/strength).

World's largest top-heading waiting for benching-down.

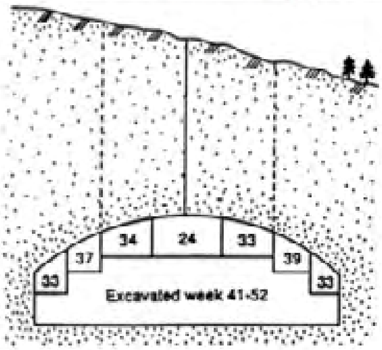
Mean 10 cm of S(fr). Note some over-break > 1 m.



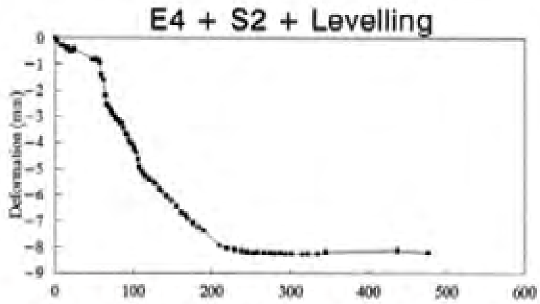
Q_{mean} ≈ 10, span 62m. B 2.5m c/c + S(fr)10cm + cables.



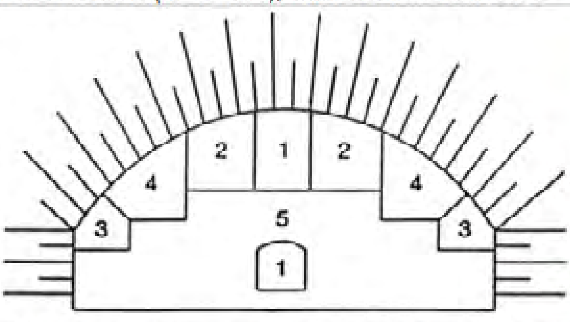
Q-values of arch (boxes): long external MPBX: in red



MPBX locations (three rows), excavation week in 1991.



Central deformation 8 mm, ends: 7.0 mm and 7.5 mm



B 2.5m c/c L= 6m, + twin-strand anchors L=12m, c/c 5m

Early games: ice hockey in Winter Olympics of 1994.

Figure 23. Some details of the Gjøvik Olympic cavern. Concept from Jan Rygh, design studies by Fortifikasjon and NoTeBy, design check modelling, external MPBX, seismic tomography, stress measurements and Q-logging by NGI, internal MPBX, bolt and cable loads, modelling, research aspects by SINTEF-NTNU. However, most important of all: construction in 6 months using double-access tunnels, by the Veidekke-Selmer JV. The cavern is an example of a drained NMT excavation.

19 GJØVIK CAVERN Q-LOGGING, NMT SINGLE-SHELL B+S(FR) SUPPORT, AND DEFORMATION

The Gjøvik Olympic cavern was a milestone event in Norwegian rock engineering and rock mechanics practice, combining as it did the experience of several of Norway's leading consulting, research institutes and contracting companies. The Q-system was well utilized.

The efficient cavern excavation and execution of single-shell NMT-style permanent support, which took just 6 months in 1991, saw the removal of 140,000 m^3 of red and grey gneiss. Mean RQD was only 60%, and the mean UCS was 90 MPa. The 62 x 24 x 90 m raw-cavern dimensions represented a large (almost 100%) jump in the world's largest-span cavern for public use, with capacity for about 5,400 people for artistic events, concerts etc., which both preceded the winter Olympic ice hockey events of 1994 (i.e. the grand opening ceremony), and have frequently followed in the years since then.

Four boreholes were used for site investigation, two of them inclined. These holes were used for Q-logging, seismic tomography (V_p range was 3.5 to 5.5 km/s, with the high values due to the 3 to 5 MPa horizontal stress). UDEC-BB modelling showed 7 mm of vertical deformation by the time that three adjacent Postal Services caverns had been completed.

In the context of engineering geology, rock mechanics modelling and rock engineering, the following are among the several technical reports describing the various aspects of the project: Løset and Bhasin (1992), Kristiansen and Hansen (1993), Morseth and Løset (1993) and Barton et al., (1994).

20 FURTHER Q-SYSTEM APPLICATIONS – PRE-GROUTING CAN CHANGE EFFECTIVE Q-VALUES

Barton, Roald and Buen, 2001 and Barton, 2002 suggested, controversially as with most innovations, that several - perhaps most - of the Q-parameters could, *in effect* be improved by the typical high pressure 5 to 10 MPa pre-injection of micro-and ultrafine cements-with-microsilica, as regularly practiced in Norway. This suggestion seems to have been proved correct over time, and others on the international mining scene are also reporting such finds.

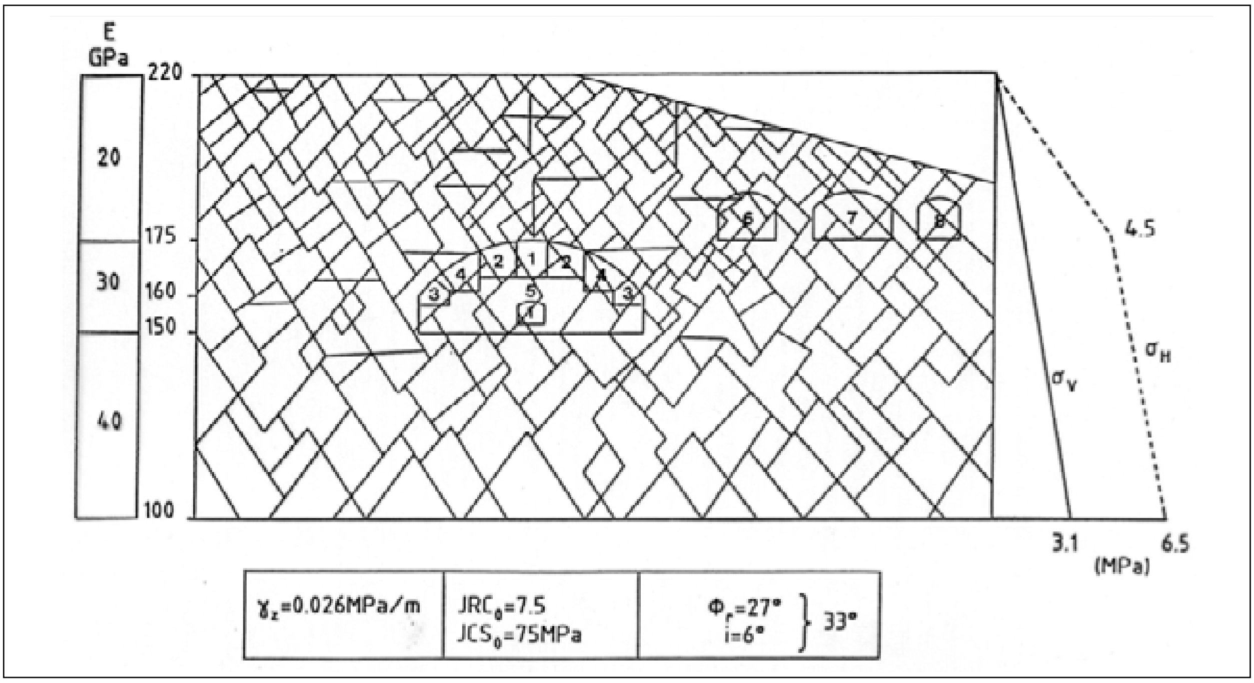


Figure 24. Representation of the conjugate jointing, the favourably high boundary stresses, the depth-dependent deformation modulus and the eight principal excavation stages. The modelled vertical deformations above the Gjøvik cavern main arch using the jointed code UDEC-BB were approximately 4 and 5 mm depending on modelled depth of 30 or 50m (relevant to each end of the cavern). The third model which is shown here had unchanged input data, but included the three Postal Service caverns (excavation stages # 6, 7 and 8). These caused the central arch vertical deformation to increase to 7 mm. The MPBX-measured results (Figure 23) incorporating internal (SINTEF) and external (NGI) extensometers, plus the results of surface levelling (downwards) were 7 to 8 mm. Barton et al., 1994.

The first author systematically Q-logged all the drill-core and analysed all the permeability measurements for the Jong-Asker and Bærum rail tunnels for JBV (Jernbaneverket). Subsequent experiences suggest that the inflow requirements for Jong-Asker were not stringent enough: care was taken of the external natural (and built-on) surface environment, but some of the pre-grouting was not sufficient for the inside-the-tunnel environment. Dripping water remained in places, when the least stringent 8 to16 litres/min/100m inflow criteria were used. However, in the case of the later Bærum rail tunnel of 5 km length, which was systematically injected at consistently high pressure, a very dry result was obtained. Several inspections of the pre-injected new rounds of tunnel advance suggested that the single-shell NMT final support, as seen in Figure 25, was on the conservative side.

In relation to the extensive (kilometers) of Q-logged core, logged to depths greater than the tunnel depths, there appeared to have been an improvement in the rock mass quality due to the pre-grouting. Not only was the shotcrete 99.999% dry, but the B+S(fr) which was applied, based on prior Q-based designs, seemed to be conservative. This can

CONSERVATIVE PRE-INJECTION MODEL	*MORE REALISTIC* PRE-INJECTION MODEL
RQD increases e.g. 30 to 50% Jn reduces e.g. 9 to 6 Jr increases e.g. 1 to 2 (due to sealing of most of set #1) Ja reduces e.g. 2 to 1 (due to sealing of most of set #1) Jw increases e.g. 0.5 to 1 SRF unchanged e.g.1.0 to 1.0	RQD increases e.g. 30 to 70% Jn reduces e.g. 12 to 4 Jr increases e.g. 1.5 to 2 (due to sealing of most of set #1) Ja reduces e.g. 4 to 1 (due to sealing of most of set #1) Jw increases e.g. 0.66 to 1 SRF improves e.g. 2.5 to 1.0
WET CONDITIONS *Before pre-grouting* *Q = 30/9 x 1/2 x 0.5/1 = 0.8*	WET CONDITIONS *Before pre-grouting* Q = 30/12 x 1.5/4 x 0.66/2.5 = 0.2
Vp ≈ 3.4 km/s *E_{mass} ≈ 9.3 GPa* *K ≈ 1.3 x 10^{-7} m/s* e.g. for a 10 m tunnel: B 1.6 m c/c, S(fr) 10 cm	*Vp ≈ 2.8 km/s* *E_{mass} ≈ 5.8 GPa* *K ≈ 5.0 x 10^{-7} m/s* e.g. for a 10 m tunnel: B 1.4 m c/c, S(fr) 13 cm
DRY CONDITIONS *After pre-grouting* Q = 50/6 x 2/1 x 1/1 = 17	DRY CONDITIONS *After pre-grouting* Q = 70/4 x 2/1 x 1/1 = 35
Vp ≈ 4.7 km/s *E_{mass} ≈ 25.7 GPa* *K ≈ 5.9 x 10^{-9} m/s* e.g. for a 10 m tunnel: B 2.4 m c/c	*Vp ≈ 5.0 km/s* *E_{mass} ≈ 32.7 GPa* *K ≈ 2.9 x 10^{-9} m/s* e.g. for a 10 m tunnel: sb (spot bolts)

Figure 25. Photographs from two (pre-injected) faces of the Bærum rail tunnels, showing top-left: the first 5 cm layer of S(fr) and the permanent CT bolts-and-washers at approximately 1.5 m c/c. Top-right: bolt heads and washers sprayed in with the final 5 cm layer of S(fr). The tunnel now has its completed single-shell NMT support and reinforcement, which appears to be conservative. The quality of the shale/limestone appears to have been improved by the high pressure pre-injection. Bottom: two hypothetical but not unrealistic 'models' for Q-parameter improvement as a result of pre-injection.

Ball-park estimation of permeability, when clay is absent. There are theoretical arguments linking Lugeon and Q, because of deformability effects.

Q_c	0.1	1	10	100
≈ Lugeon	10	1	0.1	0.01
≈ K m/s	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_p km/s	2.5	3.5	4.5	5.5

Figure 26. A well pre-injected tunnel face ('The End, Game Over' for the contractor) in the Bærum rail tunnel, which also demonstrates a 99.999% dry shotcrete, and a suspicion (based on the table on the left) that Qc (= Q x σc) for the shale might have increased from e.g. 1 to 100, with corresponding property improvements as a result: for example 0.01 Lugeon is likely, and maybe also VP = 5.5 km/s. Increases of VP of 1.5 to 2 km/s are regularly achieved even with low pressure curtain grouting at dam sites. Monitored building foundations which are cement injected demonstrate improved modulus and velocity. (Barton, 2006).

be concluded just by inspecting the photos below. Beneath these same Bærum tunnel photographs, two hypothetical models for 'before-and-after' Q parameter improvements are given to illustrate the possibilities. (The pre-grouting models have no relation to the two Bærum tunnel photos.)

Reduced tunnel support needs, reduced deformation, increased deformation modulus and increased seismic velocity as a result of pre-grouting (the latter documented in Barton, 2006) are each being suggested. Naturally one expects as a very minimum that a Jw of 0.5 or 0.66 will become 1.0 ('dry') as a result of successful pre-injection. Other parameters seem also to benefit, including the *effective* RQD, and the *effective* Jn, and perhaps even transfer of lower Jr/Ja ratios to the remaining *uninjected* (tightest?) joint sets, resulting in higher *effective* Jr/Ja ratios.

In Barton, 2002 there is documentation of the measured rotation of permeability tensors as a result of injection at a dam site. This has been measured using 3D (multi-borehole) hydro-tomography by Quadros and Correa Filho, 1995. So we know that joint sets can become sealed even at low pressures with non-ideal Portland cement and bentonite. With today's optimized grouting materials and the last 20 years of high pressure pre-injection, as often practiced in Norway, it is perhaps time to take credit for the benefits of pre-injection, as already suggested by some contractors in Norway.

21 FURTHER Q-SYSTEM APPLICATIONS – PERMEABILITY MAY BE RELATED TO 'Q-VALUES'

An interesting and pre-injection related 'property' of the Q-value, clearly a result only possible with the big numerical range of Q, is that there is *some evidence* of an inverse relation between Q and the Lugeon value. This is strictly for the case of *clay-free rock masses*. Some theoretical justification is given in Barton, 2006 (Chapter 9) based on the interpretation of a Lugeon test (with its 1 MPa over-pressure) as a slightly deforming test. A double Boussinesq 'foundation load' formulation is utilized, plus the cubic law relating flow rates to the cube of aperture, when apertures are large enough (i.e. > 0.5mm). The table below shows what to expect in approximate terms. Strong deviation, in one direction or the other, would suggest the need to expect clay-filled discontinuities, or weak deformable rock like phyllite, or de-stressed rock, causing lower or higher permeability respectively, despite lower Q-values.

The obvious 'over-simplification' of the above inter-related parameters, in particular the inclusion of the inherently complicated *stress and depth dependent permeability* has resulted in the development of a more general 'Q_{H2O}' model for estimating permeability. This has an empirically-developed depth dependence, and reverses Jr/Ja to a more logical *Ja/Jr*. In addition the 'deformability' (actually joint strength) term JCS is included in normalized form, so that a deformable weaker rock will show lower permeability. For details see Barton, 2013.

22 FURTHER Q-SYSTEM APPLICATIONS – Q IN Q_{TBM} PROGNOSIS

Although space for the Q_{TBM} prognosis model, developed in 1999, has not been made available in this NFF-23 publication, it is appropriate to give a brief glimpse of this frequently used method in the last page

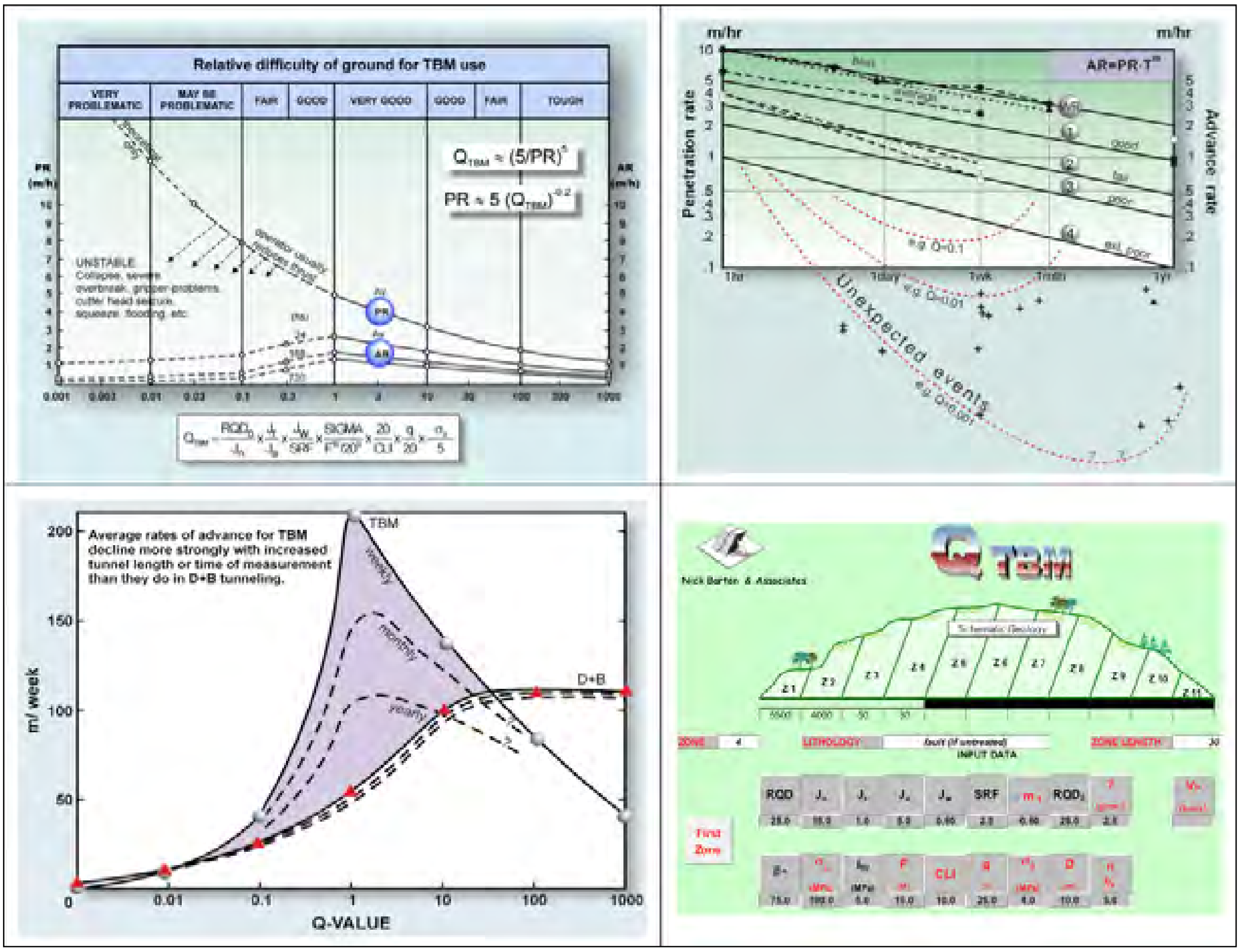


*Figure 27. From top-left, clockwise: **1.**The extended use of the six Q-parameters (with tunneling-oriented RQD_0), including comparison of cutter force F (normalized by 20 tons) with an estimate of the rock mass strength SIGMA = $5\gamma Q_c^{1/3}$ MPa, where γ= density. The last Q_{TBM} term is a tunnel depth correction, with the biaxial stress on the tunnel face (σ_θ) assumed to be ≈ 5 MPa for each 100m of rock cover. **2.** An analysis of 145 cases representing 1000 km of TBM tunnels, shows typical deceleration-with-time trends (see gradient –m) for mostly open-gripper case records. In fact double-shield case records follow similar trends, although extra efficiency can reduce the deceleration gradients in some cases. Robbins numerous world records also show deceleration. EPB projects with picks and cutters follow similar trends. The 'unexpected events' – like stoppages in fault zones – are strongly related to low Q-values. Low Q-values usually mean steep negative (-m) deceleration gradients. **3.** An example of the Q_{TBM} prognosis model input data 'keyboard', which is entered with appropriate numbers for each zone modelled, whether 1 kilometer of massive granite (slow progress predicted) or a major fault zone (also slow progress or almost stoppage). **4.** An exercise comparing drill-and-blast (based on Norwegian cycle times) with a moderate TBM performance, which shows fast weekly averages, a bit slower monthly averages, and a one year performance that needs central Q_{TBM}-values and central Q-values, if the TBM is to remain faster than drill-and-blast. Due to the normalization process, Q and Q_{TBM} can have similar magnitudes, though may differ a lot when there is high or low cutter thrust.*

of this 'Q-chapter'. In essence, Q has been extended to include TBM machine-rock interaction parameters, including the all-important comparison of rock mass strength (SIGMA) with the average cutter force F. In addition, the rock matrix porosity, quartz content, and in particular the NTH/NTNU cutter life index term CLI are each used in a normalized format, as can be seen in Figure 27a. See Barton, 2000 and Barton, 2013 for Q_{TBM} prognosis details.

23 CONCLUSIONS

1. The Q-value representing rock mass quality or lack of quality, and the Q-system linking Q to recommended single-shell permanent reinforcement and support measures (B + Sfr) has proved its value during its 40 years existence. It has been widely adopted both in Norway and in many other countries, as one of the standard empirical tools in rock engineering for assisting in tunnel and cavern design.

2. The tunnel and cavern reinforcement and support measures were originally based on systematic bolting and mesh-reinforced shotcrete, when the Q-system was first developed in 1974. The development of wet process, robotically applied steel-fiber reinforced shotcrete saw first application in Norway in a hydropower cavern in 1979, and in Holmestrand road tunnel in 1981. The development of multi-layer corrosion protected (CT) bolts followed. The Q-system support recommendations were updated in 1993 to reflect the widespread use of B+S(fr) as *single-shell permanent* support. There are about 1,250 case records behind the method since 1993, and a further 800 cases since 2002. There are of course tens or hundreds of thousands of practical applications, the number depending on whether referring to individuals or groups of engineering geologists who apply Q on a 'daily' basis in numerous countries.

3. Besides its widespread use in civil engineering, the mining industry in all the principal mining countries (USA, Canada, Brazil, Peru, Chile, Australia etc.) make active use of the Q-system for support and reinforcement of 'permanent' mine roadways, and also make extensive use of the first four Q-parameters (RQD, Jn, Jr and Ja) together with stress, stope dimensions, and structural orientation, when differentiating stable, transitional, or caving ground, meaning that the stopes are in need of temporary cable reinforcement. However, in civil engineering we recommend and need all six Q-parameters.

4. The Q-value and its modified form Q_c, obtained by normalizing with UCS/100, has many potential uses in rock engineering. It can be correlated to the seismic P-wave velocity V_P (km/s), the (static) deformation modulus M or E_{mass} (GPa), the vertical and horizontal deformation, and has been linked tentatively with the Lugeon value of clay-free rock masses. In modified Q_{H2O} form, depth-dependent permeability in the case of clay-bearing or deformable rock seems also to be predictable in approximate terms.

5. In the last 15 years the Q-value has been incorporated in a more comprehensive parameter called Q_{TBM}. This has additional machine-rock interaction parameters, and is used as a basis for TBM prognosis. On the basis of numerous (1000 km) of data linking TBM deceleration over time (also seen in world records), the Q-value in the case of poor rock conditions (Q<1) can be shown to explain delays and even stand-stills in fault zones. The gradient of deceleration relates strongly to Q-values.

REFERENCES

Austrian Society for Geomechanics, 2010. NATM: The Austrian Practice of Conventional Tunnelling.

Barton, N., Lien, R. & Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*. 6: 4: 189-236.

Barton, N., Grimstad, E., Aas, G., Opsahl, O.A, Bakken, A., Pedersen, L. & Johansen, E.D. 1992. Norwegian Method of Tunnelling. WT Focus on Norway, World Tunnelling, June/August 1992.

Barton, N. & Grimstad, E. 1994. The Q-system following twenty years of application in NMT support selection. *43rd Geomechanic Colloquy, Salzburg. Felsbau,* 6/94. pp. 428-436.

Barton, N. 1994. A Q-system case record of cavern design in faulted rock. *5th Int. Rock Mechanics and Rock Engineering Conf., Tunnelling in difficult conditions,* Torino, Italy, pp. 16.1-16.14.

Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G.. 1994. Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik. *Int. J. Rock Mech, Min. Sci. & Geomech. Abstr.* 31:6: 617-641. Pergamon.

Barton, N. 1995. The influence of joint properties in modelling jointed rock masses. Keynote Lecture, *8th ISRM Congress*, Tokyo, 3: 1023-1032, Balkema, Rotterdam.

Barton, N. 1996. Investigation, design and support of major road tunnels in jointed rock using NMT principles. Keynote Lecture, IX Australian Tunnelling Conf. Sydney, 145-159.

Barton, N. 2000. *TBM tunnelling in jointed and faulted rock.* 173p. Balkema, Rotterdam.

Barton, N., Buen, B. & Roald, S. 2001. Strengthening the case for grouting Tunnels & Tunnelling International, Dec. 2001: 34-36, and Jan. 2002: 37-39.

Barton, N. 2002. Some new Q-value correlations to assist in site characterization and tunnel design. *Int. J. Rock Mech. & Min. Sci.* Vol. 39/2:185-216.

Barton, N. 2006. Rock Quality, Seismic Velocity, Attenuation and Anisotropy. Taylor & Francis, UK & Netherlands, 729 p.

Barton, N. 2012. Defining NMT as part of the NATM SCL debate. *TunnelTalk*, Ed. Shani Wallace. Sept. 2012, 4 p.

Barton, N. 2013. Integrated empirical methods for the design of tunnels, shafts and caverns in rock, based on the Q-system. 3rd Int. Symp. on *Tunnels and Shafts in Soil and Rock*, SMIG/ Amitos, 17p. Nov. 2013, Mexico City.

Barton, N. 2013. TBM prognoses for open-gripper and double-shield machines: challenges and solutions for weakness zones and water. FJELLSPRENGNINGS-TEKNIKK – BERGMEKANIKK – GEO-TEKNIKK, Oslo, 21.1-21.17.

Bieniawski, Z.T. 1989. Engineering rock mass classifications: A complete manual for engineers and geologists in mining, civil and petroleum engineering. 251 p. J. Wiley.

Grimstad, E. & Barton, N. 1993. Updating of the Q-System for NMT. *Proc. of Int. Symp. on Sprayed Concrete - Modern Use of Wet Mix Sprayed Concrete for Underground Support,* Fagernes, 1993, (Eds Kompen, Opsahl and Berg. Norwegian Concrete Association, Oslo.

Grimstad, E. 1996, Stability in hard rock affected by high stress and supported by sprayed concrete and rock bolts. Proc., 2nd Int. Symp. On Sprayed Concrete, Modern use of wet mix sprayed concrete for under ground support. Gol, Norway.

Grimstad, E., K. Kankes, R. Bhasin, A. W. Magnussen and A . Kaynia, 2002. Rock Mass Q used in designing Reinforced Ribs of Sprayed Concrete and Energy Absorption" *4th Int. Symp. on Sprayed Concrete*, Davos, Switzerland.

Grimstad, E., K.Kankes, R. Bhasin, A. W. Magnussen and A.Kaynia, 2003.Updating the Q-system for Designing Reinforced Ribs of Sprayed Concrete and General Support. Proceedings, *Underground Construction,* London.

Grimstad, E. 2007, The Norwegian method of tunnelling – a challenge for support design. *XIV European Conference on Soil Machanics and Geotechnical Engineering.* Madrid.

Grimstad, E., Tunbridge, L., Bhasin, R., and Aarset, A. 2008: Measurements of Forces in Reinforced Ribs of Sprayed Concrete. *5th. Int. Conf. on Sprayed Concrete*. Lillehammer, Norway.

Kristiansen, J. and Hansen, S.E. 1993. Cavern Stadium – displacement measurements. *Final Report on the Research Project for the Rock Cavern Stadium,* SINTEF-NGI- Østlands-forskning, Norway.

Løset, F. and Bhasin, R. 1992. Engineering geology – Gjøvik Ice Hockey Cavern. *Research Project: 'Publikumshall I Berg',* SINTEF-NGI-Østlandsforskning, Norway.

Morseth, B. and Løset, F. 1993. Rock cavern stadium – pre-investigations – decision base – excavation. *Final Report on the Research Project for the Rock Cavern Stadium,* SINTEF-NGI-Østlandsforskning, Norway.

Quadros, E. F. & Correa Filho, D. 1995. Grouting efficiency using directional (3-D) hydraulic tests in Pirapora Dam, Brazil. Proc. 8th ISRM congress, Tokyo. Fujii, ed., 823-826.

Sjøgren, B., Øfsthus, A. & Sandberg, J. 1979. Seismic classification of rock mass qualities. *Geophys. Prospect.,* 27: 409-442.

Ward, W.H., P. Todd and N.S.M.Berry. 1983. The Kielder Experimental Tunnel: Final Results. Geotechnique 33, 3, 275-291.

Vandevall, M. 1990. *Dramix - Tunnelling the World.* NV Bækert S.A, 1991 edition.

APPENDIX A1 – Q-parameter definitions and ratings for reference. The Q-logging sheet (following page) is an abbreviated form of these tables used when logging in the field (core, exposures, tunnel).

	1. Rock Quality Designation	RQD (%)
A	Very poor	0-25
B	Poor	25-50
C	Fair	50-75
D	Good	75-90
E	Excellent	90-100

Notes: i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

	2. Joint set number	J_n
A	Massive, no or few joints	0.5-1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, 'sugar-cube', etc.	15
J	Crushed rock, earthlike	20

Notes: i) For tunnel intersections use $(3.0 \times J_n)$.
ii) For portals use $(2.0 \times J_n)$.

	3. Joint roughness number	J_r
	a) Rock-wall contact, and b) Rock-wall contact before 10 cm shear	
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5

Notes: i) Descriptions refer to small-scale features and intermediate scale features, in that order.

	b) No rock-wall contact when sheared	
H	Zone containing clay minerals thick enough to prevent rock-wall contact.	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1.0

Notes: ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
iii) J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength.
iv) J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ (where $\tau \approx \sigma_n \tan^{-1}(J_r/J_a)$)

	4. Joint alteration number	ϕ_r approx.	J_a
	a) Rock-wall contact (no mineral fillings, only coatings)		
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.	–	0.75
B	Unaltered joint walls, surface staining only.	25-35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25-30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening).	20-25°	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.	8-16°	4.0
	b) Rock-wall contact before 10 cm shear (thin mineral fillings)		
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness).	16-24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness).	12-16°	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of J_a depends on per cent of swelling clay-size particles, and access to water, etc.	6-12°	8-12
	c) No rock-wall contact when sheared (thick mineral fillings)		
KL M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition).	6-24°	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening).	–	5.0
OP R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition).	6-24°	10, 13, or 13-20

	5. Joint water reduction factor	approx. water pres. (kg/cm²)	J_w
A	Dry excavations or minor inflow, i.e. < 1 litre/min locally.	< 1	1.0
B	Medium inflow or pressure, i.e. < 5 litre/min locally, occasional outwash of joint fillings.	1-2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints.	2.5-10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings.	2.5-10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time.	> 10	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay.	> 10	0.1-0.05

Notes: i) Factors C to F are crude estimates. Increase J_w if drainage measures installed.
ii) Special problems caused by ice formation are not considered.
*iii) For general **characterization** of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33 etc. as depth increases from say 0-5m, 5-25m, 25-250m to >250m is recommended, assuming that RQD / J_n is low enough (e.g. 0.5-25) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate **characterization** values of SRF. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.*

	6. Stress Reduction Factor	SRF
	a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated	
A	Multiple occurrences of weakness zones containing **clay** or chemically disintegrated rock, very loose surrounding rock (any depth).	10
B	Single weakness zones containing **clay** or chemically disintegrated rock (depth of excavation ≤ 50 m).	5
C	Single weakness zones containing **clay** or chemically disintegrated rock (depth of excavation > 50 m).	2.5
D	Multiple shear zones in competent rock (**clay-free**), loose surrounding rock (any depth).	7.5
E	Single shear zones in competent rock (**clay-free**), (depth of excavation ≤ 50 m).	5.0
F	Single shear zones in competent rock (**clay-free**), (depth of excavation > 50 m).	2.5
G	Loose, open joints, heavily jointed or 'sugar cube', etc. (any depth)	5.0

Notes: i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterization.

	b) Competent rock, rock stress problems	σ_c/σ_1	σ_θ/σ_c	SRF
H	Low stress, near surface, open joints.	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition.	200-10	0.01-0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > 1 hour in **massive** rock.	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in **massive** rock.	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in **massive** rock.	< 2	> 1	200-400

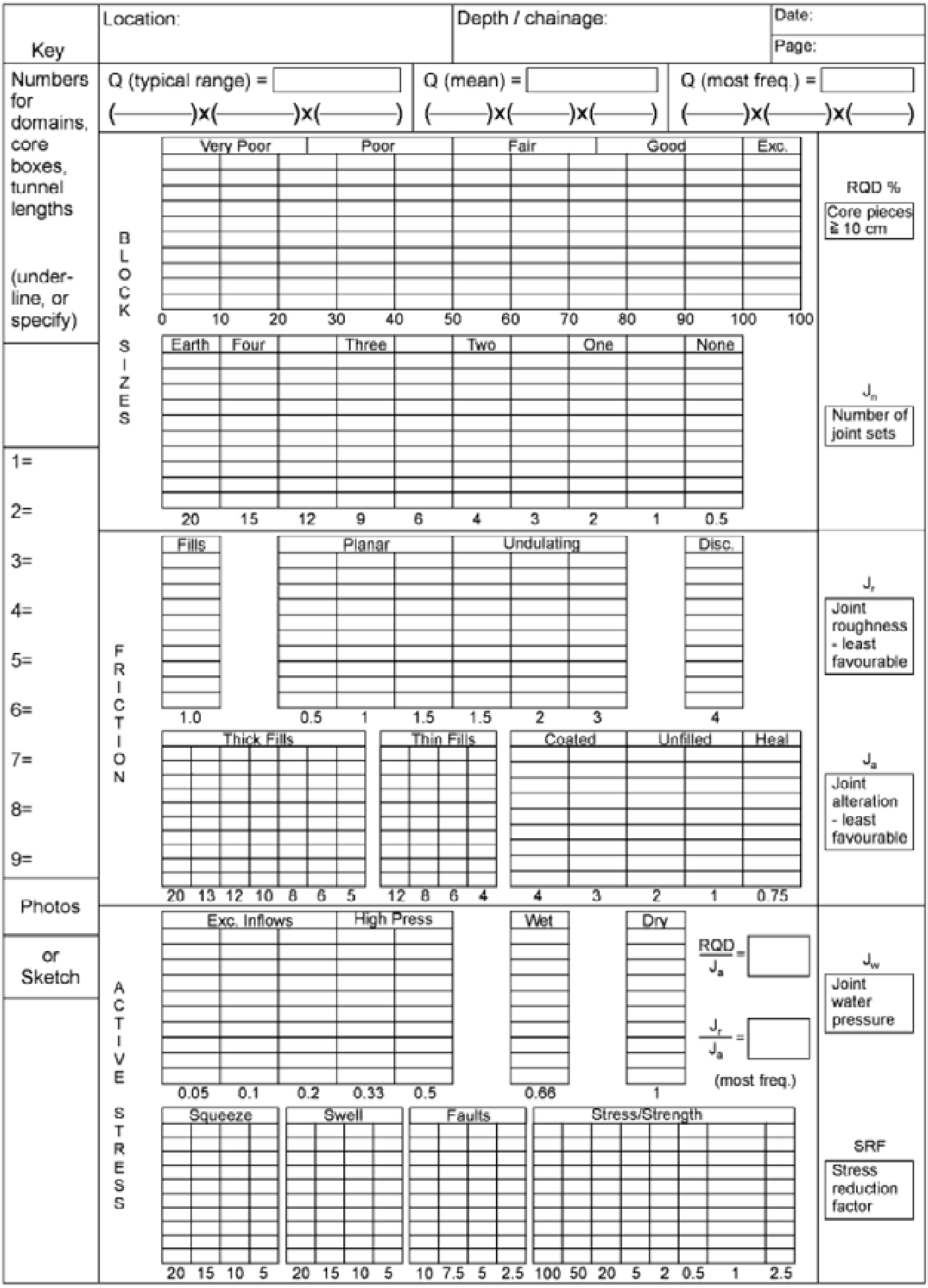
Notes: ii) For strongly anisotropic virgin stress field (if measured): When $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\,\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\,\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory).
iii) Few case records available where depth of crown below surface is less than span width. Suggest an SRF increase from 2.5 to 5 for such cases (see H).
iv) Cases L, M, and N are usually most relevant for support design of deep tunnel excavations in hard massive rock masses, with RQD/Jn ratios from about 50 to 200.
*v) For general **characterization** of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended as depth increases from say 0-5m, 5-25m, 25-250m to >250m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate **characterization** values of J_w. Correlations with depth - dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.*

	c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure	σ_θ/σ_c	SRF
O	Mild squeezing rock pressure	1-5	5-10
P	Heavy squeezing rock pressure	> 5	10-20

Notes: vi) Cases of squeezing rock may occur for depth $H > 350\,Q^{1/3}$ according to Singh 1993. Rock mass compression strength can be estimated from $SIGMA_{cm} \approx 5\,\gamma\,Q_c^{1/3}$ (MPa) where γ = rock density in t/m³, and $Q_c = Q \times \sigma_c/100$, Barton, 2000 [29].

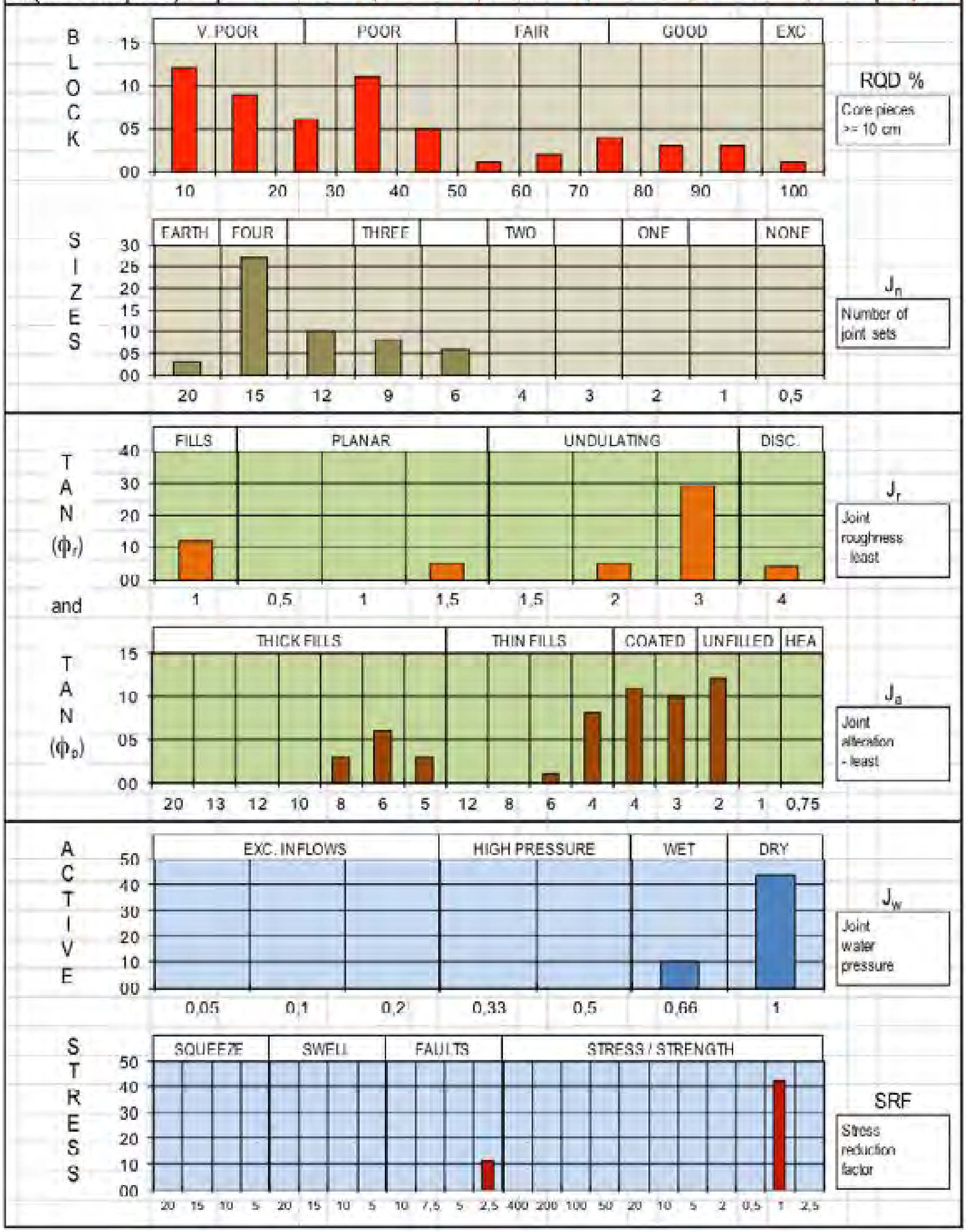
	d) Swelling rock: chemical swelling activity depending on presence of water	SRF
R	Mild swelling rock pressure	5-10
S	Heavy swelling rock pressure	10-15

APPENDIX A2 – Empty Q-parameter logging sheet. Note (brief) descriptions and ratings.

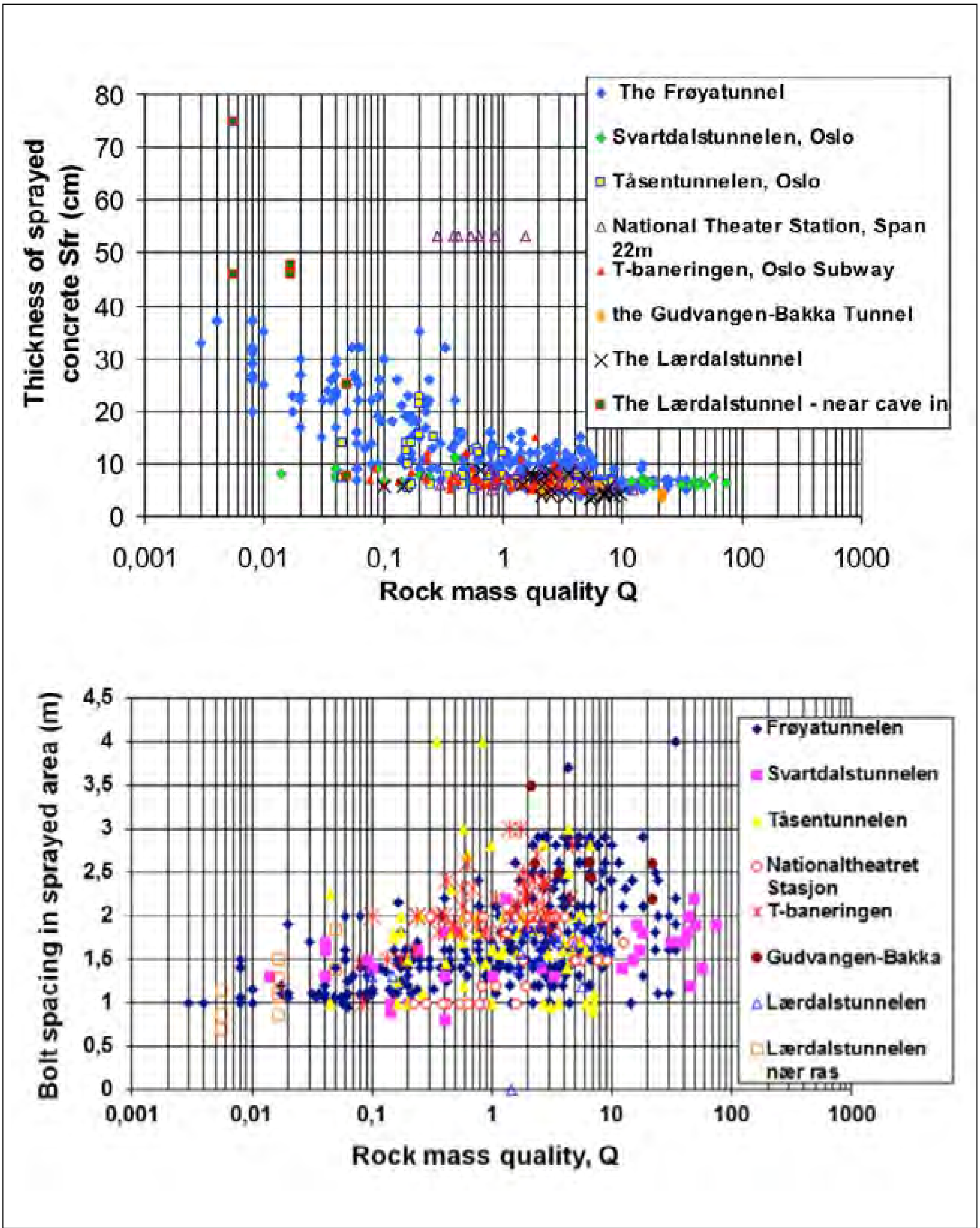


APPENDIX A3 – Completed logging sheet with EXCEL calculation of simple Q-statistics. Note weathered, heavily jointed and clay-bearing nature of this rock mass.

Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	15,0	*	1,0	/	6,0	*	0,66	/	2,5	=	0,029
Q (typical max)=	75	/	6,0	*	4,0	/	2,0	*	1,00	/	1,0	=	25,0
Q (mean value)=	38	/	12,8	*	2,4	/	3,9	*	0,94	/	1,3	=	1,29
Q (most frequent)=	10	/	15,0	*	3,0	/	2,0	*	1,00	/	1,0	=	1,00



APPENDIX A4 - Case records for shotcrete thickness and bolt spacing from some 800 cases where the Q-system recommendations were mostly not used by the designers, which also resulted in occasional failures.



APPENDIX A5 - Observations of high stresses in deep tunnels.
Table A5.1 The table shows overburden, measured and estimated compressive strengths and principal stresses, σc, σ1, σ3, calculated tangential stress σθ, and the relations σc/σ1 and σθ/σc at some Norwegian road tunnels and at two hydropower schemes in Chile and China. Grimstad, 1996.

Name	Rock type	Overburden in m	σ_1 MPa	σ_3 MPa	σ_c MPa	Max σ_θ MPa	σ_c/σ_1	σ_θ/σ_c
Strynefjellet	Banded gneiss	230-600 •	20.4	3.5	47-127	58	4.3	0.4-1.2
Høyanger I	Granitic gneiss	650-800 •	33.4	8.1	100-177	92	3-5.3	0.5-0.9
Høyanger II	Banded gneiss	900-1100 •	29	14	55-126	73	1.9-4.3	0.6-1.3
Kobbskaret	Granite	200-600 *•	26	11.5	90	67	3.5	0.7
Svartisen I	Granite	700 •	21.4	12.1	181	52	8.4	0.3
Svartisen II	Mica gneiss	500 ∇	10.9	8.1	27	25	2.5	0.9
Tafjord	Gneiss, amphib.	500-1200 *∇	24.8	6.6	82-185	68	3.3-7.4	0.4-0.8
Fjærland	Granitic gneiss	600-1200 •	25.7	6.5	110	71	4.2	0.7
Frudalen	Granitic gneiss	900-1200 •∇	30?	20?	70-150	ca. 70	2,3-6,0	0.4-1,0
Tosen	Silicate gneiss	400-600 •	20?	10?	110-200	ca. 50	5,5-10	0.3-0.5
Fodnes	Gabbro, diorite	650-1100 *∇	30?	15?	100-150	ca. 75	3,3-5.0	0.5-0.8
Amla	Gabbro diorite	100-400 •	20?	5-10?	100-150	ca. 50	5-7,5	0.3-0.5
Lærdalstunn.	Banded gneiss	800-1400 •	40?	22?	100-150	ca. 100	2,5-3,8	0,7-1,0
Stetind	Granite	300-500? *∇	9,3	3,8	90	24	10	0,3
Pehuenche, Chile	Andesite ❖	400-1200 ∇	35?	15?	100-150	ca. 75	2.9-4,3	0.5-0.8
Ertan, China	Gabbro, diorite	300 -400 *•	40?	15?	105-160	ca. 90	2,7-4.5	0.6-0.9

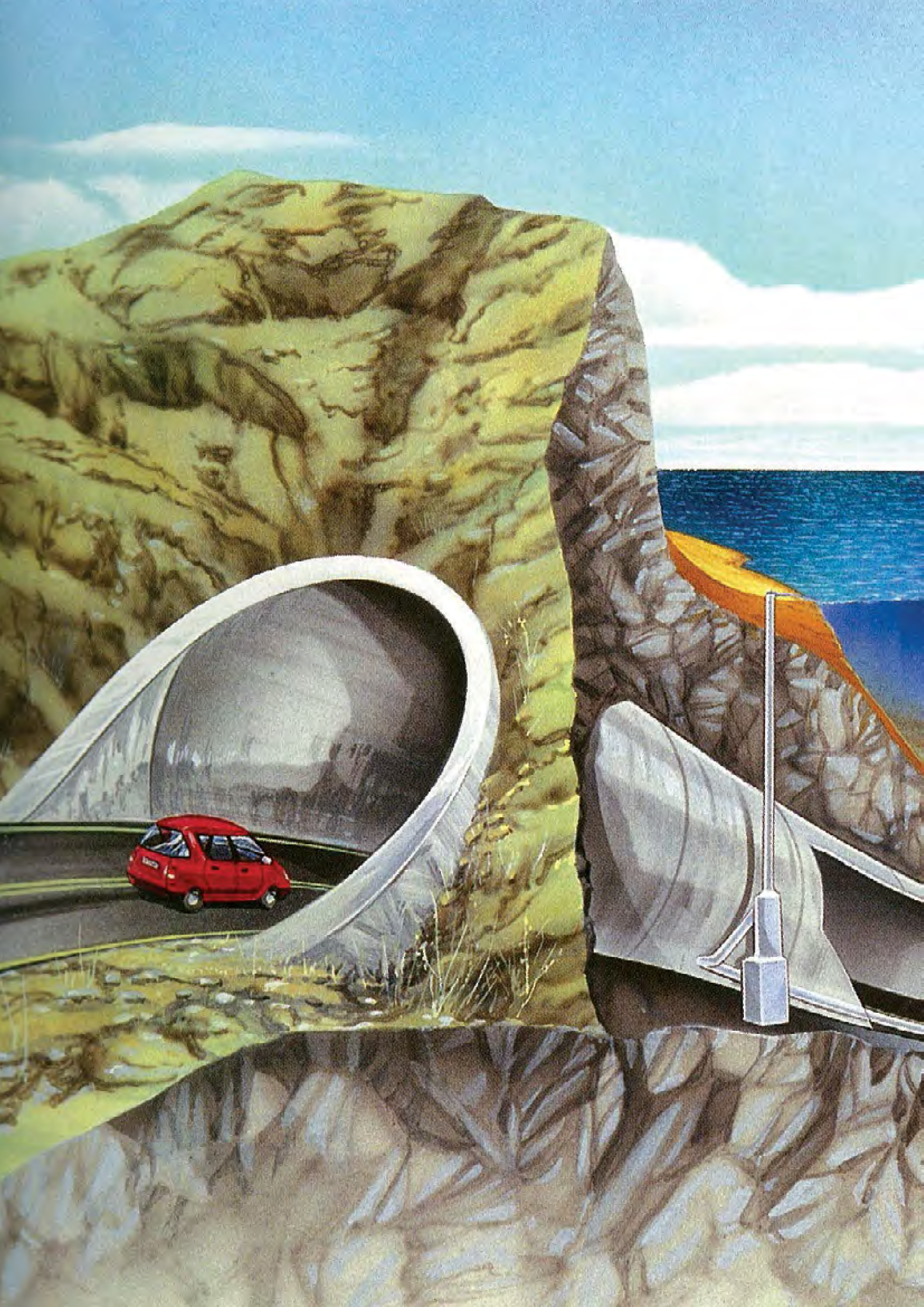
Sub-horizontal major principal stress * Valley side stress. Δ Subvertical stress. The compressive strength of the rock mass is sometimes/often less than σ_c because of jointing.

