1 INTRODUCTION

Norway extends some 2100km from its southern tip to the far north-east corner. The landscape and topography is characterised by deep valleys, high mountains and long and deep fjords, constituting numerous challenges to overcome for infrastructure construction. Therefore there is a need for tunnels to accommodate efficient communication. Most of the Norwegian electric energy is generated by hydropower, thus there is also a great number of tunnels associated with such projects. The total length of Norwegian tunnels is more than 5000km, and that means that there is about one meter of tunnel for each inhabitant. In addition to tunnels there are hundreds of caverns for different use, such as storage for oil and gas, cold storage, shelters, parking halls, sports halls and power stations.

The tunnelling industry in Norway started in the 16th century in connection with increased metal mining activities. Most metal mines are now history, but they have been replaced by mining of so-called industrial minerals, i.e. minerals used for a large range of products like cement, glass and ceramics, fillers for paper, pigments for paints etc. At the moment, 20 underground mines on industrial minerals are operating, producing about 5 million tonnes or 2 million m3 annually.

Table 1.1. Tunnels in Norway

Type	Number	Length, km
Railway tunnels	700	316
Road tunnels	881	843
Hydropower projects	>300	3500
Industrial mineral mines	20	-
Other tunnel projects	approx. 200	250
Total	>2100	≈5000

Table 1.2. Different types of caverns and tunnels

Type	Number	Length, km
Sub-sea tunnels	34	130
Sub-sea road tunnels	23	95
TBM tunnel projects	28	260
Underground hydropower stations	approx. 200	-
Air cushion chambers	10	-
Water pressure tunnels (> 150m pressure)	80	-
Industrial mineral mines	Not available	Not available
Oil and gas stores	10	-

2 ENGINEERING GEOLOGY OF NORWAY

Most of the rocks in Norway have an age of more than 350 million years. Highly metamorphic rocks of Precambrian age such as granite, gneiss and gabbro constitute about 70% of the area. The remaining bedrock consists of more or less metamorphic rocks of Cambro-Silurian age such as schist, greenstone, sandstone and marble.

Uunaltered sedimentary rocks may also be found, however, in rather limited areas, such as in the close surroundings of Oslo. Here there are also unaltered volcanic rocks such as basalt and rhombus porphyry of Permian age.

The Norwegian bedrock has a complex tectonic history, and because of intense folding and metamorphism the structural pattern and discontinuities exposed today is often complex. This means that a lot of different rock types and structural features can be encountered in a tunnel. Usually the bedrock is of good quality as far as stability is concerned, but locally there might be numerous discontinuities in terms of joints and weakness zones. Such weakness zones can be faults with crushed rocks that may be more or less altered to clay. Swelling clay (montmorillonite) is often found in such zones.

Stability problems are therefore often related to the handling of joint systems in general whilst sections of the tunnels with weakness zones may face more severe problems. Norwegian tunnellers have recently handled extreme tunnelling conditions in the Oslofjord tunnel neccessitate ground freezing and extensive rock mass grouting to stabilise a difficult zone.

Figure 2.1. Kobbvatn area in Northern Norway. The rock is a massivePrecambrian granite with high stress anisotropy. (NGI)

In a section of the Bjorøy tunnel near Bergen, the methods used had elements of soil tunnelling. See Appendix 3 for details on the Oslofjord project and the Bjorøy tunnel.

High in-situ stresses may cause poor stability over long tunnel sections. The stress induced stability problems are partly caused by high rock cover, for example in the fjord landscape of Western Norway, but several places there are in addition high tectonic stresses, thus overstressed rock masses may be encountered even at low rock cover. Solutions for dealing with high stress anisotropy by using rock bolts and sprayed concrete have been developed, as was used for the Lærdal tunnel.

Another consequence of the Norwegian nature is the generous precipitation and most of Norwegian tunnels are located in the saturated zone in the rock mass. Thus, handling water inflow and ground water has been a normal procedure of Norwegian tunnelling projects.