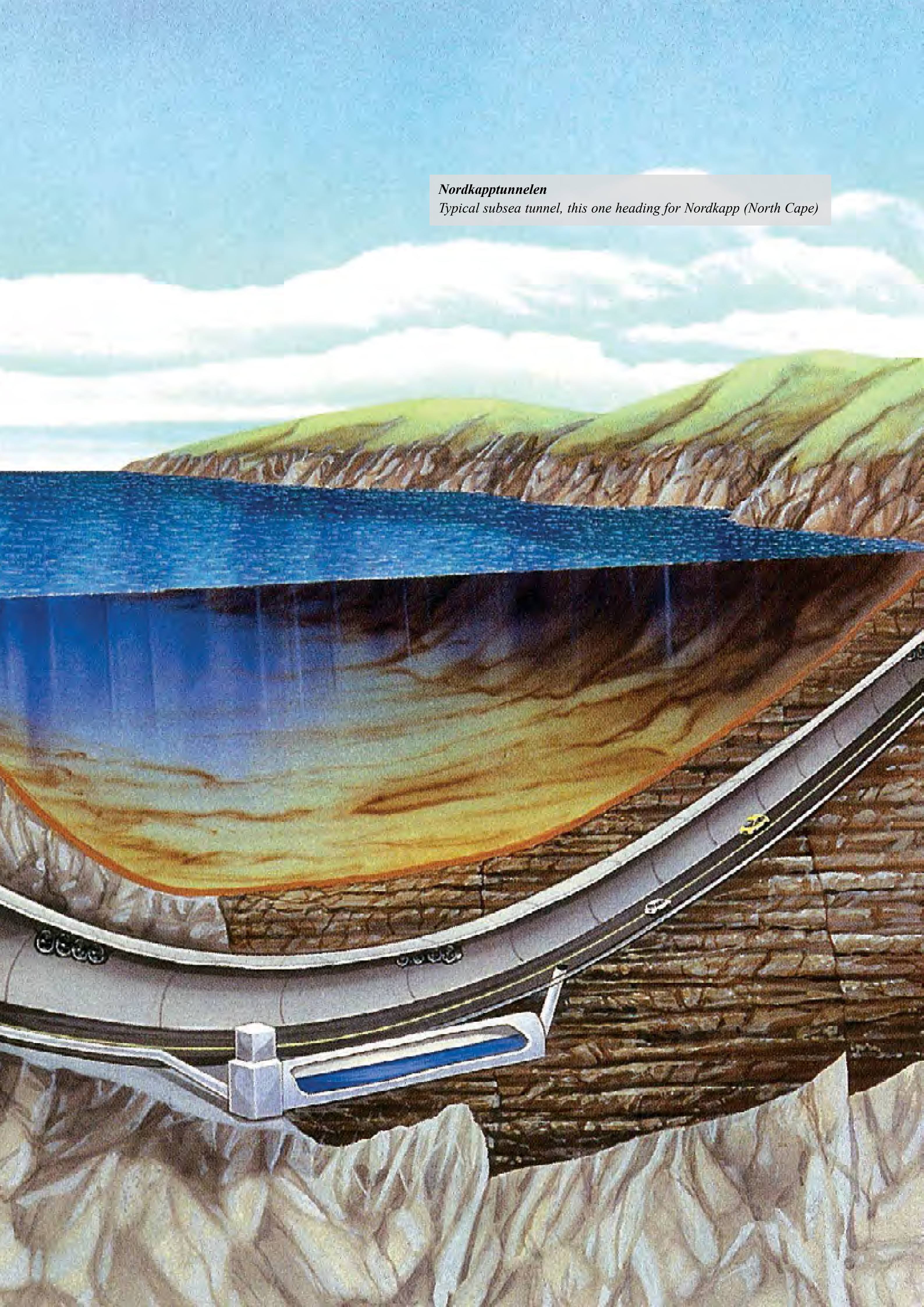
Table A5.2 Relation between uniaxial compressive strength σ_c and major principal stress σ_1, and between tangential stress σ_q and σ_c, with each compared to deformation duration time, and to estimated total deformation. Estimated SRF values are also given. Grimstad,1996.

Name/place	σ_c/σ_1	σ_θ/σ_c	SRF	Deformation time before observation	Type of damage	Estimated total deformation
Strynefjellet	4.3	0.4-1.2	5-200	16-21 years	SlR	20-60?mm
Høyanger I	3-5.3	0.5-0.9	5-150	4-8 years	SpR + DP	10-40?mm
Høyanger II	1.9-4.3	0.6-1.3	50-400	4-8 years	SpR + DP	20-100?mm
Kobbskaret	3.5	0.7	50	2-24 months	SpR	20-50?mm
Svartisen II	2.5	0.9	150	6-18 months	SpR	30-50?mm
Tafjord	3.3-7.4	0.4-0.8	5-100	2-3 years	SpR + SlS_{fr}	10-50?mm
Fjærland	4.2	0.7	50	5-7 years	SpR	10-40?mm
Frudalen	2.3-6.0	0.4-1.0	5-200	1 - 25 weeks	CrS_{fr} + SpR	20-60?mm
Tosen	5.5-10	0.3-0.5	2-5	2-12 months	SpR	1-20?mm
Fodnes	3.3-5.0	0.5-0.8	5-150	1-12 months	SpR+CrS_{fr}+SlS_{fr}	20-60?mm
Amla	5-7,5	0.3-0.5	2-5	1 week-2 years	SpR + CrS_{fr}	1-10?mm
Lærdalstunn.	2,5-3,8	0,7-1,0	50-400	1-8 weeks	SpR+CrS_{fr}+SlS_{fr}	40-200?mm
Stetind	4	0,7	50 - 100	4-5 years	CrS_{fr} + SlS_{fr}	10-60?mm
Pehuenche, Chile	2.9-4,3	0.5-0.8	200-400❖	3-16 weeks	SlS_{mr} + DP	20-100?mm
Ertan, China	2.7-4.5	0.6-0.9	50-200	1-24 months	$SlS_{fr/mrp}$ + DP	20-160mm

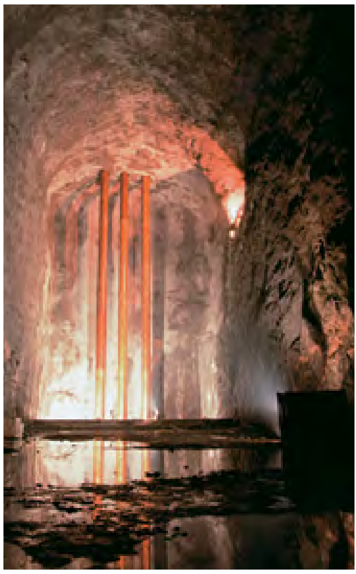
❖ The compressive strength of the rock mass is less than σ_c because of jointing

Type of damage: SpR = spalling in rock, SlR = slabbing in rock, CrS_{fr} = cracks in steel fibre reinforced shotcrete, CrS_{mr} = cracks in mesh reinforced shotcrete, SlS_{fr} = slabbing in steel fibre reinforced shotcrete, SlS_{mr} = slabbing in mesh reinforced shotcrete, SlS_p = slabbing in plain shotcrete, TB = torn off rock bolts, PB = pulled out rock bolts, DP = deformed or torn off plates on rock bolts.





Nordkapptunnelen
Typical subsea tunnel, this one heading for Nordkapp (North Cape)



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05. DECISIONS TAKEN AT THE TUNNEL FACE ADAPTED TO THE ROCK MASS ENCOUNTERED

FOSSUM, Knut
VIKANE, Kjetil
HØYEM, Kai-Morten

- The decision to modify the standard excavation and rock support may take place when needed on the bases of the "adjustment-of-support-at-the-face" principle
- Collaboration and problem solving
- One of the aspects is to utilize the competence of all parties working at the face including representatives of the contract partners and their crews and advisors.
- In a country where tunnelers are expensive it is necessary to establish crews able to handle all activities, from scaling, drilling, charging, blasting to mucking out and ordinary maintenance of machinery. reducing numbers

1 ORGANIZATION AND CONTRACTUAL RELATIONSHIPS

Tunnelling in Norway maybe characterized as informal. This method is perhaps better described as the Observational Method, as it reflects what is in fact taking place. The nature of this method is due to contractual relationships described in a separate paper below. Respect based on knowledge and know-how strongly influence decisions. The interaction between the client, contractor and consultants is based on their being equal parties, but with different perspectives, as visualized in Figure 5.1 below.

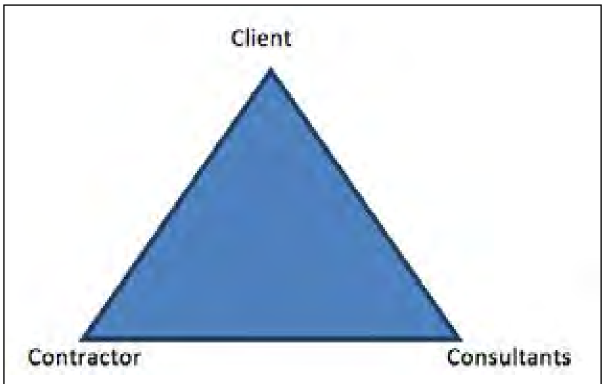


Figure 5.1: Relationship and interaction between client, contractor and consultants.

HSE, Health,Safety and Environment, is in NMT and Norwegian tradition looked upon as an important element for driving a tunnel and operating in such a challenging environment as that encountered underground. It takes into consideration the safety of all people involved and also the long-term stability of a tunnel when its construction is completed. It is through the HSE situation based on defined areas that the responsibility is clearly defined.

- **The contractor** is responsible for the safety of his people and thus has the overall responsibility for safety at the construction site.
- **The client**, who will be taking over the construction object after completion, will have been involved in all matters. He will approve the permanent rock support method and aspects that can have consequences for the permanent structure, such as the pre-grouting method.
- **The consultant**, is the advisor for what is going to be approved, and he will, on the basis of experience and requirements, recommend a solution, commonly agreed upon.

The contractor alone is responsible for work safety; he is responsible for his employees and is subject to common laws, acts and regulations in Norway and to approved standards. It will thus always be challenging to object to what those responsible for the safety of the men working in the tunnel say. This is certainly where respect for each profession is needed. The client is responsible for the permanent rock support and is given advice by his consulting engineer. The excavation and rock support method is often an issue openly discussed between all parties. The three parties involved discuss openly and feel free not to be bound by the tender document, as often experience tells us that we have suddenly a different situation from a theoretically proposed concept 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.

Figure 5.2: Challenging conditions at the beginning of Bukkestein Road Tunnel, Norway

Figure 5.3: Typical discussion at face between client, consultants and contractor.

1.1 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 are linked to a number of elements, but are 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. In recent years the need for trained engineering geologists has increased. The elements we are talking about are:

- General rock classification.
- Rock support in general; where to use:
  - Rock bolts; length of rock bolts
  - Sprayed concrete, reinforced with steel fiber or not
  - Spiling and/or reinforced sprayed concrete arches
- Full concrete lining on the tunnel face.
- Pre-injection, grouting at the tunnel face
- Or even shortening the blast length for each blast, or reduce cross-sections of length of the excavation round.

Such matters will be handled by competent representatives of the contract partners, accepting that the rock mass may change along the tunnel.

Client:
- Client's project manager: The client's project manager, (the engineer on site) is the client's representative on site . He is there to ensure that the project is running according to contract, and that all decisions are made in accordance with that. The challenge for tunnel work consists of all the elements not taken into account, which require a fast decision, or consensus for where and how to proceed. The client's representative does not necessarily need to be an experienced tunnel engineer, but some knowledge will be an advantage.
- Inspector: An inspector might follow each shift and has often authority to make decisions at the 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 underground construction work and trained to work in a tunnel team. Each team consists of three to six members. This team is capable of performing all tasks involved in excavating tunnels, including drilling, charging, blasting, excavating, scaling and installing rock support, and is able to carry out machinery maintenance and work behind the face. 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.
- Foreman: The foreman follows each shift and is the contractor's first line manager. He is responsible for daily HSE matters and makes sure that the works are executed according to the contract and client's instructions. He communicates on a daily basis with client's inspector.
- Site manager: The site manager is responsible for the entire production and makes production decisions for the contractor. Communicates with engineering geologist and client's project manager.
- Contractor's project manager: Contractor's project manager has the overall responsibility for contractor's contractual obligations and communicates with client's project manager.

2 BASIS FOR TUNNEL EXCAVATION – WHAT KIND OF DECISIONS ARE MADE ON SITE

In Norway there is focus on good geological pre-investigations, good site investigation methods and results. This will benefit all parties involved in a later decision of how to perform rock excavation and rock support. This is done up to a certain level with the main purpose of giving a general description of the geological conditions and major weakness zones and providing 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 about 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 is to 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 work starts.

During excavation the real geological conditions appear, and both parties will do their mapping for each blast. The inspector and/ or the engineering geologist does the classification, normally according to the Q-system and decides the permanent rock support.

The key indicator for expected water problems at the face, and in addition weak zones, is drill sink. Most of today's drilling jumbos have an indicator to measure the drilling sink or speed of drilling a hole. This can indicate slopes, cracks or crushed/ weather rock. Therefore, observations by the drill operator and team leader on the jumbo are of importance in judging what kind of rock conditions we might have in front of us. By utilizing a modern MWD-system, the same data can be analyzed by the engineering geologist, so to speak, simultaneously at his office as the latest development enables network transmission of such data. Thus, a lot of the forthcoming rock support might be estimated here, and certainly the estimated injection volume, especially grouting, whether it be cement, microfine cement or chemical grout. See Figure 5.4 of how MWD data can indicate fractures, water and weakness zones. But regardless of technological developments and computerized systems, we still have to rely on the human eye to determine work site safety, including the personal safety of team members. Decisions will be based on experience and judgment of those on the spot where things are happening.

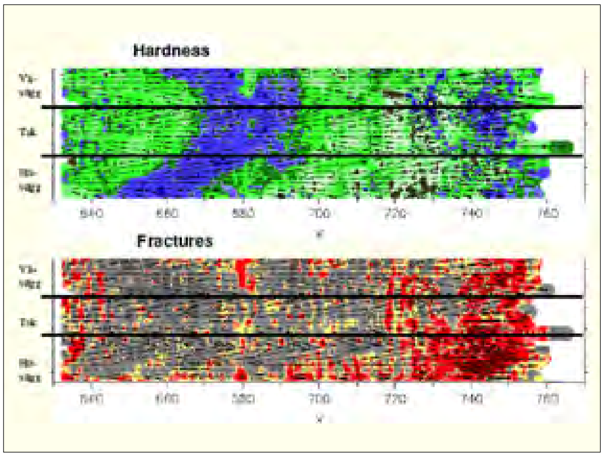


Figure 5.4: Mapping by use of Measure While Drilling data.

The temporary rock support is decided by the team leader, but normally the Inspector/engineering geologist, the foreman and the site manager discuss necessary rock support together with the team leader. They will try to implement the permanent rock support as a part of the temporary support. At an early stage, it is also usual to decide a rock support classification system, where the size and the construction and the actual rock conditions are considered and the Q-system used to give a guiding scheme to be used for planning of efficient rock support installation. We wish to stress 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 system. The Q- system is a very good tool for classifying what might occur, but we underline that it is NOT a "black and white" system to be followed in practice. An example is given in Figure 5.5. 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 a multidisciplined, hard worker who solves all tasks within the team. This results in high production and low manning cost although the Norwegian tunnel workers earn a lot above average.

It is in special geological zones, such as weakness zones or very poor rock conditions, if the pre-injection formulation does not 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 evident during construction of a sub-sea tunnel

Rock support classes	I	II	III	IV	V
Rock mass quality	Good	Intermediate	Poor	Very poor	Extremely poor
Class	A/B	C	D	E/F	G
Q-value	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 *	Cast 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 5.5: *Typical diagram for description of rock support given by mapping using the Q-system.*

in Norway. The rock mass consisted of very crushed rock, described as extremely poor. The normal procedure would be to cast concrete at the face, but instead it was decided to pre-inject the rock formation, install spiling bolts and sprayed concrete arches and blast the cross-section in two blasts. See Figure 5.6.

At a hydropower project, one of the inlet tunnels had such high water inflow that it became a threat to the feasibility of completing 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.

At the ongoing construction work for the new underground Holmestrand railway station, rock bolts of 12 meters in length have been used in the cavern. With the given set of requirements set by the client, the contractor and the supplier developed and introduced a new type of CT-bolt. This came in 2 pieces, each of 6 meters, which were connectable on site. This new type of bolt allowed a reduction in the drilling diameter, placement time and bolt weight, thus reducing the time consumption considerably. One also achieved immediate rock support. This example shows a tunneling culture in which ongoing projects can enjoy the benefits of "in situ" development, and the fact that the suppliers also play a part in the cooperation between the parties.

3 DECISION MAKING AT THE FACE – HISTORICAL DEVELOPMENT

Tunnelling in Norway has been developed over more than 100 years. It started with railway tunnels inspired by similar activities abroad, hard labour work and simple tools. Modern tunneling started say late nineteenfourties. The introduction of the air leg, pneumatic and hydraulic drilling jumbos, the introduction of dry and wet sprayed concrete were implemented by developers and contractors as soon as available as were rock bolts and other rock support products. The labourforce frequently came from the rural areas, small farms or the forestry. People that were used to hard work, simple means, natural thriftiness and inventiveness.

In order to keep costs at a competitive level, the tunneling industry always strives for increasing efficiency through the utilizing the latest technology.

Traditions from political life in society at large are also reflected in how the workers' union cooporates with contractors' association. A tradition, developed over years, has given Norwegian society a wealth 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. The recipe for this success includes cooperation, mutual respect of each party and a forum for discussions where different arguments will be listened to. All parties have the same goal: a safe workplace and to construct a tunnel of the correct quality. This is exemplified on a small scale by what takes place at the tunnel face, after a blast, often before the rock support is put in place. The respect for each party's view benefits a total solution.

Under the influence of Norwegian clients, through the introduction of risk assessments, through the need for better safety along railways and highways and through a better focus on HSE matters, the role of Norwegian tunnel workers has changed. Their influence on the design and excavation method has been slightly reduced by introduction of requirements for planning, documentation and detailed control. However, their involvement is still very strong today and includes other aspects such as risk analyses and planning. The high-tech drilling jumbos have changed the work environment and today a team leader needs a lot of IT knowledge just to operate the machinery. Still, the need for extensive knowledge about rock conditions is huge because of the way tunnels are constructed 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 replace the experienced human eye when it comes to assessing rock conditions by looking at the tunnel face. It is the responsibility for humans and long-term stability that creates a situation that is a win- win for all parties. The Norwegian Tunneling Method or the Observational Method, utilizes know-how and experience to decide an optimal construction at the actual site for the given face conditions. It is never a static description system to be followed the next day, if ground conditions change. It is always the given situation at the face that will need to be handled there at that spot/ moment.

Norwegians believe that this tradition will give the best result and low-cost tunnels. It will benefit all parties.

Figure 5.6: At this location the engineering geologist and the contractor's site manager together decided to use double spiling bolts, perform pre-injection to stabilize the rock formation, to blast the cross-section in two blasts and use reinforced sprayed concrete arches together with spiling c/c 2 meters.

Figure 5.7: At this inlet tunnel the water ingress into the tunnel was very high, with high pressure and was a threat to completion of the tunnel. A team consisting of the client, contractor, consultants and supplier came up with a new way to pre-grout ahead of face.

Figure 5.8: Startup of the Strindheim Tunnel

Kalstø shaft
"Troll" the at that time largest North Sea gas field was found 1979. The construction of the onshore facilities took place during the nineties. The picture concerns the landfall of the gas pipes, a masterpiece in underground engineering. Approach tunnels 4 km long for three import pipelines and two export pipelines. Piercing into the ocean took place at approximately 170 metres bsl. Courtesy: AF Gruppen.

06. FEXIBLE ROCK SUPPORT5 APPLICATION AND METHODS

Excerts of previous presented papers compiled by
WOLDMO, Ola
BECK, Thomas

The hydropower construction required an extensive use of tunnels and underground caverns and contributed to the development of this tunnelling concept. In the 1970'es the oil and gas era in Norway began, and underground facilities were used for transport and storage of hydrocarbon products. Norwegian tunnelling can be characterised by cost effectiveness, flexibility to adapt to changing ground conditions, safe internal environment for the users, and preservation of the external environment.

The following elements have been important for this development:

- pre-grouting of the rock mass to achieve water control,
- utilising sprayed concrete and rock bolts as rock support
- establishing drained support structures.

The rock mass itself is often an excellent barrier, having a significant capacity with regard to its tightness characteristics, but owing to its nature, it is not homogenous and its characteristics can vary greatly. The allowable amount of water inflow to a tunnel is in some cases governed by practical limitations related to the excavation process and pumping capacity, which may result in a drawdown of the groundwater level. A commonly used figure in Norwegian sub-sea road tunnels (where the water supply is infinite!) is a maximum inflow to the tunnel of 30 litres per minute and 100 meter, see Blindheim et al. (2001b).

Requirements to the surrounding environment may restrict a draw down to take place. This is applicable in urban areas to avoid settlement of buildings, and where restrictions on groundwater impacts due to environmental protection are required. Projects have been realized where the allowable inflow was in the range of 2-5 litres per minute and 100 meter.

The primary objective is to employ methods that aim at making the tunnel tight enough for its purpose.

Another aspect, which is typical in Norwegian tunnelling, is that of decision making close to the tunnelling activity, and to include the competence of the tunnelling crew in this process. A trustful co-operation between the contractor and the owner is needed, see Blindheim et al. (2001a). Based on predefined procedures for rock sup-port and rock mass grouting, the tunnelling crew is authorised to implement the design according to the rock mass conditions encountered. Contract practice in Norway has mainly been based on risk sharing through an extensive use of unit rates for different materials and activities.

1 PRE EXCAVATION GROUTING DESIGNED FOR INCREASED DURABILITY AND REDUCED RISK OF GROUND WATER DISTURBANCE

Why make the tunnel or the underground opening a dry one? The answer may be threefold:

- Prevent an adverse internal environment. For various reasons tunnels and underground openings are subject to strict requirements to obtain a safe and dry internal environment. In many cases such requirements do not allow water appearing on internal walls or the roof in the tunnel.

- Prevent unacceptable impact on the external, surrounding environment. Tunnelling introduces the risk of imposing adverse impacts to the surrounding environment by lowering the groundwater table, which may cause settlements of buildings and other surface structures in urban areas and disturb the biotypes, natural lakes and ponds in recreational areas.

- Maintain hydrodynamic containment. The concept of unlined underground openings is used for such purposes as oil and gas storage, cold storage, tunnels and caverns for pressurised air, nuclear waste repository, and other industrialised disposals. "Watertight tunnel-ling" in this context is to provide containment to prevent leakage of stored products.

Norwegian rocks are in practical terms impervious with permeability (k) in the range of 10-11 or 10-12 m/sec. Individual joints may have permeability (k) in the range of 10-5 to 10-6 m/sec. The rock mass is consequently a typical jointed aquifer where water moves in the most permeable discontinuities or in channels along them. The permeability of such rock mass consisting of competent rock and joints may typically be in the range of 10-8 m/sec. This implies that the most conductive zones in the rock mass must be identified and treated. An appropriate solution must be determined to deal with such zones to prevent the tunnel from causing a lowered groundwater table.

1.1 Groundwater control

Water control can be achieved by the use of probe drilling ahead of the face followed by pre-grouting of the rock mass, see Garshol (2001). The primary purpose of a pre-grouting scheme is to establish an impervious zone around the tunnel periphery by reducing the permeability of the most conductive features in the rock mass. The impervious zone ensures that the full hydro-static pressure is distanced from the tunnel periphery to the outskirts of the pre-grouted zone. The water pressure is gradually reduced through the grouted zone, and the water pressure acting on the tunnel contour and the tunnel lining can be close to nil. In addition, pre-grouting will have the effect of improving the stability situation in the grouted zone, also an important momentum, see Roald et al. (2001).

Pre-defined grouting criteria will govern the progress of the tunnelling works. In areas highly sensitive to groundwater fluctuations probe-drilling and pre-grouting may be executed continuously along with the tunnel advance, e.g. such as every 20 to 30 m and with a specified overlap between each round according to project specific requirements. A pre-grouting round may typically include some 30 to 60 holes, drilled in a specified pattern to create a trumpet shaped barrier in the rock mass. The length of grout holes may vary from 20 to 30 m with an overlap of 6 to 10 m between each grouting round, if continuous grouting is required.

Fig 2 Pre grouting equipment at the face. (courtesy Leonard Nilsen&Sonner, LNS)

The pre-grouting scheme must cover the complete 360 degrees of a tunnel and include regulations for control holes and success criteria for the grouting work. Pre-grouting is by far the preferable method to post-grouting. Post-grouting is often an intricate, time consuming and costly process and the result of post-grouting schemes may be rather uncertain and variable. The effect of the grouting schemes must be followed-up by an appropriate monitoring program, see Grepstad (2001). Such monitoring may typically include leakage measurements inside the tunnel, water head measurements of the ground water in the rock mass or water level measurements in neighbouring surface wells, dedicated observations holes or lakes/ponds.

1.2 Tight enough for the purpose

Why make the tunnel or the underground opening a dry one? The answer seems, as far as can be understood by the authors, to be threefold. Prevent an adverse internal environment. Tunnels and underground openings are associ-

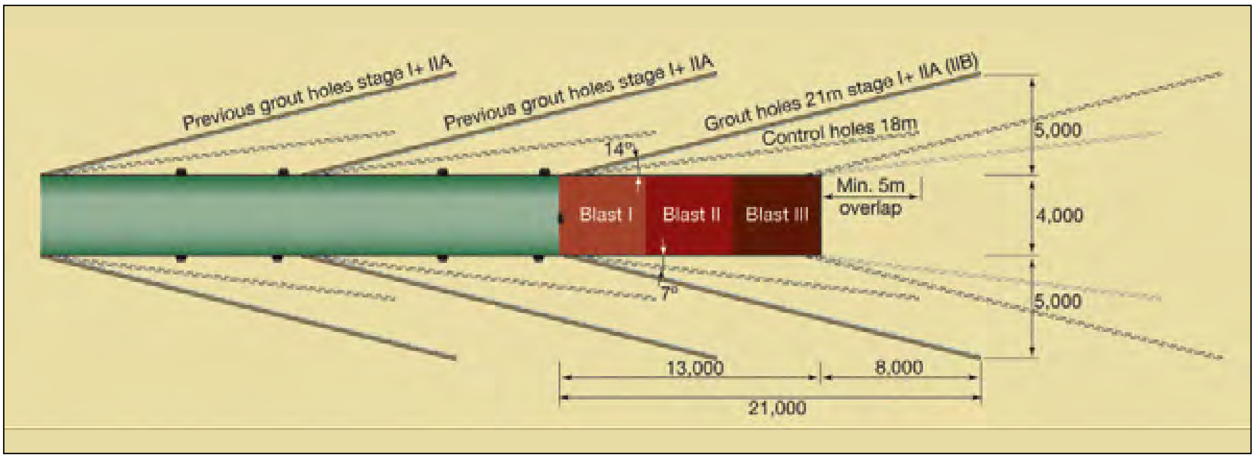


Fig. 1 Typical grout pattern in small tunnel sections

ated with strict requirements to obtain a safe and dry internal environment. In many cases such requirements do not allow water appearing on internal walls or roof in the tunnel. Prevent unacceptable impact on the external, surrounding environment. Tunnelling introduces the risk of imposing adverse impacts to the surrounding environment by means of e.g.; lowering the groundwater table causing settlements of buildings and surface structures in urban areas; and disturbing the existing biotypes, natural lakes and ponds in recreational areas. Maintain hydrodynamic containment. Unlined under-ground openings is used for such purposes as; oil and gas storage, cold storage, tunnels and caverns for pressurised air, nuclear waste and repository; and other industrialised disposals. Water control in this context is to provide a containment to prevent leakage of stored products.

1.3 Tunnelling effects on the groundwater table

The effects of tunnel excavations on the groundwater table were shown by collecting data from several wells in the close vicinity of recently built tunnels. As would be expected, groundwater drawdown becomes less significant away from the tunnels. Changes which are caused by the tunnels are not observed beyond 200 to 300 metres from the tunnel axis. The available data shows, however, no clear correlation between leakage into the tunnels and the measured groundwater drawdown. In general, leakages of more than 25 litres/minute/100 m tunnel causes significant drawdown of the groundwater table (more than 5 to 10 m), and a leakage rate of 10 l/min./100 m or less causes a groundwater drawdown of 0 - 5 metres.

1.4 Procedure to determine accepted leakage rate in sensitive landscapes

The recommended procedure for establishing leakage requirements or accepted impact in relation to consequences for the environment is summarized as follows:

- Overall analysis of vulnerable areas. Combined with a general risk assessment this gives an overview of the probability of changes and the size of the impact. This forms the basis for more detailed analyses of selected areas.
- Both a regional overview and details of specific areas are needed. Detailed investigation is performed for the vulnerable elements.
- Define a value for each of the vulnerable elements
- Describe the accepted consequences based on the obtained value
- State a figure for accepted change in the groundwater table, or water level in open sources.
- State a figure for accepted water ingress to the tunnel. Evaluate both with regard to the length of the tunnel and for ingress concentrated to a shorter section of the tunnel (least accepted change).
- Define a strategy for possible adjustment of the tunnel route, tunnelling method and measures to seal the tunnel, in the areas where tunnel leakage is likely to cause unaccepted changes or damage.

1.5 Procedure to determine accepted leakage rate in urban areas

The recommended procedure for estimating requirements for leakage rate is based on the measurements of pore pressure changes in the clay/rock interface:

- Specify accepted maximum consolidation settlement in the ground above the tunnel
- Produce a map of soil cover, type and thickness, along the tunnel
- Calculate settlements in terms of pore pressure changes for any sediment/clay filled depression identified
- Identify buildings exposed to settlements at the vulnerable sites, and calculate maximum allowable pore pressure change for this area
- Establish requirements for sealing of the tunnel based on the acceptable pore pressure change above the tunnel.

2 SYNERGY EFFECTS FROM PRE-GROUTING LIKE INCREASED STABILITY AND LESS DEMAND FOR ROCK SUPPORT

The traditional purpose of grouting in connection with tunnel excavation has been to avoid major water inflows and to reduce the amount of water that has to be pumped out of the tunnel system. In many cases, grouting has only been considered as a contingency measure, used to solve a problem only after a problem of water inflow has occurred. Some people have also been talking about the stabilizing and strengthening of the rock as a result of grouting. These effects are not well described in a scientific manner. So far, there has been limitations in the material properties and the grouting technology that have constrained the predictability of the rock mass improvement effect from grouting. The recent development in grouting materials and grouting technology in Norway are increasing the possibility to achieve tangible rock improvements (impermeability and strengthening) by grouting. The key performances offered by the recent development in grout materials are:

- Grout penetration to achieve permeability close to 0.1 Lugeon
- Stable grout in the liquid phase (<2% water loss)
- Little or no shrinkage during hardening. (<1-2% volume loss)
- Possibilities to use setting control to create a low permeability zone surrounding the tunnel.
- Environmentally safe and long durability of the grout materials.

Some of these properties (volume stability/strength) can be implied/mirrored in classification systems describing the rock quality, such as the RMR and the Q-system. By such analyses, the effect of grouting can be described in terms of rock quality:

- Rock quality before grouting.
- Rock quality after grouting.

If one knows the rock quality before grouting and is able to predict the rock quality after grouting, it is possible to estimate the time and cost benefits that can be achieved by grouting. In addition, it is possible to describe how grouting may influence the feasibility of the excavation advance in bad rock mass conditions. Taking this effect into account the use of grouting is no longer only a contingency measure, but is becoming an active part of the entire tunnelling cycle, thereby changing the concept of tunnelling.

2.1 How grouting is influencing the rock quality

Grouting is traditionally the way to control water inflow to tunnels, both for protection of the environment and for easing construction. An important effect of the grouting is of course the strengthening effect, Barton, et al.(2001).

New grouting products and techniques allow penetration in the rock masses with permeability as low as 0.1 Lugeon. An interpretation of the physical process involved, has been developed using the Q-system parameters, and using important evidence from 3D permeability testing before and after grouting. The evidence points to potential increases in the Q-sys-tem parameters RQD, Jr and Jw, and potential reduction in Jn, Ja and SRF. Permeability tensors may swing away from the permeable and least stressed joint set following grouting. This is consistent with small individual increases in RQD/ Jn, Jr/Ja and Jw/SRF. Each component may be small, but the combined result is potentially remarkable.

This implies that for the Q-system the following effects can be noted:

- Where there are dry conditions, pre-grouting may improve the rock quality with one quality class.
- Where there are wet conditions, pre-grouting may improve the rock quality with two or even three quality classes.

The overall result of efficient pre-grouting will be reduced rock mass permeability, reduced tunnel displacement and rock support requirement when tunnelling, increased deformation modulus and increased seismic velocity. Similar effects are also known from dam site investigations. Each improvement can be linked in a simple way to Q-value increases.

3 ROBOTIC APPLIED WET SPRAYED CONCRETE WITH FIBRE REINFORCEMENT FOR ROCK SUPPORT

High capacity robotic applied wet mix sprayed concrete with fibre reinforcement has been the standard method for rock support in Norwegian tunnelling since the seventies. Today 40 years later this innovation history is not very known to the industry. Some people claim that the development of the "high capacity robotic applied wet-mix method" started in Norway and other people will claim that Norwegians also had a strong influence on how the method developed around the world. Norway has from the early beginning always had a challenge in establishing communication routs between areas and island where people settled. With increased demand for efficient communication, efficient excavation and support of rock tunnels become a necessity. On the same time utilisation of the hydro power recourses in the mountains made construction of cost effective tunnels important. Later on transport and storage of oil and gas from the reservoirs in the North Sea also benefited from the "domestic" know-how in utilising the underground space. Two of Norway's largest civil contractors Høyer-Ellefsen AS and Thor Furuholmen AS both recognized the challenge in developing the use of wet-sprayed concrete for rock support as a supplement to caste concrete linings. It's no doubt that the "race" between this to very competent contractors lead to the innovations and rapid development of the wet-mix method in Norway.

4 DEVELOPMENT OF WET SPRAYED CONCRETE FOR HIGH CAPACITY APPLICATION AND FINAL ROCK SUPPORT

Today sprayed concrete is one of the most important elements when building an underground construction. The most important functional requirement to sprayed concrete used for rock support is the high early strength. After that, the sprayed concrete has to fulfil the requirements of toughness and strength. It also has to have a long-term durability.

In lack of adequate international standards in the eighties the Norwegian Concrete association made the Norwegian guideline Publication no. 7 Sprayed Concrete for Rock Support. This publication is still acting as and national supplement to the European standards used today. The publication was updated and revised and harmonised with EN standards in 2011.

Below some key information and experiences regarding possibilities and limitations when using sprayed concrete are mentioned.

Fig 3 High capacity robotic spraying (courtesy Andersen Mekaniske Verksted AMV)

4.1 Adhesion to the rock surface
Adhesion is one of the most important properties of any good sprayed concrete support. Cleaning of the surface is very important to obtain good adhesion. In some cases adding of water can lead to further rock fall and deteriorate the stability. A relevant solution could be to use compressed air.

When there are serious water leakages it is difficult to get the sprayed concrete to "hang on". A solution could then be to drill drain holes (relief holes) around the pro-file with collaring from the area further back where the tunnel has already been supported.

Sprayed concrete has its biggest limitation on thick clay zones or other places that do not have good adhesive strength. In these cases, other alternatives to provide support must be considered.

4.2 Fibre reinforcement
Today steel fibre or macro synthetic fibre reinforcement is common in all sprayed concrete that is used as rock support. The normal dosage of steel fibre is approximately 20-40 kg/m3 with a fibre length of 30-40 mm. For macro synthetic fibre the dosage is 5–8 kg/m3 at a length of 40 – 60 mm. There is a big difference in quality of the different kind of fibres on the market so that fibre types with reduced quality require high dosage in order to fulfil the functional requirements of the different prescriptions.

Macro plastic fibre was introduced to the market 10 years ago. This is a relevant alternative to use if big deformations are expected. These fibres also show constancy in aggressive environments such as for instance subsea tunnels. Regarding the highest energy absorption class, strict requirements are made when it comes to the fibre types.

4.3 Mesh reinforcement
Mesh reinforcement can be used as an alternative if sprayed concrete is used to support clay zones. The mesh is anchored safely with bolts on each side of the zone before applying the sprayed concrete. The standard mesh reinforcement when using sprayed concrete is K-131. The net has mesh openings of 150 x 150 mm and a steel thickness of 5 mm. The most common steel quality that is used in Norway is B5000NA, but an even better result can be obtain if a steel quality that is more ductile (B500NB) is used. It is important that the opening in the mesh is not too small so that there is complete contact between the concrete and the rock surface.

4.4 Additives- in general
Normal additives in sprayed concrete:
- Accelerator
- Plasticizer and super plasticizer
- Retarder
- Stabilizer
- Pumpability improving additive
- Internal curing additive

4.5 Accelerator
When working with sprayed concrete two factors are important from the beginning:
- To make sure that the concrete is bonding to the rock surface and stays there
- The hardening should start as soon as possible after the spraying to get a high early strength

In order to achieve this, an accelerator is added in the spraying nozzle. The accelerator must mix with the concrete in the nozzle and begin the hardening immediately after the concrete hits the rock. With a jet velocity of 30 to 35 metre per second and a distance between the nozzle and the rock surface of 2-3 metres this takes 0.1 second.

4.6 Alkali silicates
Earlier fluid solutions of sodium/alkali silicates were the most common accelerators on the market, but today they constitute less than 10 %.
- Sodium silicate (alkali silicate)

The qualities of these have been questioned in recent years because of:
- Alkali reactions with aggregates
- Low early strength when using sodium silicate
- Reduced final strength, especially at high dosage
- Health-risk using highly basic agents
- The safety of the working crew at the tunnel face should be improved with high early strength and better adhesion

4.7 Alkali free accelerator
The alkali free accelerator does not contain the alkali elements (Na, k, li, Cs, Fr, Rb). The properties of the alkali free accelerators are:
- High early strength, > 1 MPa after 1 hour
- No negative effect on the final strength
- No extra alkali must be added - favourable keeping possible alkali reactions in mind
- Can be applied in thick layers (40-50 cm)
- Reduced rebound and downfall
- Reduced quantity of dust
- Improved compaction of the concrete
- Does not build up on the reinforcement bar, better filling in behind the reinforcement bar
- The additives are expensive but favourable when considering the overall economy

Since alkali free accelerators are an important presumption when using sprayed concrete to build up the arch, it might be useful to keep a few things in mind when dealing with this additive:
- The equipment must be custom made for dosing control using alkali free accelerator
- Additives based on lignosulphonate (P agents) should not be used due to retarding and reduced early strength
- Some super plasticizer agents can influence the early strength, but not as much as the lignosulphonate
- The type of cement can influence the result
- Even though the adhesion on wet rock surface has improved, the water has a negative effect on the early strength and can stop the strength development
- The temperature of concrete is important, and must be at least 20 C. Lower temperatures require higher dosages
- Lower temperatures in the air and on the rock surfaces can quickly reduce the temperature of the thin concrete layer, and the early strength is then reduced
- The dosage of the accelerator is important. A trail mix should be executed
- The different suppliers have agents with different properties

4.8 Plasticiser- and super plasticiser agents (p- and sp agents)
Plasticiser- and super plasticiser agents are added to increase the ability to disperse cement and silica dust in the water. The floating properties of the fresh concrete is improved as well as the compressive strength in hardened concrete, and the water content can be reduce without harming the workability.

In general P agents must not be used together with alkali free accelerators as it will reduce the early strength, while the plasticiser- and the super plasticiser agents hardly give any retardation both at alkali silicate and alkali free accelerators.

Today synthetic, water-soluble polymer with the following properties are used:
- Increase of early strength
- No effect on the final strength

4.9 Sprayed Concrete Retarder/Stabilizers
The long-term retarding that is used in sprayed concrete is different from the one that is used in normal concrete. A controlled delay of the hardening of the concrete can be obtained, from hours to days. As soon

as the accelerator is added the effect of long-term retarding disappears.

The results are that:
- The concrete is produced at a suitable time
- The spraying can be executed when it is required at the tunnel face
- The accelerator dosage will normally remain at "normal "levels

5.10 Pumpablilty improver
The pumpablilty of the concrete can be improved by adding small amounts of floating agents. This means that the concrete will move more easily in hoses and pipes. The agents that you use are cellulose derivatives and polythene oxides (kalko floating agents). The qualities of these agents lead to:
- Lower pump pressure, giving less wear and tear
- Increased cohesiveness, which counteracts the separation of water from the solid particles in the concrete

These products are delivered as part of the super plasticizer agents. The air entraining agents are a good and cheap alternative as a pumpablilty improver.

5.11 Internal "curing"
Traditionally "external" curing agents such as spray membranes have been used. These consist of paraffin wax suspensions. They reduce drying-out of concrete and thereby prevent shrinkage and cracking, but will reduce the adhesion if you add several layers.

Internal curing agents are offered as a replacement for the spray membranes. The effect of these has not been documented very well, but they work. Especially favourable if spraying of several layers are needed.

5.12 Example of a mix design

Type of material	Kg/m3
Portland cement	430 - 485
Micro silica	15-25
Water (w/c+s) =0,4	210
Aggregates 0-8 mm	1530
Super plasticiser	3 - 6
Retarder or stabiliser	1 -2
Air entertainer	0 – 1,5
Internal curing compound	5
Pumping agent	-
Alkali-free accelerator	30 - 40
Steel fibre	20 – 40
Micro PP-fibre (spalling prevention)	0 -2
Stump	18 – 22 cm
Temperature	min 20 C

Table. 1 Typical Sprayed Concrete mix-design

5 APPLICATION OF SPILING BOLTS SUPPORTED BY IN SITU REINFORCED SPRAYED CONCRETE RIBS

This chapter describes the relevant methods used for rock support while establishing underground openings under difficult circumstances. It partly advises on which methods to use for the different situations, as well as giving practical advice on the execution. Firstly, different actions to stabilise the rock mass ahead of the tunnel face are discussed. Actions aim at rock support that satisfies safe excavation of the planned opening. These actions are mainly bolting, sprayed concrete, grouting or a combination of the same.

Subsequently methods are described to support the stability of the actual opening. These methods are different solutions of reinforced sprayed concrete ribs, support systems that can absorb big deformations, or full concrete lining at the work face.

The following support means are discussed:
- Spiling
- Injection
- Jet grouting
- Freezing
- Reinforced sprayed concrete ribs
- Sprayed concrete "vaults"
- Lattice girders
- Deformable support systems
- Different kinds of supporting concrete systems

5.1 Risk evaluation in connection with "rock support"
All the work procedures must be described in a quality plan, preferably with a flow chart that illustrates the sequence of the work activities before the risk evaluation is being prepared. Emphasis shall be put on the EHS (Environment-Health-Safety) for underground construction. § 7 in "Regulations on security, health, and work environment in rock work, "FOR 2005-06-30, no 794" gives gui-delines for EHS work in underground construction. Additionally it is required for the contractors to establish standard procedures for the execution of the various work activities. Early in the design phase high-risk activities should be identified. The risk analyses of the critical activities should be included in the engineering phase of the project and continue throughout the entire construction phase in the form of "Safe job Analyses" (SjA). SjA implies that those responsible for the design and those doing the support jointly should review the risk situation; the risk situation may be different from activity to activity. This is especially important when working with weak or altered zones or in other complicated situations where underground openings are excavated and rock support is required.

Fig. 4 Spiling bolts installed in portal area (courtesy Leonard Nilsen&Sonner, LNS)

5.2 Spiling bolts
The main purpose of pre-bolting is to maintain as much as possible of the planned theoretical cross section until the jointly should review the risk situation; the risk situation may be different from activity to activity. This is especially important when working with weak or altered zones or in other complicated situations where underground openings are excavated and rock support is required.

Below one will find guidelines regarding the various relevant methods when strengthening the rock mass.
Q value (guiding) Support ahead of the tunnel face:

0,001-0 , 0 2 Pipe screening/jet grouting/freezing
0,02-0 , 2 Bolting at face
> 0,2 Bolting at large blocks, almost horizontal stratification, low tension, and at outbreak

The main purpose of pre-bolting is to maintain as much as possible of the planned theoretical cross section until the permanent stability support is installed. Permanent support may be sprayed concrete and radial bolting, sprayed concrete arcs or full concrete lining of the profile. Pre- bolting is normally considered a temporary arrangement, which is not included in the permanent support. Therefore and normally no corrosion protection is required.

In case pre-bolting is combined with sprayed concrete arcs, the bolts will support the rock mass and the sprayed concrete between the ribs. The bolts thus will contribute to even distribution of the loads on the ribs in the longitudinal direction, and may therefore act as a part of the permanent support construction. In such cases bolts with corrosion protection must be used.

Usually the bolt material is of ordinary reinforcement steel quality (normal ridged steel bars). The bolts are installed in grout, so that the bolt and the rock mass work together.

Permanent support may be sprayed concrete and radial bolting, sprayed concrete arcs or full concrete lining of the profile. Pre- bolting is normally considered a temporary arrangement, which is not included in the permanent support. Therefore and normally no corrosion protection is required. In case pre-bolting is combined with sprayed concrete arcs, the bolts will support the rock mass and the spray-ed concrete between the ribs. The bolts thus will contribute to even distribution of the loads on the ribs in the longitudinal direction, and may therefore act as a part of the permanent support construction. In such cases bolts with corrosion protection must be used. Usually the bolt material is of ordinary reinforcement steel quality (normal ribbed steel bars). The bolts are installed in grout, so that the bolt and the rock mass work together.

If very difficult rock conditions occur, one may experience drill hole rupture. Under circumstances self-drilling bolts (for instance «Ischebeck» bolts) that can be grouted might be advantageous.

To ensure sufficient strength it is advantageous to use Ø32 deformed steel. These bolts will give lesser deflection than Ø25 mm bolts and reduce the risk of rock fall. The bolt diameter Ø25 mm is still frequently used due to easier handling. At the installation the bolt-hole is filled with expanding mortar and the bolts are squeezed in with the aid of a drilling machine on the drilling rig. The length of the bolt is usually 6 metres so that 1 meter of the bolt is used to hanging up in the rear edge. Bolts of 8 metres have been tried. The length of the blast, however, should be limited to avoid unwanted deflection.

When using 6 metres long bolts the length of the blast should be between 2.5 and 3 metres. In that way one can install a new series of bolts while there is still overlapping from the previous series. It is very important to establish safe anchoring at the rear end of the bolt prior to the next blast taking place.

The normal procedure is to use steel straps, radial bolts, and fibre reinforced sprayed concrete as back anchorage. There must be a radial bolt for each spile.If the drilling for radial bolts or the drilling for the next blast indicate that there is a weakness zone close to or right in front of the tunnel working face then the

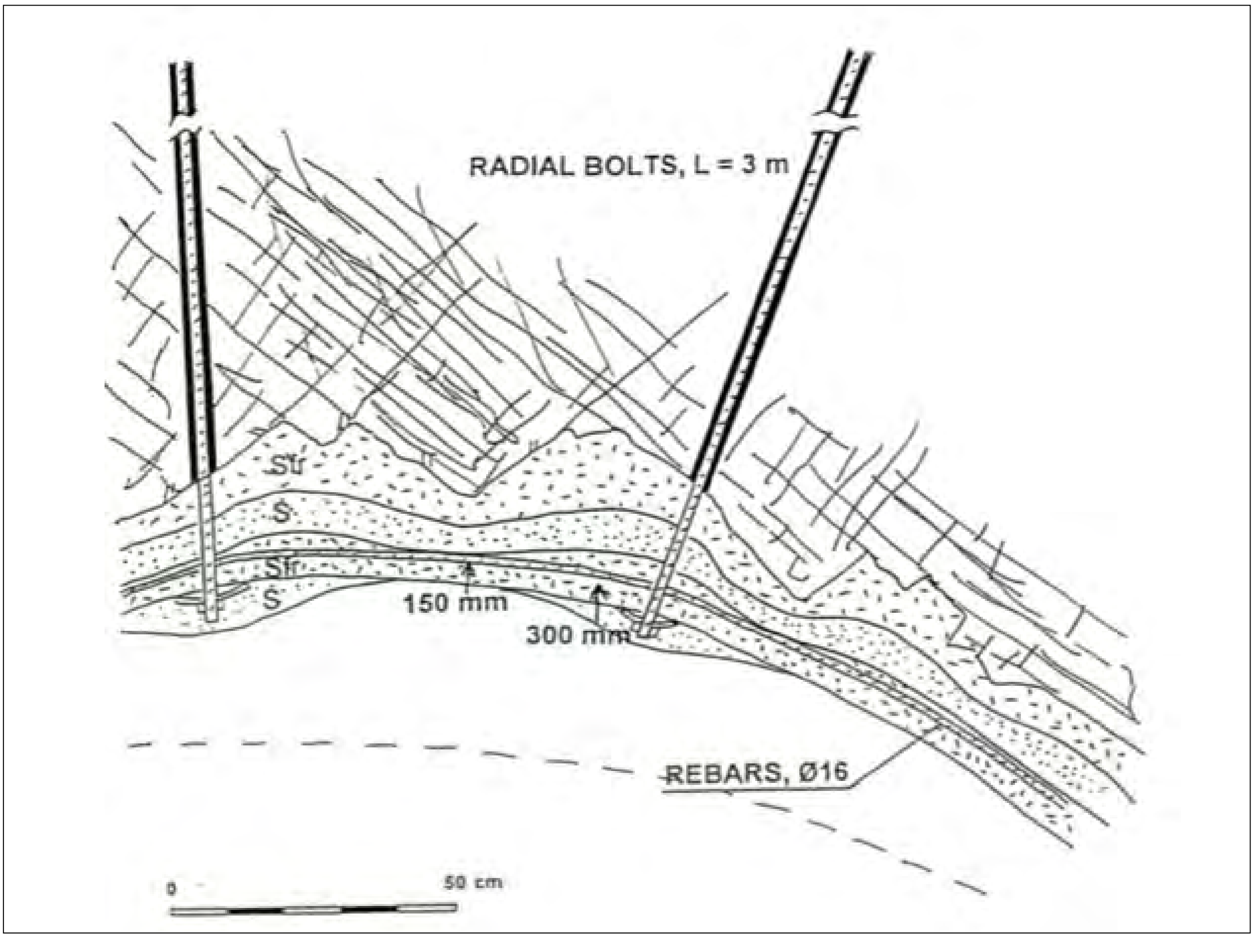


Fig 5. Typical design in-situ reinforcement with 16 mm rebar- most common rebar dimension is 20 mm

pre-bolts should be placed a couple of metres behind the tunnel face in order to get sufficient back anchorage. Normal bolt spacing is:

- The bolt spacing is normally around 0.3 metres (0.2 - 0.6) T
- The distance(burden) between bolt rows 1 and 2 is normally 2.3 -3 m
- Recommended bolt angle to the tunnel axis, 10-15o
- The distance between the bolts will then be approximately 0.5 m
- Length of bolt, 6 metres

Densely placed bolts establish a crack line. In zones where the excavated rock mass is very poor, rock fall close to the bolts usually occurs. It might be useful to adopt a method implemented for the "Frøya" subsea tunnel: The angle of the bolt to be reduced so that the crack line comes as close to the tunnel profile as possible. The spacing (distance between the bolts) should at the same time be reduced to 0.2 metres.

5.3 Geometry

a) Sprayed concrete ribs shall be constructed with a smooth curvature, equal and parallel to the theoretical cross section of the tunnel
b) The rib shall be established in a vertical plane at a right angle to the tunnel alignment. Exceptions may be minor fault zones where the rib adapts to the fault
c) Sprayed concrete ribs must be installed on adequate foundations

5.4 Reinforcement

a) Reinforcement
- Rebars shall be of quality B500NC
- Dimension Ø20 mm, pre-bended to the given curvature
- Spacing ≥ 110 mm
- Concrete cover ≥50 mm [≥ 75 mm subsea]

b) lattice girders
- Lattice girders may be used as an alternative where double reinforcement is needed.

5.5 Sprayed concrete

- Sprayed concrete used in subsea tunnels shall adapt to endurance class M40 that in turn calls for concrete quality B45. (45 MPa). For other tunnels class M45 is required (concrete quality B35)
- In areas where sprayed concrete ribs shall be installed, the first step is scaling and the establishing of a sprayed smoothing layer with fibres, E1000 (energy absorbing class) , quality B35 ≥ 150-250 mm thick.
- To safeguard the correct arch geometry a further spray-ed layer (no fibres) shall be applied as necessary.
- Installing of the reinforcement and concrete spraying.
- A compressive strength ≥ 8 MPa is required prior to the next round being blasted.

5.6 Radial bolts

- Bolts to be included in the rib system shall be grouted and of dimension ≥ Ø20 mm
- Where several ribs are installed, the rock bolts between shall be 3-6 m long with spacing c/c 1.0 – 1.5 m
- The low end of the rib shall be secured by grouted anchor bolts, dimension 25 mm, l= 4 to 6 m. Optional to install a concrete invert.
- Rock bolts may be exposed to pull-out tests

REFERENSERS

Norwegian Tunnelling Society. Publication no. 12 Water control in Norwegian Tunnelling (2000)

Norwegian Tunnelling Society. Publication no. 19 Rock Support in Norwegian Tunnelling (2010)

Norwegian Tunnelling Society. Publication no. 20 Rock Mass Grouting (2011)

Large face – ample place for three jumbos at Holmestrand station. Courtesy: Norwegian National Rail Administration

07. COMPUTER AIDED DESIGN AND CONSTRUCTION, WITH INTEGRATED HEAVY EQUIPMENT

WETLESEN, Thorvald
ANDERSEN, Peder
BERG, Heidi

1 INTRODUCTION

As a consequence of the increasing urbanization, the big cities require more and more flexible, complex solutions for underground construction. The big cities will no longer be in one or two levels, but in multiple levels. The space is valuable, and the requirement to build tight and compact is increasing. This has lead to a development of 3D design software tools for road, rail and tunnels, that let the designer be in total control of the tight geometry, to assure there is enough space. We have multiple examples of drill and blast tunnels, that in multiple sections share roof in one tunnel with road bed in a crossing tunnel.

There are multiple benefits of creating a 3D model of the tunnel.

- You are able to run clash detection towards other, existing tunnels or other constructions
- You can check inner space inside the tunnel, both before construction and during construction
- You can export the tunnel geometry to computer aided drilling jumbos, as a basis for their drill plan (using e.g. LandXML format)
- You get accurate volume calculation
- You could extract cross section drawings anywhere from the model
- You have a basis for geo referenced documentation of the geology and support measures

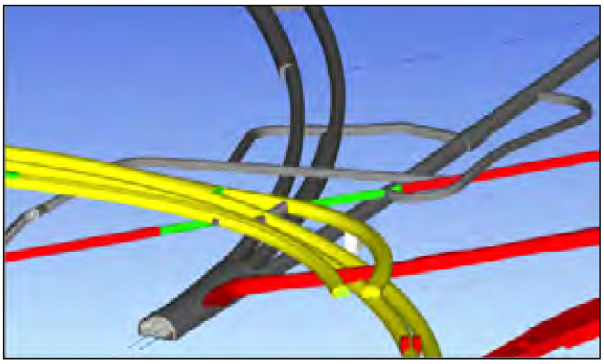
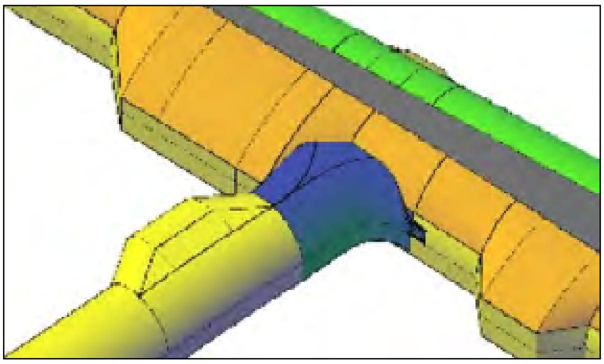


Figure 1 Example of crossing tunnels in Ekeberg, Oslo



Novapoint Tunnel design tool, for 3D design

Figure 2 Data flow from 3D tunnel model to production equipment.

2 DATA FLOW FROM MODEL TO EQUIPMENT

Software is becoming essential in all parts of development project today.

- Early planning – alignment control
- Design phase – 3D geometry models for interdisciplinary clash control
- Construction phase phase – 3D models used by machine guidance systems and surveying instruments
- Operation phase – geo-referenced information, with sufficient details/attributes

2.1 Communication underground WLAN

The Norwegian method are based on digital information flow and geo-referenced information The contractor and client define a setup for communication in the project. The client establishes a web hotel for exchange of data between the project members.

In many projects install a WL-N system in the tunnel. It will be connected to the underground equipment with different objectives:

- Ventilation control
- HMS monitoring
- Drilling jumbo WLAN connection
- Grouting equipment WLAN connection
- Cell phone communication (IP)

The communication setup is used for support, plan transfer, production data report. For the complex machinery it is also possible for supplier to do on face support and fault diagnosis from remote location.

2.2 Accurate drilling and contour quality – setting up at face

The position of the drilling jumbo at face shall be measured accurate and quick, and is done without surveyors at face. The methods which are used are:

1. Classic method with laser aligned with the feed on left or right boom (tunnel laser beam)
2. On-rig-profiler with pointing device and fixed point reference targets (Bever 3D Profiler).
3. Total station of robotic type or manual operated. Measure two fixed points on the jumbo chassis (Trimble or Leica mostly used).

We recommend method 2 and 3 and 1 as backup. The time for setting up the drill rig jumbo at face should be less than 5 minutes.

2.3 Navigation of drilling jumbo – establish a geo-referenced system

Total station or profiler navigation is used to establish a geo-referenced production system. All fixed points are saved on drilling rigs computer and a radio communication device controls the navigation. It takes ca 30 sec to navigate after the setup of the total station and accuracy is 2.3 cm.

2.4 Download design and drill pattern to data to drilling jumbo

The project is transferred in 3D formats to the contractors, also including DWG drawings as needed. Contractor prepares production plans like contours, drill pattern, detailed excavation plans Client defines the amount of rock support as needed.

Contractor establishes the project data base in BeverTeam Office or similar. Contractor will download the production plans to the underground equipment when needed via a data exchange server or web-hotel server.

2.5 Upload production data from Drilling Jumbo and Grouting rig

The drilling jumbo is equipped with advanced control and monitoring systems. Data acquisition system records data from scanning of excavated contour, drilling parameters and drifter performance. Anti-jamming features and auto positioning is available. Also the actual drilled pattern is reported.

The grouting rig has a similar system to control the grouting process. All processes is recoded in a log and presented on the rig and in the office (BeverTeam – Injection. Drilling parameters from the MWD analyses is available after download from the office. Volumes and pressures are reported for each grouted hole.

2.6 On-drill-rig contour scanner, checking overbreak and point out underbreak

Accuracy of drill and blast is monitored from the drill rig using a contour scanner that gives on jumbo control or underbreak/overbreak without any markings on the face. The scanner records the excavated surface from last round and gives the operator warning on actual underbreak.

The scanner data is transferred to office and is used as contractor and client documentation. The scanner is also used for navigation of the drill rig

Figure 3 Total station for drill jumbo navigation checks the position of 2 targets on the drilling jumbo and transfer these positions to the jumbo computer on radio, WLAN or Bluetooth. A USB stick can also be used for manual transfer The jumbo computer calculates then the drill bit position based on sensors in all moving parts of the drilling boom.

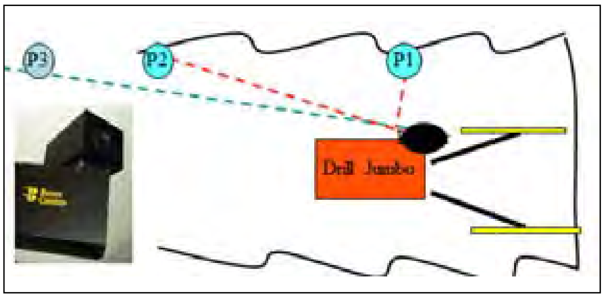


Figure 4 Profiler navigation. In most projects in Norway the drilling jumbo has a Bever 3D Profiler for scanning the last blast to give the report on overbreak and underbreak to the operator and office. The profiler is a self-contained unit and is used for navigation of the jumbo as well. Accuracy will be within 3-4 cm.

Figure 5 Geometry information downloaded to the drilling jumbo. Logging of drilling is uploaded during production. The jumbo computer has all needed project data as tunnel alignment, contour design including niches and dikes. Drill pattern design is transferred from office or designed via a parametric method direct in the combo. BeverTeam

Figur 6 Bever 3D Profiler scans the tunnel contour after shotcrete The operator will be informed on the accuracy of contour drilling

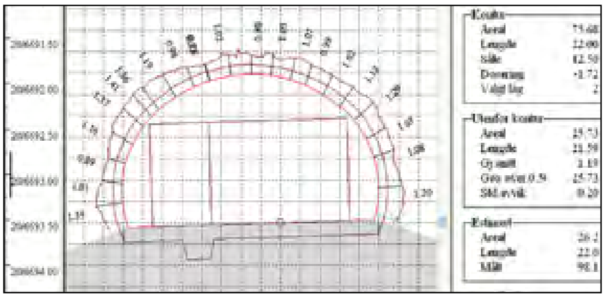


Figure 6 Profile scanning. BeverTeam

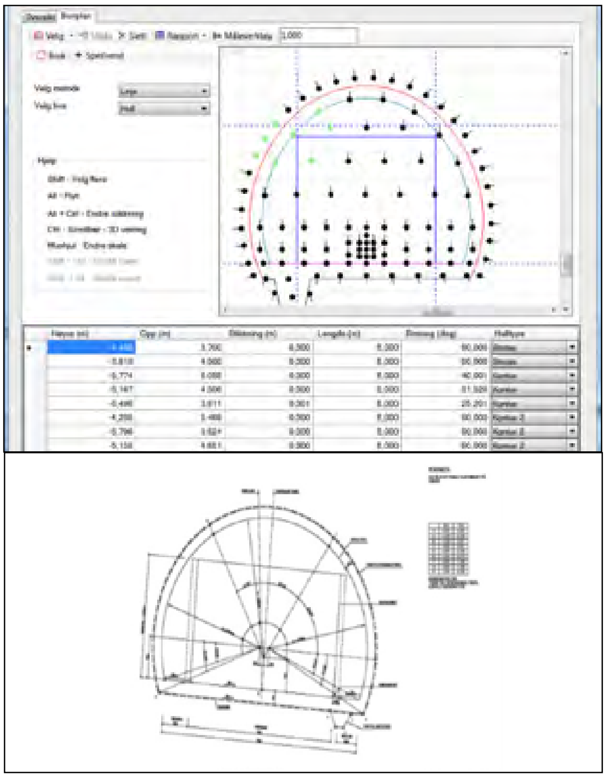


Figure 7 Drillplan and tunnel line is transferred to the drilling jumbo. It will also have defined fixpoints for navigation in the computer.

Figure 8 Standard drill rig jumbo. Three booms and a large capacity on coverage area and drilling efficiency, Automatic positioning feature.

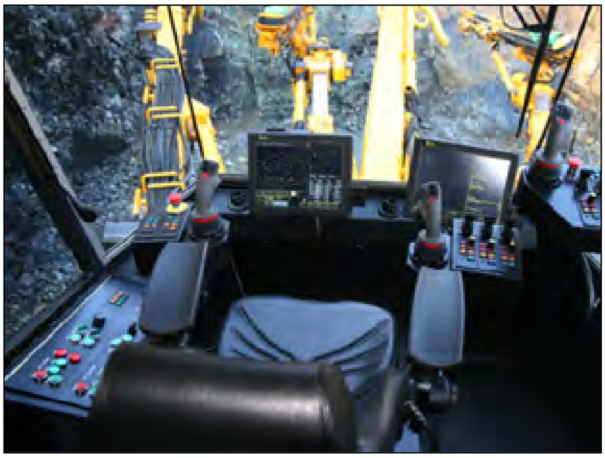


Figure 9 Cabin on the drill jumbo. One or two operators. Comfortable operations with low noise level and comfortable environment.

Figure 10 Special drilling rig for narrow shafts and caverns. Computer guiding system with the same features for computer control as the largest machine.

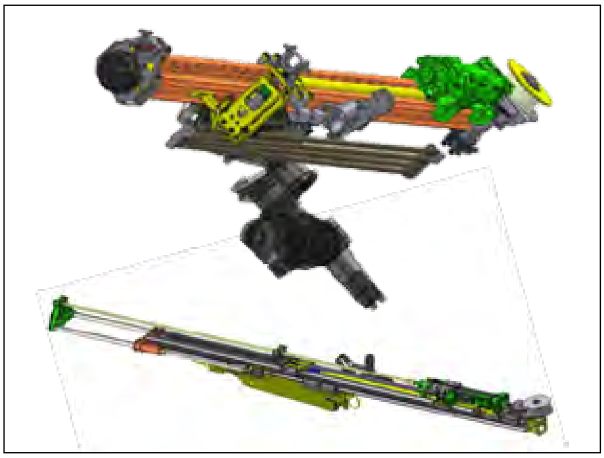


Figure 11 Special equipment, computer controlled, to improve efficiency. The jumbo can be equipped with rod handling system and telescoping feeder. This increas the flexibility of the operation at face. No man has to be in the rod handling area. The telescopic feeder gives a possible solution for drilling bolts in narrow spaces and still drill long rounds

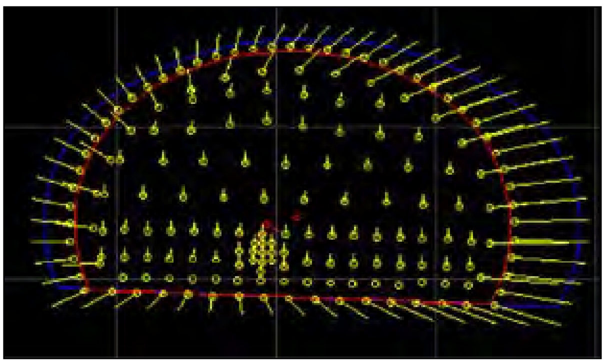


Figure 12 Drill plan automatic adjusted to a curve. You can observe that there is right curve on a highway tunnel (Ramp). Drill plan and drilling can be observed in 3D projections.

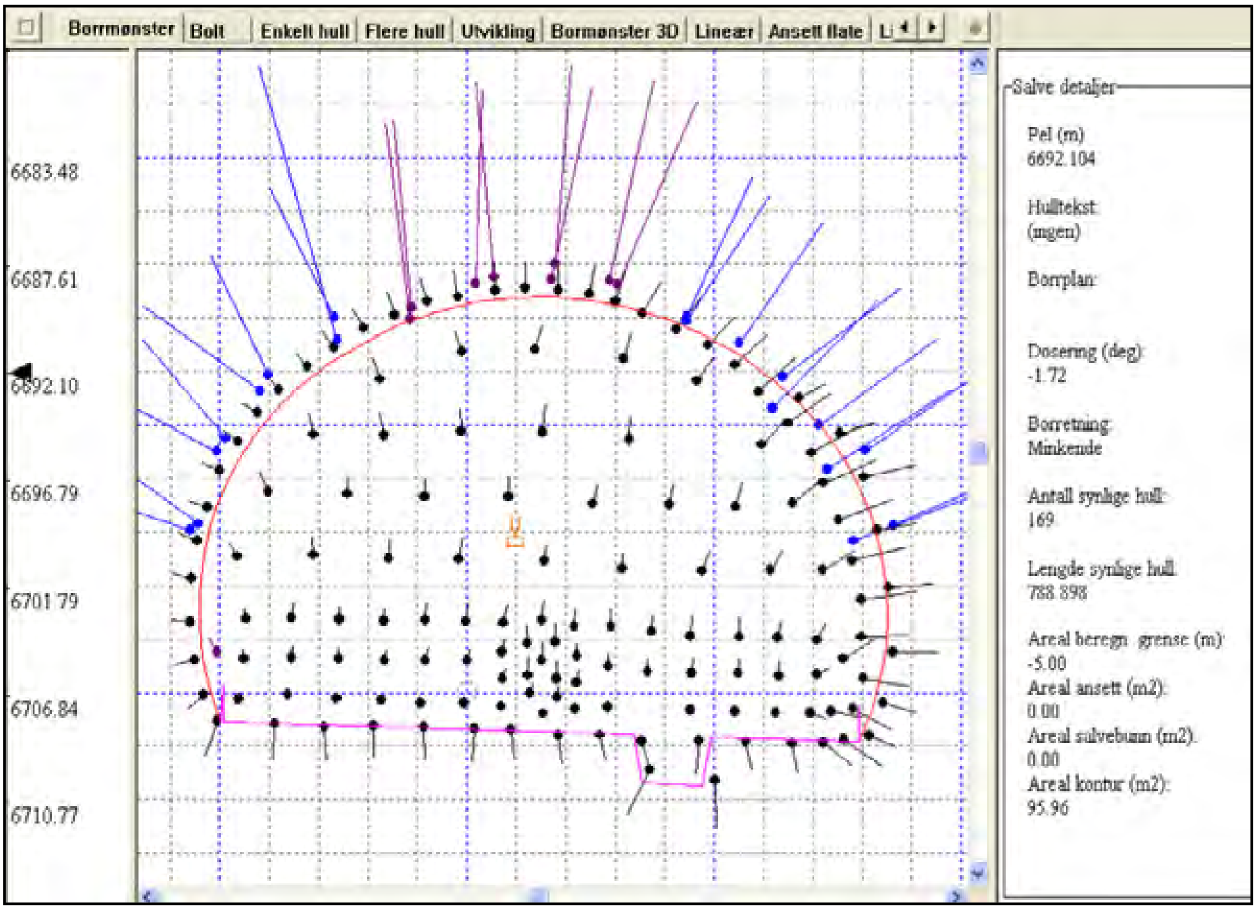


Figure 13 Drill pattern on the drill rig jumbo computer. BeverTeam

2.7 Flexibility of geometry – drill pattern
The drill jumbo computer has all project data including tunnel line contours and drill patterns. The flexibility of drill and blast makes it possible to drill any cross section and our computer tools adapts automatic to change in geometry. Drill patterns will adapt based on a parameter based drill pattern description. This is efficient at construction of ramps bypass, niches etc., and roundabouts

Drill pattern and details on drilling parameters are monitored and are used for drilling performance and quality report. We have all type of holes recorded: blast holes, probe and grouting holes. We make special reports on bolting.

2.8 Rock support bolting documentation
The drill rig drills the holes for rock support and by selecting actual hole types we present a bolt reort in a geo-referenced map including direction and length of the bolts

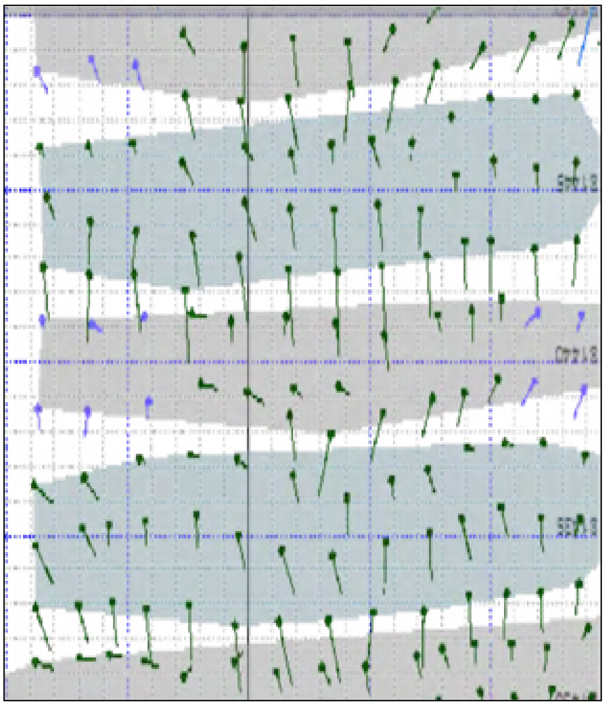


Figure 14 Bolting pattern documented by drill rig

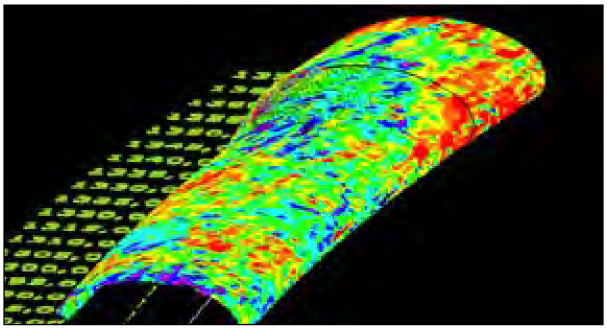


Figure 15 Example of interpretation of MWD data

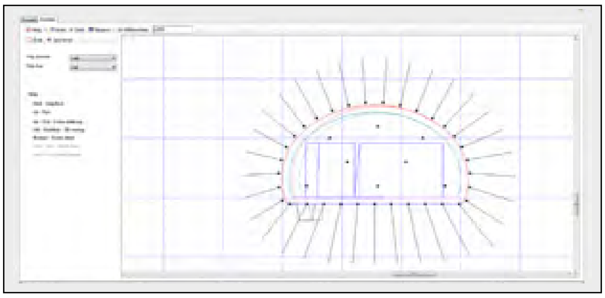


Figure 16 Pre-grouting umbrella. Guidance for the driller to drill accurate the long holes up to 25 meters. In this example we have a highway and a ramp/exit that makes it quite wide.

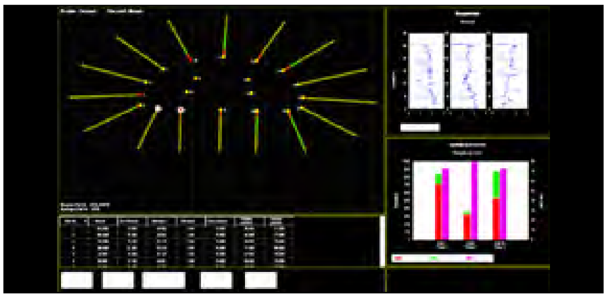


Figure 18 Grouting umbrella. On the grouting the receipts on what is injected into each hole. The MWD interpretation can aslo be displayed so any problem cases can be estimated from the rock estimation (for example faults etc).

3 MWD - LOGGED DRILLING PARAMETERS

MWD is a system for logging of parameters during drilling. The parameters can be used to interpret hardness, fractures and water disturbances when drilling. This makes it possible, based on the performance of the drilling machines to estimate the rock quality. These methods are used on probe drilling, pre-grouting drilling and blast drilling.

3.1 Probe drilling and, Pre-grouting process and equipment

Probe drilling is used for early warning of weakness zones. Typical length is 24 meters and 1 to 4 holes in front of face. Both for probe drilling and grouting umbrellas, MWD logging of data is used.

A computerized grouting system keeps track of amount of grouting material and actual grouting pressure. The MWD analysis from the drilling process gives indication of weakness zones and holes that may be critical.

3.2 Charging of explosives

Slurry based explosives is most commonly used. The charging is done via the drilling jumbo and the delivery system is mounted on the jumbo basket while drill rig is at face. Explosive charge are pre-programmed for the various types of holes (contour, bench, floor) and loaded via a hose controlled feed system. This is a very accurate system giving good documentation of actual charge per hole.

3.3 Shotcrete robot with laser scanner for thickness control

The shotcrete robot is of high capacity and most operators' preference with a cabin on the extended boom. The chassis is based on a standard truck equipped with all needed utilities. As an option is now a laser scanner mounted on the robot that produces documentation of shotcrete thickness.

Figure 17 AMV grouting system for pre-grouting at face. Jumbo drill 24 meter holes in an umbrella pattern typical 30 to 70 holes. The grouting equipment includes 3-4lines with mixing system and high pressure pump. Recipes are automatic mixed and optimized for the penetration into the rock fractures at high pressure.

3.4 Camera for geotechnical documentation – on the drilling jumbo

Norwegian methods imply that the face has to be monitored very careful for optimal rock support. The jumbo can also be equipped with fixed high density camera that takes pictures on face for each round. The pictures are marked with section no and geo-referenced coordinates.

3.5 Automation of inner lining installation

The inner shield of a Norwegian tunnel has the role of frost, water and fire protection. It is a combination of precast concrete elements and on site shield construction. To fasten this shield to the rock normally we use assembly bolts. The position of these bolts may be very accurate (3-5 cm) and a computer guided drilling system is developed for this task.

4 STATEMENT FOR NORWEGIAN TUNNELING METHOD

Modern software tools and automation are integrated in the design, construction and documentation of a modern tunneling project. Machine guidance with georeferenced coordinate system will obtain quality information as "as built" and day to day production support.

Figure 19 Laser scanner for shotcrete. A new system for shotcrete thickness measured is now available. The system is based on a well proven scanner mounted on the shotcrete robot. It will scan before and after the shotcrete session and the distance is calculated. The accuracy is proven to 10 mm in average for 1 m3 of scanning.

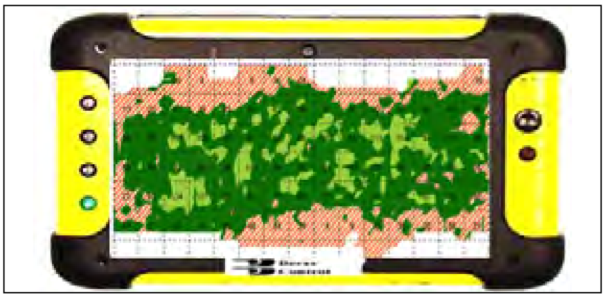


Figure 20 Operator tablet PC operate via local WLAN and gives the operator accurate feedback of thickness. Very efficient training tool and client report on thickness results- Red stripes is less than 40 mm, green is between 40 and 120 mm, light green is more than 120 mm

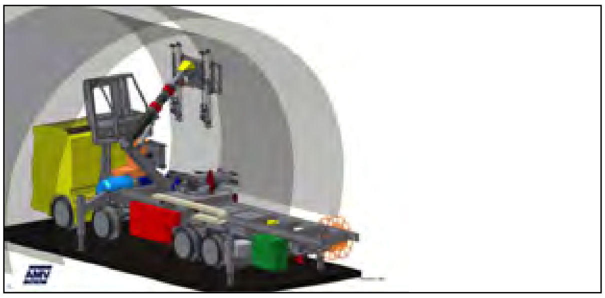


Figure 21 Bolting set automatic for fix the inner lining system. Bolting plan is typical grid of 1.2 meters for a membrane shield. With concrete elements in walls and roof only few bolts are installed. Technical bolts for fans and light is also included.

Figure 22 Installation of inner lining. This picture shows the lining in place, the bolts and reinforcement net before shotcrete layer of 80 mm.

Alta Hydropower project. Work at the damsite.

08. BLASTING TECHNIQUES

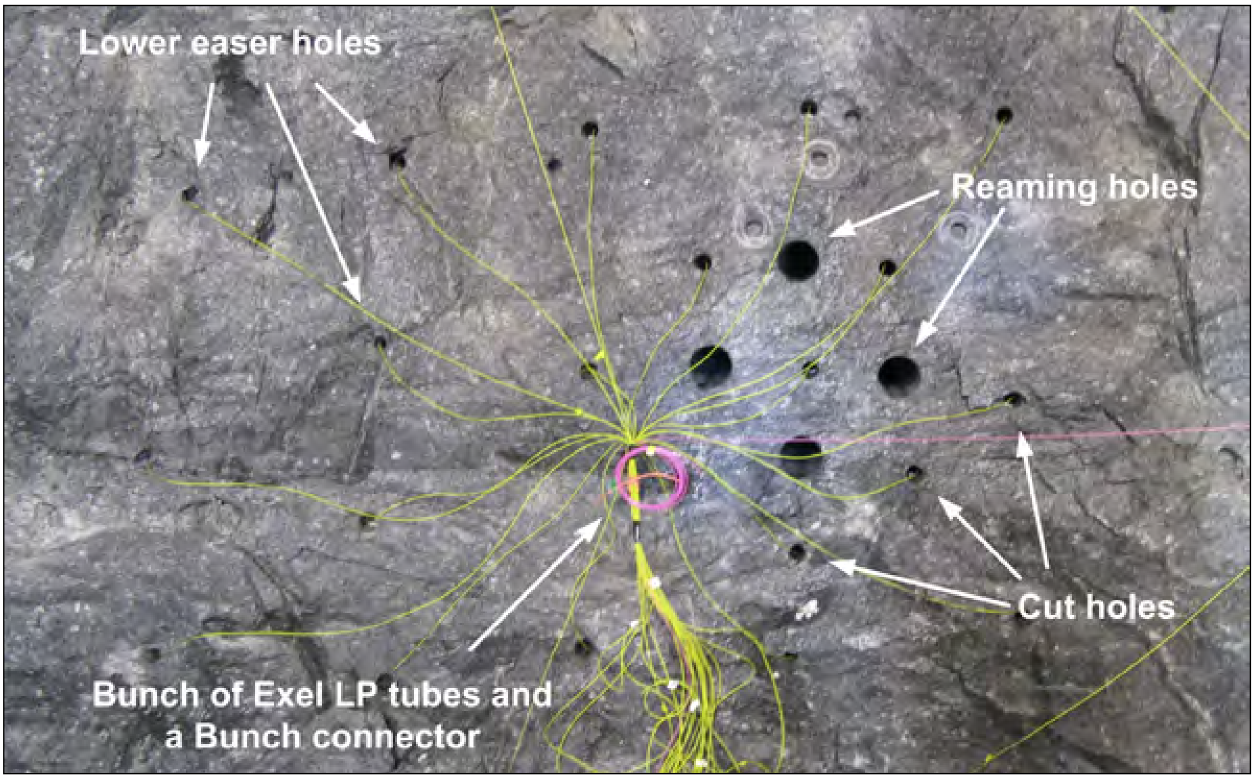
OLSEN, Vegard
RØMCKE, Olaf

1 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.

Part of the blasting history: Mechanical charging equipment Anno 1950



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).

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.

2 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 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).

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.

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

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 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.

3 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 NO_2 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

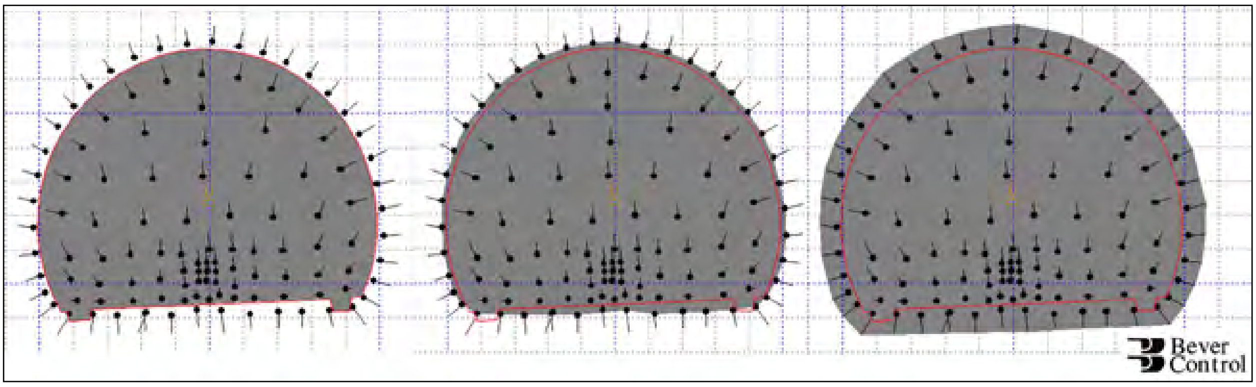


Figure 2 Typical BeverControl drill plan and drill report. Lofast tunnelling project. Left: Theoretical blasting cross section (63.12 m²).Middle: Collaring cross section (68.43 m²). Right: Cross section at the bottom of the holes due to eccentricity (85.03 m²).

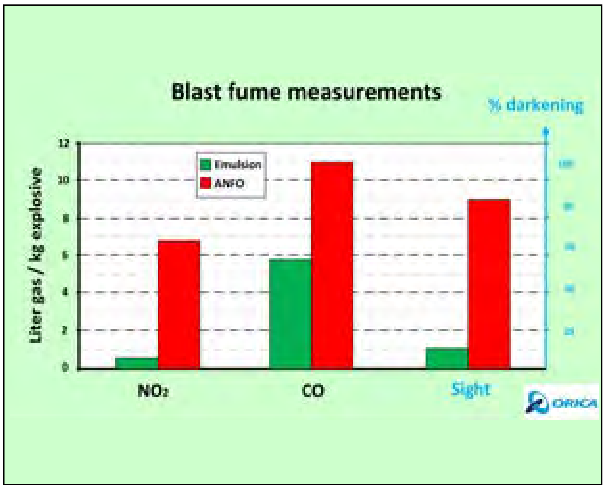


Figure 3 Blast fume measurements in Swedish test tunnel. (Petterson 1995).

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

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

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.

4 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.

5 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.

6 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 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.

7 VIBRATION CONSTRAINS

Urban tunnelling has been very relevant in Norway the latest three decades, and is more and more challenging

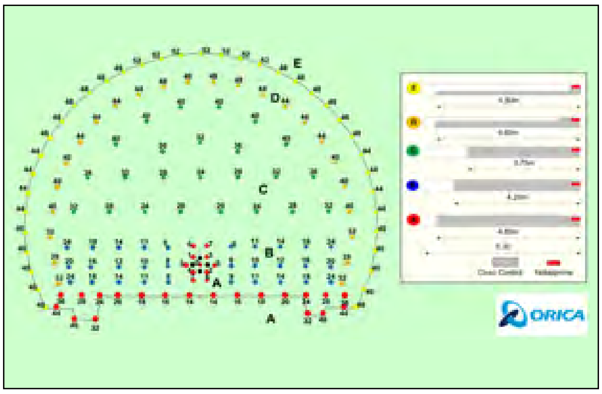


Figure 4 Typical blast design (T9.5 highway tunnel) with parallel large hole cut and string loading in the contour and the inner contour. The colouring of the holes represents different charging. The numbers show the Exel LP intervals. The blast holes are out of scale.

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.

8 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.

9 RESEARCH AND DEVELOPMENT

Since the first tunnelling projects in the late 19^{th} 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

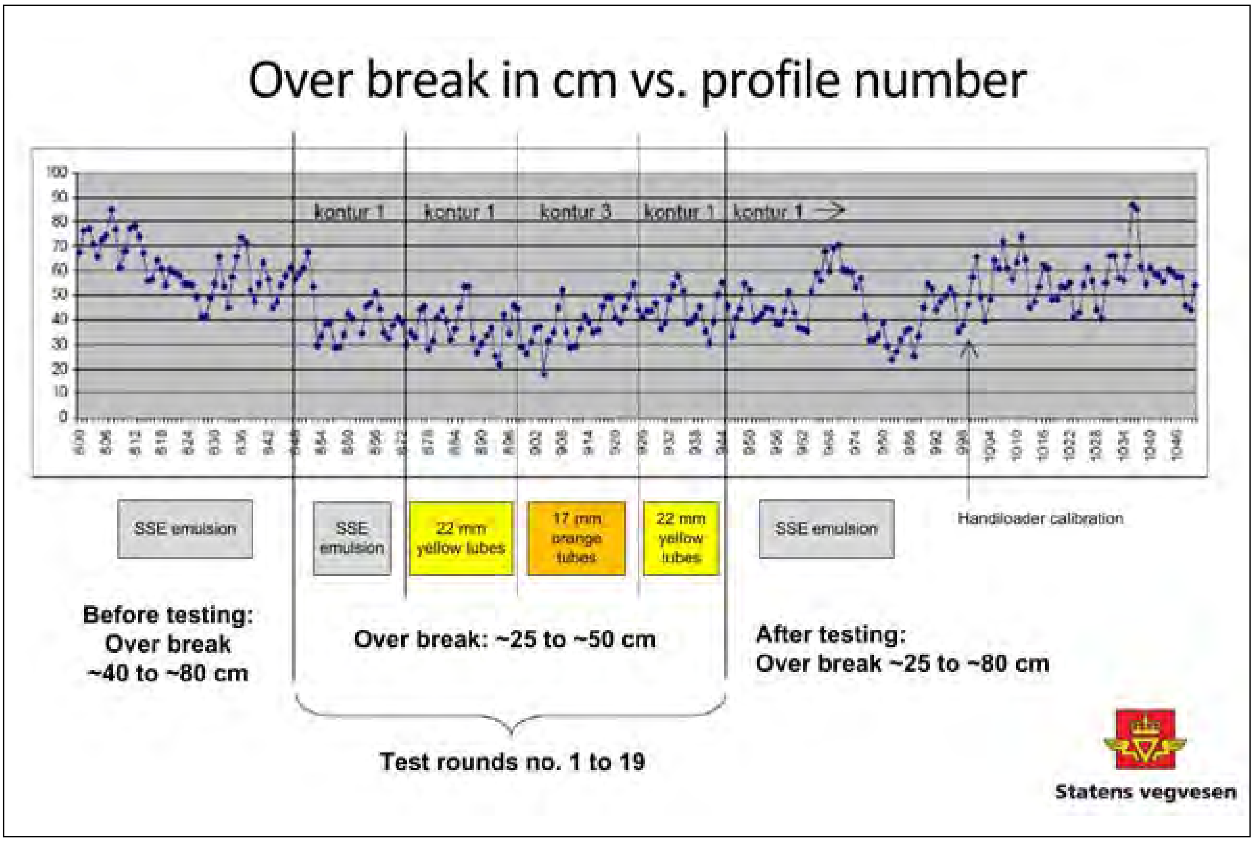


Figure 5 Results from tunnel contour testing in the Eikrem tunnel. (NPRA 2011).

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.

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.

REFERANSER

Fauske, A. 1996. Norwegian urban tunnelling: New equipment, explosives, and blasting techniques. North American Tunnelling '96, Ozdemir (ed.)©. 1996 Balkema, Rotterdam. ISBN 90 5410 802 9.

NFF 2013. Norwegian tunnel statistics. Norwegian Tunnelling Society (NFF).

Rønn P.E. 1997. Conventional Tunnelling, Norwegian University of Science and Technology - NTNU. Doctoral Thesis 1997:6. (In Norwegian).

Zare S. 2005. Drill and Blast Tunnelling, Blast Design, Norwegian University of Science and Technology - NTNU. Doctoral Thesis 2007:129.

Petterson B. 1995. Use of emulsion explosives under ground – an environmental revolution? Norwegian blasting conference 1995. (In Norwegian)

Norwegian Public Road Administration. 2012. Handbook 025 Statens vegvesen.

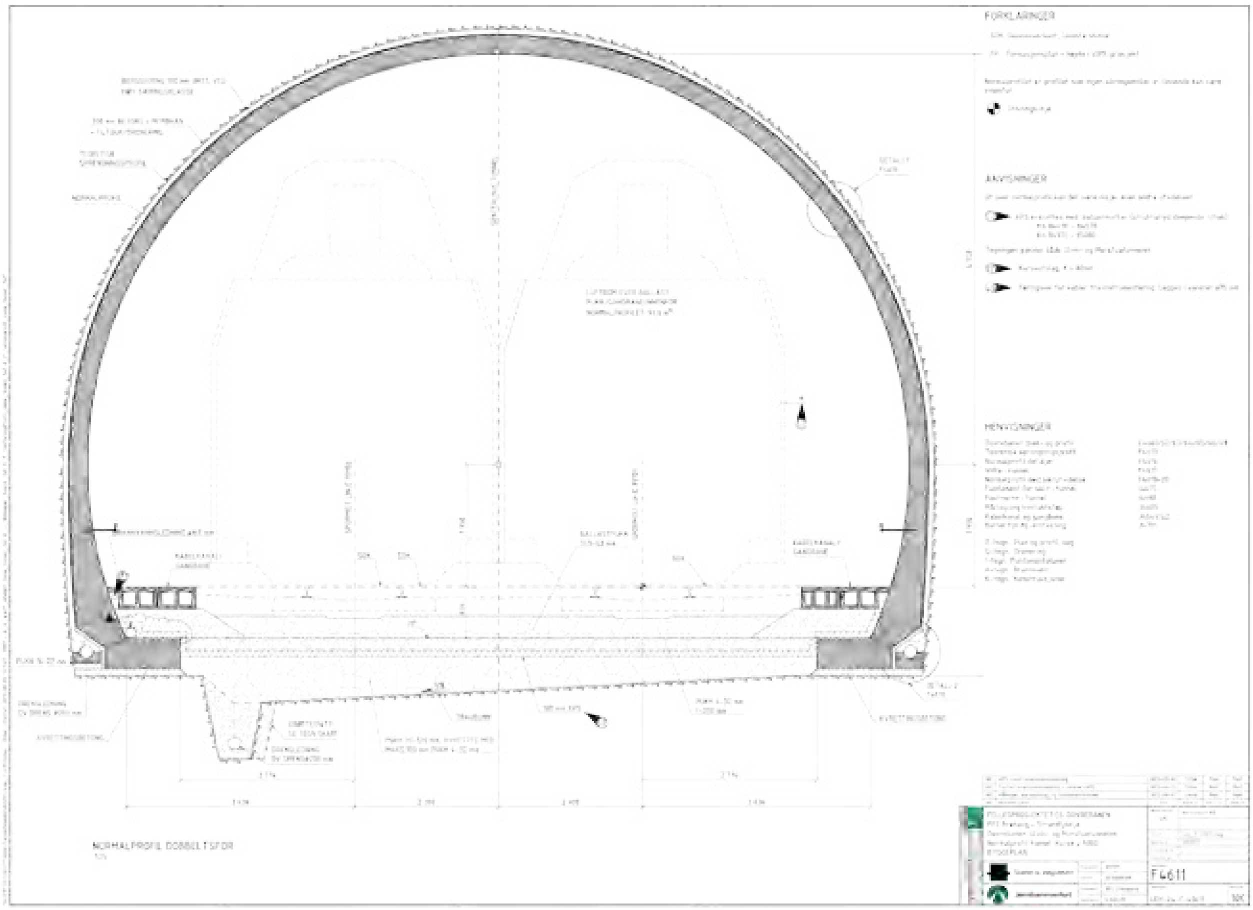
Kirkeby T. 2011. Contour blasting tests with different drilling and charging plans. Norwegian Public Road Administration, VD report 13. December 2011. (In Norwegian)

Grøv E, Kristiansen J, Olsen V. 2011. Norwegian drill & blast tunnelling. Tunnelling Journal June/July 2011 . p.36-39.

The Oset project in Oslo
Underground water processing facility.

09. INNER LINING FOR WATER AND FROST PROTECTION IN A RAILWAY TUNNEL-A CASE HISTORY. A DRAINED SOLUTION WITH MEMBRANE AND CAST-IN-PLACE LINING

AUSLAND, Jan



The design consists of non-reinforced concrete with underlying membrane and drainage layer. Niches and widened areas are reinforced with re-bars B500, 16-20 mm.

The method is well known from Central Europe where it has been used for a number of years. However, this is the first time this method is used in Norwegian railway tunnels and there is great interest in results as regards quality, production time and costs (LCC).

The solution used in Norway differs a little from the methods used in Central Europe in that the solution is in general without reinforcement.

1 TECHNIQUE

The vast majority of the production workers come from Central and Southern Europe and have experience from earlier projects. This is important to ensure good quality installation of the membrane and correct use of the concreting rig.

The concreting rig is sophisticated and is fitted with automated hydraulic systems and motor drive for movement. The rig is 12 metres long and is moved once a day during normal operations.

The concrete formulation contains 2kg polypropylene fibres per cubic metre and the thickness of the construction is minimum 300 mm. In view of the fact that the method is new in Norway, the builder has chosen to equip the construction with instruments at three points in the tunnel. This is done to document the solution, having in mind factors such as the build-up of water pressure in the vault and frost penetration/frost damage in the concrete. The measuring points are positioned in weakness zones and measurements are made of temperature, pressure and tension. As regards the build-up of water pressure, the measurements will have to be made for several years before any final conclusions can be drawn.

2 PROGRESS

Findings from the joint project show that the contractor, using two formworks, will be able to advance 24 metres per day.

In tunnel projects with crosscuts every 1000 metres it should be possible to place 80-100 metres of concrete per week on average after about 10 months of driving. (This depends on the length of the crosscut). This means that production of the lining can be completed in about 18 months in a four-km-long tunnel (55 lm/week), including concreting of portals, crosscuts and technical rooms.

3 COSTS

Preliminary costs shows that the price per running metre for a complete tunnel lining, including niches for technical systems, is approximately 10-15% higher than for concrete elements and about 20-30% higher than for controlled PE foam with 80 mm of concrete. Savings in the operating phase due to reduced maintenance requirements have not been taken into account.

Tunnel ready for installation of gas pipelines.

10. NUMERICAL MODELLING COUPLED TO DISPLACEMENT MONITORING AND STRESS MEASUREMENTS TO DOCUMENT STABILITY

BERGH, Ida Soon Brøther
RISE, Torun
LARSE, Trond

1 INTRODUCTION

In Norwegian underground projects there is a growing understanding for how numerical modelling in combination with recording and observations of displacement and stress measurements can be used as a part of advanced monitoring and stability follow-up. This concept applies to different types of underground projects, including underground mines, hydropower projects, tunnels and underground caverns for different types of facilities.

This article describes how numerical modelling is used in combination with data from measurements and observations. The Norwegian empirical method constitutes a base for this approach, and will not be described further here as it is thoroughly described in other chapters of this publication. The focus is on how the tools of measurements and numerical analysis are utilised within rock mechanics and underground technology in Norway.

The use of such tools has been widely applied for the development of Norwegian tunneling. During the years, in-situ rock stress measurements have been instrumented for the design and construction of unlined pressurised tunnels in HPD-projects, for design of storage caverns and large underground facilities like the Gjøvik hall. Use of other methods like displacement monitoring and numerical modelling applies when judged necessary to produce safe and stable caverns.

2 BACKGROUND

In Norway several projects have utilised the combination of displacement and stress measurements and numerical modelling. The information from the measuring equipment is used to verify, and in some cases calibrate, the numerical models. This process is an important step towards being able to use the numerical model as a predictive tool for stability determination of underground tunnels and caverns, in combination with other empirical and analytical tools.

The reliability of a numerical model is heavily depending on the input data. This means that when preparing for a numerical analysis one should dedicate a relatively large effort into compiling the necessary input data. As a project advances, further data should be collected for verification and/or calibration of the model based on actual data.

Several methods for in-situ stress measurements exist, including hydraulic fracturing and 2D and 3D overcoring. To enable monitoring of stress conditions over time, 2D long term monitoring doorstoppers can be applied. All the described stress measurements give valuable information about the stress distribution in the rock mass.

Hydraulic fracturing is performed in vertical drill holes. The drilling equipment should be capable of obtaining core samples in the vicinity of the planned test sections in order to examine discontinuity orientations and characteristics. An inflatable double packer system (straddle packer), through which a water flow pipe runs, is used to isolate a part of the hole, enabling a test section to be pressurized. To achieve fracturing in the rock mass and not jacking of an existing joint, the test interval must not have any pre-existing discontinuities as cracks or joints. By examination of the core samples it is possible to find test intervals with good rock mass conditions.

Hydraulic fracturing is very useful when planning underground projects or early in the construction phase. The measurements are better suited to be performed in longer and deeper boreholes, than for example 3D overcoring. This means that hydraulic fracturing is well suited for measurements from the surface level towards the deep.

From hydraulic fracturing the minor principal stress and its orientation can be directly found, while the major and intermediate principal stresses have to be theoretically calculated. Knowing the minor principal stress is often enough information when the main purpose is to check whether there are sufficiently high stresses present in the rock mass in the area, as for unlined pressurised tunnels in HPD-projects. For more comprehensive data on in-situ rock stress conditions it is often relevant to consider performing additional stress measurements, for example 3D overcoring, as the excavation process progresses.

For 2D and 3D stress measurements a diamond drill hole with 76 mm outer diameter is drilled to wanted depth [1, 2]. The core is removed and the bottom of the hole is flattened with a special drill bit.

For 2D stress measurements a two dimensional measuring cell (doorstopper) with a strain gauge rosette is installed. An initial reading (zero recording) is done before the cell is ready for overcoring. A new core is drilled with a diamond drill with diameter 76 mm, thus stress relieving the bottom of the borehole. The corresponding strains at the end of the core are recovered by the strain gauge rosette. The core is sampled with a special core catcher, and immediately after removal from the hole the second recording is done. From the recorded strains the stresses in the plane normal to the borehole may be calculated when the elastic parameters determined from laboratory tests are known. The procedure for 2D stress measurements is illustrated in figure 1 (left side).

For 3D the procedure is nearly the same as for 2D, but here the measuring cell is installed in a smaller hole drilled from the bottom of the wider hole. The cell is continuously logging the strain data which is stored in the measure cell. The cell is then overcored by the larger diameter bit, thus stress relieving the core. The strains are recorded by the strain gauge rosettes, and immediately after removing the core from the hole the recorded data is transferred to a computer. When the elastic parameters are determined from laboratory tests the stresses may be calculated. The procedure for 3D stress measurements is illustrated in figure 1 (right side).

2D stress measurements provide information about the stress components in the plane normal to the borehole, while a 3D measurement gives information on the three dimensional stress situation. Such measurements are often conducted when the excavation has commenced, because there are relative strict limitations to the length of the borehole the measurements can be performed in. This is opposed to hydraulic fracturing, which can be performed in much deeper or longer boreholes. It is therefore not uncommon that hydraulic fracturing is used in the planning phase of a project, while 2D and 3D overcoring measurements are applied as the project progresses.

The installation process of long term monitoring doorstoppers is essentially the same as for standard 2D doorstoppers. When drilling the hole standard doorstopper measurements are taken at selected intervals to obtain the initial stresses. The last measurement is taken as close as possible to the selected depth of the long term monitoring doorstopper (normally within 10 cm). Readings are taken of a standard, high quality strain gauge bridge, and continuous monitoring can easily be

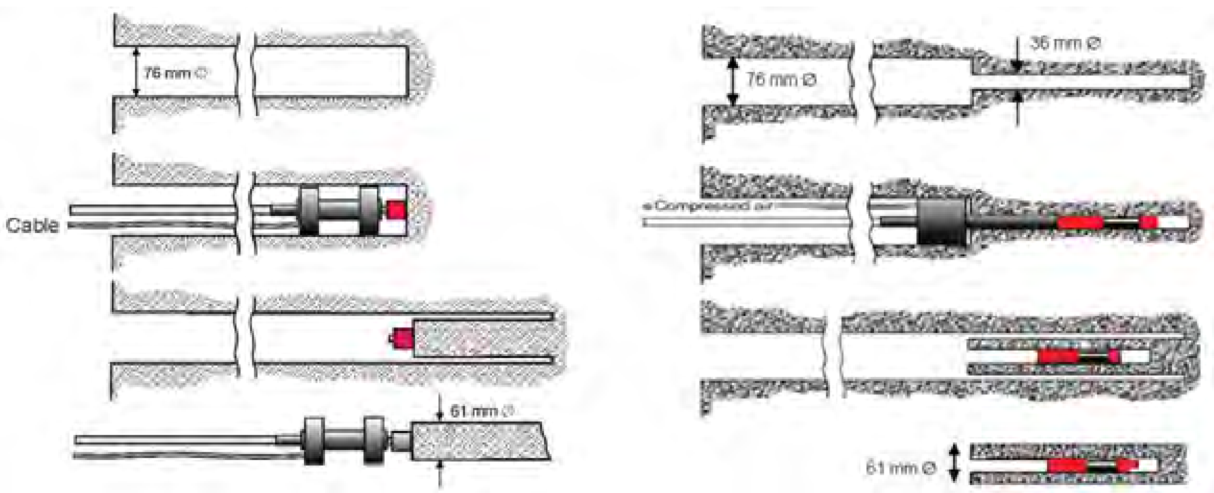


Figure 1 Procedure for 2D stress measurement (left) and 3D stress measurement (right).

established by a permanent connection to a strain gauge bridge. The stress change is calculated using the strain difference from the initial zero reading and the elastic parameters of the rock. The calculated stress change is then compared with the initial stress values obtained from the nearest standard doorstopper measurement.

Laboratory testing of rock samples to obtain more information about the properties of the rock mass are mandatory when doing overcoring measurements. In addition to rock stress measurements, results from laboratory testing are important input in the numerical model. The cores obtained from the measuring holes are used for determination of rock mechanical properties in a laboratory. The parameters that are tested are usually Young's modulus, Poisson's ration, UCS, fracture angle, sonic velocity and density.

To measure displacements in the rock mass different types of extensometers are available on the market today. The picture in figure 2 shows an extensometer before installation. Extensometers with several anchors can monitor displacement between several points along the extensometer, as shown in figure 3. In this way it is possible to identify where in the rock mass a potential displacement could happen. The extensometers are connected to a logging unit, where the displacement data is stored. The unit can be set to log information at given time intervals. From these data it is possible to compile information about displacements in the rock mass between the given anchors as a function of time.

Numerical analysis can be used to analyse the behaviour of rock masses. Rock mass properties and in-situ stress values are used as input parameters along with the geometrical configuration of the relevant underground facility. This makes it possible to use the numerical model to evaluate how the rock mass will behave as excavations progress. Data from stress and displacement measurements are used for verification and calibration of the model. This method of verification increases the quality of the output from the numerical analysis.

3 PRACTICAL APPLICATIONS

This article will look at two cases where the concept of numerical modelling and displacement and stress measurements have been utilised. The first case is the Meråker hydropower project in the middle part of Norway, and the second case is Rana Gruber, which is an iron ore mine in the northern part of Norway. Although Rana Gruber is a mining operation, the concept of the measurements and numerical modelling employed here is still applicable to underground projects for civil applications.

Figure 2 Extensometer before installation.

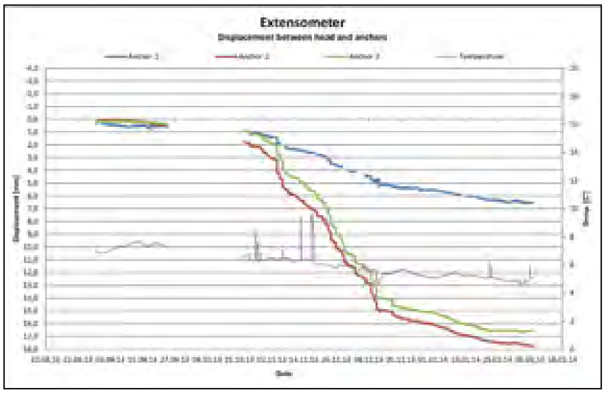


Figure 3 Example of displacement graph from extensometer with three anchors.

Although there are several differences between mining and civil engineering, there are also a multitude of shared factors. One of these factors is the basis of rock mechanics. This article tries to show how the concept of measurements and modelling is applicable in civil engineering as well as within the mining industry.

3.1 Meråker

In Meråker power station 2D stress measurements have been performed at two locations in an attempt to find the cause of fracturing of the floors inside the power station. The measurements took place inside the power station, both locations with approximately 200 m overburden.

Based on results from the 2D stress measurements, the conclusion was that the direction and size of the maximum horizontal stress were in such order that it could cause displacement in the rock mass surrounding the power station.

Numerical analysis of the condition of the power station in order to visualise how the rock mass responds around a selection of the caverns, was also performed. As an input to the numerical model the results from laboratory testing were used, as well as rock mass parameters from geological mapping in the power station. Figure 4 shows some results from the numerical model. The figure shows the results from the stress measurements done in Meråker power station, and how these results verify the results from the model. Based on this, the numerical model is assumed to be realistic.

The result from the numerical modelling shows an expected displacement in the walls due to inward directed stress around the power station of totally 50 mm (figure 5). This was thought to be a displacement which already had taken place in the past, and is today distributed and can be seen as fracturing in the floors of the power station.

To verify this, an extensometer with several anchors was installed inside the power station. The monitoring took place over a 6 month period and showed a maximal displacement of 0.4 mm, which in practical terms is considered as no displacement. This confirmed the assumption that most of the expected displacement had already happened.

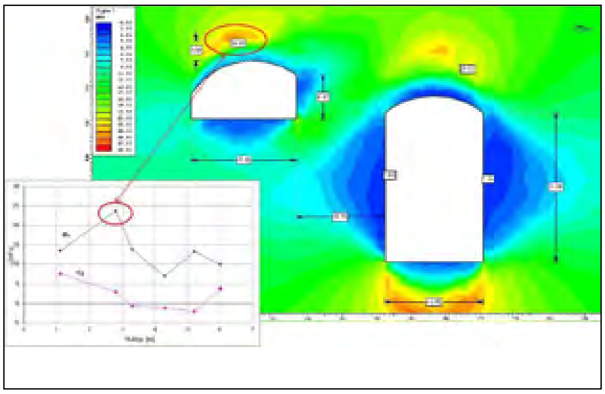


Figure 4 Example that shows verification of results from the model with data from in-situ stress measurement.