Soft rock, in any greater extent than in weakness zones, is seldom encountered in Norwegian tunnels. Such zones may, however, be several tens of meter wide and call for extraordinary support measures and tunnelling methods. Strongly weathered rocks near the surface do usually not occur since such material was mainly eroded and removed by the glaciers during the Quaternary glaciations. The rock mass appears therefore usually fresh, even on surface outcrops.

Norwegian tunnelling projects have faced a great variety of ground conditions including also adverse and problematic tunnelling, but in overall the condition for tunnels are considered being favourable in Norway.

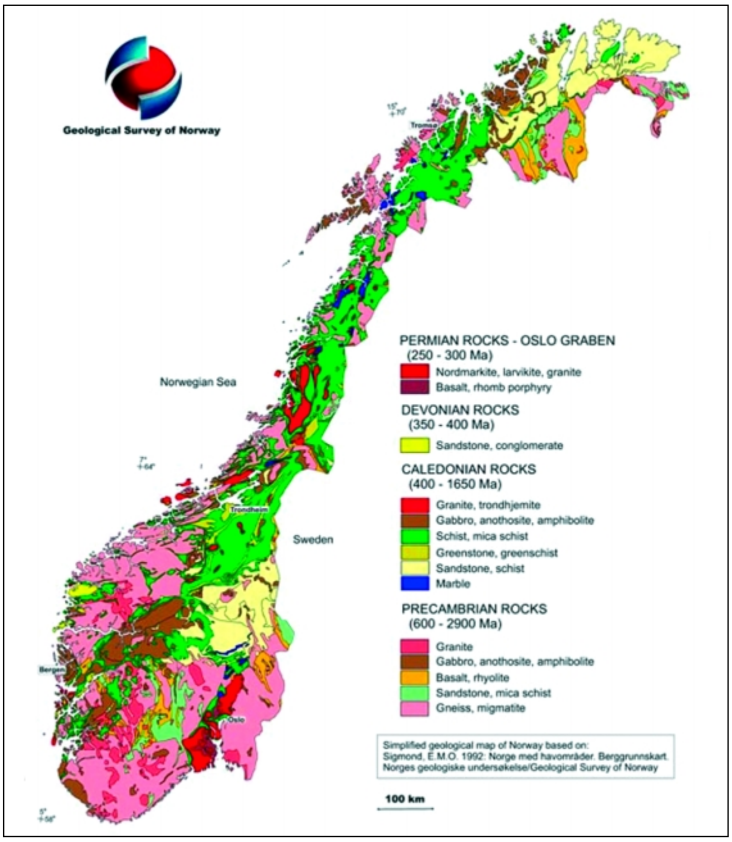


Figure 2.2. Geological map of Norway

Table 2.1. Typical mechanical properties of Norwegian rocks

Rock type	Uniaxial compressive strength (MPa)	Point load index (tensile strength) (MPa)	Young's modulus (GPa)	Poisson's ratio
Amphibolite	110	12	70	0.20
Diorite	150	14	50	0.15
Gneiss	130	15	45	0.18
Granite	170	15	40	0.15
Greenstone	100	10	55	0.20
Limestone	75	8	70	0.30
Mica schist	70	8	30	0.20
Phyllite	35	6	30	0.10
Quarzite	170	15	55	0.15
Sandstone	150	15	30	0.20

Starting in the last decade of the 19th century and continuing into the first half of the 20th century significant development of the railway system took place. The 493km long railway line between Oslo and Bergen with its 184 tunnels was opened in 1909. Most of the hydropower tunnels were built between 1950 and 1990. In the 1970's the Norwegian oil and gas era began and the experience of tunnelling enabled the application of underground storage, transport tunnels and pipeline shore approaches for hydrocarbon products. During the last two decades of the 20th century construction of many large road tunnel projects have been realised, and several sub-sea tunnels for communication to islands and crossing of fjords have been excavated.