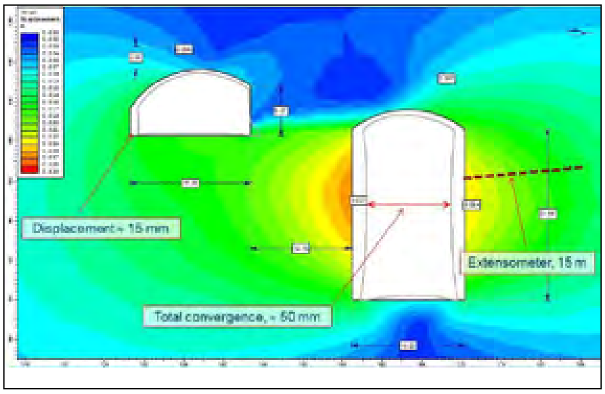


Figure 5 Results from numerical model that shows displacement around underground caverns in a hydropower project.

The results were mainly as expected, since it is about 20 years since the power station was built. Based on empirical knowledge, displacement takes place immediately after stress release and for a relatively short time period after. Displacements are seldom registered long time after stress release. Based on this, the measurements from the extensometer are mainly useful to confirm that the rock mass around the power station is stable in today's situation. As a consequence of this, it was concluded that there is no need for supplementary rock support for the floors inside the power station.

3.2 Rana Gruber

Rana Guber has historically mined iron ore in open pit mines. In 2000, mining with sublevel stoping was started underneath one of the open pits. Ten years later yet another transition in mining method was made, and this time the mining method changed from sublevel stoping to sublevel caving in the underground mine [3].

In Rana Gruber the sublevel caving configuration consists of transportation drifts along the strike of the orebody with production drifts across the orebody from the footwall-side towards the hanging wall-side. The production drifts are drilled and blasted from the hanging wall side on the upper sublevel. Caved material from the hanging wall and the orebody will build up, and blasted ore is then taken out. Continuous caving is very important, so as to prevent the creation of cavities where a sudden collapse could happen.

Stress measurements carried out in the 1970's showed horizontal stresses of up to 20 MPa, which was more than 10 times higher than the theoretical horizontal stress caused by gravity [3, 4]. Data from the long term monitoring doorstoppers shows that the horizontal stresses reach values above 60 MPa as the mining progresses. Observations of rock burst and spalling in the underground mine supports this data.

Since the transition to sublevel caving, more than 5 extensometers and 10 long term monitoring doorstoppers have been installed in Rana Gruber to monitor displacements and stress conditions in the rock mass. There are several extensometers and doorstoppers still active.

In Rana Gruber both 2D and 3D numerical models have been built to evaluate different mining alternatives as well as stability conditions. Several stress measurements have been carried out utilising 2D long term monitoring doorstoppers. These doorstoppers can give long term data about the stress conditions in the surrounding rock.

The numerical model has been continuously updated and calibrated as new monitoring data have become available. Figure 6 shows the calculated stresses from the numerical model and the measured in-situ rock stresses. To this day, only the stress measurements have been used to calibrate the model, but the displacement measurements will also be used for this purpose in the future. The data from the measurements have been of vital importance for the work with the numerical model. Today, rock mass behaviour predicted by the model is observed in several areas of the mine. The numerical model will be one tool utilised when considering mining further towards the deep. The model has also been a useful tool when considering how the rock mass might respond to different mining configurations. Pillar stability has also been an important issue, and there has been performed a numerical analysis of the pillar stability as the mining progresses.

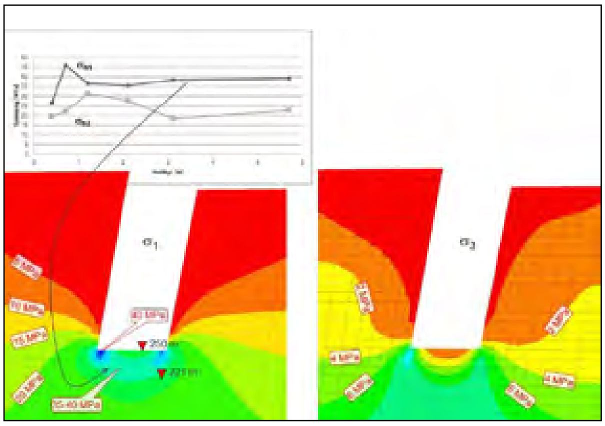


Figure 6 Example that shows results from the numerical model and data from in-situ stress measurement.

A key point in this project has been to evaluate the practical implications of the results from the numerical analysis. When the numerical model shows yielding, for example, it is important to evaluate whether this corresponds to a critical condition in the mine. To be able to say something about what different conditions in the model can mean in the mine, data from observations have been collected and utilised.

4 CONCLUDING REMARKS

Rock stress measurements are in itself useful for obtaining information about the rock mass while planning or building underground. Combined with geological mapping, the rock stress measurements give important information that can be utilised when developing a numerical model. In reality, this is essential to obtain a realistic numerical model.

As described for the two cases in this article, realistic numerical models are very useful for understanding the rock mass behaviour. The presence of rock stresses is very important when constructing caverns underground, and is one of the guideline parameters when planning how to develop infrastructure underground. When planning a new project, whether it is in developed or undeveloped areas, it is important with information about the rock mass and the rock stress conditions. This is especially true in connection with underground projects in densely populated or developed areas. In such areas it is of key importance to have a thorough knowledge of the rock mass properties, because stability issues or other challenges can have very high consequences. This applies to all types of underground projects, ranging from mines and tunnels to parking facilities and underground sports facilities.

Advanced monitoring and stability follow-up using long term monitoring doorstoppers and extensometers can provide continuous information about the conditions of the rock mass. As mention earlier in this article, a numerical model should be verified, and in some cases calibrated, based on data from supplementing measurements. In this way, the numerical model can provide for relevant and useful information about the rock mass in order to plan design, location and expected need for rock support.

5 ACKNOWLEDGEMENTS

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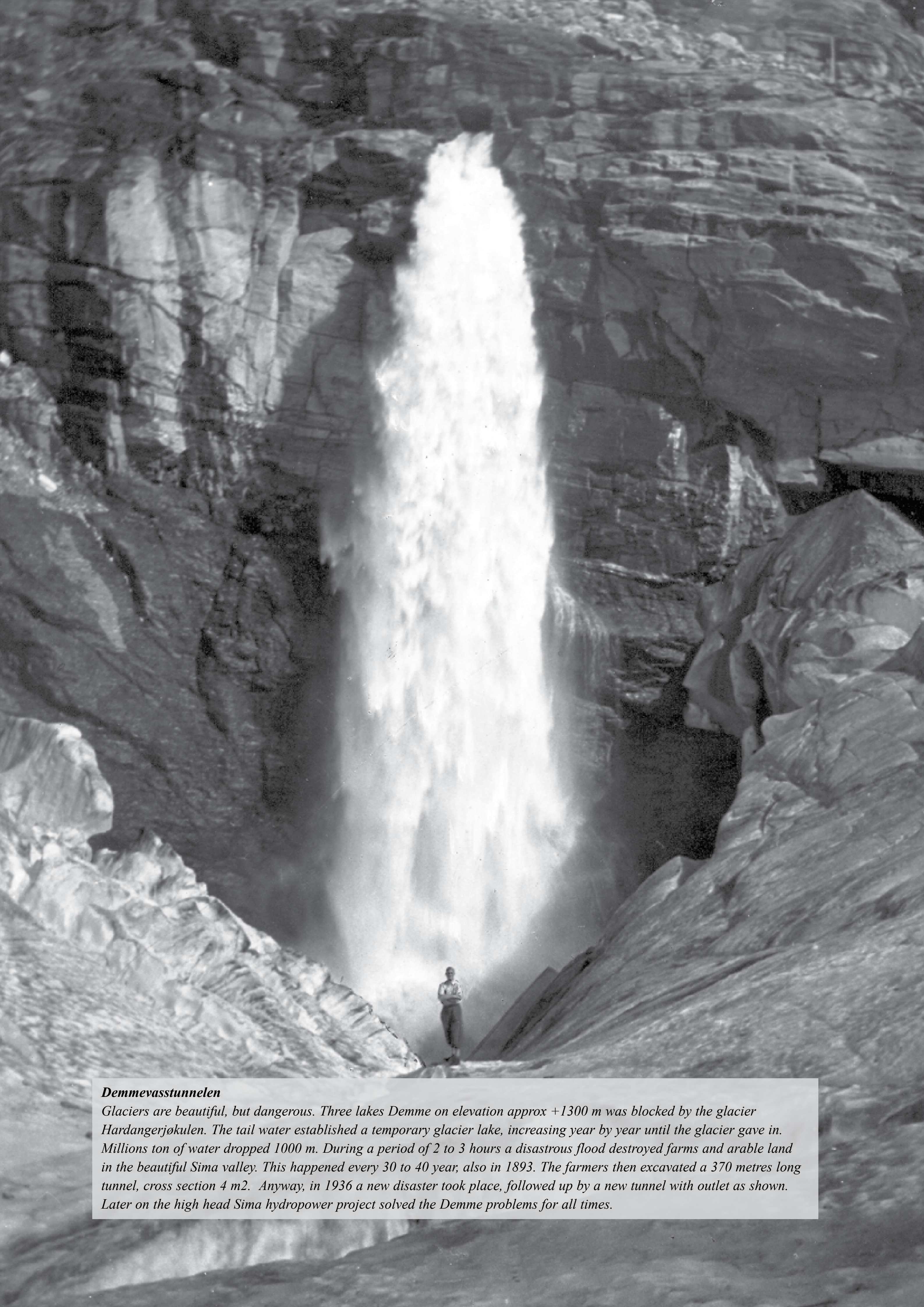
REFERENCES

[1] Trinh, N.Q., Larsen, T., Sand, S.N. and Myrvang, A., 2010. Stress measurement and rock excavation at Skaland mine, Norway. In: Rocks Stress and Earthquakes, Taylor and Francis Group, London

[2] Trinh, N.Q., Larsen, T.E., Myrvang, A.M. and Lu, M., 2013. Stress measurement and its use at Rana Gruber, Norway. In: 6th International Symposium on In-Situ Rock Stress, August 2013, Sendai, Japan.

[3] Sand, N.S. and Trinh, Q.N., 2011. Rana Gruber rock mechanics challenges connection with the establishment of a new main level and transition to new mining method. In: The 45th U.S. Rock Mechanics/ Geomechanics Symposium, June 2011, San Francisco, California, USA.

[4] Trinh, Q.N., Myrvang, A. and Sand, N.S., 2010. Rock excavation and support for a crusher hall at Rana Gruber, Norway. In: The 44th U.S. Rock Mechanics Symposium, June 2010, Salt Lake City, Utah, USA.



Demmevasstunnelen

Glaciers are beautiful, but dangerous. Three lakes Demme on elevation approx +1300 m was blocked by the glacier Hardangerjøkulen. The tail water established a temporary glacier lake, increasing year by year until the glacier gave in. Millions ton of water dropped 1000 m. During a period of 2 to 3 hours a disastrous flood destroyed farms and arable land in the beautiful Sima valley. This happened every 30 to 40 year, also in 1893. The farmers then excavated a 370 metres long tunnel, cross section 4 m2. Anyway, in 1936 a new disaster took place, followed up by a new tunnel with outlet as shown. Later on the high head Sima hydropower project solved the Demme problems for all times.

11. COORDINATION - A WAY TO ENHANCED COOPERATION IN UNDERGROUND PROJECTS

HENNING, Jan Eirik

Contractual provisions concerning coordination are intended to lay a foundation for good collaborative relationships, to build trust between the parties and to create inspiration for the further development of the project. Coordination provisions may be made the basis of all types of contracts independent of contract form and type of work or job. The scheme and prerequisites for coordination must be made clear in the tender documents for each individual contract.

1 THE OBJECT OF COORDINATION

The intention of coordination provisions is:

- to improve coordination between the parties
- to build trust between the parties
- to contribute to a shared understanding of the contract
- to help inspire innovation and development
- to help all the parties work together towards agreed goals, based on common expertise and experience.

2 UNDERLYING PREREQUISITES FOR COORDINATION

- Competent and motivated employees of all parties
- Openness
- Equality between parties
- Respect for each other
- Predictability
- Establishment of agreed procedures for personal conflicts
- Establishment of agreed procedures in the event of disagreements or disputes of a contractual nature
- Clarification of roles and responsibility
- Establishment of procedures for coordination which create trust and inspiration for development

3 BASIC CONDITIONS:

The tender competition and selection of bidder

The tender competition is conducted in accordance with standard procedures, until the selection of bidder and conclusion of contract. For public owners, this means that the whole tender competition is carried out in accordance with the Norwegian Public Procurement Act and associated regulations.

4 COORDINATION AND DEVELOPMENT PHASE

To attain a shared understanding of the contract, a common objective and agreed coordination procedures which inspire innovation and development, to the benefit of all parties, the scheme for the coordination and development phase is of major importance for all further work.

The coordination and development phase should at the very least include:

Getting to know each other

- Establishing how to involve all parties (owner, contractor, consultants, performing entities etc)
- Developing common coordination procedures, including demands and expectations on the parties
- Clarifying organisation, roles and responsibility
- Clarifying procedures for conflict resolution, for issues relating to both contract and personnel
- Clarifying procedures for technical quality control, quality assurance and HSE (health, safety and the environment)
- Clarifying routines and requirements regarding documentation, reporting etc
- Developing a shared understanding of the contract
- Developing a shared understanding and objective of the construction job

- Reviewing and optimising progress
- Analysing and determining specific development potential and development targets

The above should be specified and detailed to the extent necessary, such that all parties find the result of the process useful, and also helpful for the implementation process. The coordination and development phase should be carried out without any change to basic conditions, responsibility and risk in relation to the preconditions in the tender competition.

Sufficient time should therefore be set aside for the coordination and development phase after the conclusion of contract to allow a thorough review of all aspects of the project and the contract work.

The time necessary for the coordination and development phase must be determined after an assessment of each project, taking into account its size, complexity and development potential. As a starting point, a period of four weeks may be set aside. However, if the parties agree, the coordination and development phase can either be ended and the implementation phase started, or it can be extended. Although the coordination and development phase should basically be completed before the works are commenced, it will also carry on as a continuous process after the start of the contract work.

The intention behind having a coordination and development phase laid down in the contract, and setting aside sufficient time before the start of the works, is that involved parties will, together, be able to establish coordination procedures and a scheme for further development of the project by combining good suggestions from the contractor, consultants and owner based on common expertise and experience.

The coordination and development phase is terminated after all relevant factors relating to the contract have been reviewed and the parties have acquired a shared understanding of what is to be achieved by organising and carrying out the project as described.The expenses that consultants and contractors incur by participating in the coordination and development period will be paid in accordance with rates given in the tender, based on the preconditions in the tender documents.

5 DEVELOPMENT OF THE PROJECT AFTER THE WORKS HAVE STARTED

After the coordination and development phase has ended and the works have started, it will still possible for the parties to propose new solutions.

6 SHARING OF COST SAVINGS RESULTING FROM DEVELOPMENTS

The contract provisions must include provisions indicating the parties' sharing of cost savings. Cost savings are achieved through agreed alternative solutions which are put into practice. Improvements are usually remunerated by 50% of the net saving obtained in relation to the contract sum.

7 EXPERIENCE

Experience of the effects of coordination is gradually accumulating. The Norwegian Public Roads Administration has worked with this technique, among others, for a number of years and in view of the positive effects it has seen, it has now incorporated standard provisions in its tender documents which state that coordination prior to start-up of the contract work is to be implemented. The following may be mentioned as examples of positive effects:

- Collaboration between owner and contractor has been good from day one.
- The coordination phase led to a considerable amount of time being spent together, which has resulted in good communication.
- A coordination phase prior to construction start-up has led to well-functioning collaboration in the implementation phase.
- Strong commitment results in many proposals for developments and new solutions, but many basic conditions have led to proposed solutions not being put into practice.

8 CHALLENGES

- Being open to the process and ideas that crop up
- Low threshold for making suggestions
- Involvement
- Competent, motivated and suitable people from all parties
- Equality between parties
- Decision-making authority
- Unambiguous and clear framework for the project such as design guide, zoning plan, handbooks etc.
- Completion of the coordination phase prior to construction start-up

9 FINAL EVALUATION

The parties should, together, draw up a final report in which all aspects of the contract work and coordination are discussed.

Leirfossene Hydropower Station, Trondheim under construction 2008. Courtesy: AF Gruppen

12. NORWEGIAN CONTRACT PRACTICE SUITABLE ALSO FOR DEALING WITH UNEXPECTED GEOLOGICAL CONDITIONS

GRØV, Eivind

1 INTRODUCTION

Tunnelling contracts may vary from one country to another. The contracting practice that is normally applied for such as road tunnels being built for the Norwegian Public Roads Administration (NPRA), is in line with the general practice for tunnel construction and underground excavation in Norway. The technical specifications and contract conditions are based on the application of conventional drill and blast method, which is currently the excavation method being dominantly applied in Norway, although TBM-tunnelling also is taken care of by the same set of contract specifications. Furthermore, the bid – build model is used together with the unit price contract for tunnel projects.

In Norwegian tunnelling the rock mass is considered as a construction material with certain specific capacities and capabilities. Thus, strengthening is done by means and methods as determined by assessment of the rock mass quality as encountered at the tunnel face. This implies that the actual quantities may differ from the contract's bill of quantities, and a flexible contract is required to enable adjustment of the actual quantities related to rock mass grouting ahead of the tunnel face and support measures. In association with the variations of quantities a clause on how to adjust the construction time accordingly has been developed.

Sub-surface projects are inevitably related to the handling of uncertainties, and risks associated with geological conditions are usually significant. In subsea tunnelling the probability of so-called 'unexpected geological conditions' are naturally higher than for other tunnelling projects, and the consequences associated with such occurrences are also more severe, both for the owner and the contractor. There is a variety of contract types being applied world wide for tunneling projects, many of which though is based on a lump sum – fixed price concept with its peculiar way of distributing the risk associated with the geological conditions.

In Norway, about 40 subsea tunnels in rock has been constructed over the last 25 years with the use of unit price or unit rate type of contracts, with only one exception. The unit price contract shares the risk between the owner and the contractor as the owner retains the risk for the geological conditions while the contractor has the risk for the efficiency of his performance of the contract works. This paper has chosen to focus on sub-sea tunnels to exemplify and demonstrate the suitability of the Norwegian contract type though similar situations may have appeared in on-shore projects also putting the contract to a test.

This paper presents three cases of subsea road tunnels that cast light on the suitability of different types of contracts. Firstly, the 3.8km Godøy tunnel (the deepest subsea road tunnel in the world at the time) where a grouting effort much larger than planned was needed to prevent unexpected and unacceptable water inflow. The unit price contract proved to be suitable even with a more than 3 times increased quantity of grouting. Secondly, the 2.0km Bjorøy tunnel was expected to be a straightforward project suitable for a fixed price contract. An exceptional occurrence of a sand zone caused serious delays and cost overruns, resulting in litigation. The contractor had to take the loss. It is discussed whether a unit price contract would have been more suitable in this case. Finally, the Oslofjord tunnel is discussed. During construction an unexpected erosion channel filled with soil was detected by probe drilling at the tunnel's deepest point, i.e. under 120m water pressure. The necessary measures were dealt with outside the unit price contract regulations.

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 activities. 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 unexpected conditions occur in the form of 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 both unit price and fixed price contracts, may prove to be more suitable than fully fixed price contracts.

Requirements needed to establish suitable tunnelling contracts and the need for project specific and balanced allocation of risks are outlined. The paper is based on previous articles by Olav T. Blindheim and the author on the particular types of tunnel contracts and modified to include new developments.

2 GEOLOGICAL CONDITIONS IN NORWAY, THE BASIS FOR THE CONTRACT PHILOSOPHY

Norway forms part of a Precambrian shield. Two thirds of the country is covered by Precambrian rocks (older than 600 million years), with different types of gneiss dominating. Other rock types from this era are granites, gabbros and quartzite. Approximately one third of the country is covered by rocks of Cambrian - Silurian age. The greater part of these rocks are metamorphosed, but to a varying degree. Rock types such as gneisses, mica-schists and greenstones as well as sandstones, shales, limestones and other unmetamorphosed rocks form a mountain range, which runs through the central parts of the country. In the geologically unique Oslo region, the rock mass is partly made up of unmetamorphic Cambro-Silurian shales and limestones and partly of Permian intrusive and extrusive rocks. These are the youngest rocks.

Throughout Scandinavia, general rock mass conditions are favourable for underground utilisation. The geological setting is dominated by igneous rock types together with metamorphic rocks of various types and origins. The host rock is more or less intersected by weak zones, which may have an intense tectonic jointing, hydrothermal alteration, or be faulted and sheared, constituting significant weaknesses in the rock and making the rock mass far from homogenous. These conditions may call for grouting to reach a desired level of rock mass impermeabilization.

The host rock in Scandinavia general varies from 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 centimetres to tens of metres.

In Norway, the hydrogeological situation is dominated by a high, groundwater level, also in the rock mass. This situation is both favourable and unfavourable for rock tunnelling. One advantage of a groundwater regime surrounding an underground structure is that it provides a natural gradient acting towards the opening allowing the utilisation of unlined storage facilities. On the other hand, one disadvantage of such saturated conditions is the risk that the tunnelling activity may disturb the groundwater situation, thus imposing the potential of adverse impact on surface structures and biotypes.

The rock itself is in practical terms impervious, and the porosity is negligible. This means that the permeability of a sound rock specimen is likely in the range of 10^{-11} or 10^{-12} m/sec. Individual joints may have a permeability factor in the range of 10^{-5} to 10^{-6} m/sec. The rock mass is consequently a very typical jointed aquifer where water occurs along the most permeable discontinuities. The permeability of the rock mass consisting of competent rock and joints may typically be in the range of 10^{-7} to 10^{-8} m/sec. This implies that the most conductive zones in the rock mass must be identified and treated. Further, an appropriate solution must be determined to deal with such zones and to prevent the tunnel imposing an adverse situation in the groundwater regime, in terms of a lowered groundwater. Such an approach may not be restricted to one single measure to be executed, rather as it may consist of a series of various measures and actions to be taken during the tunnelling works. In such circumstances, to enable an effective progress the following capabilities of the rock mass can be applicable:

- It's self supporting capacity, i.e. the ability of the rock mass to maintain stability even after being subject to cavities being made, man made or natural.
- It's impermeable nature, i.e. the actual permeability of the rock mass and associated discontinuities may vary from 10^{-5}m/sec to 10^{-12}m/sec.
- It's stress induced confinement, the in-situ stress situation varying from stress released rock bodies through a pure gravitational stress situation to stresses originated by long tectonic history of the rock mass.
- It's thermal capacity, i.e. the capacity to store energy over significant amount of time.

Taking into account that "mother nature" has produced a material that is far from a perfect material, and that the rock mass may have a set of imperfections, it is most common that the construction process involves various techniques and methods to assist the design of a construction material that suits its purpose. Hard rock includes a wide variety of rock mass qualities, from competent rock mass in one end of the scale to totally disintegrated and exceptionally poor rock in the other acting merely as a soil, the latter being typically associated with weakness zones. Such weakness zones can be faults with crushed material that may be more or less altered to clay, and could be several tens of meters wide calling for extraordinary measures and tunnelling methods. Swelling clay minerals is often found in such zones. Thus, it is clear that hard rock encompasses a lot more than straight forward tunnelling, and the challenge above all is to establish a tunnelling system that is capable of handling this varying construction material, still utilising its capacities where applicable. Discontinuities represent special challenges as regards stability and proper handling to ensure a safe and sound tunnelling process.

In this context the importance is obvious as far as having a contract concept and philosophy that enables this ever changing ground to be appropriately dealt with.

3 MAPPING OF GEOLOGICAL CONDITIONS AT THE TUNNEL FACE

Crucial for the determination of the rock support is well mapped and documented rock mass conditions. 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.

4 PARTICULAR RISKS ELEMENTS OF SUBSEA TUNNELS

Characteristic risks of subsea tunnels are connected to:

- limited knowledge about ground conditions due to practical difficulties and higher costs for site investigations prior to construction;
- often poor rock quality in fault zones etc, typically occurring on the deepest points with the least rock overburden;
- the inherent hazard of tunnelling below the sea, with an inexhaustible supply of water should a collapse of the overburden occur.

Also for tunnelling on land, many of the practical problems involved are of a similar nature, although the risk profile may be lower as the consequences of something going wrong may be less. The lessons learned may still have application for many tunnelling projects.

5 USEFUL EXPERIENCE BASE

In Norway, more than 40 subsea tunnels in rock have been built during the last 25 years, almost 30 of which are road tunnels open to the public. The other tunnels are for pipelines or for water or sewage transport. They have all been excavated by open face drilling and blasting with the risks during construction reduced by extensive use of probe drilling and pre-grouting. The dominating contract type has been the typical unit price contract developed during decades of construction of underground hydropower projects, especially in the 1960's to 1980's. These contracts have a successful track record. They involve the application of tendered unit prices and pre-set regulation mechanisms for completion time according to performed quantities, thus

Tunnel	Godøy	Bjorøy	Oslofjord
Year completed	1989	1996	2000
Main rock type	Gneiss	Gneiss	Gneiss
Cross section, m^2	55	53	79
Total length, km	3.8	2.0	7.2
Subsea section, km, (%)	1.5 (39)	0.5 (26)	2.0 (28)
Lowest level below sea, m	155 *1)*	82	134
Min. rock cover, m	33	30	32 *2)*

1) World record for subsea road tunnels at the time
2) Except in eroded fault zone

Table 1: Main project data

maintaining the bulk of the risk for the ground conditions with the owner. Accordingly, the responsibility for the site investigations naturally rests with the owner, often being a public body. The principles of these contracts are discussed in more detail later in this paper.

6 CASE STORIES TO DEMONSTRATE CONTRACT DETAILS

Table 1 presents an overview of the main data for the project cases that are discussed. All were excavated as one tube tunnels (for 2 or 3 lanes) by drilling and blasting. The subsea section of the tunnels varied from 26 to 39% of the total length.

As for site investigations, the procedures for excavation, rock support, probe drilling and pre-grouting follow the guidelines set by the Norwegian Public Roads Administration (Ref. 1, and as described in a number of publications (Ref 4). Accordingly, all tunnels were constructed emphasising systematic probe drilling and pre-grouting as needed ahead of the tunnel face. This includes typically 3-5 percussive probe holes of 30m length with min. 8m overlap (see Figure 1) and pre-grouting cone-shaped fans of 15-25m length (see Figure 2), which constitute the most effective risk reducing measures during construction. Table 3 presents expected and applied measures for rock support and ground treatment.

Tunnel	Godøy		Bjorøy		Oslofjord	
Quantities	Expected	Resulting	Expected	Resulting	Expected	Resulting
Rock bolts, pc/m tunnel	2.5	5.9	4.1	2.8	3.9	3.6
Shotcrete, m^3/m tunnel	0.9	0.35	1.1	1.3	1.2	1.6
Concrete % of tunnel	1.3	0	2.5	1.5	7.4	1.6
Percussion probe drilling, m/m tunnel	3.5	6.0	3.6	2.2	2.9	6.9
Core probe drilling, m/m tunnel	0	0	0	0.1	0.4	0.2
Grouting, kg/m tunnel	83	265	51	670	220	360 *1)*
Water inflow, l/min/km	300	260	300	400	300	220

1) 30% consumed in the eroded fault zone

Table 3: Rock support, probe drilling, grouting and water inflow

7 GODØY TUNNEL

The Godøy tunnel on the west coast of Norway was built by the same private owner and contractor as the two Ålesund tunnels, which have been described in a number of publications[5]. The project was financed by private loans, without any guarantees from the government, to be repaid by the income from toll fees. The county contributed capitalised subsidies for the ferry that was replaced.

The geological conditions were well known, in particular considering the experience from the nearby Ålesund tunnels. Normal investigations without core drilling were performed. The owner had support from the local Public Roads Administration and experienced advisors[6]. The contract was a normal unit price contract with reimbursement according to tendered unit prices and regulation of construction time using the normal system of pre-set 'standard capacities'.

During excavation, the rock mass quality proved to be as expected or even better with respect to stability. Table 3 shows that more rock bolts, but less sprayed concrete were used than the estimate. No cast-in-place concrete lining was necessary. However, a joint set with open character, striking NE-SW along the coast, required far more pre-grouting than expected. This was possibly due to relatively recent tectonic movements, resulting in joint apertures from 1-2mm up to 25-30mm. The actual grouting quantity was 3.2 times the tendered quantity with respect to dry weight and took almost 6 times longer time to produce. The phenomenon of open joints is foreseeable, but the needed extensive effort was not foreseen. The potential inflows were not related to rock cover or type or lack of soil overburden, in contrast to the Ålesund tunnels.

Despite the extensive pre-grouting needed, the tunnel was opened for traffic after 16 months construction, actually 5 months ahead of schedule. This was mainly due to the reduced time for rock support and the efficient approach to changing between excavation, stabilisation and grouting. According to contract regulations, the construction time was adjusted allowing for the increased grouting time. The grouting efforts increased

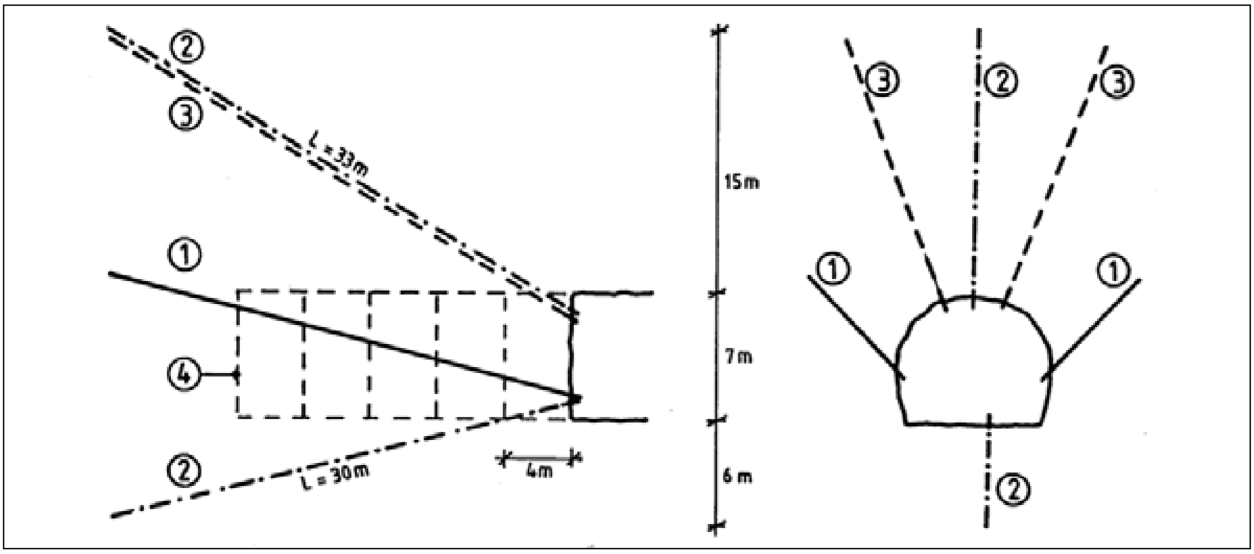


Fig. 1: Probe drilling pattern for the Godøy tunnel
(1) Probe holes at all tunnel sections below sea or loose deposits
(2) Additional holes where weak zones were expected from the seismic surveying
(3) Alternative upper holes in sections with rock cover less than 25m
(4) Min 8m overlap of probe holes drilled for each 5th blasting round

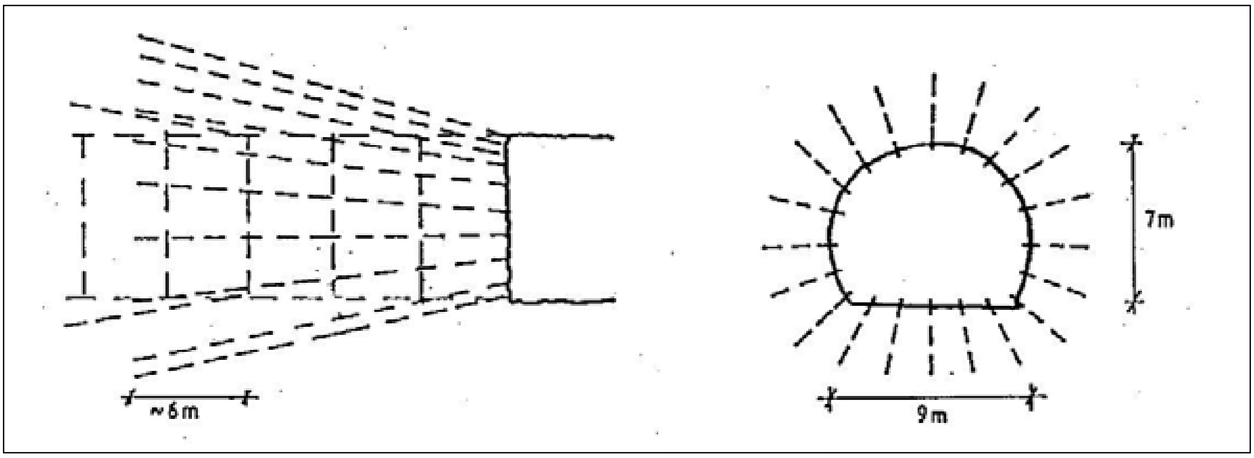


Fig. 2. Pre-grouting drill-holes for the Godøy tunnel, full cone of 18m long holes. Overlap usually minimum 6m

the tunnel cost by approx. 5% compared to the estimate based on expected quantities.

It may or may not have been possible to determine the unusually open character of the joints by long directional core drilling prior to construction. The cost of 1-2 such holes could have reached 2-4% of the tunnel cost or more. Even if the especially open character of the joint set had been realised, this would only have changed the estimated quantity in the Bill of Quantity (BoQ). This could have resulted in somewhat lower unit prices, but would not have offset the increased site investigation costs. In this way, the extent of the site investigations was cost effective.

All necessary activities were covered by quantities and corresponding unit prices and 'standard capacities'. No conflict or litigation resulted. Thus, the contract worked according to intentions, i.e. the owner kept the basic risk for the geological conditions. This is typical for most of the subsea tunnels built in Norway so far.

8 BJORØY TUNNEL

The Bjorøy tunnel is located on the west coast of Norway outside Bergen. Its purpose was to establish a fixed link between an archipelago of islands (with only 400 inhabitants) and the mainland. The plans were promoted by private initiative. There was no public financing available at the time, except by toll fees and the normal contribution from the county from capitalised ferry subsidies. This triggered the interest of one of the large contractors, who already had extensive experience with subsea tunnels, and a proposal for a fixed price project was developed.

The local Public Roads Administration prepared the detailed design and managed the contract. Regarding the geological conditions, nothing unusual was expected by either party, except a relatively high level of conventional rock support measures, i.e. rock bolts and sprayed concrete. The Pre-Cambrian gneisses and the metamorphic rocks of the Caledonian mountain-range formation were known from many projects in the district.

The contract was designed as a fixed price contract as a result of the project circumstances. Political approval was given on the clear condition that no funding could be allocated from the regular 'queue' of scheduled public projects. Accordingly, the contract contained very specific clauses regarding risk allocation. The contract sum constituted the full reimbursement to the contractor, for excavation, rock support, grouting, other civil works and installations, including any variation in quantity or change of conditions. The owner should not be entitled to a cost reduction if the content of the works proved to be of less volume than expected. Only specific change orders from the owner would lead to adjustments of the contract sum, this could apply for example if a required change in standards emerged during the construction. It was also stated that the contractor had full responsibility for any further site investigations, and that all risks in connection to the ground conditions were his, including the rock cover. In contrast to the normal unit price contract, which keeps practically all risks for the ground conditions on the part of the owner, this contract allocated all risks for the ground conditions to the contractor.

During excavation, it was found that the rock mass was generally more jointed than anticipated, and a significantly increased pre-grouting became necessary. Conditions worsened, and the main problem occurred when the tunnel reached a fault zone between the Pre-Cambrian and the Caledonian units, see Figure 3. Here a zone of Jurassic sandstone and breccia occurred, partly completely disintegrated. This sub-vertical sheet-shaped zone intersected the tunnel at an angle of 30-35 degrees giving very poor conditions over a 45m section, with a 4m wide zone of completely loose sand under 80m water pressure. The fine grained sand mixed with water gushed under pressure out of the probe holes, unless they were blocked off (Ref. 7,8).

The contractor called in external advisors to form an 'expert group' to advice on a safe tunnelling method. After 3 months preparation, the zone was tunnelled through by applying a specially developed method, combining extensive pre-grouting with microcement for sealing and compaction as well as attempts to chemical penetration grouting. The excavation through the zone was done by short rounds and extensive use of fore-bolting ('spiling'). Technically this method was successful.

The contractor completed the tunnel 10 months behind schedule, out of which about half may be directly allocated to the central sand-zone, the rest to the very poor ground adjacent to it. The contractor claimed additional reimbursement amounting to 60% of the fixed price for the adverse and unexpected ground conditions, which he characterised as 'extreme' and not compatible with the implied and applied method of 'rock tunnelling'. A settlement was not reached, and the case went to court. The first court level agreed with the contractor on basis of the exceptional ground conditions, as the two appointed co-judges with technical background voting down the professional judge. This verdict was appealed,

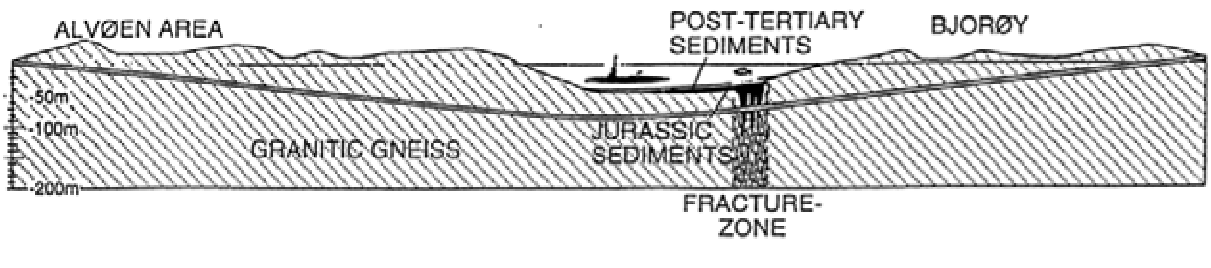


Fig. 3: Longitudinal section of the Bjorøy tunnel

and the next court level basically agreed with the owner (with the dissent of one of the two co-judges) primarily on the grounds that the contract was very specific about risk allocation, and that both parties were experienced.

The Jurassic zone, occurring in this area and in this manner, was indeed unforeseen and was characterised by geologists as 'sensational'. Based on the geological interpretations, the occurrence may be considered as a rare case of 'unforeseeable' conditions. The exceptionally poor ground conditions might have been determined by long directional core holes, but this is not certain. Due to the overall confidence in the geological conditions, none of the parties wanted to pay for such investigations, which would have had a significant cost.

Paradoxically, the exceptional use of a fixed price contract coincided with the occurrence of exceptional ground conditions. A fixed price contract was applied due to the limited economic foundation following the small traffic base. If the tunnel had served a larger population, and public financing had been available, it is possible that a normal unit price contract would have been used. The generally poorer conditions would then have been handled routinely by the regulations for increased quantities. The exceptional sand-zone would likely have been taken out of contract and paid according to a special agreement. By this the owner, and the public through the toll fees, would have taken most of the extra cost.

As it was, the contract can be said to have worked according to intentions from the owner's point of view, but not from the contractor's, who took a heavy loss. Agreeing with the courts' verdicts or not, it appears to the authors that in hindsight the 'all-inclusive' risk allocation to the contractor was not suitable for this project. A less solid contractor might have gone bankrupt in the process and left the tunnel uncompleted. The owner would then have had two options: either to complete the tunnel at the expense of delaying other public projects or leave it uncompleted. The latter option might have been politically difficult. This demonstrates that in any case the owner is exposed to significant risks, although the fixed price contract was intended to minimise his exposure to risks.

9 OSLOFJORD TUNNEL

The Oslofjord tunnel is located about 40km south of Oslo, linking the main highway system on each side of the fjord. It was partly financed by the government, partly by capitalised ferry subsidies from the counties and partly by county guaranteed loans to be repaid by the toll income.

The tunnel passes through different kinds of gneisses and under the fjord it crosses three major fault zones along the N-S striking Oslo graben. The geological conditions were well known in general. In addition, extensive pre-construction site investigations were performed including directional core drilling through the western fault zone at tunnel elevation, see Table 2. This fault zone was the one expected to be worst; in Figure 4 it is marked as the 'Oslofjord zone'.

The owner (the local Public Road Administration) organised a project management team with experienced key staff and advisors to follow-up the construction. The contractor had extensive underground experience. The contract was a normal unit price contract with time regulation according to performed quantities.

During construction, less rock bolts were used than expected, more sprayed concrete (Ref. 9), but significantly less cast-in-place concrete lining, see Table 3. As tunnelling advanced from land out below the sea, percussive probe drilling ahead of the face revealed that the expected major fault zone on the western side of the fjord had been eroded to an un-expected depth. This included a section of 30m found not to be passable by normal open face excavation, as it contained loose soil deposits under 120m water pressure. The tunnelling had started from a steep access tunnel close to shore (below Hurum, seeFigure 4); the purpose of this access was to deal with potential problems in this fault zone early in the project time schedule.

Preparatory grouting followed by ground freezing was considered to be the best method to enable safe passing of the zone. The technical handling of the zone was done in full co-operation by the parties, supported by external advisers in a 'task force' (Ref. 10). The freezing took a longer time than anticipated at first, but the tunnel was completed on schedule because the zone was encountered at an early stage of the project.

The geological conditions as encountered and the methodology applied in the 'freezing zone' were acknowledged as being outside the scope of the contract. A special agreement was made for the bypassing of the zone with a deeper lying temporary transport tunnel. It was later possible to utilize this by-pass as the pumping buffer reservoir, replacing the designed reservoir.

The unit price contract was not intended to cover such conditions, as the depth of the deep erosion was not foreseen in spite of extensive site investigations. Freezing was not included in the BoQ. The phenomenon of deep erosion was indeed foreseeable, and was

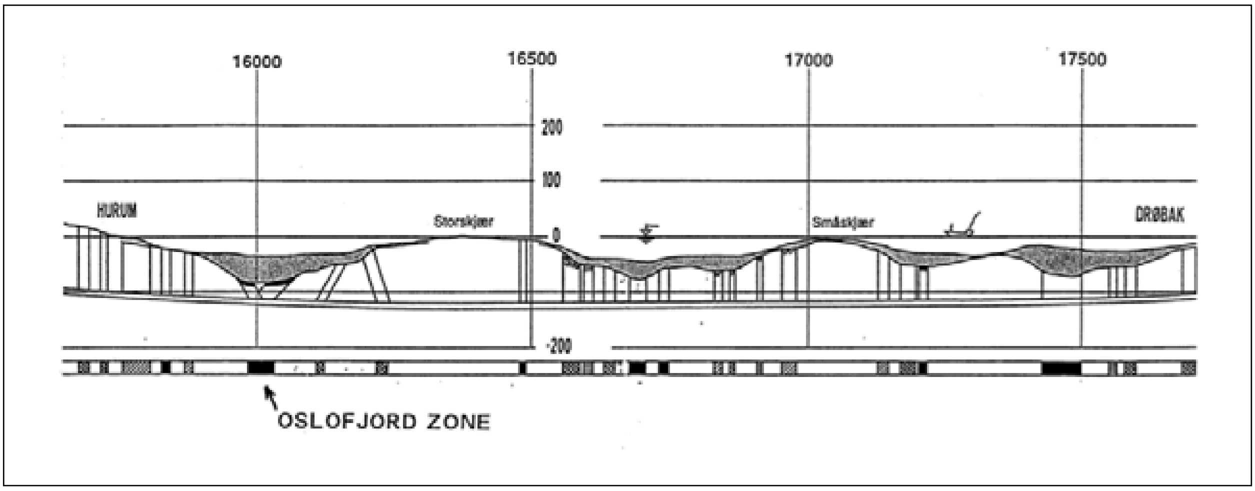


Fig. 4: Weakness zones at the Oslofjord tunnel, looking north. The 'Oslofjord zone' proved to be deeply eroded

the very reason for targeted investigations. Still, the interpretations proved to be inaccurate. In retrospect, the extent of site investigations prior to construction was sufficient, but the directional core drilling should have been targeted above the tunnel alignment in order to verify the rock cover.

It was demonstrated that it is possible for experienced parties to make an agreement outside the contract to deal with such unforeseen circumstances. However, after the successful technical completion, litigation still followed. This was due to disagreement about the payment for crossing the zone and extra costs for the transport through the by-pass tunnel. The cost increase, which remains to be finally settled, is in the order of 10-20% of the expected tunnel cost. The actual costs related to the litigation itself were significant. The verdict criticised the parties for not trying harder to settle the economic aspects out of court.

10 LESSONS LEARNED

From the above case stories, these lessons may apply to other tunnelling as well:

- 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 suitable.

- 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.

11 SOME REMARKS ON THE CHOSEN EXAMPLES

The Norwegian tunnelling contract philosophy is based on using the rock mass as a construction material, therefore a thorough geological assessment is essential. The aim of making a safe tunnel for both the construction and operation phases relies on the geologist's ability to prescribe the correct measures at the tunnel face during a hectic construction

We have experienced unfortunate incidents in Norway proving that rock fall accidents and tunnel collapses may also happen in tunnels opened to traffic. This has forced the industry to review in detail whether or not the Norwegian tunnelling concept is good enough.

Some modifications have been enforced, but basically the industry sticks to proven tunnelling philosophy. An effect the review has had, is improving the specification to set apart time in each drill and blast cycle to do the necessary geological registrations, so it is known exactly what is behind the visible surface before it is covered with concrete, water and frost insulation, fire insulation etc. Another modification is classifying the rock mass in order to prescribed rock support measures designed and verified to be sufficient in the actual cases.

The Norwegian Public Roads Administration and other main tunnel owners have used the drill and blast method for Norwegian tunnels for many years, and are confident with the method. The drill and blast method has proven effective in the hard rock environment. It is also a very flexible method, allowing changes in the tunnel shape or diameter easily, and giving unrestricted access to the tunnel face which is a useful facility to be able to deal with changing ground conditions. The use of TBM method were typically applied for the hydro power development and only in a couple of instances in the 1980'ies for road tunnels. Conventional drill and blast is likely to continue to dominate road tunnelling in Norway, whilst the Norwegian Railway authorities is currently viewing TBM as an alternative for some of their future tunnel projects.

Keeping in the mind the tremendous amount of tunnel projects being executed in Norway every year the number of cases that requires court decision is few. This proves that the Norwegian contract philosophy works and that it holds various elements enabling a fair risk sharing principle allocating the various risks to the parties best fit to handle them. Taking also into account that the general cost level for Norwegian tunnels, such as for road and railway tunnels, are significantly lower than tunnelling costs arrived at in other countries having similar ground conditions. Some articles even propose that the cost level of such Norwegian tunnels is one tenth of the cost level in central Europe. Last but not least, hard rock tunnelling may not always mean good rock mass conditions and therefore it is a need to have a contract system that has the flexibility to work appropriately in a frequently changing ground..

12 CONTRACT PHILOSOPHY

As mentioned above the tunnelling concept regards the rock mass as being a construction material and pre-construction assessments of the expected rock becomes extremely important. These pre-construction assessments are used for the determination of predicted quantities for such as rock support measures and grouting measures. These assessments are also forming the basis for the prediction of the construction time. It is important that the contracts include quantities which are representing the best estimate and not being subject to tactics from any of the parties in the contract.

During the construction phase it is important that both the client and the contractor have competent people at the tunnel face to determine the support measures needed and assess the rock mass conditions ahead of the tunnel face. [1] This latter being especially relevant for the execution of the grouting works. 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. Cooperation at site is a matter that is as importance, including in short such as; respect for the different roles and values, experienced professionals participating in the decision making, conflicts being solved at the construction site.

Planning for the construction implies that contingency and precaution need to be included in the contract to handle conditions that are expected and foreseen, and also unexpected conditions. Unforeseen conditions occurring during the construction need to be dealt with specifically in any way as to our knowledge no contract type includes sufficient flexibility to cope with the unforeseen. Handling of truly unforeseen conditions have been experienced in some Norwegian sub sea tunnelling projects. The Norwegian contract practice applies the unit price contract, which places the risk for varying ground conditions on the owner. Though the contract does normally not include different prices depending on rock quality encountered, separate unit prices are almost always included for such measures as reducing the blasting length to half the normal, or dividing the tunnel face into various sections. [2]

Before contracting, the ground conditions are carefully mapped and a geological report is being compiled which later becomes part of the contract documents. This report is not a geological base line report. The geological report describes what have been recorded in terms of factual data and in addition a part of it presents a description of the expected ground conditions, that is an interpretation of the factual data. This gives the owner a basis for assessments on measures and quantities to be specified in the contracts. It also provides the contactor an information basis for his own judgement of the ground conditions that he may use for calculations and planning purposes. Predictability is a key issue and it is important that information is provided from one phase of the project to the next and that nothing is getting lost in the process.

In the bill of quantities, the owner is specifying various support methods and stipulates the quantities, trying as accurately as possible to stipulate the amounts that we expect will be carried out, as this gives the least surprises, and the truest picture of the scope of work. The contractor is paid according to the actual amounts carried out. [3]

Important objectives of the Norwegian philosophy are safety under construction and safe tunnels in operation, all the time bearing costs in mind. It therefore becomes important that:

1. the risk sharing principles are clear and fair
2. the tunnel contracts are flexible in handling changing ground conditions
3. the owner and the contractor cooperate to achieve the common goals in the project.

Tunnelling is a construction method that is inevitably associated with risk and risk taking. No matter the level of investigations done prior to commencing tunnelling the risk cannot be nil, there will always be some remaining risk in a tunnel project. The remaining risk level is project specific and is associated with a number of different aspects. These aspects include some of the following:

- Geological conditions and the geological complexity
- Localisation of the project
- Project requirements and project complexity
- Project organisation, owners competence and experience
- Financing and project solidity
- Tunnelling method including support and water tightness

To mention a few aspects, and the project owner has to identify the acceptable level of risk for the project.

13 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. The risk sharing 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 problem.
- 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.

By this, the owner keeps the risk of increased cost if the ground conditions prove to be worse than expected; after all he has chosen the site location. He will also earn the savings if the conditions are better than expected. The contractor keeps the risk of his 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, saving money by this and increasing 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. 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 an unexpected and unforeseeable geological feature occurs. This system, its development and application was described by Kleivan (Ref. 11) who coined the term NoTCoS – the Norwegian Tunnelling Contract System. In Figure 5 it is illustrated how this risk allocation produces the lowest cost possible in average for a number of projects.

14 CHARACTERISTICS OF UNIT PRICE CONTRACTS

The typical unit price contract in Norway is characterised by the following:

- 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. Traditionally it also contained interpretations, not being limited to factual data, but this practise 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.

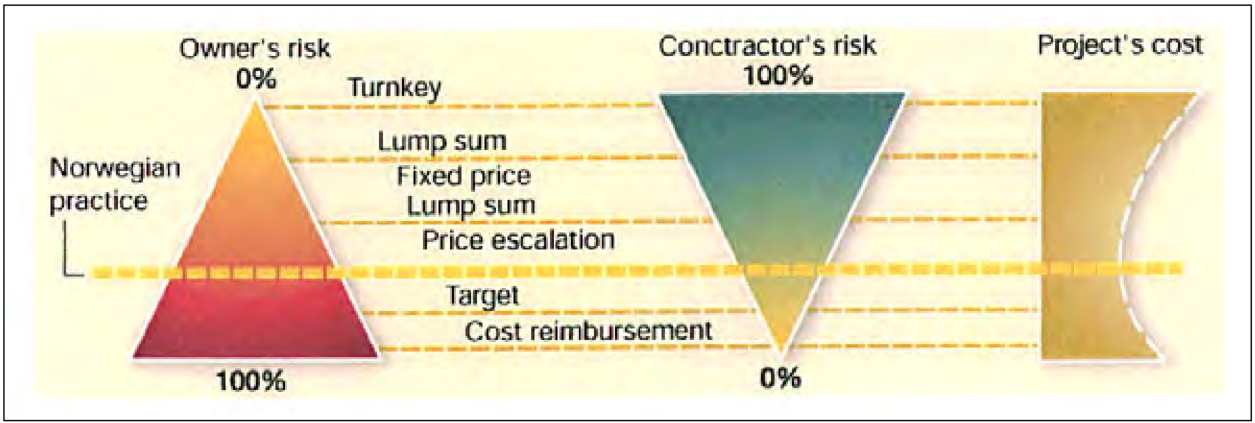


Fig. 5: Risk allocation principles (Kleivan, Ref. 11)

- 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 these have been set by negotiations between the contractors' and owners' organisations. 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.

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

- Experienced owners and contractors. The parties must be experienced with underground works and the site management teams from both sides must have the necessary authority to take decisions, allowing technical and contractual issues to be solved at site as they occur. This requires respect for each others tasks.

- 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.

- 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 utilise his special skills. Some owners do not ask for, or even allow, alternative solutions to be introduced. However, this is not due to the type of contract, but to how it is applied.

15 CONTRACT CLAUSES TO TACKLE VARYING QUANTITIES AND CONSTRUCTION TIME FOR EXPLORATORY DRILLING AND 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.

To tackle this, the contract has defined "*the 100 % rule*" in the specification describing support [3]:

- *The unit prices apply even if the sum of actual quantities differs from the bill of quantities 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 geological conditions, 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 [2]. 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 m^3/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 bear is consider 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 1 below 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.

16 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 contractor 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 criticised, 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.

17 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. [2]

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.

18 EXPERIENCE WITH 'ADJUSTABLE FIXED PRICE' CONTRACTS

Modified or 'adjustable fixed price' contracts may be a suitable tool to balance the risk in another manner than by unit price contracts. By clauses and incentives specifically adapted to the project, it may be possible to combine a reasonable predictability of the total cost for the owner, with the option for the contractor to accept larger risks, but at the same time having the potential for good profits.

Such contracts were developed and applied on two recent subsea road tunnels in the Nordic countries, i.e. the Hvalfjördur tunnel in Iceland and the Vága tunnel in the Faroe Islands. In both cases the rock conditions were significantly different from Norwegian conditions, including basaltic rocks and geological features that could potentially cause large water inflows. The general public confidence in subsea tunnels was lower, and, most important, the owners had a strong need for a reasonably predictable cost due to the available financing methods. These cases and the circumstances are described in detail by the authors of this paper (Ref. 12), and the following conclusions can be included here:

- It is possible to customise a contract to the specific circumstances of a project, considering the need for predictability of costs for the owner, while keeping a reasonable risk on his hand for the more uncertain elements like rock support and water control measures.
- This can be achieved by applying elements from unit price contracts to account for variations in the geological conditions, which may prevent that unnecessary high contingencies are built into the bids. Unless the completion time is adjustable, it must not be set too tight.
- Such 'adjustable fixed price' contracts may include incentives for the contractor to use innovative methods in order to improve efficiency without sacrificing safety. However, such incentives may also be built into unit price contracts.
- The potential benefits of 'adjustable fixed price' contracts were identified already at the planning stage, and helped the realisation of the projects. As the encountered conditions were more or less as expected, the contracts were not put to the test in unexpected or unforeseen conditions.

Adding to these examples listed above on modified contracts, the tunnel construction for the Harbour Area Treatment Scheme (HATS2A) in Hong Kong employed a contract format that merged the traditional fixed price contract in Hong Kong with new elements of unit rates. The base case was a fixed price contract for the tunneling works but the uncertainty associated with rock mass grouting during the time duration from blasting to final concrete lining installation needed additional focus. A solution was achieved where the works related to pre-grouting were paid as per unit rates that the contractor tendered. This has showed to be a well balanced way of reimbursement for the project although the concept was not necessarily in line with Hong Kong traditions.

19 REQUIREMENTS TO THE CONTRACT

The author believee 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 stabilisation 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.

20 CONCLUSIONS

In order to achieve success according to these criteria, the following requirements to the contract
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 in agreement with the recommendations by the International Tunnelling Association(Ref. 13).

REFERENCES

1. NORWEGIAN PUBLIC ROADS ADMINISTRATION: "Road Tunnels", Handbook No. 021, Directorate of Public Roads, 129p, Oslo 1992.
2. O. T. BLINDHEIM & B. NILSEN: "Rock cover requirements for subsea road tunnels", 4th Symposium on Strait Crossings, Bergen, Norway, 10 p, 2001.
3. O. T. BLINDHEIM & A. BEITNES: "Investigation strategy for sub-sea rock tunnels", Int. Symp. on Strait Crossings, 18p, Stavanger, 1986. Short version in Tunnels & Tunnelling, p 35-39, September, 1986.
4. O. T. BLINDHEIM & E. ØVSTEDAL: "Design principles and construction methods for water

control in subsea road tunnels in rock", *in* Water control in Norwegian tunnelling, Publication No. 12, Norwegian Soil and Rock Engineering Association, Norwegian Tunnelling Society, NFF, 2002, *(can be downloaded at www.tunnel.no)*.

5. O. T. BLINDHEIM & A. B. OLSEN: "Prevention is better than cure. Experience with probe drilling, stability control and pre-grouting in the Ålesund-Giske subsea road tunnels", Tunnels & Tunnelling, March, 1989.
6. O. T. BLINDHEIM, A. B. OLSEN & A. BEITNES: "The deepest subsea road tunnel in the world – Experiences with the Godøy tunnel", *Presented at:* Int. Symp. on Unique Underground Structures, Denver, 1990, *(copy available on request to: otblindheim@compuserve.com)*.
7. HOLTER, K. G., JOHANSEN, E. D. & HEGRENÆS, A. (1996): Tunnelling through a sandzone - Ground treatment experiences from the Bjorøy Subsea Road Tunnel, Proc. North American Tunneling '96, Washington, USA, Ozdemir (ed.), Balkema, pp 249-256.
8. B. AAGAARD & O. T. BLINDHEIM: "Crossing of exceptionally poor weakness zones in three subsea tunnels, Norway", *in* Alten (ed.): Challenges for the 21st Century, Proc. World Tunnel Congress '99, Oslo, pp 457-466. Balkema, 1999.
9. O. T. BLINDHEIM & H. BERG: "Sprayed concrete rock support on the Oslofjord Connection", 3rd Int. Symp. on Sprayed Concrete – Modern Use of Wet Mix Sprayed Concrete for Underground Support, Norwegian Tunnelling Society, Gol, Norway, 1999.
10. L. BACKER & O. T. BLINDHEIM: "The Oslofjord Subsea road tunnel. Crossing of a weakness zone under high pressure by freezing", *in* T. Alten et.al., Challenges for the 21st Century. Proc. World Tunnel Congress '99, Oslo: pp 309-316. Balkema, 1999. *Also published as:* "Oslofjord challenge", Tunnels & Tunnelling International, December, pp 39-42, 1999.
11. E. KLEIVAN: "NoTCoS – Norwegian Tunnelling Contract System", *in* Norwegian Tunnelling Today, Publication No. 5, Norwegian Soil and Rock Engineering Association, 4p, 1987.
12. E. GRØV & O. T. BLINDHEIM: "Risk allocation in 'adjustable fixed price contracts", Tunnels & Tunnelling International, June 2003.
13. INTERNATIONAL TUNNELLING ASSOCIATION (Working Group on Contractual Sharing of Risks): "ITA Recommendations on Contractual Sharing of Risks", Tunnelling and Underground Space Technology, Vol. 3, No. 2, pp. 103-104, 1988.

Oset water processing plant, Oslo. Control at site. Courtesy: AF Gruppen.

13. WATER PROOFING AND FROST INSULATION IN NORWEGIAN ROAD TUNNELS – CURRENT STATUS AND FUTURE TRENDS

BUVIK, Harald
HOLTER, Karl Gunnar
LINDSTRØM, Mona

ABSTRACT
In Norway more than 1000 road tunnels have been excavated, adding up to a total length of nearly 900 km. At present, 33 sub-sea tunnels are in operation, and more such tunnels are planned. Most of the tunnels are shorter than 1km and the longest is 24,5 km.

20 % of the tunnels have an annual average daily traffic (AADT) > 5000 and the highway tunnels in Oslo have an AADT up to 95000.

According to national guidelines from the Norwegian Public Roads Administration (NPRA) road tunnels are currently designed with a tunnel lining system consisting of a permanent rock reinforcement lining according the sprayed concrete and rock bolt method, and an inner insulation and waterproofing shield system. Road tunnels must be especially protected against water and ice. This is normally done by erecting a lining which directs water to the invert and into a trench. If the frost exceeds given limits the lining is carried out as a thermally insulated construction. Currently, a planned service life of 50 years assuming normal maintenance is required. The choice of frost insulation system is based on traffic volume (tunnel category) design traffic speed, tunnel length, standard of the tunnel, aesthetics, frost level, maintenance requirements and finanacial aspects. In addition local conditions are always considered in the design. This paper reviews the current design practice and gives an overview over some suggested and possible future technical solutions for such tunnel linings.

Longer service life time, higher quality of the constructed water proofing and frost insulation concepts, as well as minimising down time of the tunnels is required for modern road tunnels in Norway. This has led the NPRA through a process to consider improved technical solutions for tunnel linings in Norwegian road tunnels. Modern tunnel lining systems from Central Europe are being evaluated.

1 INTRODUCTION
Keeping the construction costs low has traditionally been considered the main issue in the total cost-effectiveness of a new road tunnel project in Norway. The costs related to maintenance and refurbishment has only to a limited extent been considered in the planning, decision and design process. In most cases one has accepted significant and frequent time slots with closure due to required maintenance. The vast portion of Norwegian tunnels has therefore been constructed with a tunnel lining system which has had a low investment cost, but also a limited service lifetime in a number of cases.

Since the beginning of the 1960's some 20 different methods and systems for water proofing and frost insulation have been tested in Norwegian traffic tunnels, including both light weight solutions and various concrete solutions, as well as combined solutions.

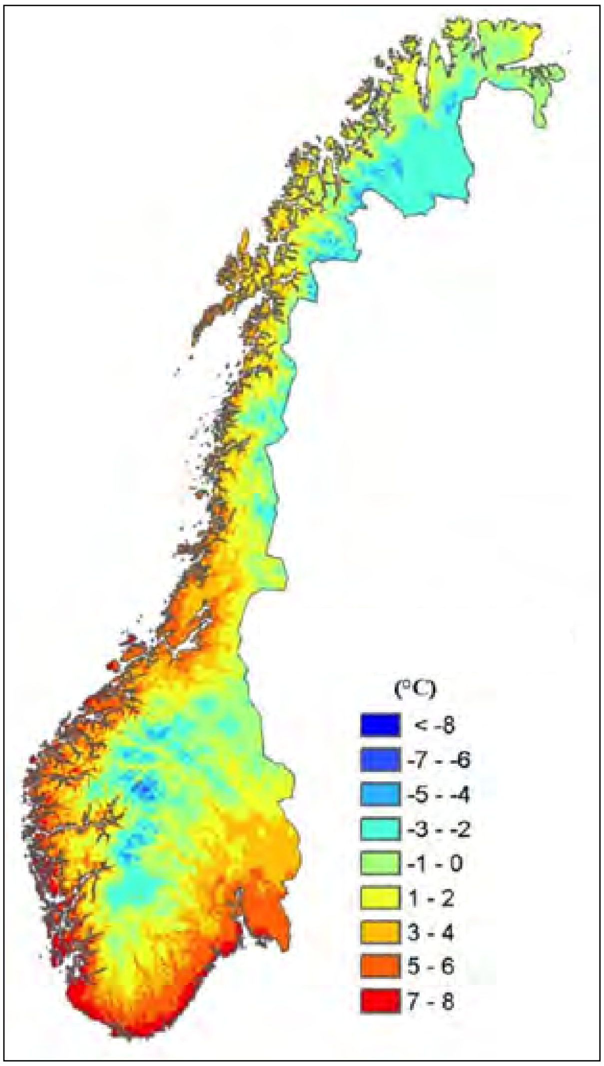
Thorough investigation of the performance record of these systems has shown that some of the systems do not fulfil the requirements for long-term durability. Only a few have proved to withstand the complex loads caused by air pressure and suction loads from the traffic.

The use of concrete linings for water proofing and frost insulation in road tunnels have increased, partly because an increased amount of tunnels are built in urban, high-traffic areas, the maintenance and repair of more light-weight structures have proven costly over time, and there is a demand for more durable and solid constructions allowing for lower maintenance and less closing time for the tunnels.

2 CLIMATE CHALLENGES
With its northern location, Norway is often regarded as a cold and wet country. In some aspects this is true, because we share the same latitude as Alaska, Greenland and Siberia. But compared to these areas , Norway has a mild climate. Due to the steady warming from the Golf current in the northeastern Atlantic Ocean, Norway enjoys a much warmer winter climate than the latitude would indicate..

However, Norway's climate shows great variations. From its southernmost point, Lindesnes, to its northernmost, North Cape, there is a span of 13 degrees of latitude, or the same as from Lindesnes to the Mediterranean Sea. Furthermore there are great variations in received solar energy during the year. The largest differences are found in Northern Norway, having *midnight sun in the summer months and no sunshine at all during winter. Furthermore, the rugged topography of Norway is one of the main reasons for large local climate differences over short distances.*

The normal temperature distribution in winter show two main features: firstly, the mean temperature in the winter months are above freezing all along the coast from Lista (Vest-Agder) to the Lofoten area (Nordland). Secondly, the lower inland areas, both in the southern and northern part of Norway, have very low mean temperatures in winter. The Finnmark Plateau in northern Norway is the coldest area with mean monthly temperatures around -15 °C. The highest temperature ever registered during winter, is 18,9 °C in February. The lowest temperature ever measured in Norway is -51,4 °C, recorded on on the Finnmark Plateau. **(met.no)**



Source: met.no

Examples of challenges that operation of tunnels has to deal with as a result of winter conditions:

Accumulation of snow not considered (NPRA)

Lack of waterproofing and thermal insulation (NRK)

Challenging design of tunnel portals (NPRA)

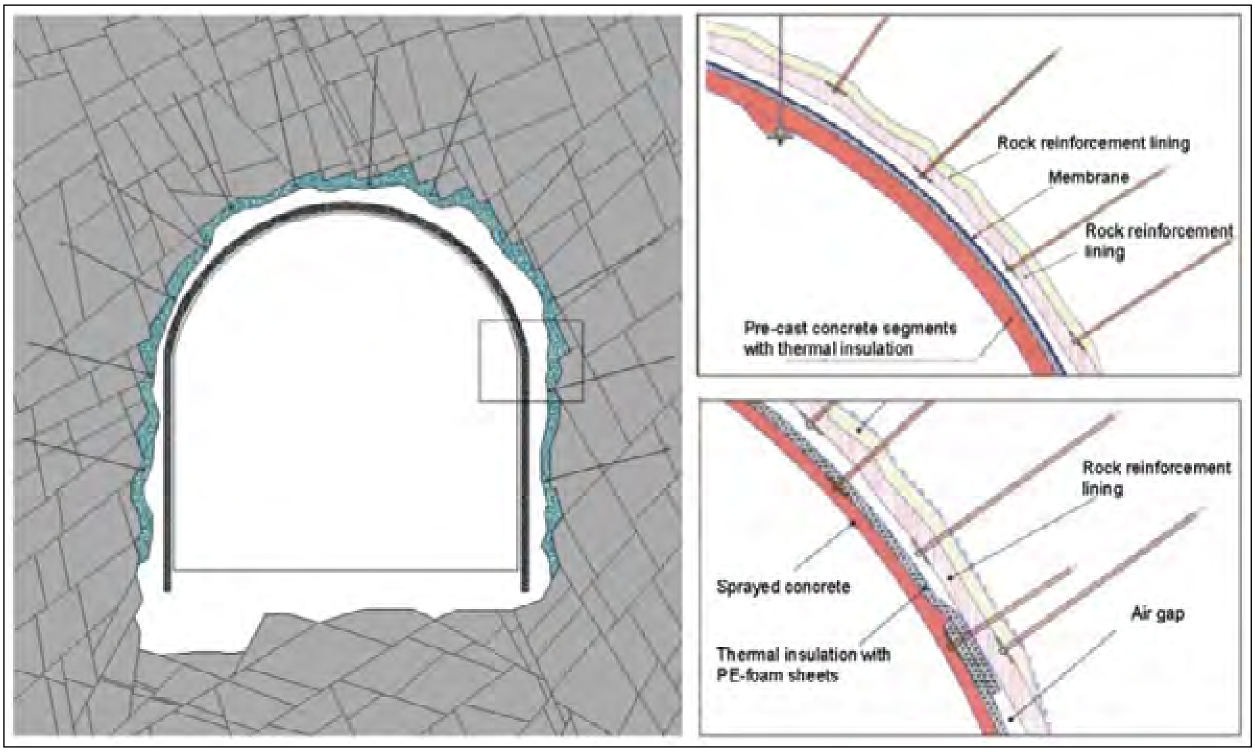
3 CURRENT TUNNEL SUPPORT PRACTICE

Norwegian road tunnels are currently being designed with a functionally divided tunnel lining system. The rock reinforcement lining is designed with sprayed concrete and rock bolts to provide permanent stability of the rock mass. The design of this lining is carried out according to the Q-system (Barton et al. 1974). This procedure is also referred to as the Norwegian Method of Tunneling NMT (Barton et al. 1994). An important feature is that the rock reinforcement lining is not waterproof. Hence, the tunnel structures are designed as globally drained structures. Water seepage control is handled with the pre-grouting method (Garshol 2003).

The pre-grouting method essentially utilizes a systematic pressure grouting of cementitious and mineral grouts ahead of the advancing tunnel face. Todays practice enables hard rock tunnel pre-grouting to achieve water ingress rates down to 1-2 litres per 100 linear m tunnel per minute (Hognestad et al. 2005). With rock overburdens in the range of 10-100 m this implies hydraulic conductivities after pre-grouting of the rock mass in the range of 10^{-8} to 10^{-9} m/s.

The remaining seepage has been allowed to enter into the tunnel. This implies a global drainage of the immediate rock mass around the tunnel. The globally drained tunnel structure has been a fundamental principle of Norwegian road tunnel construction. Since 1982 a large number of subsea road tunnels in rock have been successfully designed and constructed according to this principle (Nilsen and Henning 2009). This technical solution for tunnel linings implies the need for an inner lining structure which collects and drains the water down to the invert.

In areas exposed to freezing thermal insulation to prevent formation of ice is an important issue. For road tunnels the inner lining has also been designed to obtain a proper esthetic design of the traffic area. Examples of the traditionally employed tunnel lining systems are shown in figures below.



Layout of traditional Norwegian tunnel lining system with two options. A: shield system with thermally insulating pre-cast concrete elements. B: shield system with PE sheets (after NPRA 2012).

Recently constructed tunnel lining systems with drainage and thermal insulation shield structures as shown above. Left: Concrete segment shield structure for a highway tunnel. Middle: PE-foam shield structure in a high speed rail tunnel. Right: 3D image of the concrete segment and PE foam lining system highway tunnels (left and middle photos: Ådne Homleid/Byggeindustrien).

4 WATER PROOFING AND FROST INSULATION SYSTEMS

Rock support on one hand and the water proofing and frost insulation on the other hand are dealt with separately. Road tunnels must be especially protected against water and ice. This is normally executed by installing a water- and frost protecting system (lining) which directs the water into the drainage system of the tunnel. If the amount of frost on the site exceeds given limits, the tunnel lining is insulated.

Any new type of water-and frost-protecting systems shall be approved individually by the Directorate of Public Roads, following specific guidelines. This procedure includes documentation of insulating properties of materials, fire resistance, load resistance etc.

In tunnels with annual daily traffic (AADT) above 8000 vehicles a day, the lining shall include concrete wall elements. In tunnels with lower traffic amounts, more lightweight constructions may be installed. The most commonly used is PE-foam with sprayed concrete. A number of systems for water proofing and frost insulation systems have been tested and implemented over time, but the experiences show that many of these have been repaired and replaced due to materials (e.g. corrosion) and stability.

4.1 Polyethylene foam

In Norway, due to the cold winter climate experience shows that freezing temperatures penetrate deep into the tunnels. Thus, to prevent or reduce the risk of ice formation in the tunnels, different means and methods have been used. The most successful water proofing and frost insulation system has been the use of insulating mats of PE-foam (polyethylene).

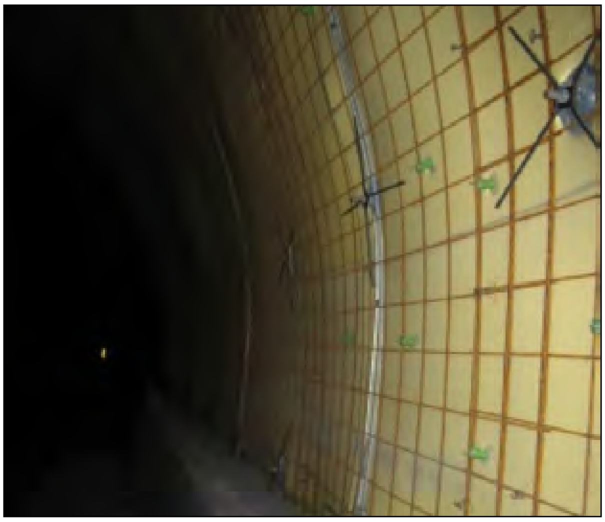
The PE-foam lining consists of mats of extruded polyetylene foam suspended by rock bolts mounted along the tunnel cross section. The lining is fire protected by sprayed concrete, with a minimum thickness of 80 mm and added polypropylene (PP) micro-fibers to prevent spalling in the case of a fire.

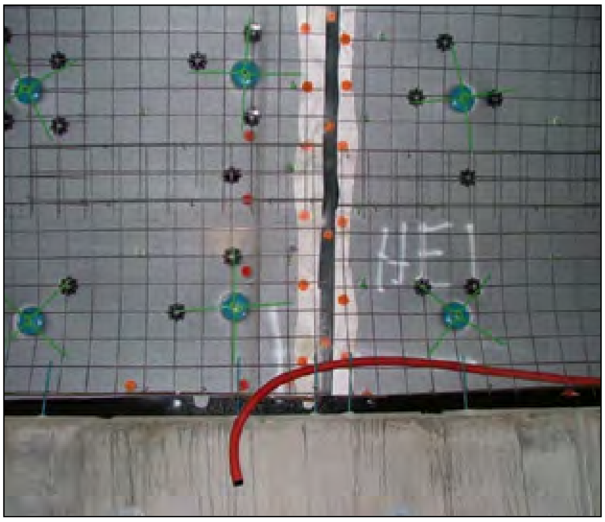
The distance between the bolts is 1.2 x 1.2 m. The bolts are coated to prevent corrosion, and the bolt holes through the PE-foam are sealed with metal plates. The thickness of the PE-foam mats is minimum 45 mm. This thickness is sufficient for insulation up to about F_{10} 10 000 hºC. With higher frost quantities, thicker mats are used.

The sprayed concrete protection lining is reinforced using a mesh, and additional steel reinforcement is used to prevent cracks due to temperature changes. Expansion joints are installed every 30 - 40 m to allow for movement.

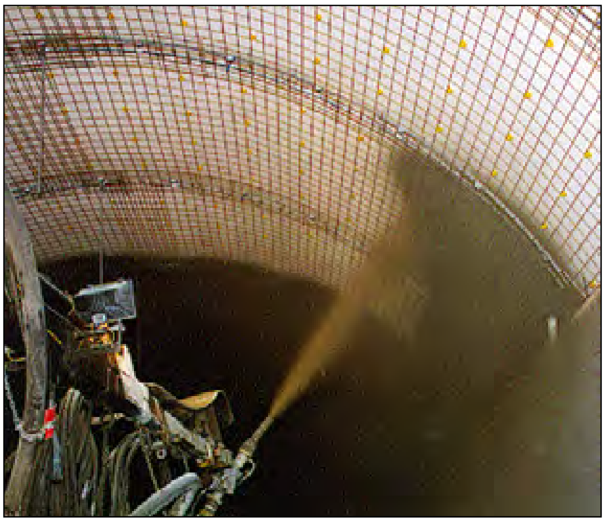
The mats are flexible and low-weight, the lining is easy to install and easy to adapt to different tunnel cross sections and transitions between different tunnel profiles.
The PE-foam/sprayed concrete lining is also used in combination with concrete wall elements.
The sprayed concrete surface has proven to be difficult to keep clean.

For the combined PE and pre-cast concrete solution, pre-cast concrete segments are used along the walls, while PE-foam is applied in the roof. The PE-foam is kept in place by fixation bolts, drilled radial to the tunnel contour. In combination with the fixation bolts installation frames consisting of vertical arches are located at a spacing of 2 - 3 m to fit the width of the PE-foam sheets. They are designed and manufactured to fit the theoretical tunnel contour. The PE-foam sheets are placed edge to edge, without overlap, along the vertical arches.





PE above concrete elements (H.Buvik, NPRA) (Mona Lindstrøm,NPRA)
The details include an expansion joint, reinforcement mesh, structural reinforcement and plastic coils for control of distance between PE-foam mat and the reinforcement.



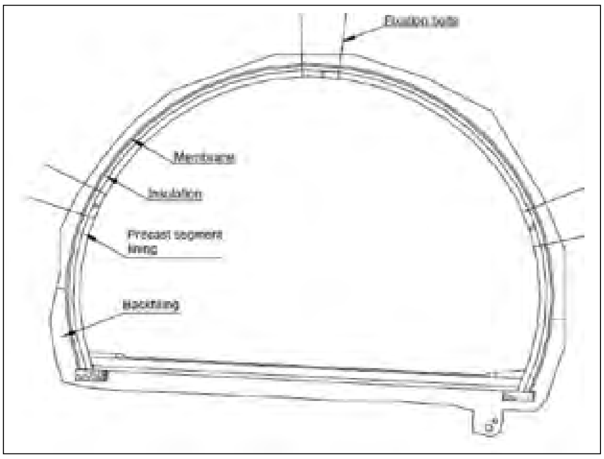
Fire protection of PE-foam with sprayed concrete (NPRA)

Protection details (H.Buvik, NPRA)

This system provides an inner lining which has proved to be watertight as there is no puncturing by rock bolts. Sprayed concrete with a thickness of 80 mm including polypropylene fibres, in combination with wire mesh, covers the whole of the PE-foam installation and provides a complete fire resistant system.

A 600 mm X of reinforcement bar has to be assembled to secure the necessary power transmission between the arch of sprayed concrete and the rock bolt.

4.2 Inner waterproofing anf insulation lining systems with pre-cast concrete segments
Water proofing and frost insulation systems, also referred to as inner lining, saw the introduction of pre-cast concrete segments for use in high traffic tunnels in the 1990's. An inner lining of precast segments could be applied either as insulated or as not insulated. The inner lining should prevent water from penetrating the lining and enter the carriageway. Consequently, the use of water proof membranes has been enforced. Various modifications on the pre-cast concrete segment lining have been tested, including sandwich solutions, lightweight concrete segments etc. to provide an inner lining which is a stable, water proof, durable and frost insulated installation. The figure below shows a typical cross-section of an inner lining of pre-cast concrete segments.



Pre-cast concrete segment lining (NPRA)

For the successful implementation of an inner lining of pre-cast concrete segments there are some basic requirements. Some of these are related also to the tunnel excavation and rock support application. The following requirements should be addressed:

- Strict control on the drilling accuracy to reduce overbreak
- Careful perimeter blasting to prevent unnecessary reduction of the self-supporting effect of the rock mass.
- Probe drilling ahead of the tunnel face in potentially difficult areas to predict rock mass and groundwater conditions.
- Criteria for acceptable ingress of water into the tunnel.
- Pre-grouting to prevent too high amounts of water entering the tunnel.
- Inner lining designed as an independent structure without any interaction with the rock support.
- Rock support and inner lining designed for a service lifetime of 50 years.
- Ventilation fans, cable bridges, illumination etc. installed without any interaction with the inner lining or the rock support.

The installation of full contour of pre-cast concrete segment liningisdescribed below. If necessary, a frost insulated inner lining can be installed at relevant sections of the tunnel where frost is expected, such as for the entrance areas. The segments should be equipped with insulation material before being erected. Insulation can be PE-foam, XPS (expanded polyester) or similar material attached to the segments on the side facing the rock.

The inner lining consists of 4 or 5 pre-cast concrete segments which form an arch covering the walls and roof in the tunnel. Normally the length of each segment is 5 to 6 m for the walls and 2 to 3 m for the roof, measured longitudinally in the tunnel. The width (transversal measure) may vary. Normally wall segments are 4 m wide. Roof segments must fit to the actual cross section and number of segments to be used, but they are often 6 m wide. The pre-cast concrete segments are constructed to meet the criteria of 45 MPa strength after 28 days and have a thickness of e.g. 150 mm. Reinforcement is designed on a project specific basis to include the impact from vehicle collision and use of PP micro fibers to prevent concrete spalling in the case of a fire.

The pre-cast concrete segments are stabilised by the use of fixation bolts drilled into the rock mass. The bolts are fully grouted steel rebars, galvanized and epoxy coated, normally with a diameter of 25 mm. Pre-made holes in the segments are required. The fixation bolts will ensure a uniform installation. The joints between neighbouring segments are sealed by waterstops.

A water-tight membrane of PVC or similar material is placed along the entire segment section. The fixation bolts penetrate the entire structure, thus a sealing should be in place at the interfaces between the various elements in the structure. These could be rubber rings or gaskets.

The wall segments are placed on a concrete foundation which is cast-in place. A backfill of pea gravel or lean concrete is placed between the lower part of the wall segment (0,5 m) and the rock surface to stabilise the wall segment in case of being bumped into by vehicles. This is done before the installation of the roof segments.

5 EXPERIENCED SHORTCOMINGS WITH THE EXISTING WATERPROOFING AND INSULATION SHIELD SYSTEMS

A basic design concept for using an inner lining for water proofing and frost insulation in Norwegian tunnels is that there is no interaction between the lining and the rock support. This clearance area of various volume means that there are requirements for inspection of the rock support. Such regular inspections are time consuming over the years, expensive and includes closure of the tunnels. Independent of the different water proofing and frost insulation systems, the question of future use of this concept has been raised the last years.

Experiences has shown that some of these systems has too short service life time. In some cases one has experienced only 15 -20 years before the need of expensive rehabilitation. The polyethylene foam system has been questioned for years for its combustible material even if it is well protected with sprayed concrete and polypropylene fibers and fire tested and proven in full scale.

A general demand of longer service life time and higher quality of the constructed concept and an increasing demand of minium down time of the tunnels has led to the discussions about the future strategy of water proofing and frost insulation.

6 FUTURE TRENDS

The Norwegian Public Roads Administration is adopting the central European approach by designing tunnel linings with either cast-in-place or segmental concrete structures. The main reason for this is the expected high service lifetime and low need for maintenance for such tunnel lining systems compared to the performance of the tunnel lining systems which have traditionally been in use. A lowest possible down time is very important for tunnels for major rail and highway portions. Hence, tunnel linings which require a minimum of maintenance are suggested for such projects. For road connections with less importance, one has chosen to accept the latest developed versions of the existing drainage and insulation shield lining systems.

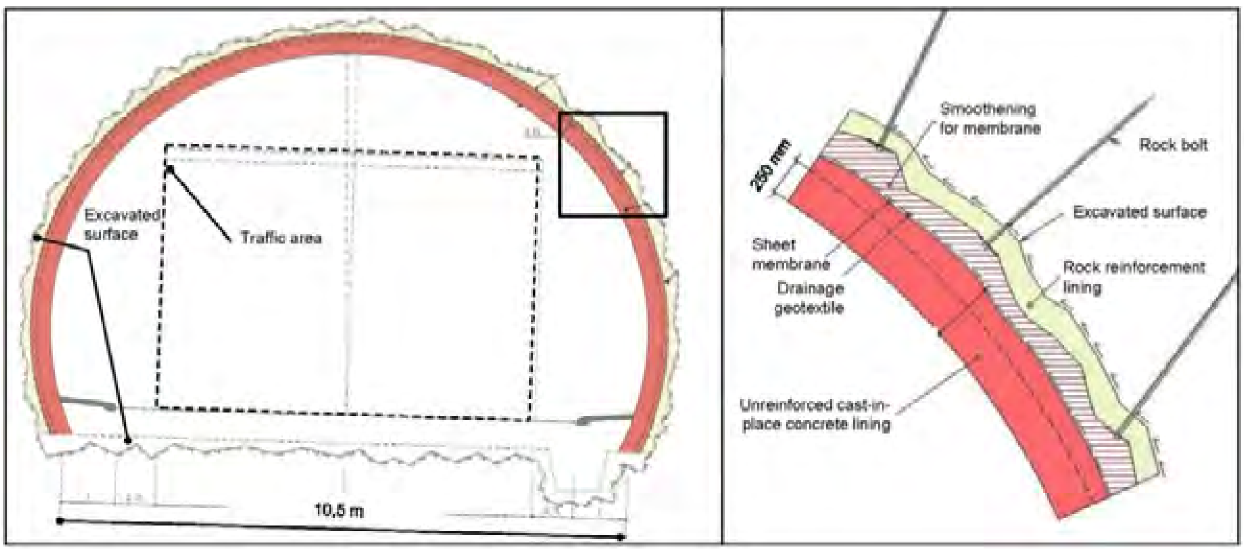
6.1 Rock reinforcezent and water control design

For both road and rail tunnels, the proven and established rock reinforcement method and water control philosophy with the pre-grouting method is suggested for the future geomechanical and hydrogeological design (NPRA 2012). For geomechanical stability of the tunnel lining this implies that sprayed concrete and rock bolting will still be the main rock mass reinforcement lining method (Barton et a. 1974, Barton et al. 1994, Norwegian Concrete Association 2011). Hence, the possible use of cast-in-place concrete tunnel linings with sheet membrane waterproofing has an esthetic and waterproofing function only and is not considered to have a structural or geomechanical function (NPRA 2012). The groundwater control philosophy with pre-grouting with strictly evaluated allowed water ingress amounts will still be the main approach. The main concept is to allow controlled and maximum defined amounts of water ingress. This water control philosophy implies the tunnel structure being a globally drained structure, with no loads imposed by water pressure acting on any parts of the tunnel lining.

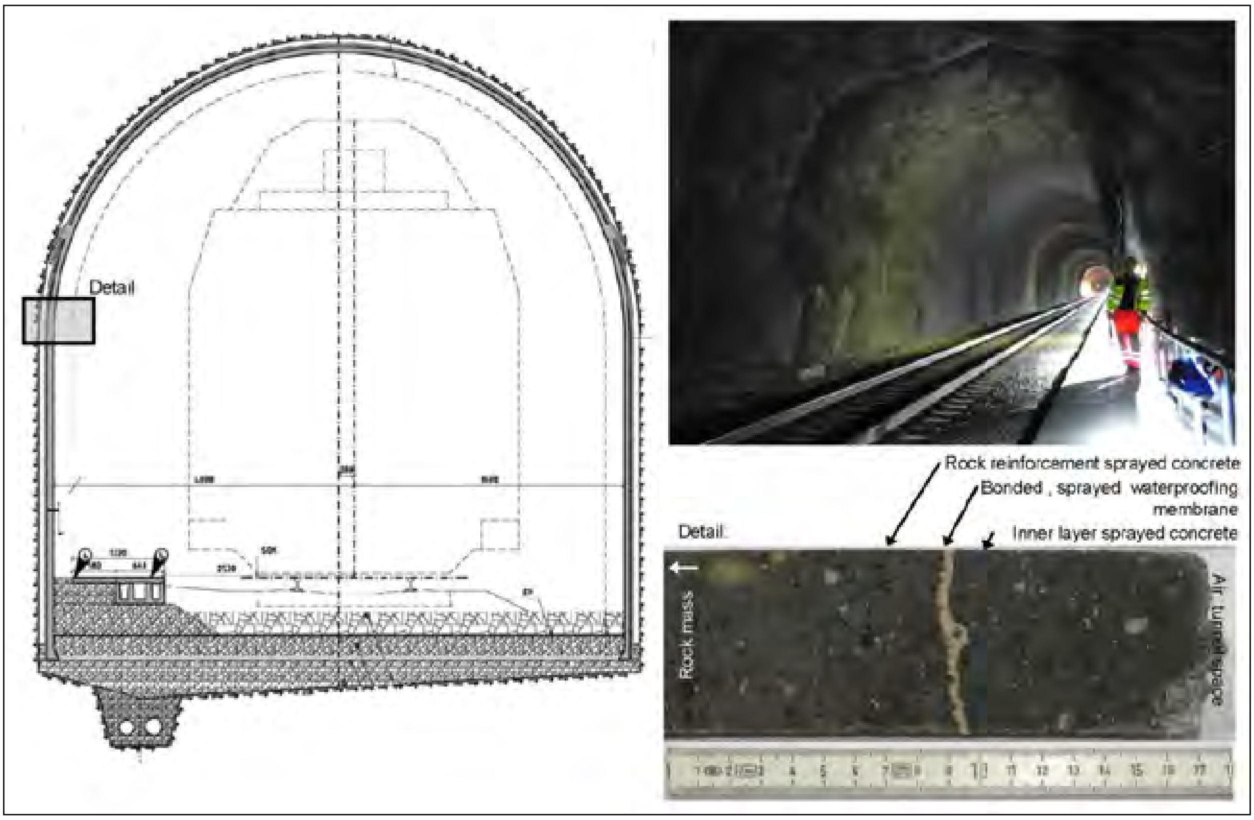
6.2 Trends for design of permanent linings in road tunnels

Two and three lane highway tunnels in hard rock in Norway will also in the future very likely be excavated by the drill-and-blast method. For major highway connections one recommends the cast-in-place tunnel lining system as shown in figure above for tunnels with annual average daily traffic movements (AADT) more than 4000. The lining system is adopted in a globally drained context, using a geotextile fleece for drainage, a sheet membrane for waterproofing and leaving the invert without waterproofing.

For road connections with less traffic density one will accept the shield systems as described. With the recent technical improvement of these tunnel lining systems one can realistically require a service lifetime of 50 years.



Suggested design with cast-in-place tunnel lining for modern two-lane highway tunnels in hard rock in Norway. Left: full cross section. Right: detail of lining structure (NPRA, 2012)



Innovative design option for rail and road tunnels in rock with sprayed concrete and bonded waterproofing membrane. Design and finished lining from the recently constructed Gevingås rail tunnel in central Norway

6.3 Sprayed concrete and sprayed membrane tunnel linings

An innovative tunnel lining system with sprayed concrete and bonded waterproof membrane is currently subject to detailed research for possible use in modern rail and road tunnels in Norway. This system has already been successfully used on several rail and road tunnel projects in central Europe (Holter et al. 2010) and most recently in Norway for a significant section of the recently constructed single-track 4 km long Gevingås rail tunnel near Trondheim (Nermoen et al. 2011).

The design process for the tunnel lining system included a RAMS and LCC analysis (DNV, 2010), in which the total cost-effectiveness of this tunnel lining method was verified. Example of a tunnel lining layout for a rail tunnel using the sprayed concrete and bonded membrane method, with system detail and example of finished tunnel lining in a modern rail tunnel, is shown in figure above. This tunnel lining method has proven to have a high maintainability without requiring long periods of down time.

7 CONCLUDING REMARKS

Pre-cast concrete lining systems for the waterproofing and thermal insulation have been successfully used in a large number of tunnels in Norway. For low traffic tunnels this approach has proven to be a cost-effective approach.

For high density road tunnels around the major cities strict requirements on service lifetime and minimising of downtime, the pre-cast lining systems do not completely meet modern demands. For this reason cast-in-place tunnel linings with sheet membranes for waterproofing are being considered as the future technical solution for such tunnels.

The pre-cast waterproofing and insulation systems have been improved over the latest years. This comprises improved structural performance, improved insulation and waterproofing performance, as well as more cost-effective installation procedures.

REFERENCES

Barton, N., Lien, R., Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics, 6(4):* Springer Verlag, Vienna, pp 189-236

Barton, N., Grimstad, E. 1994. The Q-system following twenty years of application in NMT support selection. *43rd Geomechanic Colloquy,* Salzburg. *Felsbau, 6/94:* pp 428-436

DNV Det Norske Veritas. 2010. RAMS and LCC analyses for the waterproofing and frost insulation for the Gevingås rail tunnel. DNV report no 2009-1968 for NNRA (Norwegian)

Garshol, K. 2002. Handbook: Pre-excavation grouting in hard rock underground construction. MEYCO Underground Construction, Zürich

Holter, K. G., Bridge, R., Tappy, O. 2010. Design and construction of permanent waterproof tunnel linings based on sprayed concrete. *In: Proc 11th Int'l Conference Underground Constructions Prague 2010. City and Transportation Tunnels – Zlámal, Butovič, Hilar (eds).* Czech Tunnelling Society, Prague. pp 121-126

Hognestad, H.O., Frogner, E. 2005. State-of-the-art microcement pre-injection for the Jong-Asker rail tunnel, Norway. *In: Proc 31st ITA-AITES World Tunnel Congress, Istanbul. Underground Space Use:Analysis of the Past and Lessons for the Future – Erdem, Solak (eds).* A.A Balkema, London. pp 925-930

Nermoen, B., Grøv, E., Holter, K.G., Vassenden, S. 2011. Permanent waterproof tunnel lining based on sprayed concrete and spray-applied double-bonded membrane. First Norwegian experiences with testing under freezing conditions, design and construction. *In Proc 6th Int'l Conf. on wet-mix sprayed concrete for rock support.* Tapir. Oslo, pp 317-338

Nilsen, B., Henning, J.E. 2009. Thirty years of experience with subsea road tunnels. *In Proc Int'l Conference Strait Crossings 2009.* Tapir, Trondheim. pp 35-44

Norwegian Concrete Association. 2011. Publication No 7 Sprayed Concrete for Rock Support, Norwegian Concrete Association, Oslo

NPRA Norwegian Public Roads Administration. 2012. Report No. 127. Major Research and Development Project: Modern Road Tunnels 2008-201, NPRA, Oslo (Norwegian)

The giant Swedish LKAB with iron ore mining operations in Kiruna and Malmberget in the northern part of Sweden depends on rail transport to coastal cities in Sweden and Norway for export of its products. The picture shows construction of a unique project in Narvik on the coast of the Atlantic Ocean. 12 underground silos under construction, each 60 m with a diameter of 38 m.Additional tunnels further below. Courtesy: LNS AS

14. PROJECT DOCUMENTATION DURING PLANNING AND CONSTRUCTION

GRASBAKKEN, Elisabeth
BERG, Heidi
WETLESEN, Thorvald

1 INTRODUCTION

Rock mass is a nature made construction material. It differs from man-made building materials with regard to documentation of its quality and state. Concrete and steel has defined parameters and qualities, derived from standard laboratory procedures, which enables reliable calculations as basis for construction, given that the performance is according to best practice.

Geology is nature's own work and can vary greatly both within rock types e.g. minerals, micro fractures, porosity, strength and abrasivity. In addition rock masses exhibits jointing, weakness zones, joint minerals, water ingress and rock stresses which are not possible to determine in a laboratory.

In addition to the geology itself a lot of surrounding parameters will affect underground project design, for instance rock overburden, soil depth over bedrock, vulnerable nature, lakes and wet areas, groundwater level, buildings and infrastructure and in some cases also existing underground constructions.

It is therefore crucial for the understanding of the building material present and the evaluation of the consequences for the surroundings, to map and document the properties of the rock types and the rock mass as well as all surrounding elements that may be affected. This mapping and documentation has to be a continuous process during planning and construction as the knowledge of the rock mass and the effect on the surroundings increases throughout the project.

After completion, all existing geological documentation should be compiled, along with construction experience, to improve competence and knowledge. This information is needed to plan future maintenance. It is also valuable information for future projects of the same kind or in the same geographical area.

2 DOCUMENTATION DURING PLANNING

The level of pre-investigations is related to project type, project stage and evaluations of risk and economy. There has to be a clear understanding of what the purpose of the ground investigations are.

For all underground projects there will be some level of risk. To be able to reduce this risk it is necessary to perform pre-investigations. However there will always be a residual risk which the project will have to act in accordance with. The level of residual risk will depend on the type of project, location, the difficulty of the ground conditions, requirements and complexity. A water tunnel in remote areas will have different external conditions than a road tunnel in an urban environment.
For owners and contractors a good documentation of the ground conditions in the planning phase of a project is important in order to be able to take the correct decisions regarding location and geometry, demonstrate complex issues that need special attention during construction, and to be able to calculate cost within acceptable uncertainty.

The documentation of the geological conditions at the planning stage will be:

- Investigation reports
    - Engineering geological mapping and drawings
    - Rock soundings
    - Core drillings
    - Soil investigations
    - Geotechnical laboratory tests
    - Rock wells
    - Pore pressure measurements
    - Seismic refraction profiling
    - Resistivity profiling
    - Rock stress measurements
    - Rock sampling and laboratory tests
- Engineering geological reports including drawings
- 3D-models
- Numerical modelling

The extent of desk studies, geophysical investigations, drillings, field mapping and laboratory investigations will increase throughout the project stages in order to be able to choose e.g. a suitable location for a rock cavern or for a tunnel alignment. Ideally one should aim at sufficient information at reasonable cost at the different stages during the project development.
The geological model should be under continuous evaluation and change as new and more detailed information is obtained.

2.1 Rock support design
A basis for planning of underground structures is to consider the rock mass as a self-bearing material. Hence the purpose of rock support is to enable the rock mass to be self-bearing, and not to establish a construction for a passive rock mass to be stable.
The design of rock support for underground projects at a planning stage of a project is always a challenge, as both loads and bearing capacity is unknown. Owners and third parties with little knowledge regarding engineering geology often require absolute answers in the form of calculated rock support design, for instance like calculation of reinforcement in concrete structures. The bedrock however is not fully "opened" for inspection and investigations and will always provide surprises. Effect and interaction between different factors as low overburden, proximity to existing underground structures, unfavorable stress situation, poor rock mass quality, intrusions, weakness zones and ground water can be very difficult to predict.

"As long as the myth persists that only what can be calculated constitute engineering, engineers will lack incentive or opportunity to apply the best judgment to the crucial problems that cannot be solved by calculation"-Dr. Ralph B. Peck, 1980

To be able to make a calculation of a rock mass behavior, a model has to be established. The modeling can be done by simplified manual calculations considering the different data available or by establishing a numerical model. The key components of any satisfactory modelling approach are data and understanding. A model will always be a simplification of reality rather than a perfect imitation of nature. Therefore establishment of a simple model with few parameters may be more appropriate than a complicated one, with an overload of more or less uncertain parameters, in order to analyse different aspects of a problem. This will make it easier to address the same questions from different perspectives.

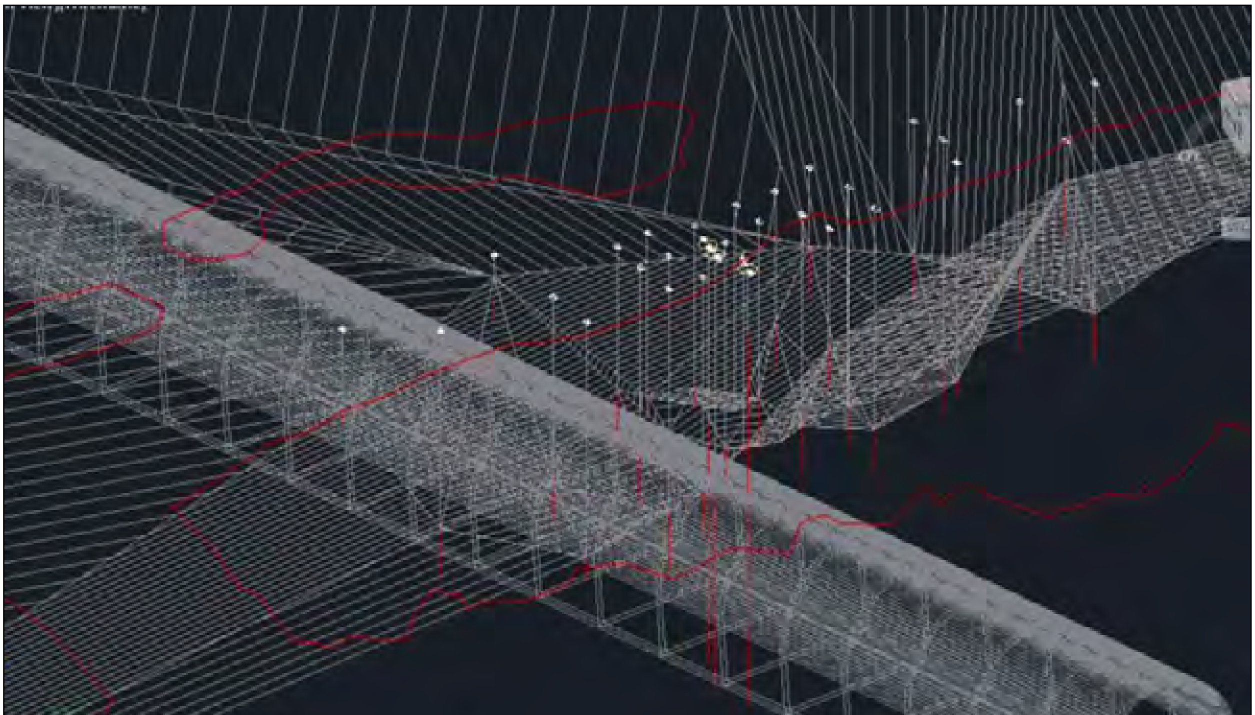


Figure 1 3D interpretation of rock surface based on rock soundings. Example from The Follo Line project. AutoCad

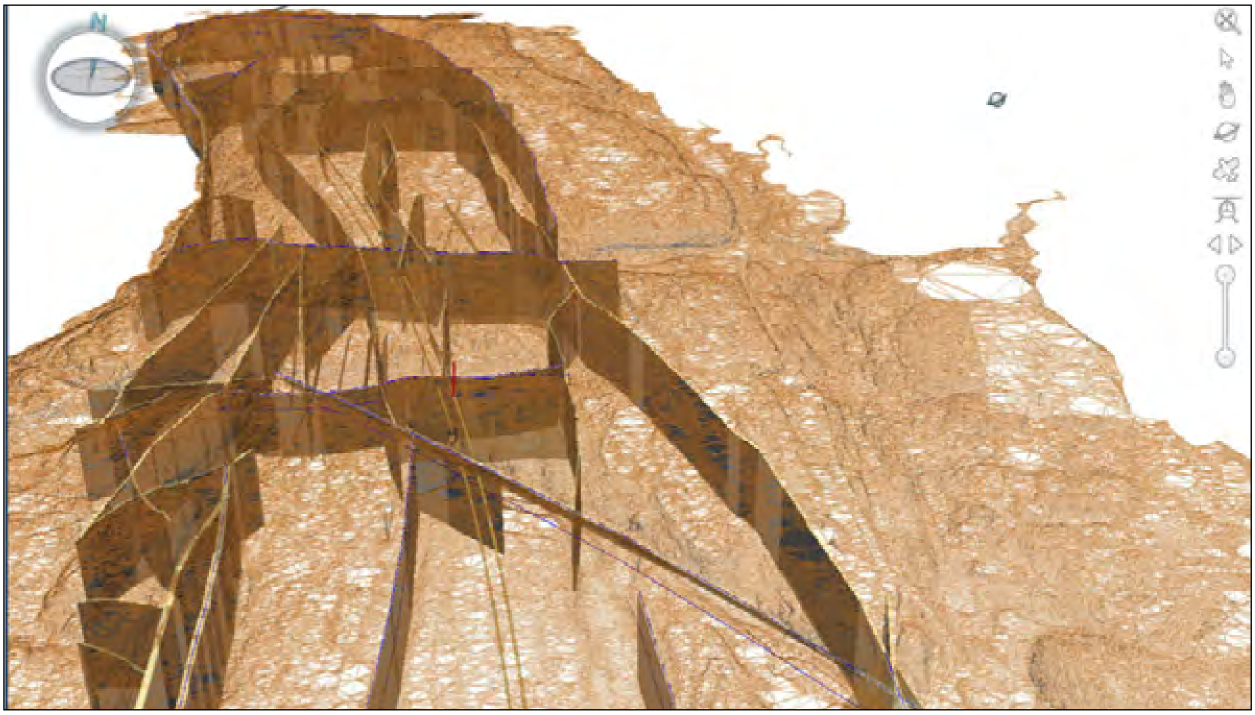


Figure 2 3D-model geology. Example from The Follo Line project. Novapoint

Figure 3 Example of different type of investigations assembled in Google Earth along with interpretation of soil depth. For each point data regarding type of investigation, location, soil depth for instance can be imported from a database in ArcGIS and shown as information when clicking on the actual point of interest. Example from The Follo Line project.

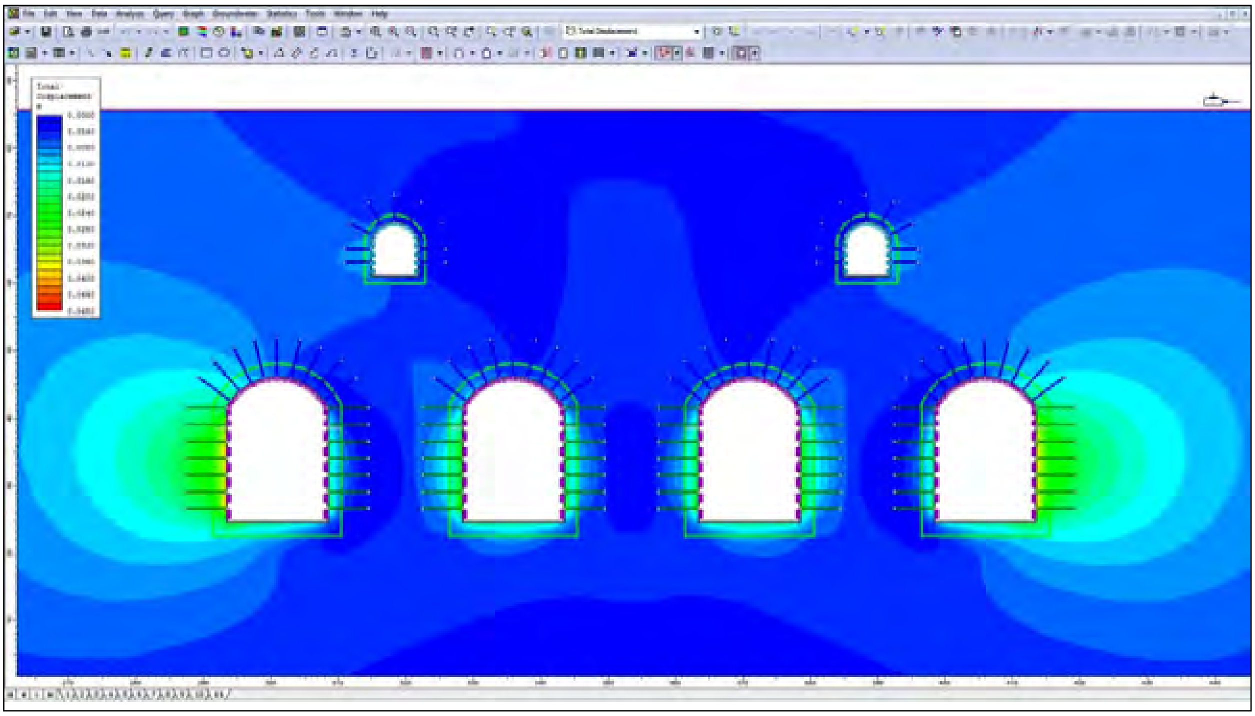


Figure 4 Modelling of rock caverns with installed rock support showing displacements in Phase2, Rocscience

"It is better to be approximately right than precisely wrong" – Dr. Evert Hoek

Used correctly and with caution, numerical modelling can be a helpful tool used in connection with all other available information. However both numerical and empirical techniques share the same limitations with respect to the need for an accurate representation of the structural character of the rock mass and the assumed joint properties.