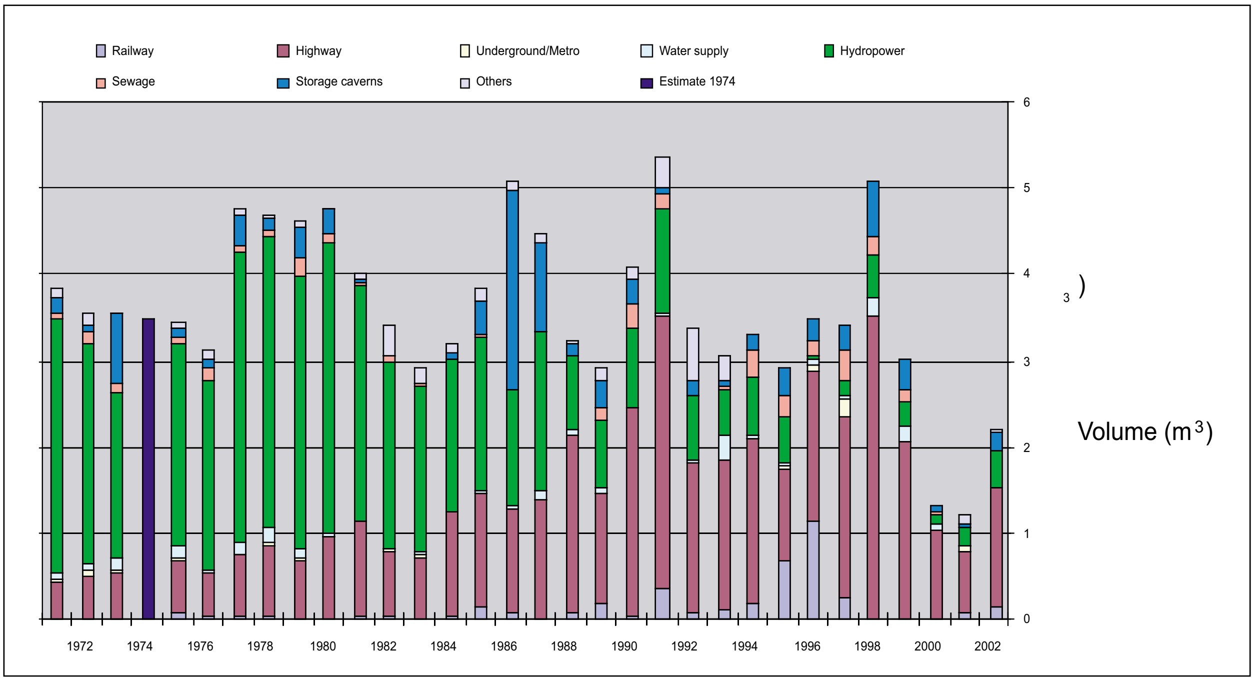


Figure 1.1. Tunnel excavation in Norway over the last 32 years

3 NORWEGIAN TUNNELLING PROJECTS

3.1 Hydropower schemes

3.1.1 General

The climate and nature in Norway allows for extensive development of hydropower schemes and the annual output from hydropower production is about 121TWh, constituting nearly 100% of the electric power supply. The development of hydropower started back in 1885. Tunnels have been used to various extent in some 300 hydropower projects with a total tunnel length of about 3500km.

See Appendix 1 for details on reference projects and list of publications on this theme.

3.1.2 Underground stations

Some 200 underground hydropower stations have so far been constructed in Norway, and this is about half the total number of underground stations found world wide. The advantages of underground hydropower stations are mainly associated with better sheltering and reduced maintenance cost compared to facilities placed above ground. Especially during the cold war the security aspect was of importance. It is also usually possible to select a location for the underground station where the geological conditions are favourable.

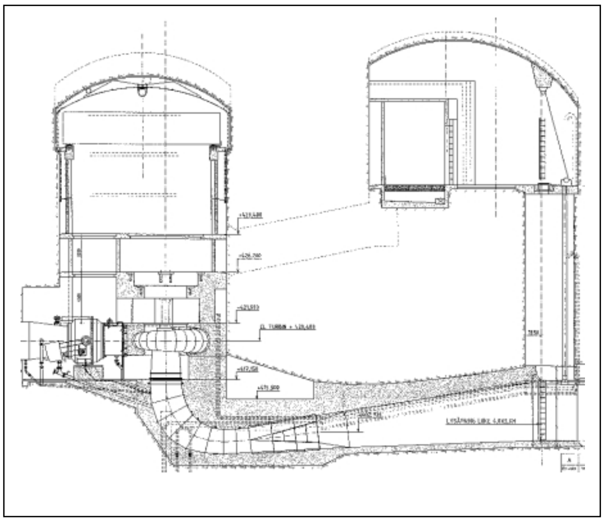


Figure 3.1. Cross section of a Norwegian power plant

The first underground hydropower station that was built in Norway, Bjørkåsen near Narvik, was built in the years 1919-1923, and since 1950 the typical design has been to locate hydropower plants underground.