In Norwegian practice there is always a prerequisite that rock support should be "designed as you go". This means that the final decisions shall be made at the tunnel face when all possible information is collected and evaluated by competent personnel.

3 DOCUMENTATION AND DESIGN DURING CONSTRUCTION

In traditional Norwegian tunneling the owner is represented on site with skilled engineering geologists mapping geology for each round throughout the entire excavation phase. Norwegian contracts have therefore recently included a half-hour after each blast round for the owner's representative for inspection and mapping during which the contractor shall be at hand with equipment and personnel. In addition to traditional engineering geological mapping, the geologist will get information from the drilling both verbally from the crew and in the form of MWD-data. Any water ingress shall also be reported and measured. The owner's engineering geologist will take active decisions on site together with the contractor regarding need for grouting and rock support. The engineering geologist shall also follow up and control the actual performance and quality of the rock support works.

It is common now to use the Q-system, ref /1/, as a basis for rock support design and to document rock quality. The Q-system is based on empirical data for many different existing underground projects. By calculating the Q-value it is possible to find the type and quantity of rock support that has been applied previously in rock masses with similar qualities. The purpose of the Q-system is to use it as a guideline in rock support decisions and not as a schematic way of designing final rock support. The joint orientation and rock strength are for instance not included in the system. The Q-values can also vary a lot within one blasting round. It is therefore important to use sound engineering judgment and make sure that weakness zones, blocks and clay zones have correct support.

«When developing classification systems and other tools to evaluate or "calculate" nature, it is of crucial importance to keep in mind the innumerable variations that occur in rock masses and the uncertainties involved in observing and recording the different parameters. During planning and construction of tunnels and caverns it is thus of great importance to apply sound judg-

ment and practical experience. A good overview of how the different parameters may have an influence is vital, as well as respect and understanding of the complexity of the task." Arild Palmstrøm og Einar Broch, ref /2/

Particularly difficult condition which requires heavier rock support may need documentation in the form of calculations. Some of the traditional Norwegian rock support will be impossible to calculate accurately, e.g. sprayed concrete ribs and spiling, because the load acting of reinforcement elements is almost always unknown. As mentioned above, all calculations are challenging since there are so many parameters interacting, but simplified, conservative calculations will document the validity of the chosen support system. Empirical experience from numerous projects in Norway also document these methods as suited and cost-efficient systems.

A thorough mapping of geology including, weakness zones, water ingress, Q-values and calculations is common practice. This is documented in reports, photos and drawings together with data from MWD-drilling, applied rock support and grouting. Together this documents the basis for rock support design and the final applied design.

4 IMPORTANT DOCUMENTATION FOR THE FUTURE

A good system for storing the documentation from an underground project is of great value for the future. Either it is for planning and controlling future maintenance or for sharing experience.

Since the rock mass is a secretive substance only revealing itself when opened, a good documentation of all geological investigations and recordings during planning, design and construction is valuable information also for later projects. Maybe some of the problems and solutions are relevant for later projects of the same kind or maybe the geological information and experiences are relevant for later projects in the same area.

4.1 Geo-referenced documentation

Since a lot of the optimization when it comes to selection of support measures is done at tunnel face for the Norwegian method, it is very important to do a thorough documentation to create a complete picture of the as-built of the tunnel. Since it's a lot of documentation that

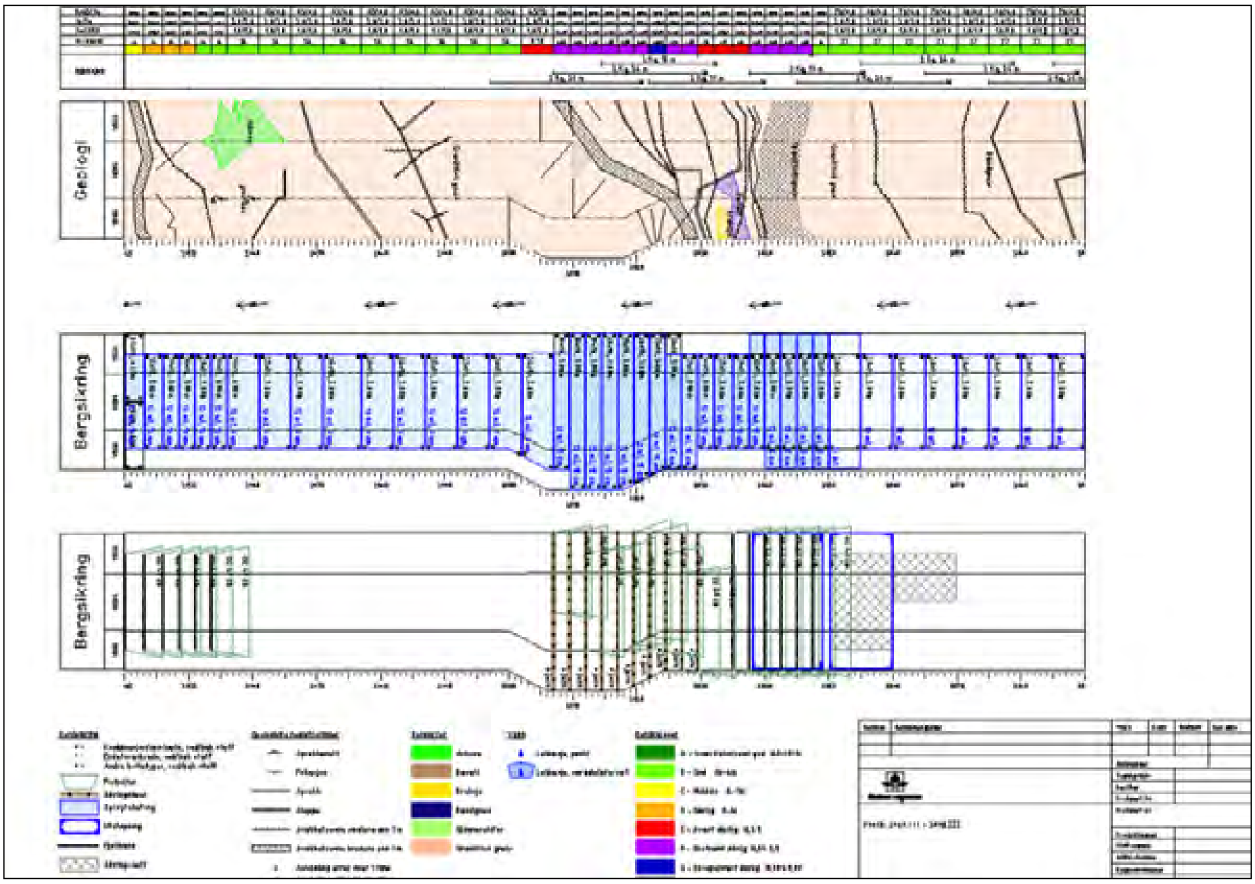


Figure 5 Example of documentation of geology and rock support for a road tunnel generated with Novapoint.

together will form the complete picture, geo-referencing of the documentation is used to systemize the data.

New technology has made it easier to do more automatic logging of documentation directly from the production equipment. Combining manually logged documentation and automatic logging, gives less risk of missing important information regarding geological conditions that may be crucial for the final rock support design. A richer documentation will also give a more complete picture.

The Norwegian Public Road Administration have through the project “Moderne vegtunneler” (“Modern road tunnels”) helped develop a software for collecting all the documentation in one, geo-referenced database. The documentation software let the engineering geologist add all documentation to the same geo-referenced database:

- Automatic logging from production equipment e.g. result from MWD during drilling
- Volume and material type of pre grouting (m3) for each drill hole
- Manual geological mapping done at face
- Amount and type of support measures set for each round
- 3D positioned photos
- Digital note book, for daily descriptions
- Laser scanning of as-built tunnel

Since it’s stored in a database it is easy to view only the documentation you want to view in the same window, and it is also easier to combine different documentation in DWG prints etc. More than 30 road-and rail tunnels are now documented in this way in an experience database for the future. The Norwegian Public Road Administration and the Norwegian Rail Administration both require this type of documentation for all their tunnel projects.

4.2 Automatic logging from the production equipment

Norway has always minimized the staff on jobsite for operation of the tunnel production. High tech equipment is utilized and planning and as built documentation is integrated in the production equipment. This is typical for

- Contour scanning. Control of overbreak/underbreak
- Bolting pattern for rock support
- Pre-grouting parameters, volumes and process control. Estimation of optimal grouting sequence and process
- Shotcrete thickness using laser scanning
- MWD data for geology estimation based on drilling parameters

Geo-referenced information will give us the opportunity to collect all data in with the same reference independ-

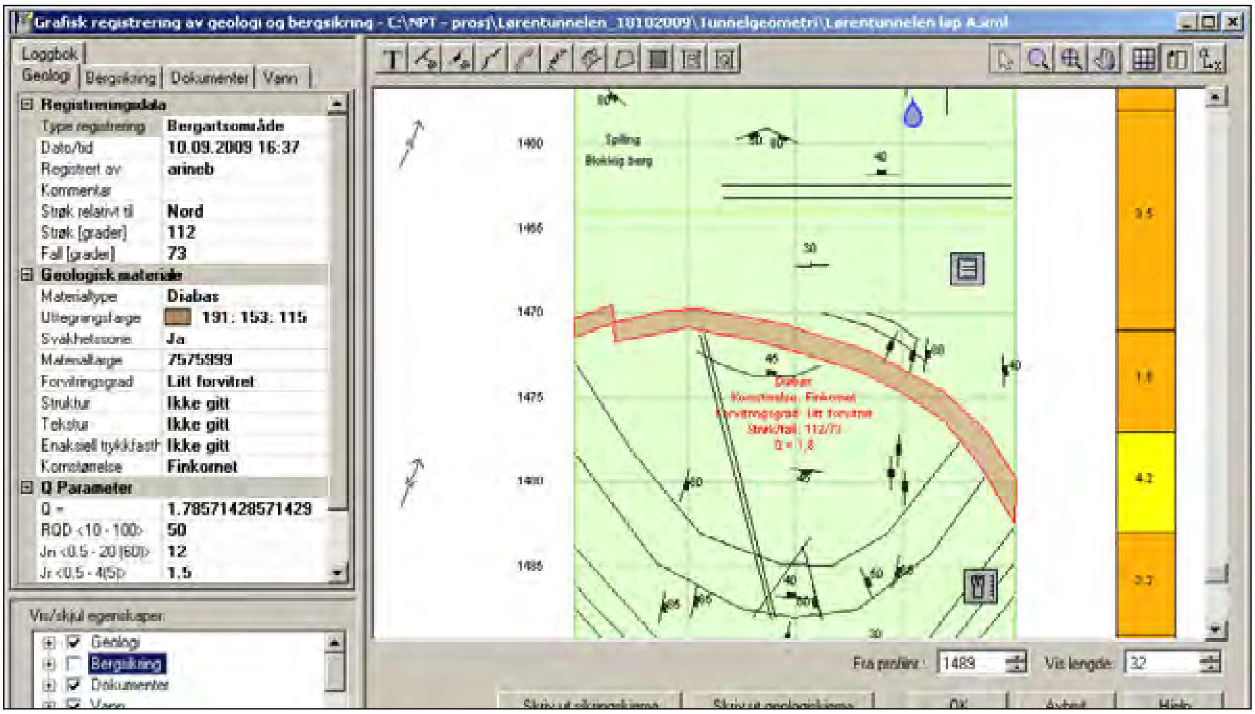


Figure 6 Database view, showing the result of the manual mapping, Novapoint

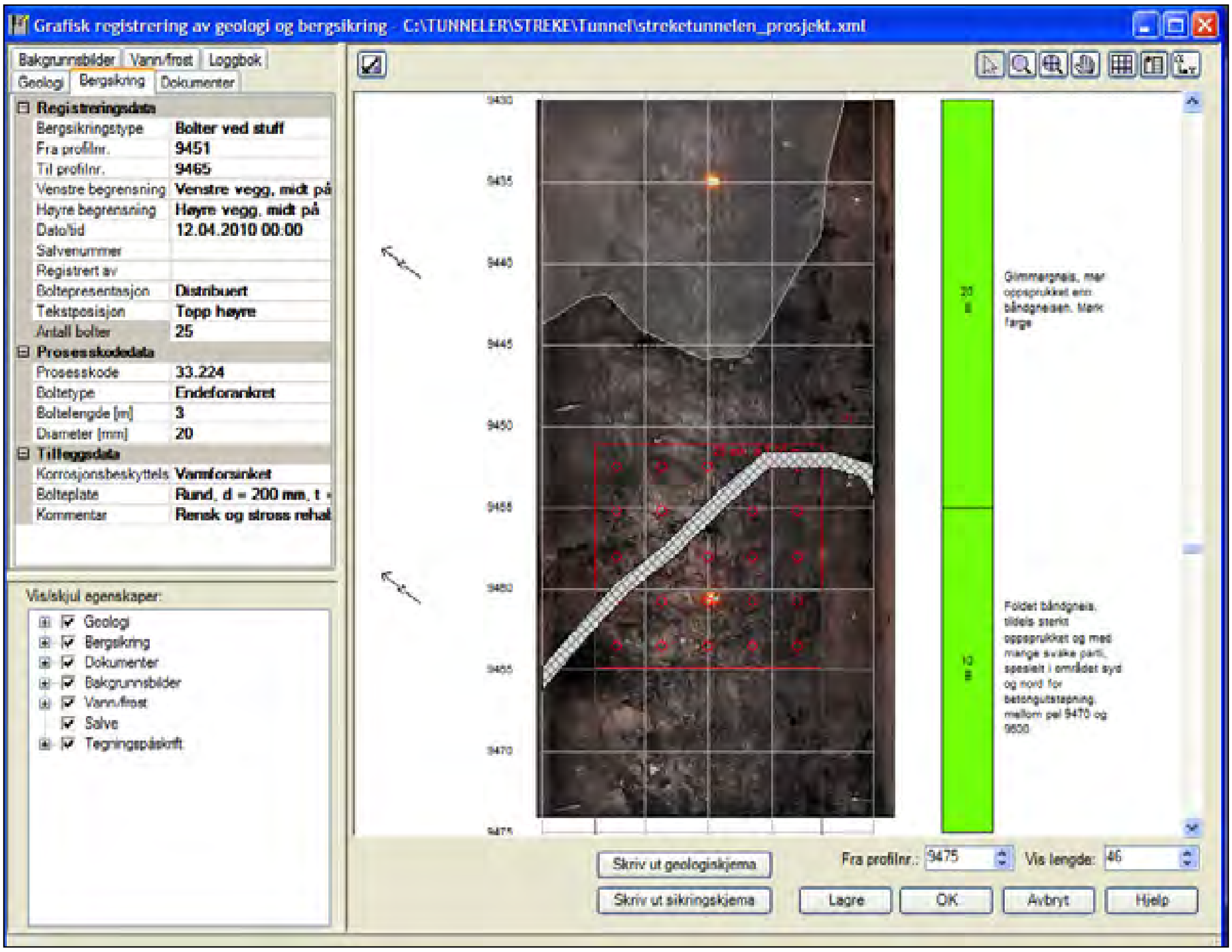


Figure 7 Example of geo-referenced photo, together with support measures and a weakness zone, Novapoint

ent of production equipment that is used. The main production machine is the drilling jumbo. This machine has a navigation system based on alternative of these 3 methods:

- Laser alignment with one of the booms. Classic method now mostly used as backup, and on project with low requirement to accuracy.
- Profiler navigation. This tool combines scanning of tunnel contour and navigation. Accuracy typical within 3-5 cm
- Total station navigation. Two fixed point on the machine is measured manually or a robot/radio-controlled total station

Geo-referenced coordinates allows a combination GPS coordinates with contractors coordinates for production planning and control.

Contour scanning. Client documentation.
There are three different systems used for contour scanning and under-/overbreak control. The profiler and the supporting software Bever Team Office is an efficient alternative to scanning using total station or Lidar scanning. All methods are used in Norway today.

5 SHOTCRETE THICKNESS CONTROL LASER SCANNER FROM ROBOT

Shotcrete thickness is one of the important parameter to control the quality of the shotcrete works. Normal requirement is 80 mm thickness or 100 mm where there is bad rock. Normal procedure for thickness control is to drill holes or probes after an agreed scheme. In Norway the regulations require a test for every 250 m3.

Another approach is to use laser scanner control. The laser scanner is mounted on the shotcrete robot and the surface after last blast is scanned once before shotcrete and a second scan after shotcrete is applied. The motions of the robot is tracked and compensated. Laser scanning gives very detailed and quick feedback to the Shotcrete operator, given the possibility to adjust the amount of shotcrete and save volumes. Savings of 20-30% could be obtained.

6 MWD - GEOLOGY ESTIMATION BASED ON DRILLING PARAMETERS

The MWD data and data from the pre-grouting, bolting, shotcrete and manual mapping can all be combined in the final documenation of the project. For new projects in Norway today, MWD logging is required.

In the MWD analysis there is indicated weakness zones in the grouting umbrella. This information can be useful when planning the grouting process. After grouting, the blast drilling shows the same weakness zone. If the water disturbance diminishes, this may indicate that the weakness zones are sealed.

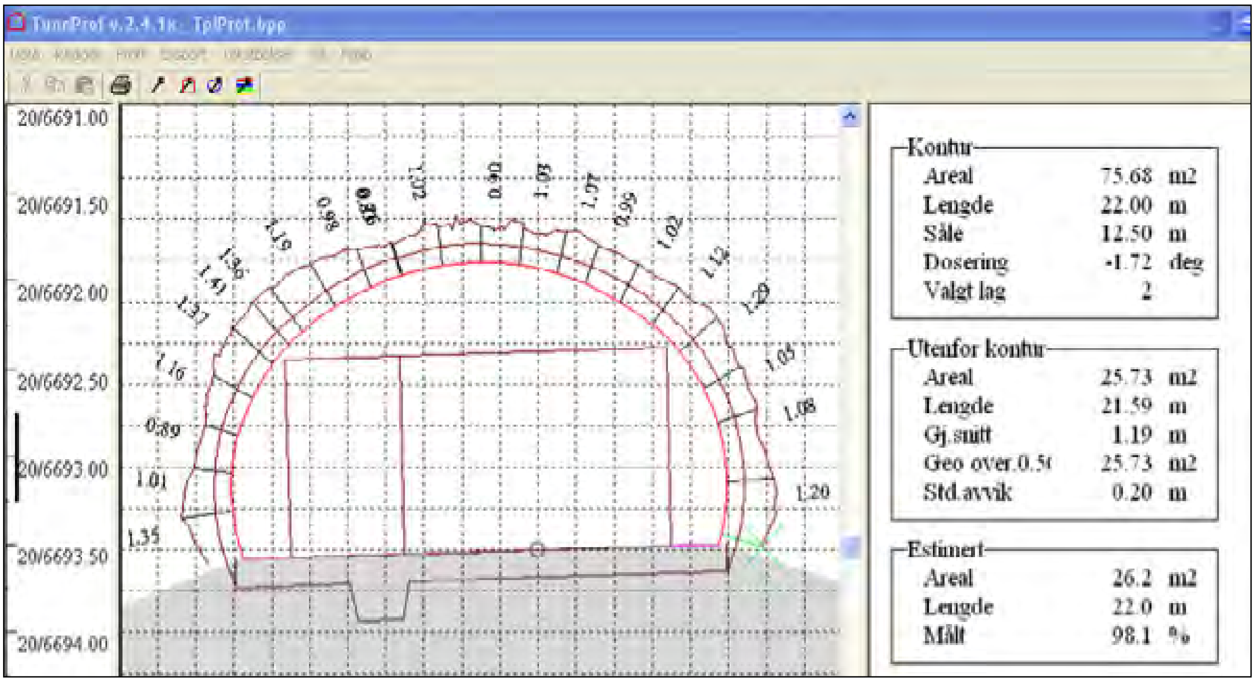


Figure 8 Contour control with indication of blasting contour, final contour and excavation. BeverTeam

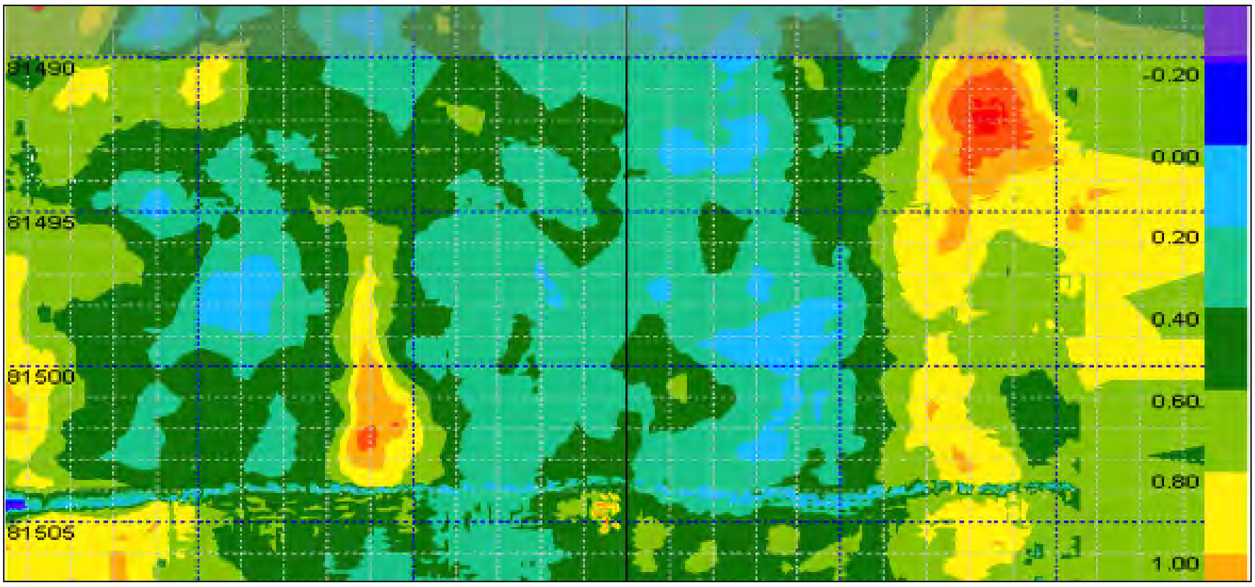


Figure 9 Topographic map with contour as reference. Shows 20 meter of surface mapped tunnel. Marine Blue is underbreak and red is more than 120 cm overbreak. BeverTeam

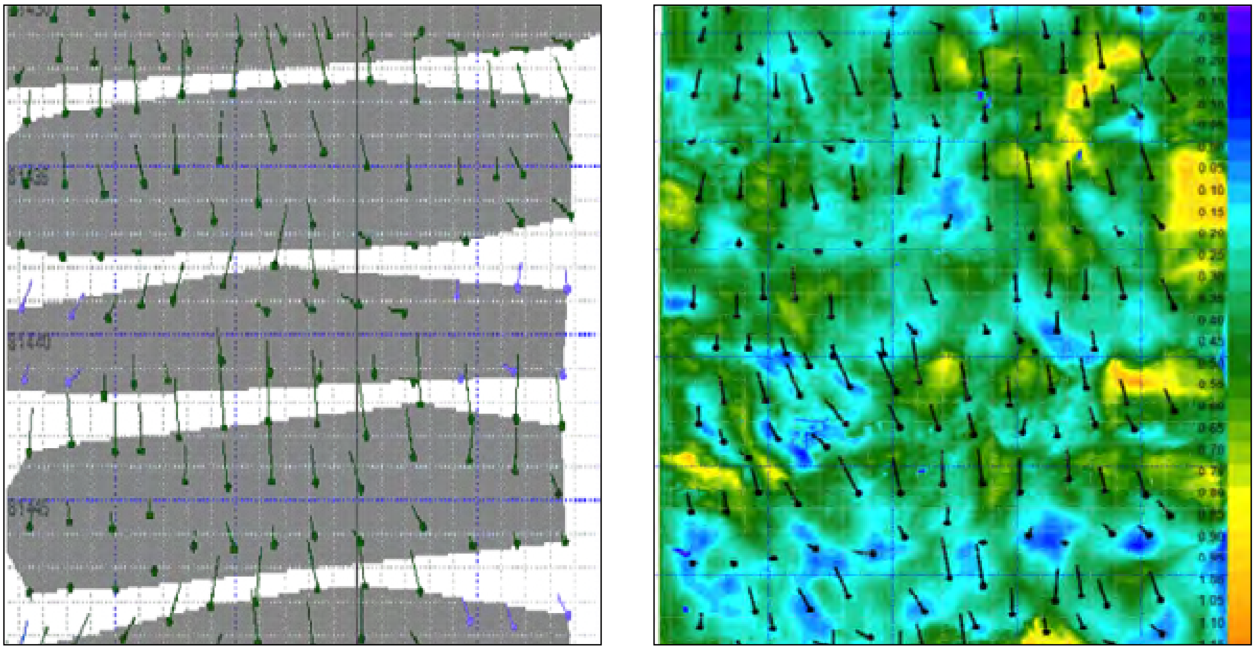


Figure 10 Automatic logging of rock bolt position, orientation and length. BeverTeam

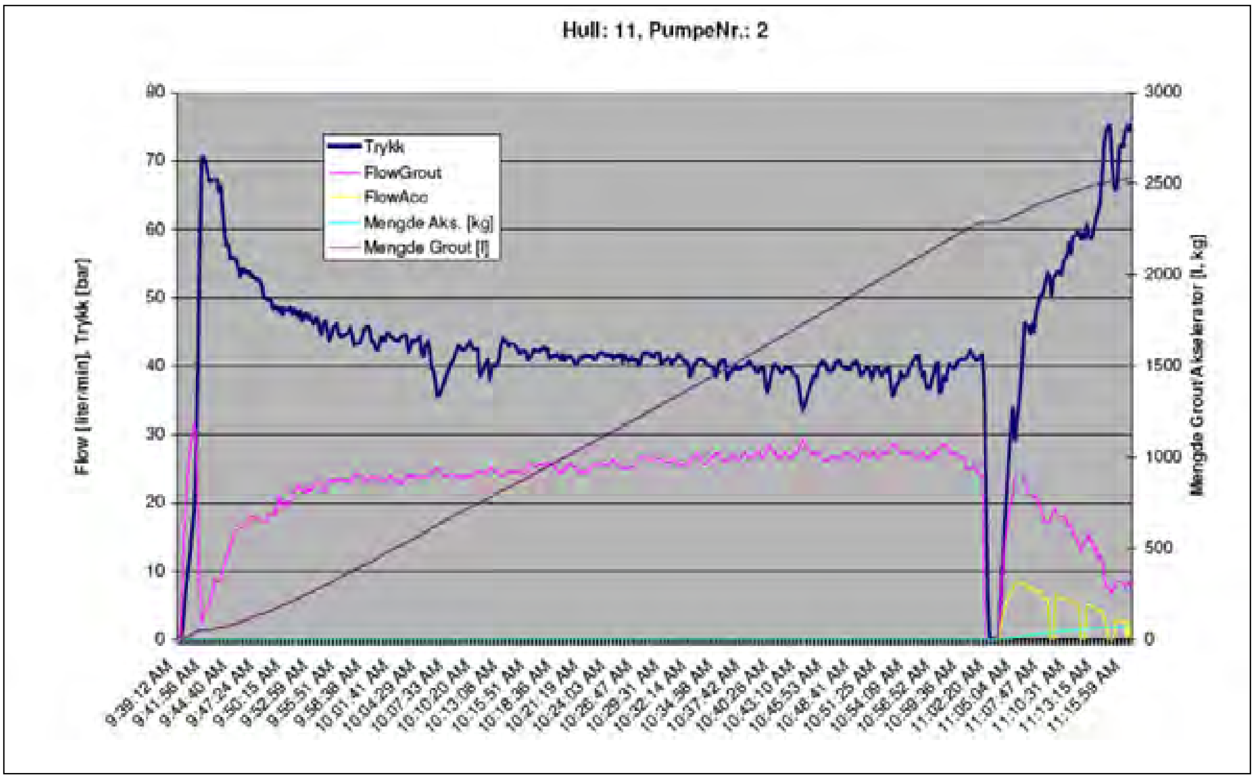


Figure 11 Automatic logging of Pre-grouting parameters from the grouting rig. BeverTeam

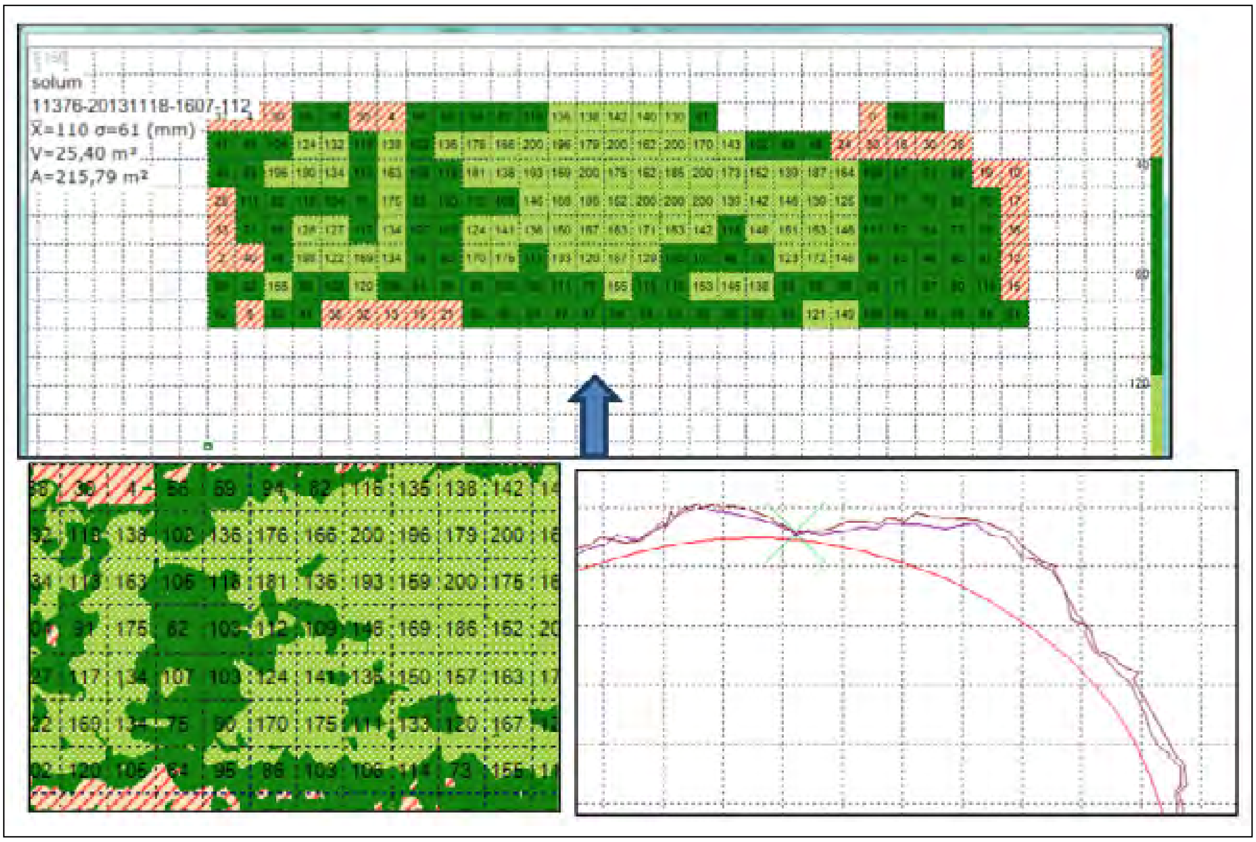


Figure 12 Shotcrete thickness. Example from a test on the railway tunnel Larvik-Porsgrunn. BeverTeam

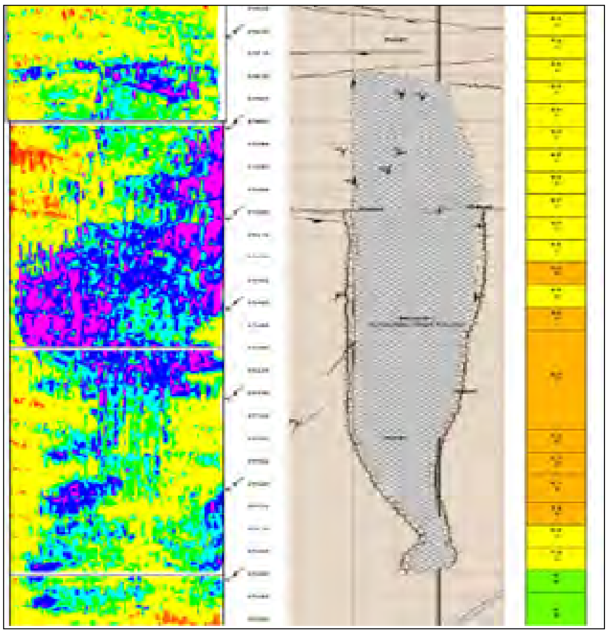


Figure 13 Interpretation form MWD data together with manual mapping. BeverTeam. To the right the manual mapping of the geology, to the left the automatic MWD interpretation. In this case there is a horizontal sand layer in the roof of the tunnel.

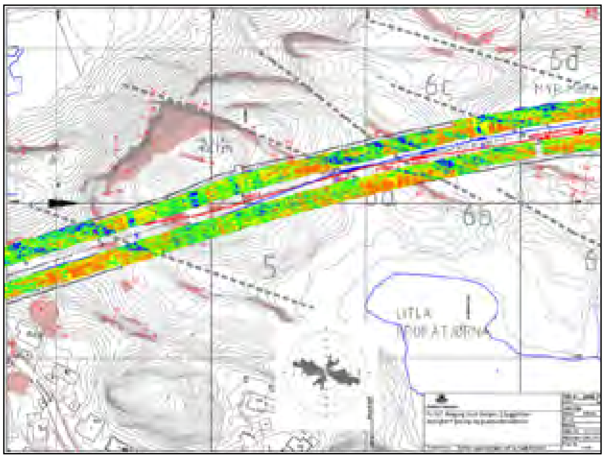


Figure 14 Figure 15 MWD interpretation from Ringvei vest Fase 2 in Bergen. It is a twin tube highway. Blue is bad rock. The rock hardness is estimated and transferred to a pdf map with weakness zones estimated in the preface of the project. BeverTeam

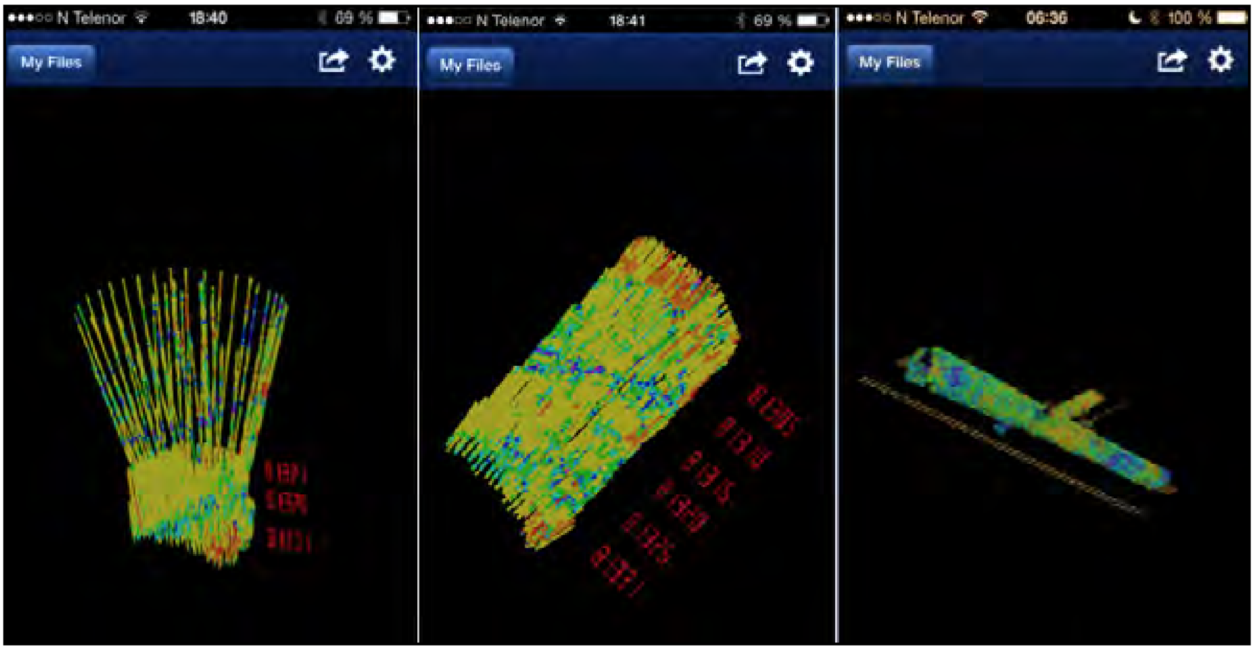


Figure 16 The 3D displays from the Railway project in Holmestrand, Norway. Shown on a smartphone display. BeverTeam. In the center picture you can look into the drilling round and the pre-grouting umbrella at the same time. This will give a good reference for estimation of problems in the pre-grouting phase and the advance in drill and blast operation. To the right you see the drill analysis after grouting and blasting. BeverTeam

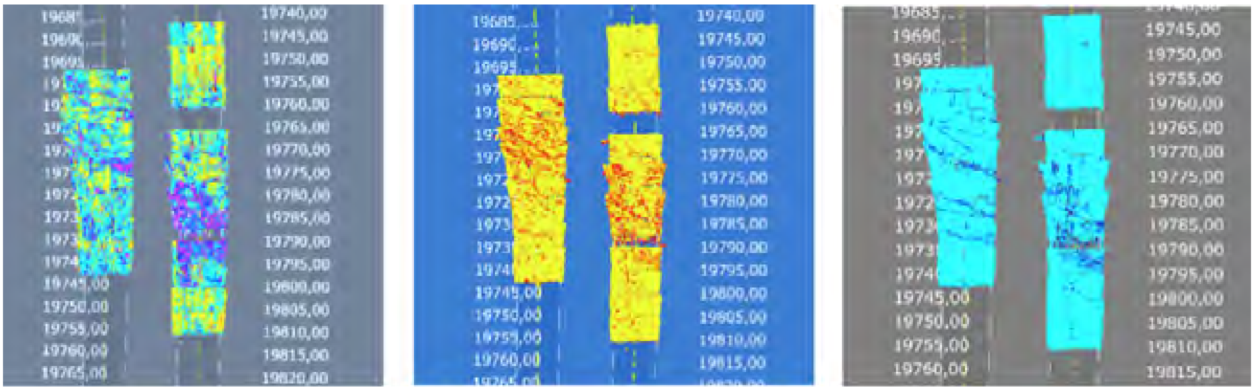


Figure 17 MWD interpretation showing from the left hardness, fracturing and flush water disturbance during drilling. The fractures are quite visible and to the left we see a well-defined area with bad rock (blue/pink area). BeverTeam

REFERENCES

/1/ NGI, Using the Q-system, Handbook 2013

/2/ A. Palmstrøm og Einar Broch, Use and misuse of rock mass classification systems with particular reference to the Q-system, Published 2006 in Tunnels and Underground Space Technology, vol. 21, pp. 575-593

/3/ Bever Team Office functions, Bever Control product systems www.bevercontrol.com

/4/ AMV products applied in the drill and blast process. www@amv-as.no

/5/ Norwegian Public Roads Administration Handbook

Approaching Sandvika west of Oslo the wiresawing method was successfully used along the lake. Courtesy: Norwegian National Rail Administration

15. NORWEGIAN TBM TUNNELLING – MACHINES FOR HARD ROCK AND MIXED FACE CONDITIONS

HANSEN, Arnulf M.

1 OVERVIEW

A total length of about 260 km of tunnels has been bored by TBM in Norway, with diameters ranging from 2.3 m to 8.5 meter. 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 so far, have been of the Open Hard Rock 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 Building and Construction 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".

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.

The Glommedal transfer tunnel, Ulla-Førre hydroelectric power project. Massive granite and gneiss formations. Photo: NTH (NTNU)

5 HIGH PERFORMANCE TBM

In 1988 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.

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

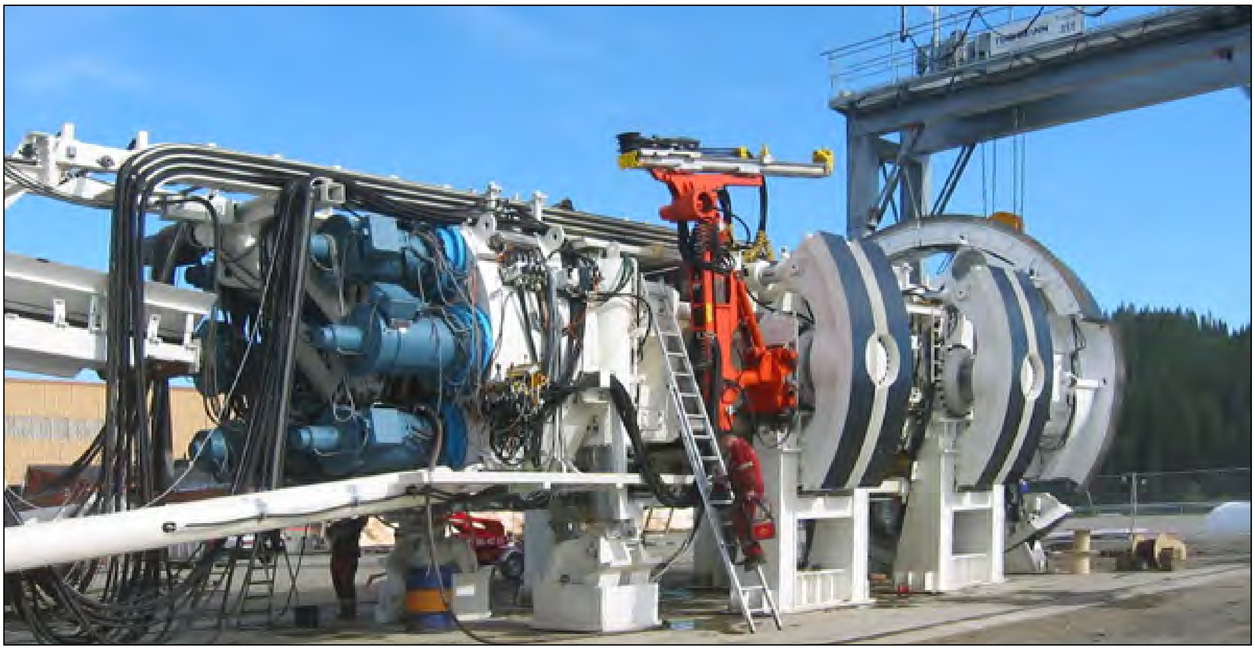
In August 1992 Merkraft, a joint venture of Eeg-Henriksen Anlegg AS and AS Veidekke, completed the boring of a 10 km transfer tunnel at Meråker Hydro Electric Power Project with a 3.5 m diameter Robbins High Performance TBM in less than 11 months. The tunnel boring was finished six months ahead of schedule. 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

The record performance Merkraft TBM. Photo: The Robbins Company



Atlas Copco Robbins TBM MK27 – Rebuild and diameter change by NCC International AS in Fosdalen, Malm for a hydroelectric power project in India. Photo: Egil Engesrønning

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.

10 THE RETURN OF TBMS TO NORWAY

10.1 The Røssåga Hydroelectric project

The two power stations Lower and Upper Røssåga in northern Norway some 200 km south of the Arctic Circle are now 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 hydro-power 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, will increase the capacity by 100MW and annual production by 200GWh are expected as a result of the project.

10.1.1 Geology

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

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 is

Robbins HP TBM 236-308. Photo: The Robbins Company 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 has 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 evaluation of all the bids, lead to the contract being awarded to LNS.

In conclusion, the new headrace tunnel of Lower Røssåga will be excavated at a slight uphill gradient of 0.02%, and its 450 metres long adit will be bored on a 1:9 decline and on a curve radius of about 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 is equipped for probe- and grout-hole drilling as well as rock support work. The primary rock support consist of two roof drills dedicated to rock bolting and mesh installations, the McNally roof support system using steel slats, and sprayed concrete systems.

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 started 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 is using 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 is 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 312 kN 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 are working 24 h/ day, six days a week. Two of the four crews are 12 days on site

Assembly of the LNS Røssåga TBM. Photo: The Robbins Company while the other two crews are 16 days off. LNS is expecting all of the tunnelling to be complete in summer of 2015 and the new power station to go online in summer of 2016

10.1.2 Hard rock and mixed face condition
During the first 150 metres of tunnelling the TBM drive encountered extremely hard, tough and abrasive rock with quartz lenses and hardbands of quartzite in the tunnel face that caused a lot of cutter changes due to ring chipping. The core samples tested by SINTEF shows UCS: 190 – 280 MP

Mixed face conditions. Photo: Kristian Repstad, LNS

As a rule of thumb in hard rock tunnel boring one should use as slim cutter ring tip-width able to sustain the cutterloads required to gain highest possible rate of penetration. In split-face situations the impact on cutter rings may cause ring chipping. An answer to this might often be to increase the tip-width of the constant-cross-section cutter ring. During the excavation of the access tunnel at Lower Røssåga, ring chipping problems have been overcome by increasing the tip-width and thus reducing cutter consumption.

10.2 The Follo Line Project
The new double track line between Oslo and Ski is currently the biggest infrastructure project in Norway and includes a 19.5 km twin-tube single track tunnel which will be Norway's longest railway tunnel to date. The high-speed railway tunnel is designed for maximum tunnel capacity, safety and availability for maintenance.

Jernbaneverket, the Norwegian National Rail Administration (NNRA) has decided to construct the main part of the tunnel using four large tunnel boring machines (TBMs), as well as traditional drill and blast excavation methods. This is the first time that tunnel boring machines will be used in Norway to build a railway tunnel.

Four shielded tunnel boring machines will excavate the major part of the Follo Line tunnel, a rock tunnel with a total length of 19.5 km. From a construction site at Åsland south of Oslo, two TBMs will bore southwards to the tunnel portals at Langhus, while two TBMs will bore northwards to the suburb of Bekkelaget.

It has been provisionally planned to use conventional drill and blast (D&B) methods for the inner part of the tunnel, approx. 1 km section towards Oslo Central Station.

Drill and blast excavation will also be used for all cross passages between the two tunnel tubes, access tunnels to the main tunnel, a large chamber for underground TBM assembly and service area for construction work, and a rescue station. The project includes, as well a new single track tunnel on the old Østfold Line for traffic to Oslo Central Station. The construction of the Follo Line's twin tube single track tunnels is an extensive project, significantly larger than any previous railway project in Norway. The use of TBMs as the primary method of construction paves the way for major national and international participation in the construction of what will be Norway's longest railway tunnel to date.

10.2.1 Geology
In general terms the rock conditions for the tunnel are considered good, with generally little fracturing and good stability. However, poor rock conditions will be encountered in local weakness zones intersecting the tunnel alignment. The rocks of the project area consist predominantly of Precambrian gneisses. A significant number of intrusives from the Permian period, as well as amphibolite dykes/sills, occur. The amphibolite dykes/sills are more prevalent in the project area than the Permian intrusives. Most dykes/sills are a few meters thick, a few thicker than 10 m. Sedimentary schistose rocks occur in a very short part in the North toward Oslo Central Station. Investigations carried out so far show moderately to low fractured rocks (with the exception of limited parts with fractures, gouges and weakness zones), which are fresh and not weathered.

The area is known for large anisotropic stress with high horizontal stress (σH is measured to 9.4 MPa). However, problems with spalling, rock burst or squeezing are not expected. It is not anticipated that low stress will cause stability problems outside areas with very low rock cover. An important observation is that the tunnel axis is parallel

Assembly of a 9.8m diameter Double Shield TBM and Backup. Photo: The Robbins Company

or sub-parallel to the foliation and a prominent jointing direction. Some areas include tick layers of marine clay and silty clay. Lowering the ground water level in these areas may cause development of settlements in buildings on top of the tunnel sections. Hence it is important to limit water leakage into the tunnel during the construction period and for the operating period.

10.2.2 Preliminary works
Crucial preliminary works will start on the Follo Line Project in springtime 2014 before commencement of the main works in 2015. In April 2014, the contract for construction of the long tunnel will be issued to tender both nationally and internationally.

10.2.3 TBM tunneling
In Norway, drill and blast is the traditional method for constructions of road- and railway tunnels. Internationally it is more common to use tunnel boring machines for long tunnels.

Both conventional drill and blast and TBM excavation methods have been considered. The Norwegian National Rail Administration has concluded that tunnel boring machines are the most suitable option for the construction of the almost 20 km long twin-tube tunnel.

The decision in favour of TBM excavation is taken after extensive study, geological survey and expert consideration of the technical, logistical and environmental issues. An earlier decision by NNRA ruled out design of a single tube, double-track tunnel of 118 m2 as this would have required either escape adits to the surface at regular intervals, or a parallel 25m2 service tunnel with escape connections every 1,000m. Single-track parallel tunnels of 75m2 with cross passages at 500m intervals fulfil the operating safety requirements. Drill and blast or TBM excavation was appropriate for single-track tunnels and the final decision might have been left to the successful EPC construction group. However, after considering the pros and cons of each, NNRA has specified TBM excavation.

TBMs of approx. 9.8m diameter will be needed to create the tunnel designed for 250 km/h high speed train services. A bolted and gasketed precast concrete segmental lining is also specified as part of the TBM excavation operation to meet strict specifications for the control of ground water inflows. Using single or double shield TBMs and erecting the segmental lining as excavation progresses also adds programme advantages to the TBM method for the long drives.

The single central working site at Åsland, eliminates up to six additional adits that would have been necessary if the project had decided to divide D&B-sections in the tubes into 6 x 4 headings of about 2 km each.

D&B will be used on the project for nearly one quarter of the excavation required. This includes excavation of the two central working adits, each up to 1,000m long, and the emergency cross-passages between the two TBM drives. D&B is also likely to be used for excavation of the innermost part of the tunnel, a 1 km section of the twin running tunnels on the approaches to the Oslo Central Station. The alignment through the Ekeberg Hill passes by existing road tunnels and other underground infrastructure and final decision about the most appropriate method of excavation are under consideration. This might lead to an extension of the TBM bored section between Åsland site and Oslo.

10.2.4 Twin-tube, single-track tunnel
This is the first time in Norway that a twin-tube railway tunnel is to be built. There are three main reasons why the twin-tube option for the 19.5km long rock tunnelled section was recommended and approved:

- The twin-tunnel solution has significant advantages regarding safety:
  - Limitation to one tunnel of the operational consequences of an incident. The other tunnel will then serve as an evacuation tunnel and provide access for rescue activities from the outside.
    - In case of an emergency it avoids the risk of passengers being run over by trains travelling in the opposite direction
    - Facilitation of ventilation and smoke removal devices are simpler
- Improved operation and maintenance conditions
- Optimum train/traffic handling

10.2.5 Lining of the tunnels
The TBM system to be used for the tunnel comprises impermeable gasketed pre-cast concrete segments installed to ensure protection from rock fall, as well as water and frost. The space behind the concrete segments will be filled with mortar to seal the gap towards the tunnel wall.

Production and installation of concrete segments will form part of an industrialised process. This will help ensure a high and consistent quality to components, as well as the actual installation process. From a life cycle

Pre-cast concrete segmental lining in a 10m diameter tunnel. Photo: The Robbins Company

Inspection of a 10m diameter back-loading cutterhead.
Photo: The Robbins Company

perspective, precast concrete segments will require less maintenance than the more traditional form of rock support using rock bolts and sprayed concrete.

For the D&B tunnelling systematic pre-excavation grouting for control of ground water ingress as required is mandatory. Primary support of rock bolts and sprayed concrete would be installed as excavation progresses. The water and frost protection shell to be adopted throughout the tunnel will consist of a full waterproofing membrane system and a final cast-in-place concrete lining.

Choosing machinery that is suited to the ground conditions and crews with TBM experience from similar rock conditions is crucial. Extensive knowledge of ground conditions is an important prerequisite to success.

10.2.6 A major construction site

At Åsland, nearby the European highway E6 southwards out of Oslo, a large rigging area will be established. Concrete segments, to be used for the tunnel lining, will be manufactured here. Large areas will be required for this production work and for temporary storage of the pre-cast concrete segments. Space will also be required for other tasks and other logistics.

Two declined access tunnels and an underground erection chamber for the assembly of the TBMs will be constructed utilising conventional drill and blast methods. Together, the two tunnels will cater for incoming and outgoing traffic, as well as transportation of TBM muck out of the tunnel via conveyor belts.

The Norwegian National Rail Administration is planning the construction of a rescue station in the service area at the main tunnel level between the two declined access tunnels.

The access tunnels are also important for ventilation of the main tunnels during the construction phase and for the rescue station during the operation of the tunnel.

The four large shielded TBMs and Back-up will be assembled underground. The TBM muck haulage will be handled by using continuous conveyor systems throughout the main tunnels to the junction of the access tunnels. There the muck will be loaded on inclined conveyors and transported to a temporary deposit area at Åsland site.

10.2.7 Tunnel spoil

Around 10 million tons of rock spoil will be removed during tunnel construction. Following an invitation of interest, the Norwegian National Rail Administration has established contact with various public and private sector parties who can make use of the spoil. It is important that the rock spoil is recycled in a socially and environmentally acceptable manner.

It is assumed that the Follo Line project could recycle up to 20 % of the TBM excavation material for concrete production purposes.