Underground hydropower stations are designed with a special type of cavern layout. The cavern span is usually 15-20m and the height is 30-40m, and a two-cavern solution with the transformer station in a cavern parallel to the powerhouse is also often used, see Figure 3.1.

3.1.3 Unlined pressure shafts and air cushion chambers

Figure 3.2 shows the development of the general layout of hydropower plants in Norway.

Before 1950 underground power stations were rather unusual, and from the headrace tunnel there was a penstock on the surface.

From 1950 and onwards most of the installations were moved underground and the surface penstock was substituted by a steel-lined pressure shaft.

To obtain cost savings, alternative solutions to reduce the length of the steel-lining were looked at, and unlined pressure shafts were tried as early as 1920, but it was not until 1960 that this became the common solution. At present, more than 80 unlined shafts/tunnels with a water head larger than 150m are in operation, with 1080m water head as the highest. To prevent water seepage into the powerhouse, the last part of the headrace tunnel nearest to the powerhouse must be equipped with a steel lining. The length of this lining depends on the water pressure, but is usually in the range of 30-80m. The access tunnel leading to the unlined pressure shaft must be confined with a concrete plug, normally 10-25m long, to withstand the water pressure. A manhole is installed in the concrete plug to allow later access. Around the concrete plug and the upper part of the steel-lined shaft high-pressure grouting is carried out, see Figure 3.3. The grouting pressure must be in the same range as the water pressure or even higher.

The headrace tunnel system for a hydropower plant needs a surge arrangement, either an open tunnel/shaft or a closed underground chamber. In most cases this arrangement has been solved with a surge tunnel from the top of the pressure shaft up to the surface. An air cushion chamber near the power station may sometimes constitute a less expensive solution, and in some cases also the only practical solution. After 1975 ten air cushion chambers have been built. The largest of these chambers with a total volume of 120 000m^3 is found at Kvilldal power station. They are all unlined, but normally extensive rock mass grouting has been undertaken and in some cases water curtains (artificial ground water compensation) are in use.

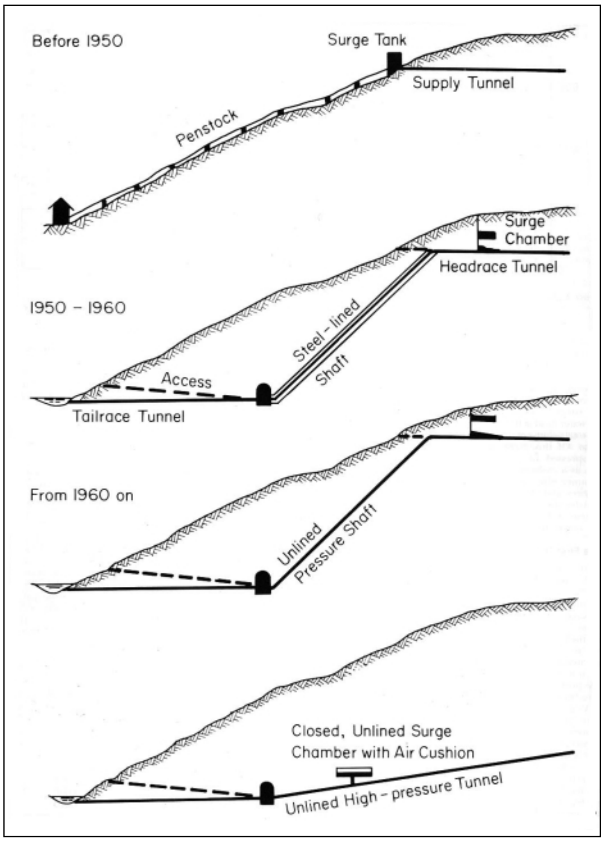


Figure 3.2. The development of the general layout of hydropower plants in Norway (from Broch 1985)

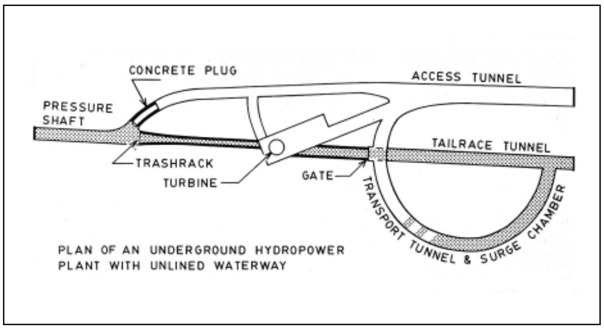


Figure 3.3. Plan of an underground hydropower plant (Broch 1985) Lake tap

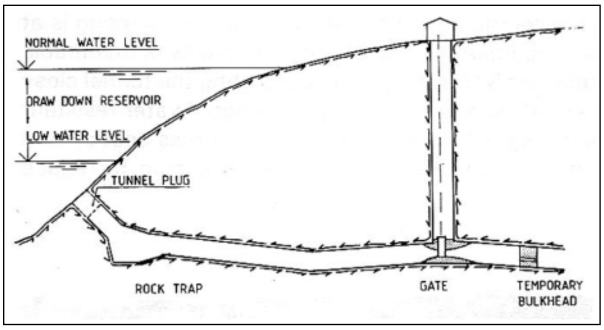


Figure 3.4. Principle of lake tap (Buen 1988)

The headrace tunnel in hydropower projects is coming from the reservoir that often is a natural lake. The tailrace tunnel from the station is usually ending in a lake or in a fjord. In such cases it will be necessary to pierce the bottom of the lake or the fjord with the tunnel, see Figure 3.4 and 3.5. The method is also named lake tap.

Figure 3.5.Piercing of the tailrace tunnel for Jostedalen hydropower plant at 40m depth in Gaupnefjorden (Statkraft)

This has been a regular practice in Norway for nearly 100 years. It is assumed that about 500 such lake taps have taken place. The deepest piercing for hydropower projects was carried out at a depth of 105m at Jukla power plant in Western Norway. A piercing is located where the rock quality is favourable and soil thickness is small. Exploratory drilling, grouting and careful blasting are techniques applied to bring the tunnel close to the rock surface. The final blast has at least two parallel holes cuts and high-strength water-resistant explosives are used. In the most common piercing method an air cushion is established below the final plug to reduce shock waves on gates and bulkheads. The principles of lake taps have also been applied for shore approaches where pipelines with oil and gas from the oilfields in the North Sea approach land, see chapter 3.4.

3.2 Railway tunnels

More than 700 railway tunnels with a total length of 316km have been constructed in Norway. Most of these tunnels were built before 1950. The longest railway tunnel is the double track 14km long Romeriksporten tunnel on the line between Oslo and the airport at Gardermoen and it was opened for use in 1998. At the same time the extension to double track at the southbound railway from Oslo included excavation of several tunnels. Tunnelling works are still going on (2004) at major railway projects west of Oslo.

The railway through the city centre of Oslo is also partly underground with a 3.6km long double track tunnel, including the National Theatre station. This station consists of two parallel caverns, each with a span of 20m, and both partly with only a few metres of rock cover. In addition the metro system in Oslo has about 10km of tunnels and several underground stations.

See Appendix 2 for details on reference projects and list of publications on this theme.

3.3 Road tunnels

The Norwegian road network consists of more than 800 road tunnels with a total length of 840km. Most of these tunnels were built during the last 50 years. The tunnels include a great variety in lengths, cross-sections, traffic density, standard and geological conditions. The Norwegian Public Roads Administration has issued a guideline for use in the design and construction of road tunnels, see Figure 3.6. This guideline (Handbook 021) provides recommendations on tunnel classes, including alignments (vertical and horizontal curvatures), cross-sections and installations. Typically, the annual traffic density is a governing parameter for the determination of the tunnel class.