Safe transportation of spoil is also crucial in order to minimise the impact on the local environment. From an environmental perspective, it is advantageous that most of the spoil ends up at Åsland site, which has direct access to the European highway E6, rather than being removed from eight access tunnels and transported along local roads in dense populated areas. Rock spoil, from what will be Norway's longest railway tunnel to date, will be removed over a period of around 3-3.5 years.

10.2.8 A more sustainable method, design and construction of rail-infrastructure

The Norwegian National Rail Administration/Follo Line Project is the first organisation in Norway to formulate a green assessment method governing the choices of materials and the construction methods. The project will be planned with input factors such as materials and energy, which after an assessment of factors such as quality, environmental impacts, safety and cost, ensures low environmental impacts throughout the project lifecycle. The project's environmental impacts (including climate change) will be documented in a Life Cycle Assessment environmental report.

The Follo Line project is scheduled for completion in summer of 2021.

10.3 Ulriken Railway Tunnel Project

On the Bergen line, the Norwegian National Rail Administration has started planning for extending from single to double track between Bergen and the suburban town Arna, which will include the new 7.8 km long Ulriken tunnel. The alignment of the new 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 will be allowed for.

In 2013 NNRA decided to tender the project based on TBM excavation, as well.

Prequalified contractors/ Joint ventures submitted their bids in February 2014. NNRA has received quite a few bids for both conventional D&B and for the TBM excavation method. The contract will be a negotiated unit price contract and it is expected that the contract will be awarded in May 2014.

10.4 A new TBM era in Norway

The choice of Statkraft and LNS of using TBM for the construction of the headrace tunnel at Lower Røssåga project and the NNRA decision of using four TBMs to excavate the major part of the Follo Line tunnels have brought TBM excavation expertise back to Norway after 22 years since the last TBM project. In some ways one may call it the start of a new TBM era in Norway. Hopefully, the evaluation team of the Ulriken railway tunnel project might also come to the conclusions using TBM excavation method for their project.

REFERENCES

Hansen, A.M. (1998): "The History of TBM Tunnelling in Norway". Norwegian Soil and Rock Engineering Association, Publ. No.11

Hansen, A.M. (2004): "TBM Tunnelling". Norwegian Tunnelling Society, Publ. No.14

Kalager, A.K., Smith, E. «Das Follo-Line-Projekt – Bau des längsten Eisenbahntunnels in Norwegen». Beton- und Stahlbetonbau Spezial – Europas lange Tunnel (Juli 2013)

Wallis, Shani, TunnelTalk, 26 Sep. 2013: "Hydro brings TBMs back to Norway"

The Robbins Company Press Release, January 8, 2014: "Robbins Røssåga Machine marks the Return of TBMs to Norway"

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Underground car park in Arendal. Courtesy: Veidekke ASA

16. THE NTNU PREDICTION MODEL FOR TBM PERFORMANCE

BRULAND, Amund

ABSTRACT
Prediction models for excavation rates and costs of tunnelling are used for several purposes, e.g. time scheduling, cost estimates, tendering, budgeting and cost control, and for choice of excavation method. This paper treats the prediction model for hard rock tunnel boring developed by NTNU, the Norwegian University of Science and Technology. The model estimates advance rate, cutter life and excavation costs based on rock mass and machine parameters.

1 INTRODUCTION
The complete prediction model is published in /1/ and /2/, which is the latest revision of a series of prediction models on TBM tunnelling published by the Norwegian University of Science and Technology since 1976. The current model is based on data from 35 projects with more than 250 km of tunnels, partly in very demanding rock conditions. The engineering geology of the tunnels has been carefully back-mapped, and production and cost data have been analysed.

The NTNU TBM model estimates:

- Net penetration rate (mm/rev, m/h)
- Cutter life (h/cutter, sm^3/cutter)
- Machine utilisation (%)
- Weekly advance rate (m/week)
- Excavation costs (NOK/m)

The net penetration rate and cutter wear depend on rock properties and machine parameters.

The prediction model is available as a software, FULLPROF /3/, which may help when analysing the effect of variation in one or more input parameters on penetration rate, cutter wear etc.

2 ROCK MASS PARAMETERS
Through the input parameters, the following rock mass properties are considered:

Rock Mass Parameters	Machine Parameters
• Fracturing; frequency and orientation • Rock type drillability or strength • Rock type porosity	• Cutter thrust • Cutterhead rpm • Cutter spacing • Cutter size and shape • Installed cutterhead power

Table 1. Machine and rock mass parameters influencing the net penetration rate. /1/

Rock Mass Parameters	Machine Parameters
• Rock type abrasivity • Content of abrasive minerals	• Cutter diameter • Cutter type and quality • Cutterhead diameter and shape • Cutterhead rpm • Number of cutters on the cutterhead

Table 2. Machine and rock mass parameters influencing the cutter wear. /1/

- Degree of fracturing by Fracture Class and the angle between the tunnel axis and the planes of weakness.
- Drillability by the Drilling Rate Index DRI /4/.
- Abrasiveness by the Cutter Life Index CLI /4/ and the quartz content.
- For some rock types, porosity is also an input parameter.

Of the above mentioned parameters, the rock mass fracturing is by far the most important. The estimated penetration rate (m/h) may vary by a factor of five from homogenous (i.e. non-fractured) to well fractured rock mass. For homogenous rock mass, estimated penetration rate may vary by a factor of two from very low to very high DRI values. Since the net penetration rate is an important factor for the weekly advance rate, the cutter life and the excavation costs, it is obvious that great efforts should be made to establish a best possible model of the rock mass fracturing of a tunnel project.

3 MACHINE PARAMETERS

The estimation model uses the following machine parameters:

- Average cutter thrust (kN/cutter)
- Average cutter spacing (mm)
- Cutter diameter (mm)
- Cutterhead RPM (rev/min)
- Installed cutterhead power (kW).

When boring in hard rock, the average cutter thrust is the most important machine parameter. Hence, the technology development has focused on larger cutters to be able to sustain the required thrust. In strong and homogenous rock masses, a TBM equipped with 483 mm diameter cutters may typically have a net penetration rate (m/h) that is 30-50 % higher than that of a TBM with 432 mm diameter cutters. At the time being, the limiting factor for more efficient boring in hard rock is the material quality of the ring steel of the cutters. Further improvement of penetration rate and cutter life in hard rock conditions may benefit from improved cutter ring material and cutterhead design.

4 NET PENETRATION RATE

Figure 1 shows a general penetration curve of a TBM. The penetration curve illustrates the effect of increased thrust in a given rock mass. M_B is the average cutter thrust and b is the exponent of the penetration curve. In strong and non-fractured rock mass, the exponent b has been measured as high as b = 6. In more frequently occurring rock masses, the b is typically 2.5, i.e. the penetration rate will increase by 27 % if the cutters may sustain an increase of 10 % in the applied thrust.

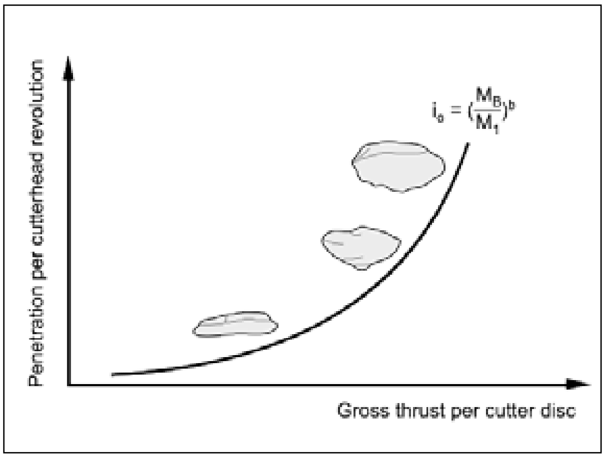


Figure 1. Penetration in mm/rev as a function of cutter thrust in kN/cutter. /5/

The basic philosophy of the NTNU prediction model is to simulate the penetration curve based on rock mass and TBM parameters. The reason for this approach is that efficient breaking of rock is an interaction between the rock face and the cutterhead. The model combines the decisive rock mass parameters into one rock mass boreability parameter, i.e. the equivalent fracturing factor k_{ekv}, and the relevant machine parameters into one machine or cutterhead parameter, i.e. the equivalent thrust, M_{ekv}.

4.1 Equivalent Fracturing Factor

The equivalent fracturing factor is found by combining:

- Rock mass type of systematic fracturing (Fissures or Joints)
- Rock mass degree of fracturing (Fracture Class 0 to IV)
- Angle between planes of weakness and the tunnel axis (0 to 90 degrees)
- Rock strength (drillability) expressed by DRI.

The type of systematic rock mass fracturing is divided into Fissures and Joints. Joints are continuous, i.e. the planes of weakness may be followed all around the tunnel contour. One example is bedding joints in granite. Fissures are non-continuous, i.e. the planes of weakness can only be followed partly around the tunnel contour. A typical example is bedding planes in mica schist.

The degree of fracturing is divided into Fracture Classes for practical use when mapping.

The nature is of course not as simple as the classification above can express, as illustrated in Figures 2 and 3. Sometimes, continuous joints predominate, at other

Fracture Class (Joints = Sp / Fissures = St)	Spacing between the Planes of Weakness [cm]	Grouping of Classes with Regard to Spacing [cm]
0	∞	240 - ∞
0-I	160	120 - 240
I-	80	60 -120
I	40	30 - 60
II	20	15 - 30
III	10	7.5 - 15
IV	5	4 - 7.5

Table 3. Fracture Classes for systematically fractured rock mass./6/

times bedding plane fissures or foliation planes are more dominant. In schistose rock, it may be difficult to distinguish between the schistosity of the rock type and fissures along the schistosity planes. Hence, one has to simplify and use sound judgement to avoid making too complicated rock mass models.

The angle between the tunnel axis and the planes of weakness is calculated as follows:

$$\alpha = \arcsin(\sin\alpha_f \cdot \sin(\alpha_t - \alpha_s)) \qquad (°)$$

α_s = strike angle
α_f = dip angle
α_t = tunnel direction.

The fracturing factor is shown in Figure 4, as a function of Fissure or Joint Class and angle between the tunnel axis and the planes of weakness.

Figure 2. Sometimes it is easy. Gneiss, fissure class II-, i.e. about 25 cm average spacing. /6/

Figure 3. Sometimes it is more difficult. Mica gneiss, fisure class I-, i.e. about 80 cm average spacing. /6/

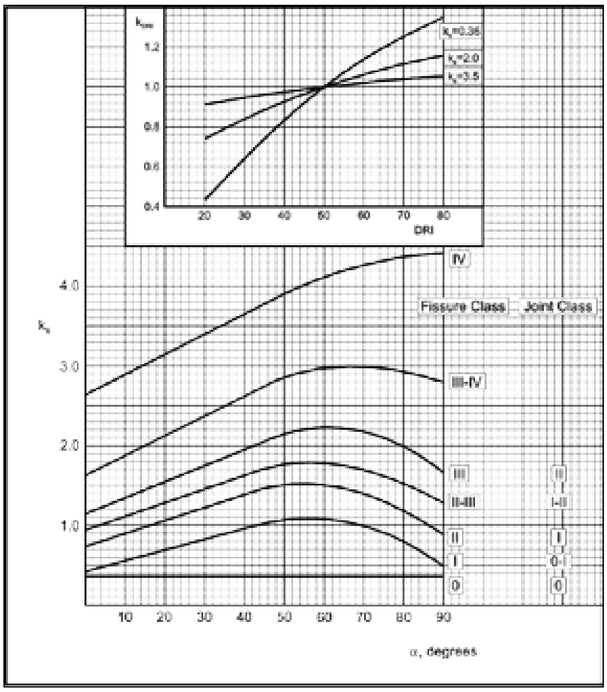


Figure 4. Fracturing factor. Correction factor for DRI ≠ 50. /1/

If more than one set of weakness planes are included in the model (maximum 3 recommended), the total fracturing factor is:

$$k_{s-tot} = \left(\sum_{i=1}^{n} k_{si} \right) - (n-1) \cdot 0.36$$

k_{s-tot} = total fracturing factor
k_{si} = fracturing factor for set no. i
n = number of fracturing sets.

The equivalent fracturing factor is

$$k_{ekv} = k_{s-tot} \cdot k_{DRI} \cdot k_{por}$$

k_{ekv} = equivalent fracturing factor
k_{DRI} = correction factor for DRI of the rock type
k_{por} = correction factor for porosity of the rock type.

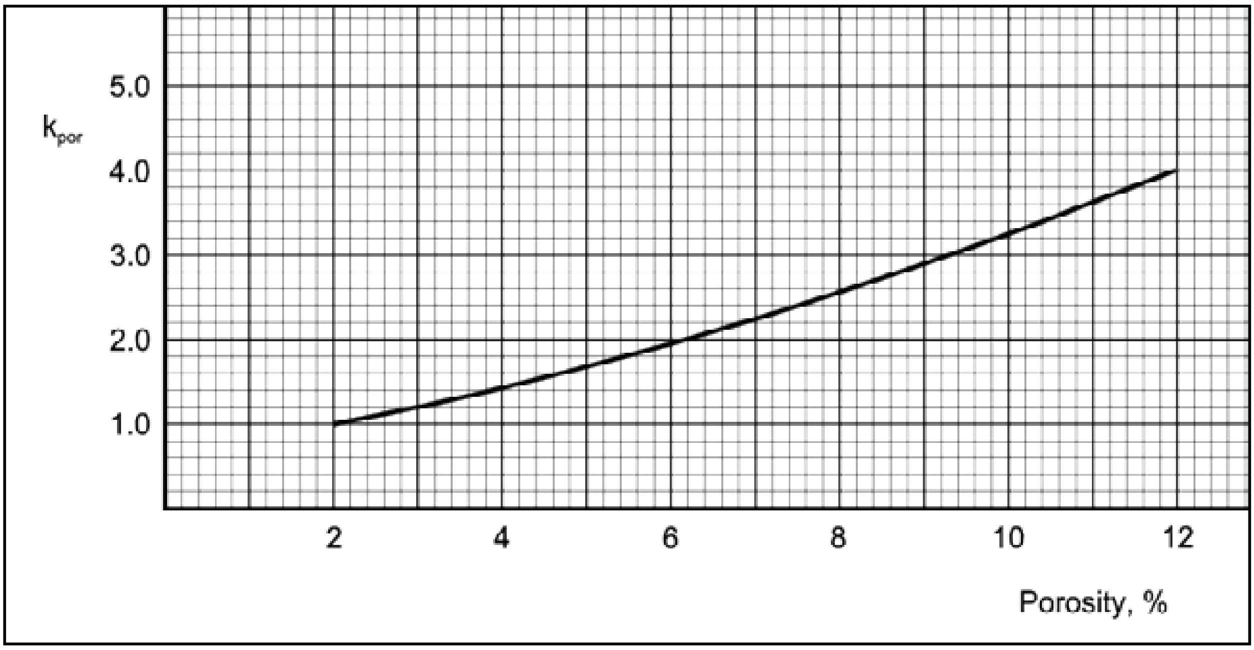


Figure 5. Influence of rock porosity on the equivalent fracturing factor. /1/

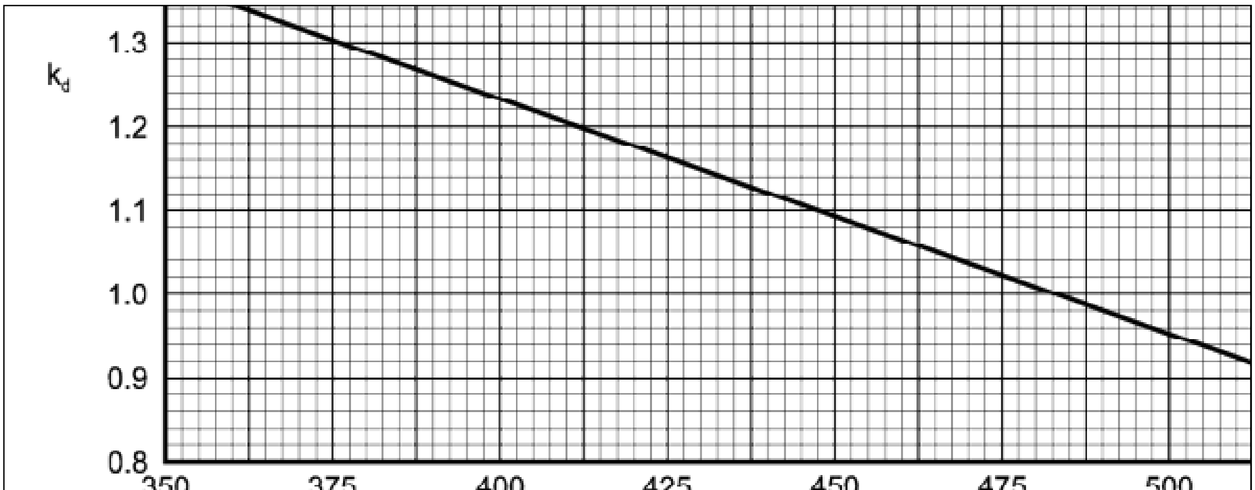


Figure 6 Correction factor for cutter diameter. /1/

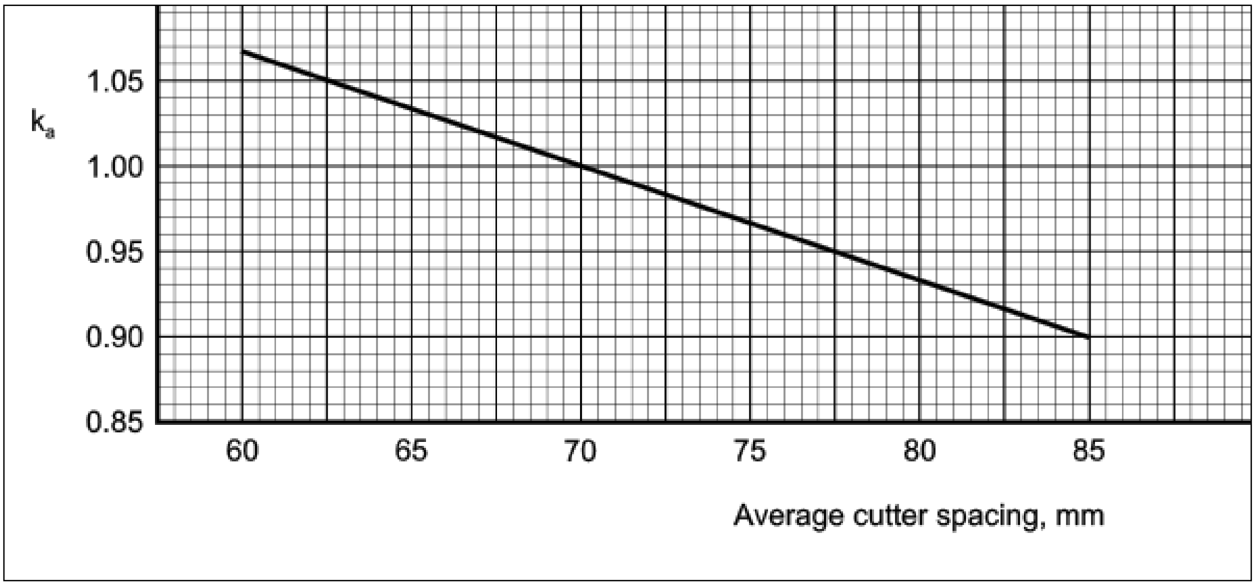


Figure 7 Correction factor for average cutter spacing. /1/

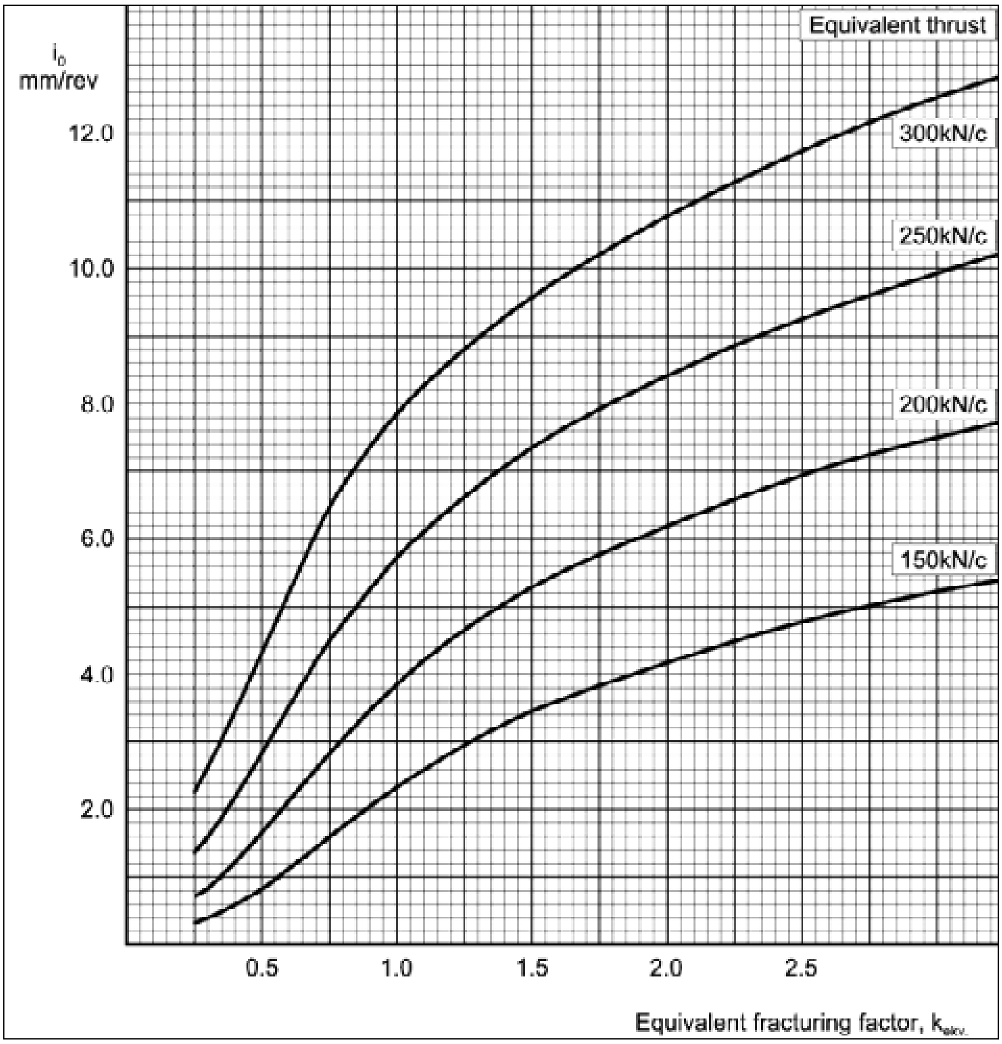


Figure 8 Basic penetration rate. /1/

4.1.1 Equivalent Thrust

The basic penetration rate is expressed in mm per revolution of the cutterhead. Hence, the decisive machine characteristics are independent of the TBM diameter. The equivalent thrust is expressed as:

$$M_{ekv} = M_B \times k_d \times k_a \qquad \text{(kN/cutter)}$$

M_B = gross average thrust per cutter, i.e. not the available thrust capacity of the machine, but the actual thrust (to be) used.
k_d = correction factor for cutter diameter; an indirect expression of the contact area under the cutter.
k_a = correction factor for average cutter spacing.

Average cutter spacing is found by dividing the cutterhead radius by the total number of cutters on the cutterhead.

Basic penetration rate i_0 as a function of equivalent thrust and equivalent fracturing factor is shown in Figure 8. The graphs in Figure 8 assume that the cutterhead rpm is according to the following:

- Rolling velocity of the gauge cutter is 2.62 m/s for 483 mm diameter cutters.
- Rolling velocity of the gauge cutter is 2.30 m/s for 432 mm diameter cutters.

The estimated basic penetration rate should be checked with regard to the torque capacity of the cutterhead drive system. A detailed model for torque capacity is presented in /1/.

Net penetration rate in m/h is found by

$$I_n = i_0 \cdot RPM \cdot \left(\frac{60}{1000}\right) \qquad \text{(m/h)}$$

RPM = cutterhead revolutions per minute.

5 ADVANCE RATE

The gross advance rate is given as bored metres per week as an average for a longer period, and depends on net penetration rate, machine utilisation and the number of working hours during one week. The machine utilisation is net boring time in percent of total tunnelling time. The total tunnelling time includes:

- Boring, T_b
- Regripping, T_t
- Cutter change and inspection, T_c
- Maintenance and service of the TBM, T_{tbm}
- Maintenance and service of the back-up equipment, T_{bak}
- Miscellaneous activities, T_a
  - Occational rock support in good rock conditions, i.e. rock support that may be installed while boring and without increasing the tunnelling crew
  - Waiting for transport
  - Tracks or roadway; installation and maintenance
  - Surveying, moving of laser
  - Water, ventilation, electric cable; installation and maintenance
  - Washing and cleaning of the TBM and the backup
  - Other (change of crews, incidental lost time, etc.).

In addition to the listed items, miscellaneous includes time loss connected to the tunnelling method and organisation, and unforeseen time consumption. The time consumption is estimated in h/km, and the machine utilisation is given by:

$$u = \frac{100 \cdot T_b}{T_b + T_t + T_c + T_{tbm} + T_{bak} + T_a} \qquad (\%)$$

$$T_b = \frac{1000}{I_n} \qquad \text{(h/km)}$$

$$T_t = \frac{1000 \cdot t_{tak}}{60 \cdot l_s} \qquad \text{(h/km)}$$

l_s = stroke length, typically 1.5 - 2.0 m
t_{tak} = time per regrip, typically 4 – 5 minutes.

$$T_c = \frac{1000 \cdot t_c}{60 \cdot H_h \cdot I_n} \qquad \text{(h/km)}$$

t_c = time per changed cutter, typically 45 - 60 minutes
H_h = average cutter ring life.

The time consumption in Figure 9 is based on 100 working hours per week. Hence, it is presupposed some available time outside the standard working hours to handle unforeseen and critical incidents like major repairs. Some parts of such time consumption are not recorded in the shift logs and are therefore not included in Figure 9.

Figure 10 indicates that the possibilities to handle unforeseen and critical incidents in a flexible manner are fewer as the weekly working hours T_u increase

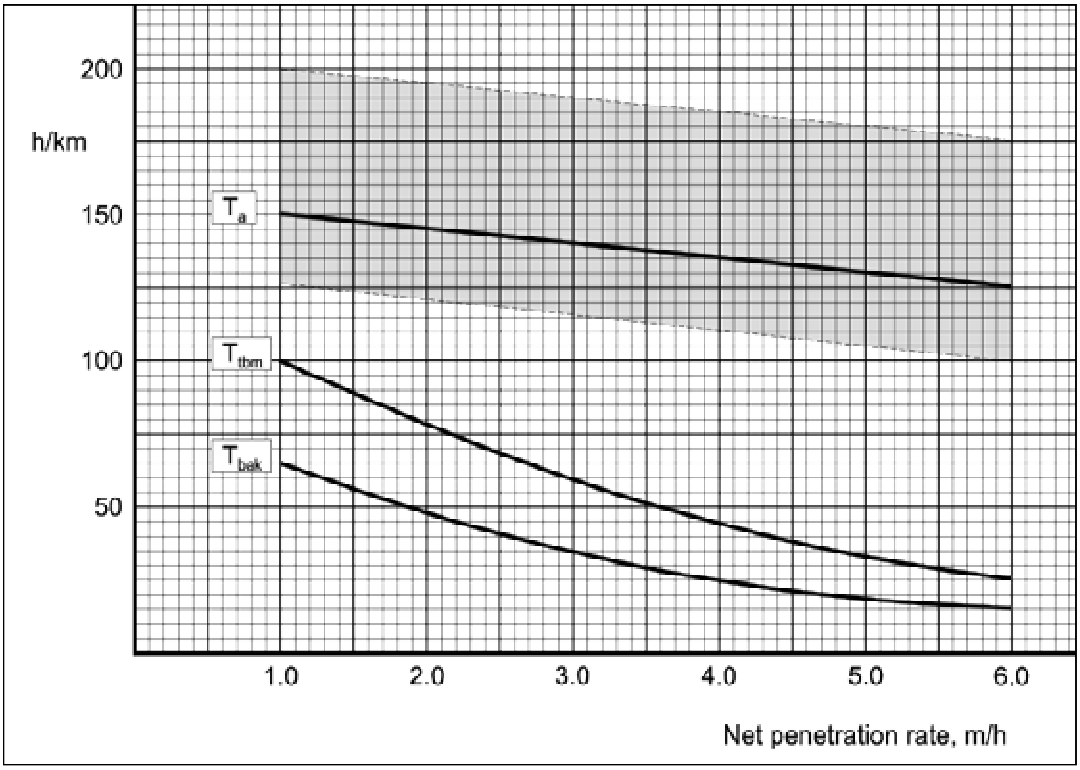


Figure 9. Time consumption for various activities. /1/

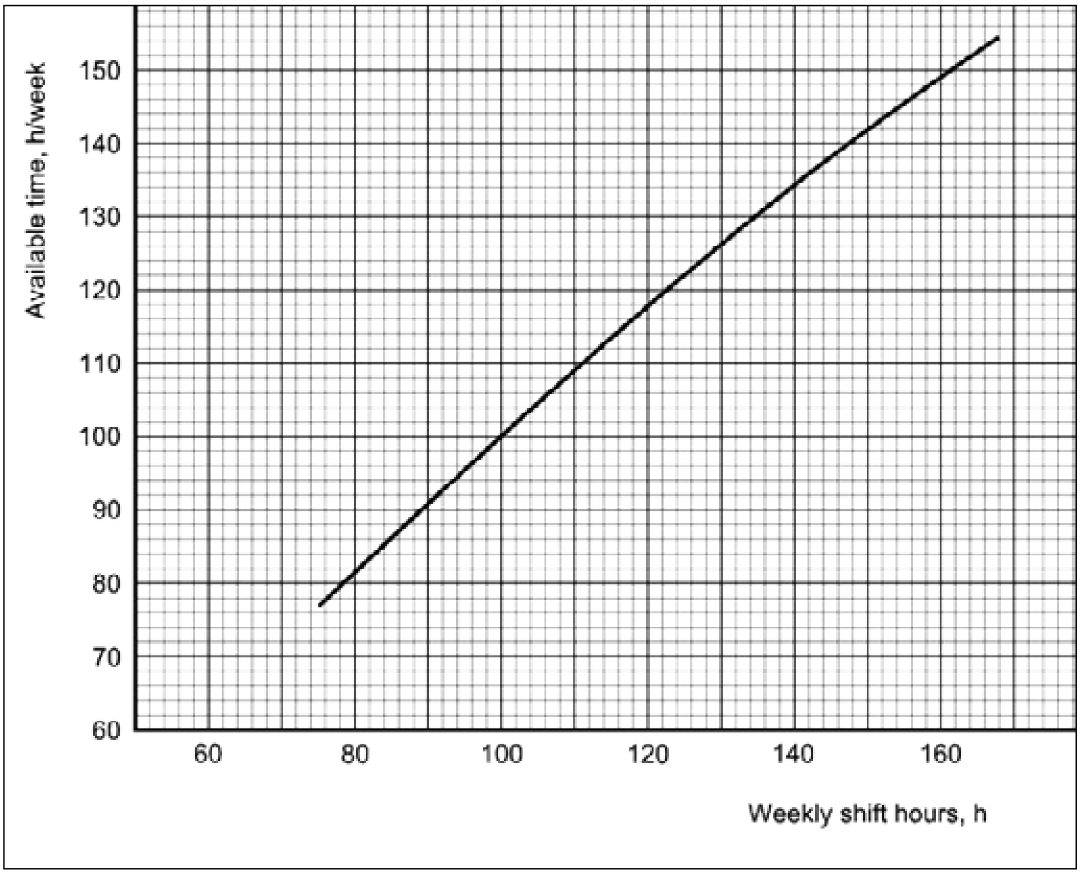


Figure 10. Effective working hours per week. /1/

towards 168 hours. T_e expresses the available effective working hours (in prediction model terms) when the weekly working hours differ from 100. The curve is based on relative few observations, but is believed to be a conservative estimate regarding the loss of effective working hours.

For the total tunnelling time, extra time must be added for:

- Excavation of underground assembly and start-up area, tip station, etc., if applicable.
- Assembly and disassembly of the TBM and the backup equipment at site, normally from 4 to 8 weeks, mainly depending on the TBM diameter.
- Excavation of niches, cross passages etc.
- Boring through and stabilising zones of poor rock mass quality.
- Additional time for unexpected rock mass conditions.
- Permanent rock support and lining work.
- Downtime TBM (additional time for possible major machine breakdowns).
- Dismantling of tracks, ventilation, invert clean-up, etc.

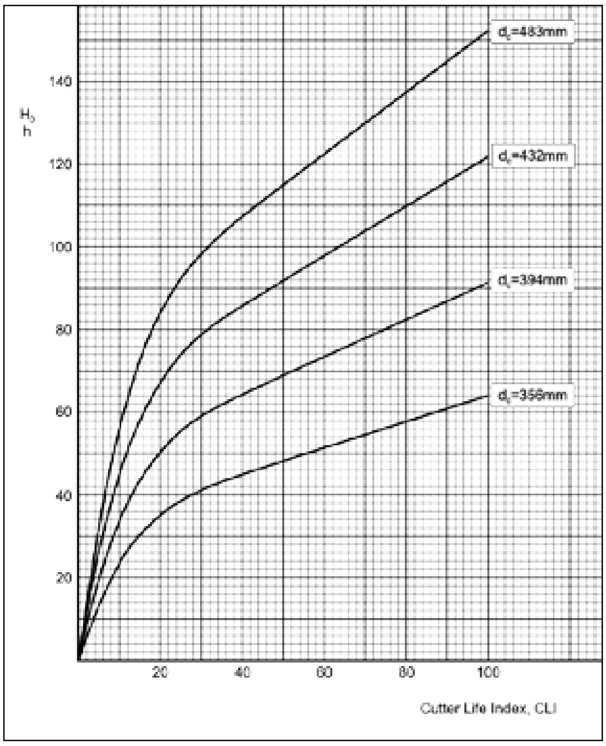


Figure 11 Basic cutter ring life H_0. /1/

6 CUTTER CONSUMPTION

The NTNU model estimates cutter ring life in boring hours, i.e. the number of boring hours it will take to wear down one cutter ring in the average position on the cutterhead. From the life in boring hours, cutter ring consumption expressed as m per cutter ring (i.e. tunnel metres per cutter ring) or sm^3 per cutter ring can be derived.

The cutter life estimation model presupposes that the TBM is operated at a thrust level resulting in mainly abrasive wear of the cutter rings. The amount of blocked cutters and cutter rings worn by ring chipping or other destructive wear should be less than 10 - 20 % of the total number of changed cutter rings.

The average life of cutter rings is given by:

$$H_h = (H_0 \cdot k_D \cdot k_Q \cdot k_{rpm} \cdot k_N) / N_{tbm} \qquad \text{(h/c)}$$

H_0 = basic cutter ring life
k_D = correction factor for TBM diameter
k_Q = correction factor for rock quartz content
k_{RPM} = correction factor for cutterhead rpm
k_N = correction factor for number of cutters
N_{tbm} = number of cutters on the cutterhead.

$$k_{rpm} = \frac{50 / d_{tbm}}{RPM}$$

d_{tbm} = TBM diameter (m)
RPM = cutterhead rpm.

$$k_N = \frac{N_{tbm}}{N_0}$$

N_{tbm} = actual number of cutters
N_0 = normal number of cutters.

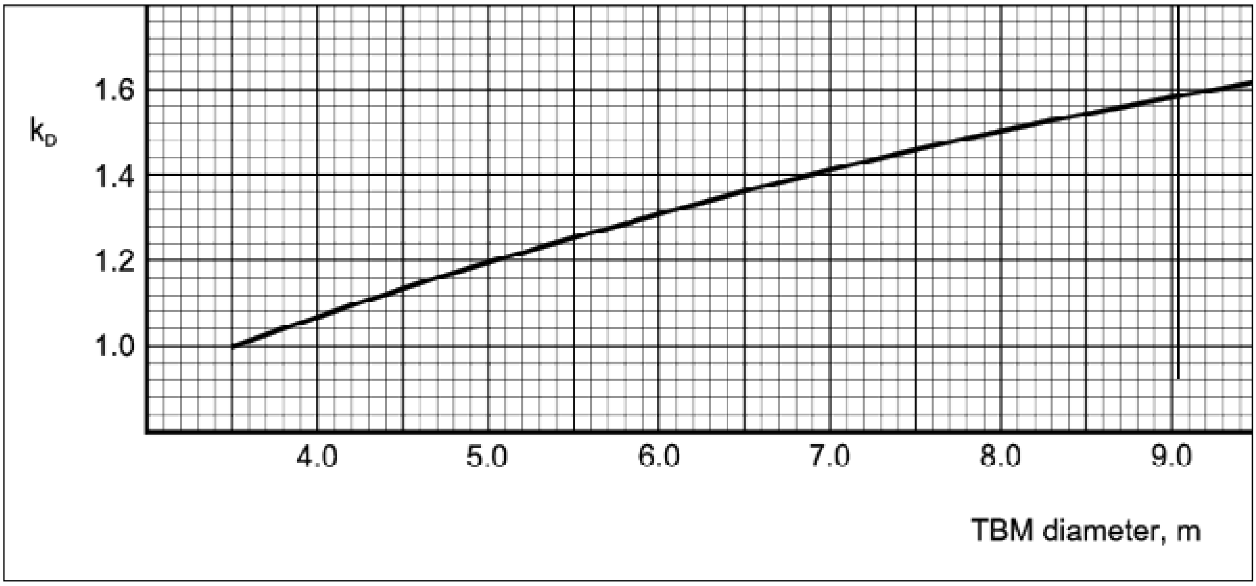


Figure 12. Correction factor for TBM diameter. /1/

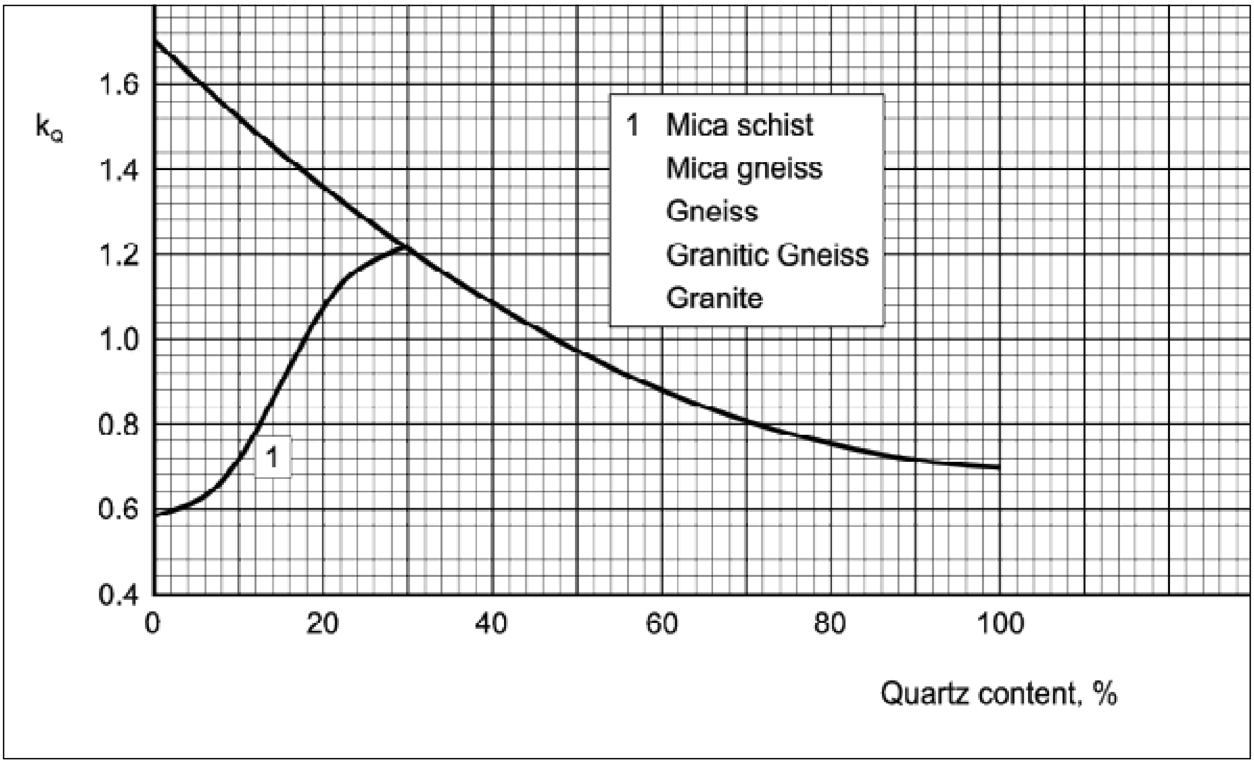


Figure 13. Correction factor for quartz content. /1/

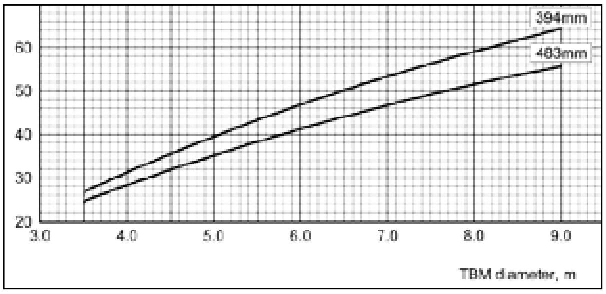


Figure 14. Normal number of cutters on the cutterhead. /1/

7 GEOLOGICAL PARAMETERS

The background data of the NTNU model have been utilised to make summaries of the observed geology of more than 250 km of TBM tunnels. The following shows the distribution of these parameters for the most frequently observed rock types.

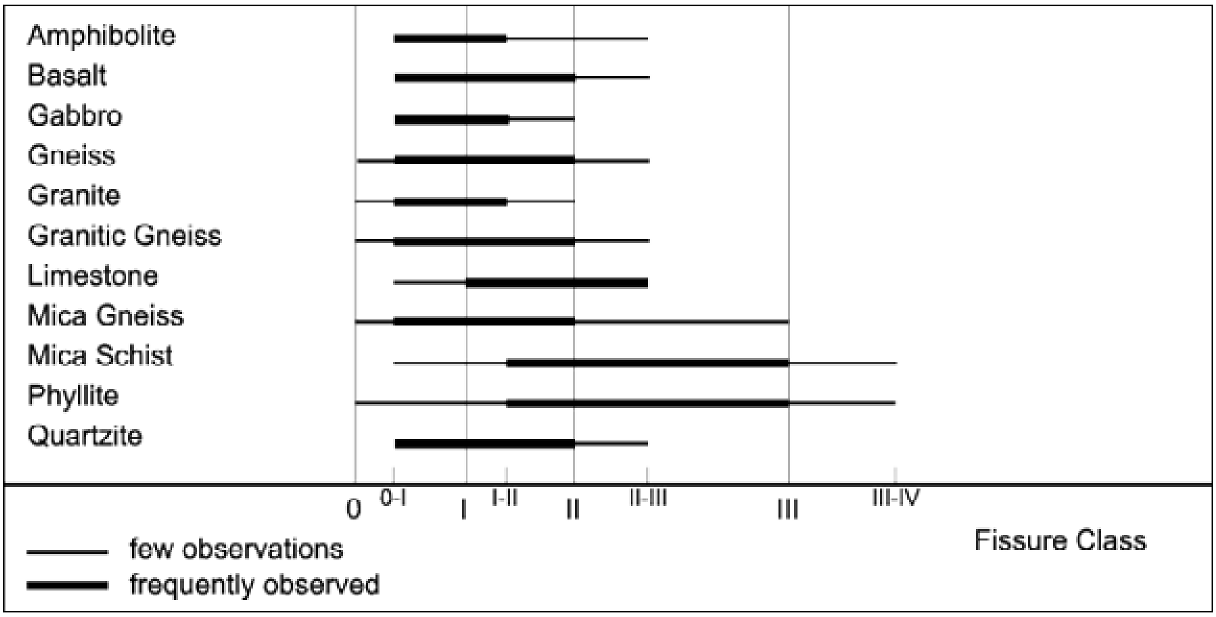


Figure 15. Recorded degree of fracturing for some rock types. /1/

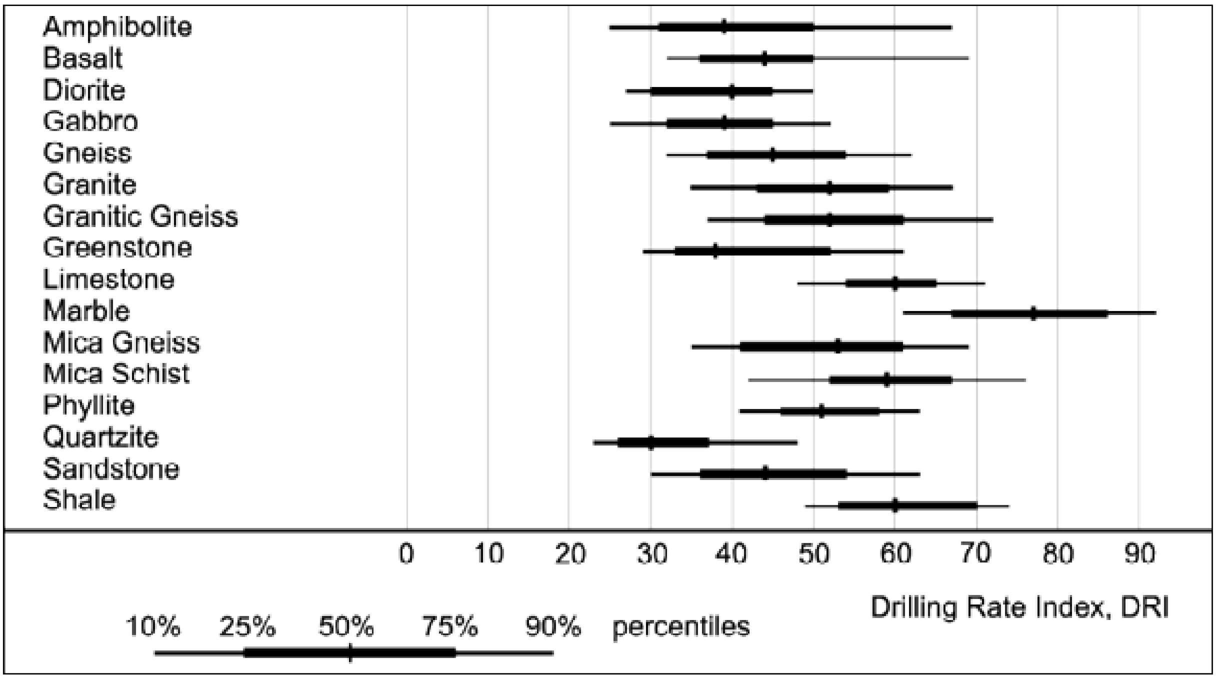


Figure 16. Recorded Drilling Rate Index for some rock types. /1/

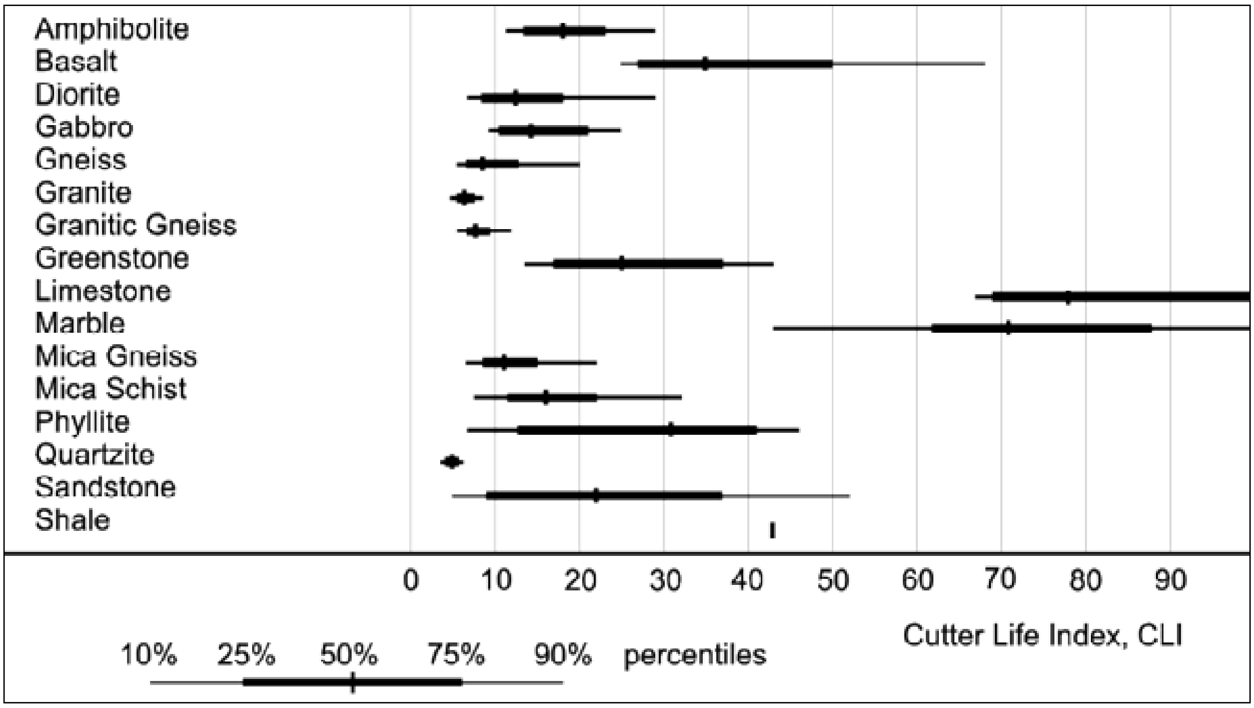


Figure 17. Recorded Cutter Life Index for some rock types. /1/

REFERENCES

Bruland, A. (1998) Project Report 1B-98 Hard Rock Tunnel Boring, Advance Rate and Cutter Wear. Doctoral theses at NTNU 1998:81, Vol. 3 of 10. Trondheim: Norwegian University of Science and Technology.

Bruland, A. (1998) Project Report 1C-98 Hard Rock Tunnel Boring, Costs. Doctoral theses at NTNU 1998:81, Vol. 4 of 10. Trondheim: Norwegian University of Science and Technology.

Anleggsdata. (2010) Fullprof v4.0. Software developed by Anleggsdata AS. http://www.anleggsdata.no/, visited 07.04.2014.

Bruland, A. (1998) Project Report 13A-98 Drillability, Test Methods. Doctoral theses at NTNU 1998:81, Vol. 8 of 10. Trondheim: Norwegian University of Science and Technology.

Bruland, A. (1998) Project Report 1F-98 Hard Rock Tunnel Boring, The Boring Process. Doctoral theses at NTNU 1998:81, Vol. 7 of 10. Trondheim: Norwegian University of Science and Technology.

Bruland, A. (1998) Project Report 1D-98 Hard Rock Tunnel Boring, Geology and Site Investigations. Doctoral theses at NTNU 1998:81, Vol. 5 of 10. Trondheim: Norwegian University of Science and Technology.

Artist's impression of Holmestrand station now in a finishing stage. Courtecy: Norwegian National Rail Administration

17. FOCUS ON OPERATION, MAINTENANCE AND REPAIR - VALUE FOR MONEY

GJÆRINGEN, Gunnar

1 INTRODUCTION

A mountainous country with people living all over the available land, in valleys, on islands, along the coastline, in the highlands . Five million people with concentration in urban areas; this is the way it is, a well supported policy, but challenging – people need access to roads.

Sometimes one must accept that a simple road is better than no road and a tunnel without best standard better than no tunnel. With limited economy cost reductions were a necessity. Allocations were made by the politicians.

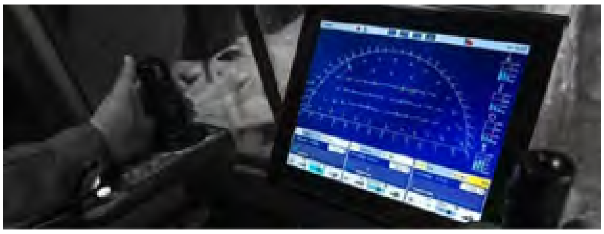
Even in a well-off country there are always insufficient means for new projects and the necessary maintenance of the existing system. Maintenance was frequently the looser.

Roads and tunnels are built to national standards, mostly governed by the traffic intensity: low volumes of traffic - low standards, high traffic density - higher standard.
In all cases , safety and security tunnel safety shall match the actual road safety.

2 UPGRADING AND COSTS

With reference to the recommended maintenance standards a maintenance backlog has accrued during the years. The Roads Authority has assessed the costs for eliminating the backlog on the county road network to be in the range of NOK 45 - 75 billions (109)
About 46 percent of the needed funds are related to pavements (incl. drainage), about 24 percent is related to tunnels, 17 percent is related to bridges and ferry quays and about 14 percent for environmental improvements.

A similar assessment for the national road network assumed some NOK 25 - 40 billion.
About half of the needed funds are related to tunnels, about 25 percent is related to pavements (incl. drainage), about 15 percent is related to bridges and ferry quays and about 10 percent is related to road furniture and environmental measures.



3 DEFINITION OF OPERATION AND MAINTENANCE

The objectives of the operation and maintenance of road tunnels are:

- Functional requirements for the tunnel and its equipment should be maintained
- Functional security should be attended to
- Safety equipment must comply with the given requirements
- To aim for equal standards for comparable systems
- New constructions should ensure future needs

- To aim for an optimization of the level of maintenance.
- Operations are all the tasks and procedures that are required for a building or an installation to work as planned
- Maintenance is all of the measures that are necessary to maintain a plant or a building on a fixed-quality

The aims must be more closely linked to:
- maintenance costs
- operational availability
- life span
- accessibility
- injuries
- security

An optimal maintenance means the lowest possible maintenance costs, short shut-down time, good operational availability and operational security, the longest possible life span of constructions and equipment and the safeguarding of the level of security. In order to realize the optimization of operation and maintenance in a tunnel, it is important that both the availability and maintenance-friendly solutions are sufficiently emphasized from the moment the planning begins. It is operations included inspection procedures and control by competent personell.