The 24km long Lærdal tunnel, opened in 2000, is the longest road tunnel in the world, see Figure 3.7 (Grimstad, E., Kvåle, J.1999). It was designed and constructed according to Handbook 021 to provide a tunnel focusing the drivers confidence, comfort and safety, today it is an important link of the road connecting the Eastern and Western parts of Southern Norway. The tunnel goes through Precambrian gneisses with rock cover up to 1500m. The rocks have usually few joints but major fault zones and very high stresses locally caused severe rock stress problems.

For communication to islands and crossing under fjords numerous sub-sea tunnels have been excavated, the first was opened for traffic in 1983. Since 1983 a total number of 23 sub-sea road tunnels have been built, the longest being the Bømlafjord tunnel close to Bergen, 7.9km long. The deepest tunnel, connecting the islands of Hitra and Frøya reaches 264m below sea level. The critical parameter of minimum rock cover is governed by regulations in Handbook 021. A new sub-sea tunnel, the Eiksund project, is soon to commence construction and it will reach as deep as 280m below sea level, whilst a record long sub-sea road tunnel (approximately 24km) crossing the Boknafjord is in the planning stage.

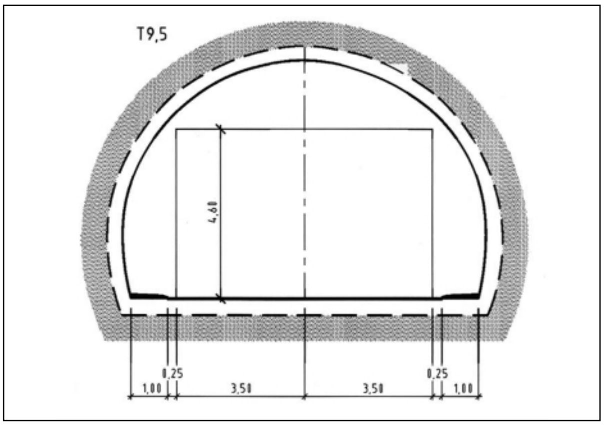


Figure 3.6. Cross section of a typical Norwegian road tunnel

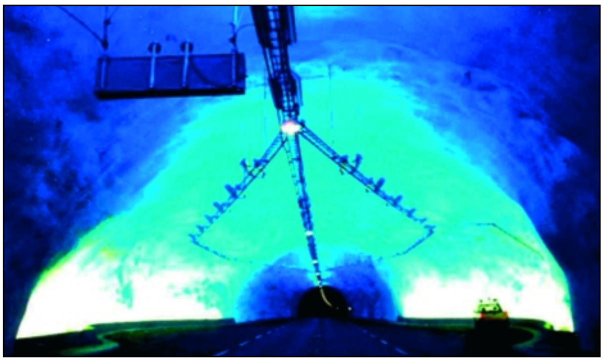


Figure 3.7. From the Lærdal tunnel (Grimstad)

See Appendix 3 for details on reference projects and list of publications on this theme.

3.4 Underground oil and gas installations

The traditional way of storing oil products in Norway as well as in other countries has been in steel tanks above ground. In the late 1960's the first projects were realised where oil and gas storages were placed in rock caverns in Norway. During the last 35 years rock caverns have been used commonly for oil storage. The total volume of oil stored in underground caverns is about 5 million m^3. In connection with the refinery at Mongstad north of Bergen there is a large number of caverns storing different types of oil products, including a crude oil terminal with a capacity of 9.5 million barrels. In addition the Mongstad project involves several gas storage caverns, according to both the chilled and pressurised concepts.

Particularly the technique of unlined caverns has been preferred, taking advantage of experience gained from unlined pressurised tunnels in hydropower projects. Rock mass grouting is needed for sealing of the rock mass and water curtains for hydrodynamic control.

Large combined storage, refinery and terminal facilities utilising underground facilities are also in use at Sture (see Figure 3.8) and Kårstø. In the Oslo area there are several unlined caverns storing oil products, and at

Figure 3.8. Oil cavern, Sture

Rafnes in South-eastern Norway there is an unlined cavern with a total volume of 100 000m^3 storing propane. In Glomfjord, at the Arctic Circle, there is an unlined underground storage for ammonia.

In connection with the landing of oil and gas from the North Sea several sub-sea tunnels and shore approaches for the pipelines have been constructed. These have been excavated through varying geology reaching 180m below the sea level. Piercing the sea bottom has been necessary to enable these shore approaches. At Kollsnes northwest of Bergen the gas pipeline from the Troll Field was taken into a sub-sea tunnel at a water depth of 161m. The rock mass was thoroughly grouted before the last blast rounds. The drilling was performed with a variance of less than 10 by a special constructed hydraulic boring equipment, and the explosives were especially prefabricated for each hole.

See Appendix 4 for details on reference projects and list of publications on this theme.

3.5 Water supply and sewage

Tunnels and underground openings have been used for various purposes in projects related to water supply and sewage handling. The four major cities of Oslo, Bergen, Stavanger and Trondheim have extensively developed their subsurface systems for such purposes.

In Trondheim the main water supply is from the lake Jonsvatnet 10km away from the city centre. A system of water tunnels and four underground water tank facilities has been constructed to supply the city's population with potable water. The Høgåsen water tank, see Figure 3.9, was excavated in greenstone next to a major fault zone. The rock mass quality was rather poor and it was decided that instead of using two independent single caverns, two double caverns should be constructed. In this way the span width could be reduced from 12 to 8m. The sewage system in Trondheim has also moved underground and consists of several tunnels and two underground treatment plants.

Oslo has more than 20km of tunnels for water supply and two underground plants for water treatment. In the Oslo area there are also about 50km of sewage tunnels, most of these are relatively new and have been bored by TBM, and two underground sewage treatment plants. One of these plants is in Precambrian gneiss of good quality whereas the other is in Ordovician sedimentary rock of rather poor quality.

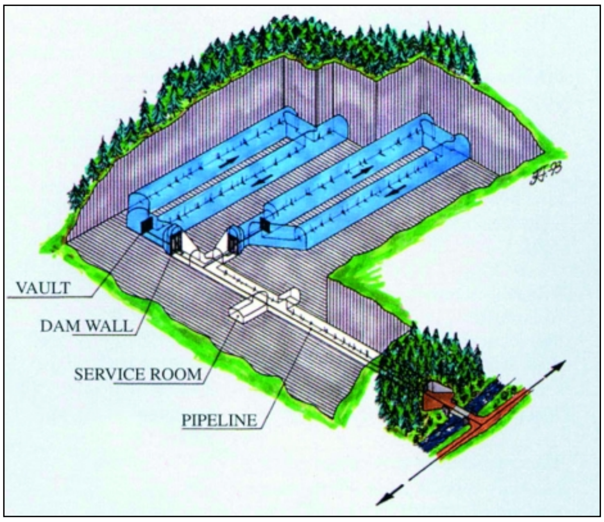


Figure 3.9. The Høgåsen underground water tank in Trondheim. Commissioned 1993. Capacity: 22 000m³

See Appendix 5 for details on reference projects and list of publications on this theme.

3.6 Waste storages

The largest underground waste storage in Norway is located in Odda, in Western Norway, see Figure 3.10. The project encompasses storage of waste from the zinc production in form of slurry which is deposited in rock caverns. The caverns have a width of 17.5m and a maximum height of 23.5m. The length is varied and the storing capacity for each cavern varies between 70 000m³ and 140 000m³. One year of zinc production equals 70 000m³ of waste, which means that excavation of caverns must be a continuous process. At the moment about ten rock caverns have been commissioned. This method of waste storage is considered safer than above ground deposition of the waste.