4 MANAGEMENT, OPERATION AND MAINTENANCE - MOM

If operational and maintenance tasks are to be resolved in the most cost-effective manner, the utilisation of the resources must be optimal. This requires a certain degree of predictability, which currently is not quite satisfactory. This predictability can be improved by continuously preparing guidelines for "The administration, operation and maintenance of tunnels". Such guidelines will provide the basis for new strategies, which in turn ensure the predictability of further production tasks. This will also make choices of strategy for production considerably easier. The following objectives of such guidelines are suggested:

- Management, operation and maintenance (MOM) should ensure that the accessibility in the tunnel should be as good as on the roads in general.
- MOM should ensure that:
  - the level of security in the tunnel's entry zones (that is 100 m before and the first 100 meters inside the tunnel) should be at the same level as the adjacent road
  - he level of security inside the tunnel should remain at the same level as on the adjacent roads without junctions and pedestrian-/bicycle traffic
- MOM should ensure appropriate preparedness for unforeseen events in the tunnel
- MOM should ensure that current environmental standards are complied with
- MOM should ensure that the tasks are carried out in an economic way for the community (LCC)

4.1 Management

Management consists of:

a. An administrative section.
b. A contingency plan section.
c. An operation and maintenance section.
d. Action plans
e. Documentation

Administrative section
The administrative section should contain a description of the actual structure.
- Name of tunnel
- Type
- Year of construction
- Key data

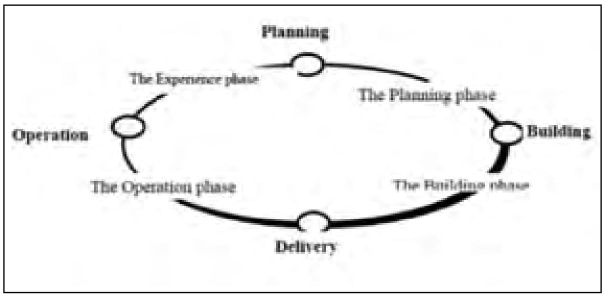
Moreover, it should provide an overview of the organization and the different areas of responsibility. The plan of the organization should describe the boundaries of responsibility for personnel related to the tunnel. The administrative procedures for quality-assurance should be described the filing and preparation of documents and the maintenance of the administrative system.

The contingency plan section
In road tunnels there is a need for fixed procedures for the determination of who should respond to an emergency call, and how fast this should happen when errors are discovered or accidents occur. This kind of contingency plan should contain a list of who is to be notified and when notification should occur depending on which events take place as normal practice say..

The operation and maintenance section
All of the elements of the tunnel are gathered and registered in the MOM-programme *PLANIA* with the preparation of maintenance procedures and the documentation of these procedures.

Action plans
For tunnels with so much technical equipment and costly solutions, plans should be drawn up based on a condition assessment including anticipated future repair- and replacement needs.

Documentation
This can consist of:
- Technical documentation



Figur 6. Phases in tunnelling

- Legal documentation
- Financial documentation
- Quality assurance material

4.2 Operations and maintenance

Operation and maintenance should be organized so that the fundamental conditions that existed when the tunnel was planned, are continued throughout the entire production phase. The standards and solutions that are selected here will affect future operation and maintenance needs.

- the preparation of a register for the individual tunnel
- a register for maintenance tasks, including a description of the individual elements
- a register for historical data, where experiences can be taken from.
- function agreements
- washing
- cleaning
- inspection
- electric power
- the flow of traffic
- clearing
- supplemental safety measures
- drainage plant
- maintenance of water safety measures
- maintenance of electrical safety measures
- pumps
- ventilation and waste water treatment plants

One must always be aware of the operational phase of 100 years.

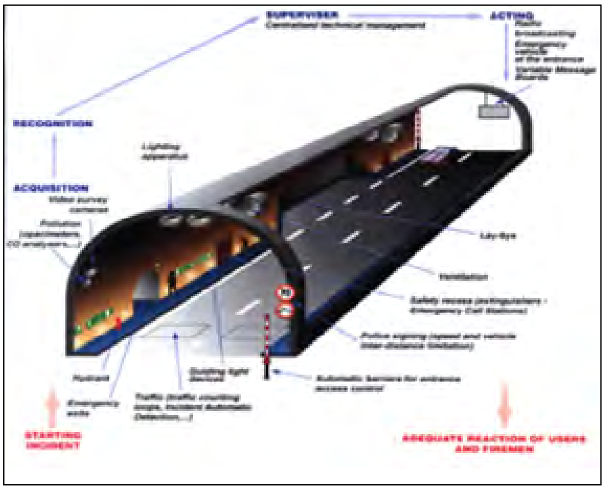
5 LIFE CYCLE COSTS

It has been gradually acknowledged that operation and maintenance costs to a large extent are determined by the decisions that are made as early as in the planning phase. This means that the need to provide a tool that makes it possible to focus on an overall operational-optimization is becoming more and more of current importance.

Fig 11: Fire in Seljestad tunnel.

Fig 12:. Queue in a tunnel.

Fig 12: From the fire in the Gudvanga tunnel



Figur 8. Old ventilator after corrosion.

Figur 9. New ventilators

Throughout the life span of a tunnel project renewals and upgrades of the technical equipment are carried out due to wear and tear and / or technical development. The main aim of any acquisition is an optimal life span, with the lowest possible costs.

6 METHODS

The operation and maintenance work in a tunnel should take place as far as possible without unnecessary obstacles to road users. The safety of both road users and maintenance personnel should be assured through the equipment that should be built into the tunnel's security system.

To achieve this, a systematic maintenance is necessary. The MOM-programme *PLANIA* should be used to set the correct maintenance routines. These maintenance routines will to a large extents be based on empirical data from a corresponding tunnel element and on the documentation of equipment that the builder gains access to at the acquisition of the completed tunnel. What kind of maintenance routines that should be used, depends on, among other things, length, profile, YDT (Year Daily Traffic) and what kinds of elements that are present in the tunnel. The NPRA defined as early as in 1988 that we should develop a system to manage the work with operation and maintenance in road tunnels. A system was developed together with a private company, where NPRA as owner of the tunnels, defined how often and how extensive the work should be performed in each tunnel. Frequency of action is based on YDT, length, the amount of equipment, function, and then of course, type of equipment found in the various tunnels. It has both a web version and a normal version.

7 CONTRACTS

All works will be advertised for bids. In this way the contractors must compete to get these works.

Figur 10. and 11. Broken cable bridges shown as an example after corrosion in Norwegian tunnels.

In these contracts it is listed a range of tasks in each tunnel. It is also specified how often, and when to be performed. The contracts last for up to 5 years and a contract may cover several tunnels. There are several types of contracts. These are divided according to subject areas as electrical work, work with the road and of course work with the tunnel construction. The contractor must document that the pre-defined tasks are performed. They do so by signing out of the so-called work orders. In this way we get a clear overview of all types of work being carried out, when they are performed and also about some tasks have not been done at the right time. If not, the contractor gets a fine. Statistics and graphs are produced to illustrate the performed work.

8 MAP

We have made a module map showing all road tunnels in the country with the specific location and other data. Clicking on the tunnel name, the layout with the tunnel data appears.

9 DOCUMENTATION

These data give a good documentation of what kind of operations done in each tunnel, that the tasks are performed as well as the status of the equipment - functionality, etc. This gives us then a relatively simple overview of replacement needs. It also provides a simple overview of the costs.

Special conditions are reported from contractor to owner immediately. It is also an invaluable tool in relation to the requirements concerning fire and electrical installation and systems. We can thus easily provide the documentation of these requirements.

10 ORGANIZATION

The operation manager must choose an optimal maintenance organization that also must take into account other traffic-related tasks in addition to the technical issues connected to current maintenance tasks. Guidelines and instructions for all kinds of routine tasks, the repair of safety equipment and for larger rehabilitation assignments are necessary. The cost to society in the form of traffic problems will generally increase when the tunnel is closed because of long-term maintenance. Through planning and construction, one can take into account ways to carry out the operation and maintenance reducing the need for closure.

11 REDUCTION OF COSTS

In order to reduce the costs of operation and maintenance an optimal maintenance will primarily depend on the type of tunnel and the investments (that have been) made in the planning phase. Moreover, the conditions for and accessibility to operation and maintenance will be central issues.

12 CONCLUSIONS AND CLOSING REMARKS

The registration of experience has formed the basis and conditions for the proposed measures that have come forward in order to contribute to more optimal tunnel maintenance with optimal costs. Choices and decisions that are made in the planning phase have a decisive influence on the costs in the operation- and of maintenance phase.

These choices must be selected based on experience! The optimization of maintenance requires life span calculations based on data from previous experiences throughout the different processes. This requires a high degree of availability and the selection

The management system should in other words answer all questions regarding the administration, operation and maintenance of a tunnel. A systematic execution of operation and maintenance ensures good workmanship, a long life span and lower costs.

Operation and maintenance requires a necessary choice of profile that provides sufficient space for installations and the availability of rational methods of maintenance. A significant amount of capital has already been invested and will in the future be invested in Norwegian road tunnels.

The administration of this capital should first and foremost consider a long-term ownership. If this is to happen, it is difficult to see how it can be done without adding a much stronger emphasis to life cycle costs in both investment and operation and maintenance.

In order to work towards an optimal operation and maintenance, one needs knowledge about which conditions one should manage according to, which factors affect this management and which factors need to be influenced in order to achieve the desired results. Therefore we use the MOM system PLANIA. In Norway we use the same system to plan and to document our maintenance in subsea tunnels as in other road tunnels.

Fig13: Tunnel with white walls

Future structures will be designed and constructed based on the lessons learned from the past – whether or not they be successes or failures. Structural engineering has a proud history of learning from failures, with many of the contributions to engineering expertise occurring as a result of significant failures.

REFERENCES

i. Handbook 021 Roadtunnels in the NPRA.

ii. Nilsen, B. 1993. Empirical analysis of minimum rock cover for subsea rock tunnels. vii. Blindheim, O.T. & B. Nilsen 2001. Rock cover requirements for subsea tunnels.

iii. Proc. Int. ITA Conf. Options for Tunneling, Amsterdam: 677-687. Proc. Int. Symp. Strait Crossings 2001, Bergen: 439-448.

iv. Dahlø, T.S. & B. Nilsen 1994. Stability and rock cover of hard rock subsea tunnels.

v. Nilsen, B. & A. Palmström 2001. Stability and water leakage of hard rock subsea tunnels.

vi. Tunneling and Underground Space Technology, Vol. 9, Number 2: 151-158.

vii. Proc. ITA Conf. Modern Tunneling Science and Technology, Kyoto. 497-502.

viii. Kocheise, R-C. 1994. Svelleleire i undersjøiske tunneler.

ix. Blindheim, O.T., E. Grøv & B. Nilsen 2005. Nordic subsea tunnel projects.

x. Dr.ing.-avhandling 1994:124, NTH, Institutt for geologi og bergteknikk. 321 s.

xi. Tunneling and Underground Space Technology, Vol. 20, 570-580.

xii. Nilsen, B. 1994. Analysis of potential cave-in from fault zones in hard rock subsea tunnels.

xiii. Holmøy, K. 2008. Significance of geological parameters for predicting water leakage in hard rock.Rock Mechanics and Rock Engineering, Vol. 27, Number 2: 63-75. rock tunnels. Phd-thesis 2008:291, NTNU, 224p.

xiv. Nilsen, B. 1999. Key factors determining stability and water leakage of hard rock subsea tunnels.

xv. Nilsen, B. & A. Palmstrøm 2009. Engineering geological key factors for planning…

xvi. 37 US Rock Mech. Symp., Vail: 593-599. constructing hard rock subsea tunnels.

xvii. Proc. Strait Crossings 2009, Trondheim 2009, Tapir, 6p. Nilsen, B., A. Palmström & H. Stille 1999.

xviii. Quality control of a subsea tunnel project in complex Nilsen, B. & J. E. Henning 2009.
Thirty years of experience with subsea tunnels.

xix. Proc. ITA World Tunnel Congress, Oslo. 137-144.

18. EXPOSURE TO AEROSOLS AND GASES IN MODERN TUNNELING OPERATIONS AND LUNG FUNCTION DECLINE

Bakke, Berit
Ulvestad, Bente

1 ABSTRACT

1.1 Objective
Personal air measurements of aerosols and gases among tunnel construction workers were performed as part of an 11-days follow-up study on the relationship between exposure to aerosols and gases and respiratory effects.

1.2 Methods
Ninety tunnel construction workers employed at 11 construction sites participated in the exposure study. The workers were divided into seven job groups according to tasks performed. Exposure measurements were carried out on two consecutive working days between the two health examinations. Ninety tunnel workers and 51 referents were examined with lung function tests and questionnaires before their work period started and again 11 days later.

1.3 Results
The GM air concentrations for the thoracic mass aerosol sub-fraction, α-quartz, oil mist, organic carbon (OC), and elemental carbon (EC) for all workers were 561 μg/m^3, 63 μg/m^3, 210 μg/m^3, 146 μg/m^3, and 35.2 μg/m^3, respectively. Statistical differences of air concentrations between job groups were observed for all contaminants, except for OC, EC, and ammonia ($p>0.05$). Air concentrations of OC were correlated to air concentrations of oil mist ($r_{Spearman}$= 0.56 ($p<0.0001$)). The shaft drillers, injection workers, and shotcreting operators were exposed to the highest levels of thoracic dust (7061 μg/m^3, 1087 μg/m^3, and 865 μg/m^3, respectively). The shaft drillers and the support workers were exposed to the highest levels of α-quartz (GM= 844 μg/m^3 and 118 μg/m^3, respectively). Overall, the exposure to nitrogen dioxide and ammonia were low (GM=120 μg/m^3 and 251 μg/m^3, respectively). After 11 days of work, lung function declined significantly in the tunnel workers, not in the referents. Lung function decline was associated with exposure to organic carbon.

1. 4 Conclusion
The current study strengthens previous findings that use of emulsion explosive has reduced exposure to NO_2 in all jobs compared to using ammonium nitrate fuel oil which was previously used. Diesel exhaust air concentrations seem also to be lower than previously assessed (as EC). Nevertheless we conclude from the study of lung function that the air exposure in today's tunnel work still appears to have a detrimental impact on the airways. We can only speculate that repeated, short-term loss of lung function, probably due to inflammation caused by exposure, still may be linked to the risk of developing chronic lung disease.

2 INTRODUCTION

Studies in the 1990ties revealed that tunnel construction workers are exposed to aerosols and gases while operating drilling machines and through detonation of explosives in confined spaces. In addition to particular matter, diesel exhaust, α-quartz, nitrogen dioxide (NO_2), ammonia (NH_3), oil mist and oil vapour are air contaminants that dominate in tunnel construction work. Associated health effects include airway inflammation, lung function decline and chronic obstructive pulmonary disease (COPD) (Ulvestad *et al.*, 2000; Bakke *et al.*, 2001; Bakke *et al.*, 2001; Ulvestad *et al.*, 2001; Ulvestad *et al.*, 2001; Bakke *et al.*, 2002; Ulvestad, 2002).

During the last decade efforts have been made to reduce occupational exposure to aerosols and gases among tunnel construction workers through careful planning of the work and improved ventilation of the tunnels. Today most construction projects use emulsion explosives because of its higher resistance to water. Such explosives have been shown to generate less gases following detonation compared to the former first choice Ammonium Nitrate Fuel Oil explosive (ANFO) (Bakke *et al.*, 2001). Electrically powered drilling equipment and machines are preferred to reduce the emission of diesel exhaust. In addition, new technologies such as diesel exhaust particulate filters and catalytic converters have been implemented in this industry. Also, there has been focus on use of personal respirators when performing known high-risk tasks such as spraying of mineral oil and wet concrete. Demands for reducing the construction time increases the number of parallel activities and amount of traffic movements within the tunnel. This may introduce new risks for the workers. The aim of a recent study was to characterise and assess exposure to aerosols and gases among tunnel construction workers in modern tunneling operations as part of a 11-days follow-up study on the relationship between exposure to aerosols and gases and possible respiratory effects in tunnel construction workers.

3 MATERIAL AND METHODS

3.1 Work characteristics

These tunnel construction workers work 12 days consecutively and are then off for nine days. A typical work shift lasts 10-12 hours and includes two breaks of 30 min each. Tunnel construction workers are engaged in rock drilling, charging of explosives, and various support - and finishing work. Occupational job groups in tunnel construction work have previously been described (Bakke *et al.*, 2001). Briefly, the excavation process starts off with drilling and charging of explosives. After blasting, the rock is loaded and transported out of the tunnel using dump trucks. Finally, removing of loose rocks using a scalar and various types of rock support is carried out. Rock support includes, e.g., fastening of unsafe rock with steel bolts and sealing of the rock by spraying wet concrete onto the excavated surface. Other important tasks during excavation are mounting ventilation ducts, maintenance and repair of machines, and installation of electrical power supply. If the risk of water leakage into the tunnel is considered high, injection workers carry out rock consolidation with micro concrete to prevent leakage. All tunnels investigated in this study had forced ventilation systems using fans and ventilation ductings to dilute aerosols and gases for workers in all areas of the tunnel. Excavation of the shaft followed the same sequence as for tunnels, however, instead of using an underground drilling rig, pneumatic hand held equipment for rock drilling and a raise climber were used. The only ventilation in the shaft was from pressurized air used to power the drills.

3.2 Study design

All tunnel construction workers (n=91) employed at 11 available tunnel construction sites in Norway were invited for this study in 2010-2011. Participation was voluntarily. One worker decided not to participate. Health effects assessments were performed shortly before the work shift on the first day back on site after nine days off. After 11 days of work, the medical tests were performed again at the same time of the day. In total, ninety tunnel workers and 51 referents were examined with lung function tests and questionnaires.

The workers were stratified into job groups according to tasks performed. Job groups included in this study were drill and blast workers, drill and blast mechanics (a subgroup of the drill and blast workers), support workers, loaders (a subgroup of support workers), injection workers, shotcreting operators, and shaft drillers. Personal air measurements were carried out on two consecutive working days prior to the day of the second health examination. Each worker was sampled twice . Thoracic dust, elemental carbon (EC), organic carbon (OC), α-quartz, and NO_2 were measured in all workers. Oil mist, oil vapour, and NH_3 were measured in a subsample of workers from all job groups (N= 57), except shotcreting operators and injection workers. All samples were collected in the breathing-zone outside personal protective respirators. The sampling time varied between 270 and 855 minutes (arithmetic mean (AM) =569 minutes).

Sampling methods and analysis of the samples are described elsewhere (Bakke et al., 2014)

3.3 Data analysis

The frequency distribution was examined visually using probability plots and indicated that a log-normal distribution provided a better fit to the exposure data. The data were therefore ln-transformed before statistical analysis. The measured air concentrations were used without further adjustments. Air concentrations were summarized by geometric means (GM), geometric standard deviations (GSD), minimum concentrations (Min) and maximum concentrations (Max) using maximum likelihood estimation (MLE). Arithmetic mean was estimated from the expression $EXP[\ln GM+0.5 \ln GSD^2]$ (Seixas *et al.*, 1988). The SAS procedure NLMIXED was used to perform MLE for repeated measures data subject to left censoring for all contaminants except for NH_3 where the SAS procedure LIFEREG was used because there was no repeated measurements (Jin *et al.*, 2011)2011.

Correlations between exposure variables were evaluated using Spearman's correlation coefficient. No correlation coefficient exceeded 0.6. The highest correlations were between air concentrations of OC and oil mist, and between EC and NO_2 ($r_{Spearman}$= 0.56 and 0.60, respectively (p<0.0001).

Statistical analyses were carried out with SPSS 21.0 (SPSS Inc, Chicago, Illinois, United States) and SAS version 8.2 (SAS Institute Inc., Cary, NC, USA).

3.4 Results

A total of 90 tunnel construction workers carried personal sampling equipment in the exposure study, and all workers were monitored twice. Few workers reported use of personal protective respirators, except shotcreting operators who partly used disposable half-masks with filters for particles (3M™) during sampling. In total, six samples of α-quartz and 20 samples of EC and OC were discarded because of technical failures.

In total, 79 samples of NO_2 using direct reading instruments were evaluated. Only 8 of these samples of NO_2 were above the LOD of 376 µg/m^3. The median NO_2 concentrations of the samples above LOD were 565 µg/m^3 (range 376-1317 µg/m^3). However, in 17 of the measurements maximum observed peak value incidents of high air concentrations of NO_2 were detected (>3764 µg/m^3).
The GM air concentrations for the thoracic mass aerosol sub-fraction, α-quartz, oil mist, OC, EC, NO_2, and NH_3 for all workers were 561 µg/m^3, 63 µg/m^3, 210 µg/m^3, 146 µg/m^3, 35.2 µg/m^3, 120 µg/m^3, 251 µg/m^3, respectively. Statistical differences of air concentrations between job groups were observed for all contaminants, except for OC, EC, and NH_3 ($p>0.05$). On average, OC accounted for 76 % of the total carbon measured, and total carbon accounted for 49 % of the thoracic aerosol mass. Also, statistical differences of air concentrations of α-quartz between construction sites were observed ($p<0.05$). The arithmetic mean (AM) percent of α-quartz in the thoracic mass aerosol sub-fraction ranged from 3 to 40 percent between sites.

The shaft drillers, injection workers, and shotcreting operators were exposed to the highest air concentrations of thoracic aerosol mass (GM=7061 µg/m^3, 1087 µg/m^3, and 865 µg/m^3, respectively). The shaft drillers and the support workers were exposed to the highest concentrations of α-quartz (GM= 844 µg/m^3 and 118 µg/m^3, respectively). Shotcreting operators were the highest exposed workers to NH_3 (GM= 2927 µg/m^3). The highest levels of NH_3 were found during loading of mass following detonation of the explosive. An example is shown in Figure 1.

3.5 Respiratory effects
After 11 days of work, lung function, measured by mean forced expiratory volume in one second (FEV_1), had declined 73 ml (SD 173), $p<0.001$ in the tunnel workers, compared to 6 ml (SD 33), $p=0.9$ in the referents. Also forced vital capacity (FVC) had declined significantly. Decline in both FVC and FEV_1 were significantly associated with exposure to organic carbon (Ulvestad *et al.,* 2014).

3.6 Discussion
As part of an 11 days follow-up study on the relationship between personal exposure to aerosols and gases and possible respiratory effects in Norwegian tunnel construction workers, an exposure survey was performed in 2010 and 2011. Tunnel construction workers are a mobile workforce, who perform a number of tasks and are in contact with many different materials at different worksites. These characteristics challenge the exposure assessment process and measurements performed at a single worksite may not be valid at other sites or time periods. In this study we measured air concentrations of selected contaminants at 11 different worksites. Overall, the results indicate that the air concentrations have slightly decreased for some contaminants and have been reduced for some jobs compared to measurements in this industry 10-15 years ago (Bakke et *al.*, 2001). However, challenges remain especially with regard to airborne dust concentrations. Particles are generated by drilling, blasting, crushing, grinding, shotcreting, and transport operations. The mass of particles in the thoracic aerosol sub-fraction was substantial during shaft drilling (GM= 7.1 mg/m^3). Among other jobs, the GM air concentration varied between 0.42 and 1.1 mg/m^3 (drill and blast and injection workers, respectively). In former studies of shaft drillers and drill and blast workers we found that the GM of "total" dust was 6.1 and 2.3 mg/m^3, respectively, and the GM of respirable aerosol sub-fraction 2.8 and 0.91 mg/m^3, respectively (Bakke et *al.*, 2001). This indicates that focus on better ventilation and work practices have resulted in decreased air concentrations.

The thoracic aerosol mass sub-fraction was chosen because it is considered the most relevant health-related aerosol sub-fraction with regard to studying cardiovascular and respiratory effects (Vincent *et al.*, 2001). This aerosol sub-fraction, which penetrates below the larynx gives a better estimate of the dose to the lung than the inhalable-, respirable- or "total" dust (Brown *et al.*, 2013). Alpha-quartz was measured in the thoracic aerosol sub-fraction. The main work task, in which exposure to α-quartz occurs, is rock drilling. In addition, inhalation of the aerosol generated during blasting may also increase α-quartz exposure. The air concentration of α-quartz during shaft drilling was very high (GM=0.84 mg/m^3). The support workers who works about 100 meters from the tunnel face, also experienced high air concentrations of α-quartz (GM=0.12 mg/m^3). Statistical significant differences in air concentrations of α-quartz between construction sites were observed, probably due to differences in geology. Underground project planning requires detailed geological documentation. Information in these reports could be used in risk assessment of geological hazards, such as α-quartz. Alpha-quartz may cause serious pulmonary diseases (Hnizdo and Vallyathan, 2003; Tjoe and Heederik, 2005).

The solid particle fraction of diesel exhaust is predominately composed of EC. EC has been proposed to be the most reliable marker of this particle phase of diesel exhaust (NIOSH, 2003). Few countries have regulated occupational exposure to diesel exhaust particulate matter, measured as EC. In Austria the occupational exposure limit (OEL) is 100 µg/m^3 (Austria Arbeitsinspektion, 2013). In our study the overall GM air concentration of EC varied from 31-54 µg/m^3 for all job groups. This is considerably lower than what was previously reported where we found an overall GM of 160 µg/m^3, and 340 µg/m^3 among drill and blast workers (Bakke *et al.*, 2001). This indicates that diesel exhaust exposure have considerably been reduced among workers in these jobs. Other studies in Sweden and Switzerland among tunnel construction workers have reported EC levels of 80 - 90 μg/m^3 (Sauvain *et al.*, 2003; Lewne *et al.*, 2007).

Particulate emission rates of EC and OC from diesel engines may vary greatly depending on the mode of vehicle operation. Typically EC/OC ratios under normal operating conditions are approximately 2.5 (Shah *et al.*, 2004). In our study, OC constituted on average 76 % of the total carbon and the main source is therefore probably not diesel exhaust. The air concentrations of OC were moderately correlated to the air concentrations of oil mist ($r_{Spearman}$=0.56), and OC may therefore partly be an expression of exposure to oil mist in tunneling. Oil mist was, however, only measured for 2 hours during drilling. This also explains why the measured air concentration of oil mist is higher than the air concentration of OC. Alternatively, since the sampling duration of EC/OC was eight hours it is also possible that OC to some extent may have evaporated from the filter.

Machines used for drilling of shafts and tunnels require that the cutting head is lubricated. Oil mist and oil vapour may therefore be released into the work atmosphere. The GM air concentration of oil mist varied between <50 and 9100 μg/m^3. The highest GM air concentrations of oil mist were measured during shaft drilling using pneumatic drilling equipment (9100 μg/m^3). Such levels are known to affect lung function and should be prevented (Skyberg *et al.*, 1992). All individual measurements of oil vapour was < 620 μg/m^3, indicating that the oils in use were of low volatility.

The main sources of NO_2 during tunnel construction are blasting and exhaust from diesel powered machinery and vehicles. The amount of gases released during blasting depends on the type of explosive used (Bakke *et al.*, 2001). In this study emulsion explosive was the explosive of choice in all construction sites, and this may explain the relative low levels of NO_2 (GM= 120 μg/m^3) which was similar to findings in former studies where the same emulsion explosive was used (GM=226 μg/m^3) (Bakke *et al.*, 2001). However, the analytical methods were different and as shown in this study direct reading instruments do not have the sensitivity that is required for measuring full-shift NO_2 air concentrations during tunnel construction.

Use of respirators was not mandatory and was not used on a general basis, except among shotcreting operators who partly used respirators. The actual inhaled air concentrations may therefore be lower than measured among workers in this job.

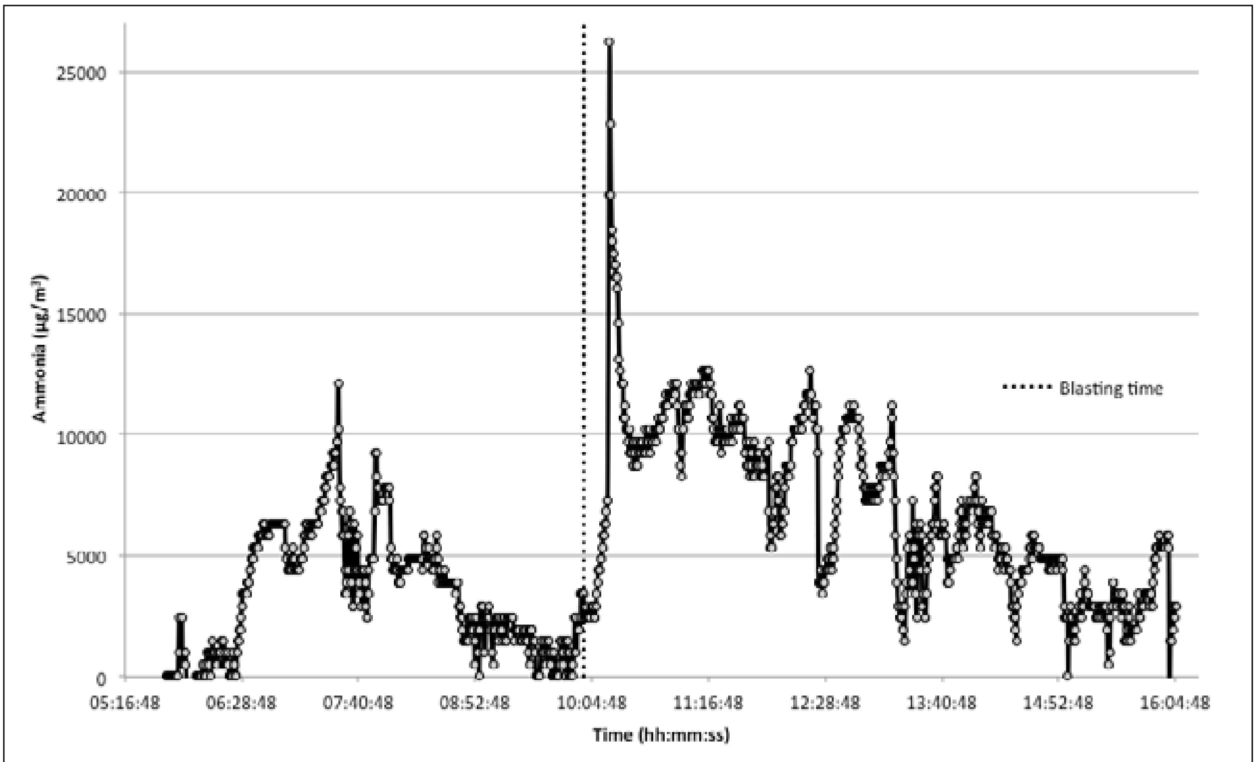


Figure 1. Example of personal real-time measurement of ammonia *while loading of blasted rock*

In conclusion, the current study strengthens previous findings that use of emulsion explosive has reduced exposure to NO_2 in all jobs compared to using ANFO which was previously used. Diesel exhaust air concentrations seem also to be lower than previously assessed (as EC). Nevertheless we conclude from the study of lung function that the air exposure in today's tunnel work still appears to have a detrimental impact on the respiratory system. We can only speculate that repeated, short-term loss of lung function, probably due to inflammation caused by exposure, still may be linked to the risk of developing chronic lung disease such as COPD.

REFERENCES

Austria Arbeitsinspektion (2013). Verordnung des bundesministers für Arbeit, Soziales und Konsumentenschutz über Grenzwerte für Arbeitsstoffe sowie über krebserzeugende und überfortpflanzungsgefährdende (reproduktionstoxische) Arbeitsstoffe (Grenzwerteverordnung 2011 - GKV 2011). Austria, Arbeitsinspektion. Available from: URL: http://www.arbeitsinspektion.gv.at/NR/rdonlyres/F173280B-D4FB-44D2-8269-8DB2CB1D2078/0/GKV2011.pdf (accessed 19 Dec 2013).

Bakke B, Stewart P and Eduard W. (2002 Determinants of dust exposure in tunnel construction work. Appl Occup Environ Hyg; 17: 783-796.

Bakke B, Stewart P, Ulvestad B et *al.* (2001) Dust and gas exposure in tunnel construction work. Am Ind Hyg Assoc J; 62: 457-465.

Bakke B, Ulvestad B, Stewart P et *al*. (2001) Effects of blasting fumes on exposure and short-term lung function changes in tunnel construction workers. Scand J Work Environ Health; 27: 250-257.

Bakke B, Ulvestad B, Thomassen Y et al. (2014) Characterization of occupational exposure to air contaminants in modern tunneling operations. Ann Occup Hyg (submitted 2014).

Benbrahim-Tallaa L, Baan RA, Grosse Y et *al.* (2012) Carcinogenicity of diesel-engine and gasoline-engine exhausts and some nitroarenes. Lancet Oncol; 13: 663-664.

Brown JS, Gordon T, Price O et *al.* (2013) Thoracic and respirable particle definitions for human health risk assessment. Part Fibre Toxicol; 10: 12.

Hnizdo E and Vallyathan V. (2003) Chronic obstructive pulmonary disease due to occupational exposure to silica dust: A review of epidemiological and pathological evidence. Occup Environ Med; 60: 237-243.

Jin Y, Hein MJ, Deddens JA et *al*. (2011) Analysis of lognormally distributed exposure data with repeated measures and values below the limit of detection using sas. Ann Occup Hyg; 55: 97-112.

Lewne M, Plato N and Gustavsson P. (2007) Exposure to particles, elemental carbon and nitrogen dioxide in workers exposed to motor exhaust. Ann Occup Hyg; 51: 693-701.

NIOSH (2003) Manual of analytical methods (NMAM NIOSH Schlect PC and O'Connor PF, editors. (Suppl. 3) 4th edn. Cincinnati, OH: Department of Health and Human Services, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health DHHS (NIOSH) Pub. No. 2003-154.

Sauvain JJ, Vu DT and Guillemin M. (2003) Exposure to carcinogenic polycyclic aromatic compounds and health risk assessment for diesel-exhaust exposed workers. Int Arch Occup Environ Health; 76: 443-455.

Seixas NS, Robins TG and Moulton LH. (1988) The use of geometric and arithmetic mean exposures in occupational epidemiology. Am J Ind Med; 14: 465-477.

Shah SD, Cocker DR, III, Miller JW et *al*. (2004) Emission rates of particulate matter and elemental and organic carbon from in-use diesel engines. Environ Sci Technol; 38: 2544-2550.

Skyberg K, Ronneberg A, Christensen CC et al. (1992) Lung function and radiographic signs of pulmonary fibrosis in oil exposed workers in a cable manufacturing company: A follow up study. British J Ind Med; 49: 309-315.

Tjoe NE and Heederik D. (2005) Risk assessment of silicosis and lung cancer among construction workers exposed to respirable quartz. Scand J Work Environ Health; 31 Suppl 2: 49-56.

Ulvestad B (2002). Exposure and obstructive pulmonary disease in heavy construction workers. An epidemiological study of norwegian heavy construction workers, University of Oslo, Norway.

Ulvestad B, Bakke B, Eduard W et *al.* (2001) Cumulative exposure to dust causes accelerated decline in lung function in tunnel workers. Occup Environ Med; 58: 663-669.

Ulvestad B, Bakke B, Melbostad E et *al.* (2000) Increased risk of obstructive pulmonary disease in tunnel workers. Thorax; 55: 277-282.

Ulvestad B, Lund MB, Bakke B et *al*. (2001) Gas and dust exposure in underground construction is associated with signs of airway inflammation. Eur Respir J; 17: 416-421.

Vincent JH, Ramachandran G and Kerr SM. (2001) Particle size and chemical species 'fingerprinting' of aerosols in primary nickel production industry workplaces. J Environ Monit; 3: 565-574.

Rallare
A synonym for a worker active within heavy construction. The picture presents a group of Rallare anno 1895. Honest, hardworking, free to move. The crew flocked around the female assistants handling the team catering.

19. HEALTH AND WORK ENVIRONMENT

MYRAN, Tom

1 INTRODUCTION

Tunnelling, mining and other underground works have always been seen as a tough branch. Dust, gases from diesel and blasting, soot, oil mist, noise and vibration, radon and radon daughters, different chemicals, rock fall, temperature and air humidity, heavy and uncomfortable working positions with arms over head, reduced light and visibility, danger of explosion, traffic accidents etc are typical but heavy loads for the workers. Automation and mechanisation have clearly brought about a reduction of the physical stress, but the psychological stress is more predominant now than it was earlier.

If we look at today's tunnel workplace in a historical HSE perspective, (e.g. the development during f.i. the last 30 years), today's workplaces have become technically safer, but not necessarily healthier. This arouses uncertainty about the health hazards and risks that this activity entails.

Injuries from rock works can be fatal (rock-falls, drilling into remaining explosives, etc). The accidents that cause injuries to personnel may cause damage to material or to the external environment. All such losses are undesirable. However, most accidents causing injuries to personnel and/or material damage can be prevented.

Many examples can be found where companies have achieved remarkable improvements in their Health and Safety work, including reduced accident rates, and have simultaneously experienced good economic returns.

The environmental preconditions and requirements for tunnelling, mining and other rock engineering are constantly being made more stringent. At the same time, the mass media, environmentalists, action groups and others are continually focusing on factors which may influence neighbourhood and environment.

2 EXPERIENCE WITH THE WORKING ENVIRONMENT

Underground excavation has a high and steadily increasing degree of mechanisation. This development is caused partly by an increase in the capacity of current equipment, and partly by the introduction of new technology to replace what already exists. It is here important to emphasise the significance of obtaining a balance between desired technical development and the implementation of necessary initiatives in the work environment. A lack of such balance will lead to a situation in which certain employee groups will be more vulnerable than others.

Today's problem situations in regard to preventing damage to the working environment in rock-blasting activities is not about finding new problems, but about dealing with those that already exist. Noise, mineral dust and vibration cause three out of four work illnesses in Norway.

The working environment is usually described as being constituted of physical and chemical environmental factors on the one hand and psychological and social factors on the other. There can be considerable differences as to how individual employees experience their own work situations and working environments. The experienced risk is often different from the actual risk.

In recent decades, we have generally had a development which in many ways has positively affected Health, Safety and Environment (HSE) in tunnelling. This includes increased interest, attitude and motivation for HSE work, and at the same time the understanding of the relation between production work and HSE has been improved. Further, the legal responsibility of top-level management has been made clearer and more transparent.

Better blasting agents and techniques, reduced explosive concentration, more effective watering of the stockpile, other energy forms and methods for loading and transportation, cleaner engines and fuel, better driving techniques, more systematic service and maintenance of equipment and roadway and more effective ventilation methods are all efforts to improve the air quality underground. These efforts also affect productivity and economy in a positive way. Training and control functions have improved. A motivated and determined management is necessary for this development.

Analyses of sick leave and, not least, rehabilitation measures have in recent years improved the understanding of the health problems and methods to reduce the problems. Measurements of work hygiene and product development in relation to protective equipment are important key factors in the work designed to prevent illness at work and personal injuries.

3 WORKING ENVIRONMENT AND MEASURES

The stimulation to a greater degree of technical and medical co-operation aimed at more purposeful and cost-saving supervision and inspection procedures, regarding both prevention technique and health services, will contribute to an improvement of the work environment in line with society's general expectations.

Because of the demand for reduced construction times for tunnel projects and increased mechanisation and productivity, the amount of air pollutants produced per time unit is also increasing. If the right measures are not taken, the air quality underground could be dramatically worsened and the workers highly exposed to a variety of airborne substances.

With reference to the working environment generally, the chemical/physical factors such as dust, gas, noise, vibrations and lighting are very similar in the construction and mining industries. However, conditions related to psychological pressure or stress can be rather more dominant in construction activities (tunnelling) than is usually the case in mining. This has also been confirmed by employees with experience from both the construction and the mining industry.

On account of steadily increasing productivity in rock-blasting work, the amount of air pollution produced per time unit has significantly increased in recent years, something which can easily lead to poorer air quality should the follow-up phase in the enterprise be carelessly carried out.

Construction times for tunnel projects are constantly pressed downwards. The result of this is that much work on the face is carried out at the same time as the work behind the face in order that deadlines can be met.

The work environment and air quality on the working face is only slightly affected by increased activity behind the face. To obtain an acceptable work environment for the tunnel as a whole, the working patterns in the tunnel should therefore attempt to eliminate or even out the environmentally harmful activities over the building period as a whole, and also over the shifts during the day.

4 CHEMICAL AND PHYSICAL PARAMETERS

Since early in the 1970s, Sintef Rock and Mineral Engineering and the Norwegian University of Science and Technology in Trondheim have been investigating the exposure, chemical and physical, to which the worker may be subjected in both the mining industry and tunnel construction works. This includes exposure to both gases and aerosols. In addition to occupational exposure to nitrogen dioxide (NO_2 and NO), carbon monoxide (CO), carbon dioxide (CO_2), ammonia (NH_3) from blasting and diesel exhaust, there is also exposure to particulate matter such as mineral dust, alpha-quartz, and asbestos and non-asbestos fibers, and also oil mist when using pressure drilling machines. In 1971 the first measurements were made of exposure to ionizing radiation from radon or radon daughters, and to some extent thoron and thoron daughters, to which workers underground were subjected.

Incentives to greater technical and/or medical co-operation aimed at more objective, cost-cutting, supervisory and monitoring routines for both safety techniques and health services will help to improve the environment in keeping with society's requirements for HSE. In collaboration with the mining industry, among others, SINTEF and NTNU are developing the database, BEI (Background Exposure Index), which is a tool for describing relationships between health (medical studies) and environment (exposure).

The relationship(s) between exposure and risk and/or symptom are complex. For some factors there seems to be a linear relationship; for other and combinations of factors the relationships seem to be of a more varying, exponential nature. The state of health of today's industrial employees is an expression, among other things, of past work environment conditions and exposure. It is important that both the medical information and the exposure data that exist be employed in order to be able to continuously improve the future work environment.

4.1 Blasting and explosive gases

Workers in tunnelling have for many years been exposed to unacceptable concentrations of gases when passing through a smoke pocket smoke plug after a blast.
Slurry applications in tunnelling have been in commercial use in Norway since March 1996. Since 1993

technical investigations have been carried out on blasting fumes, dust, smoke, visibility and other factors concerning the slurry techniques compared with ammonium nitrate/oil (ANFO).

Slurry, unlike powders such as ANFO, is a liquid blasting agent, which is transported by tanker truck directly to the site, or building area. Slurry is based on a two-component system where the product is pumped into and mixed together in the drilling holes.

The working environment (air quality) at the face and behind the face is highly improved because of a much lower concentration of nitrogen dioxide (NO_2) in particular, but also carbon monoxide (CO), when using slurry instead of ANFO. In addition, less dust, both total and respirable, is produced using slurry, which gives lower airborne smoke and dust concentration and increases the visibility. Using slurry gives less spill, waste and pollution of the water in the tunnel, and the need for transport and storing of blasting agents is reduced.

The health risk associated with explosive gases in tunnelling has also clearly been reduced in recent years, not least where two-way ventilation is employed as prescribed. This blower-exhaust-combination ventilation method was introduced in Norwegian tunnelling in 1986/87, and results in significantly better conditions concerning gas, dust and visibility compared with the blowing ventilation method normally used. The blower-exhaust method is suitable in long tunnels, but has limitations in tunnels with small and very large cross sections.

Because of the highly improved air quality and visibility when using slurry instead of ANFO, the use of the blower-exhaust ventilation methods has been reduced in the last year.

Naturally occurring gases can cause risks, and because of that they have to be continously in focus. Methane is normally a coal mine problem, but like other natural gases, such as radon and hydrogen sulphide, it might be detected in tunnels. They are seldom a problem during construction in mainland Norway, both because of the geological formations that prevail, but also due to the normally high ventilation volume applied for other reasons.

4.2 Diesel exhaust

Since heavier wheeled diesel machines were introduced in mines and tunnels in Norway in the early 70s, exposure to gases and particles has been among the dominating burdens in these industries. There is a suspicion that occupational exposure to diesel fumes gives an increased risk of cancer. This risk is thought to be primarily associated with particles.

For a long time, the focus was exclusively on fuel consumption (and waste gas quantity) from diesel engines, not on composition. During the 90s, increasingly more focus was placed on pollution and air quality for underground rock work. It was acknowledged that conditions connected to gas/exhaust, particle pollution, ventilation, back-flush of stone piles, the quality of roadway, etc. have a decisive influence on the working conditions, both on and behind the face. This must be taken into consideration when organizing plants and determining operational arrangements. Today, however, as some decades ago, questions are still being asked regarding threshold limit values (TLVs), marginal values, exposure/emission and measuring methods (what is to be measured, when, and how?). Efforts have been made to find an (or a few) exposure indicator(s) that is/are simple to measure using functional methods, such as light instruments with direct indication. This applies to the working environment as well as the external environment. So far, though, no-one has succeeded in finding such criteria or proposals for such indicators.

4.3 Ionizing radiation

Some decades ago, employees underground (especially miners) were on average the group of workers in Norway most exposed to the highest doses of ionizing radiation. This is caused by the presence of radioactive components in the rock types, e.g. radon and radon daughters, and to some extent thoron and thoron daughters. Occupational exposure to radon daughters gives an increased risk of lung cancer.

Normally it is seldom a problem in tunnels, because of the high ventilation volume applied for other reasons (blasting, loading and transportation). But if the ventilation is reduced, or stopped, the concentration of radon can increase rapidly to high levels. When starting the ventilation system again, the concentration of radon will in the course of a few hours return to the background level.

4.4 Particle pollution

In spite of the introduction of dust-dampening efforts and ventilation, particle pollution (mineral dust, organic particles like soot and oil mist) is a dominating air-polluting environmental factor in rock-blasting work. Investigations reveal illness from dust in the lungs (reduced lung function and lung diseases, among others silicosis). Since the early 70s, testing and documentation of dust and other air pollutants associated with mines and tunnelling has been carried out. This includes sampling of dust, with quartz content and types of particles associated with the underground work exposure. The objectives have been both to document the exposure level to dust for individual workers and to determine the background level in different work tasks and within different work areas.

Investigations show that the threshold limit values (TLV) occur more frequently in tunnelling than for instance in mining. This is especially linked to differences in the quartz content of the dust, but also to the fact that the work momentum in tunnelling operations can be rather more intense than in mining.

The content of alpha-quartz in the dust in tunnels varies between 0 % and more than 70 %. Dust investigations carried out indicate that the danger of silicosis must still be regarded as real in a number of tunnelling and mining works. Therefore, vigilance, knowledge and information are essential, as are checking and preventative measures, together with regular and correct health supervision of employees. The TLVs for rock dust depends on the quartz content.

4.5 Noise
Noise is probably the single factor which causes the most injuries at work. In the tunnelling and mining industries hardly any 50 to 60-year old workers in rock blasting and underground construction work have normal hearing. The great majority are more or less hearing-impaired.

4.6 Whole body vibration and lighting
Whole body vibration and lighting (poor light) are important, but often neglected environmental factors, which can cause personal injury and accidents. In particular rubber-wheeled production equipment (loading machines, loaders and carriers) combined with poor roadways/foundations can cause heavy whole-body vibrations.

4.7 Tunnelling Boring Machines (TBM)
The first Tunnelling Boring Machines (TBM) was put into use in Norway in 1972. Since then some 260 km has been bored., Compared with conventional tunnel-blasting, the use of TBMs has reduced or completely eliminated environmental pressures such as blasting fumes, diesel exhaust, oil mist at the face and blockfall, but at the same time has resulted in increased pressures elsewhere.

Questionnaires show that TBM operators perceive the following environmental situations as most troublesome: noise, mineral dust, vibrations, heat (when changing cutters), repetitive work and ergonomic pressures. A clear majority fear health risks on account of long-term effects of dust, noise and vibration more than the risk of acute, mechanically caused or pressure injuries.

Still there is potential for improvements to be obtained with relatively small adjustments and modification of current operating methods. This relates in particular to mineral dust, noise, whole body vibrations and ventilation.

4.8 Long tunnels and ventilation
In 1995 work commenced on the longest road tunnel in the world, the Laerdal tunnel, which is 24.5 km in length. The tunnel was completed in 2000. It was a great challenge to the work with health, environment and safety, and especially concerning the choice of an optimal ventilation method. In the Laerdal Tunnel, the single-tube version was developed further by means of a bypass air system for surface fan, mobile fan and auxiliary fans that will eliminate frictional losses through these fans. Both the single and twin-tube systems were used in the Laerdal tunnel. Today the blowing-exhaust ventilation method is seldom in use in Norway.

It is interesting to note the fact that during the five years of the Laerdal Tunnel project, the ventilation system in the tunnels transported more than three times as much air in tons as rock has passed through the tunnel. This is a fact in nearly all tunnels and even mines in Norway.

5 CHALLENGES AND AREAS OF CONCERN

This section summarises a number of conditions that are important, or that may become important and possibly can have consequences for the HSE work and measures for rock engineering work both today and in the future:

- HSE work must become an activity with the same priority as planning, progress and financial performance.
- HSE work must become more visible.
- HSE delegates or representatives are an important, positive resource. They have to be involved, f.i. in safety job analyses.

- More and updated information about the use, servicing, utility of and limitations on various types of safety equipment, e.g. personal safety equipment for use against fumes, dust or noise.
- New requirements, directives and new threshold limit values have to be followed up, and necessary information given to employees.
- Methods for control, monitoring and sampling vary greatly in frequency, content and quality both relating to medical and exposure research studies. Equipment and procedures must be homogenous and harmonized, including calibration routines for both technical and medical monitoring.
- During excavation, ventilation is a crucial factor. Ventilation is expensive, and care must be taken to choose a solution that gives the maximum utilization in that every cubic metre of air is brought to the right place, at the right time and in sufficient volumes.

The industry is confronted with many difficult and challenging HSE tasks linked to underground work. However, the industry has competent workers and leaders who will ensure that the laws and regulations required for conforming to environmental standards will be complied with through a collective and responsible effort.

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Appendix I.

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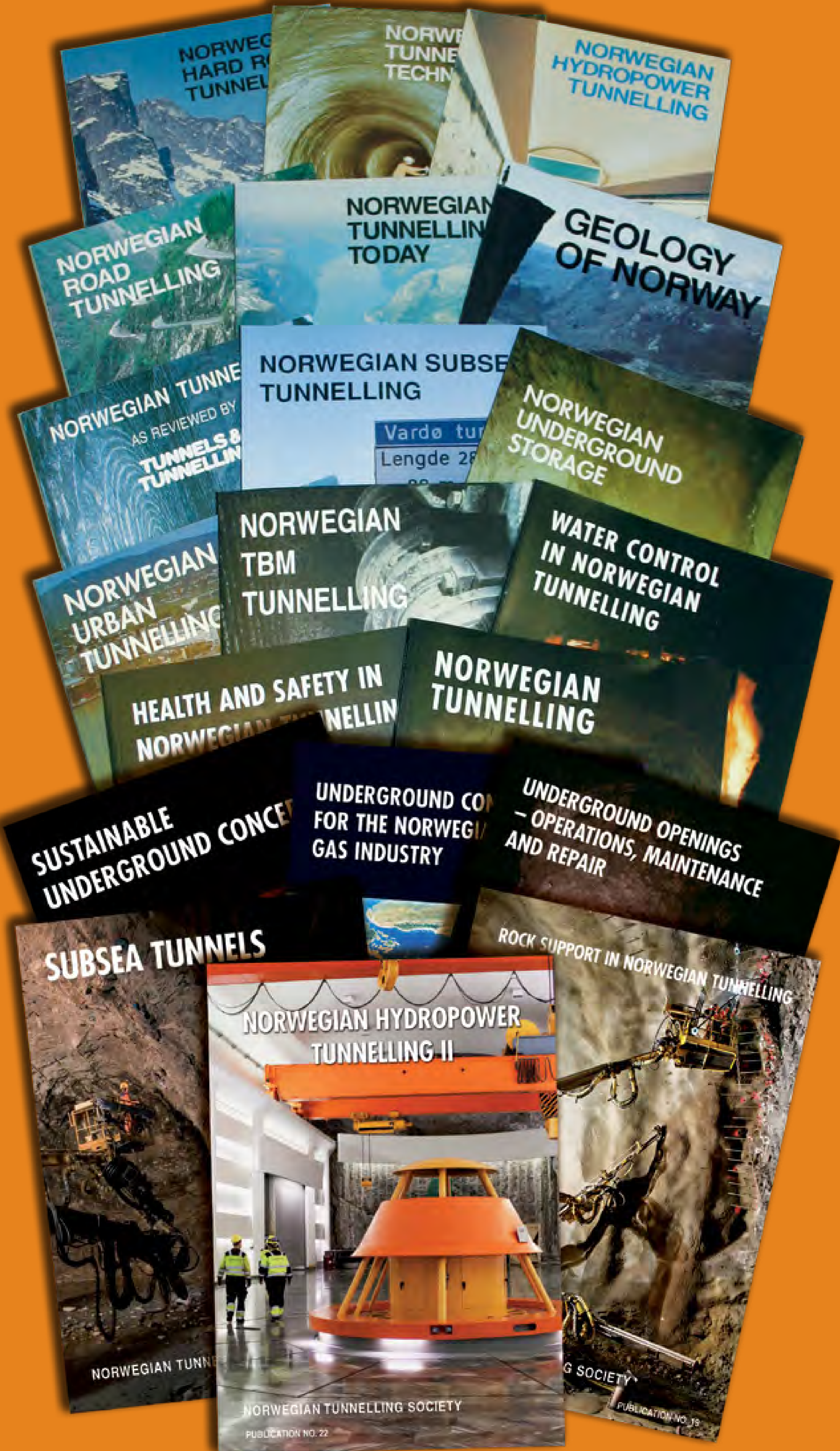
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