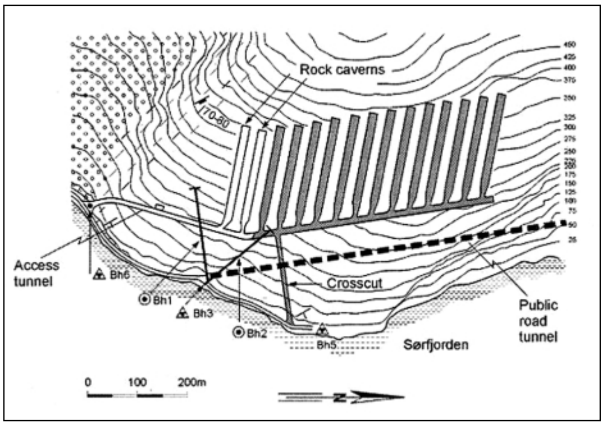


Figure 3.10. Schematic layout of the disposal caverns in Odda (Myrvang 1996)a

A similar, but relatively smaller facility has been constructed in connection with a nickel smelter near the city of Kristiansand. An underground repository for the waste mud was built underneath the factory. The facility consists of two caverns with volumes 11 000m³ and 34 000m³ and the rocks are Precambrian gneisses of good quality.

Since Norway has no commercial nuclear power plants (only two research reactors) the quantity of radioactive waste is small. The Himdalen repository, which is located approximately 40km east of Oslo, was built to provide a long term disposal and storage for low and intermediate level radioactive waste arising up to the year 2030. The repository consists of four rock caverns with cross-section of 138m2 and 55m length.

See Appendix 6 for details on reference projects and list of publications on this theme.

3.7 Sports halls

Several caverns for various sports activities have been built in Norway. Most of these caverns have a span of 20-25m and can be used for handball. The Gjøvik Mountain Hall, built for the Olympic Winter Games in 1994, is the largest cavern for public use in the world. It has a span width of 61m and a length of 90m located only 25 to 50m underground, see Figure 3.11. Feasibility studies included mapping in existing nearby caverns by use of the Q-system, stress measurements and FEM and UDEC-BB modelling. Subsequent site investigations included cross-hole seismic tomography, stress measurements, logging and testing of joints in cores and detailed numerical analysis using UDEC-BB for predicting performance for comparison with distinct element non-linear FEM for verifying performance and comparison with the distinct element results (Athanasiu and Heimli 1994, Barton and Hansen 1994).

Measured deformations of only 6 to 8mm were very close to those predicted, and the stability situation is mainly a result of the high horizontal stress of 4 to 5MPa at cavern depth.

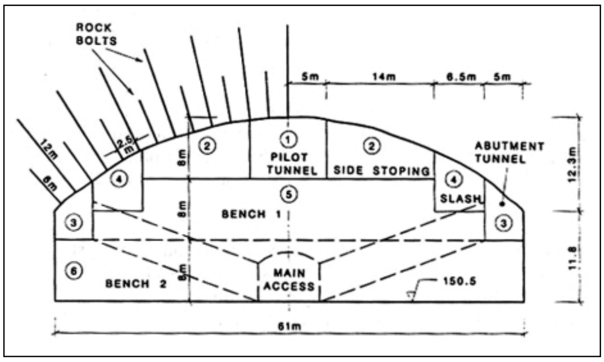


Figure 3.11. Cross-section of the Gjøvik Mountain Hall with excavation sequences (Bollingmo, Heimli and Morseth 1994)

The construction cost of an underground sports hall will usually be somewhat higher than for a similar building above ground. Experience indicates that reduced maintenance costs and energy saving often balance the higher construction costs. The fact that above ground space is a scarcity and highly priced in urban areas, is also important factors favouring underground facilities. In addition the underground sports halls can be combined with air-raid shelters.

See Appendix 7 for details on reference projects and list of publications on this theme.

3.8 Rock caverns for civil defence and military purposes

3.8.1 Use of underground solutions

As a member of NATO, Norway has always put large efforts in a well updated and modern defence system. Underground caverns and tunnels with adequate rock cover, have been experienced as a very good starting point for protection of defence facilities. Full protection can only be obtained by application of certain special equipment and systems. A short description of the Norwegian experience with rock caverns for defence purposes is given in the following.

Experience gained during World War II showed that rather simple rock caverns gave surprisingly good protection, even against very large conventional bombs.

Throughout the decades that followed, the development in the technique of blasting and excavating rock, made rock caverns a cost effective alternative also for defence purposes. This coincided with a very active rearmament period that started in 1950 and came to a preliminary halt in 2002. This rearmament took advantage of the significant technical development in the tunnelling industry and involved extensive use of rock caverns for

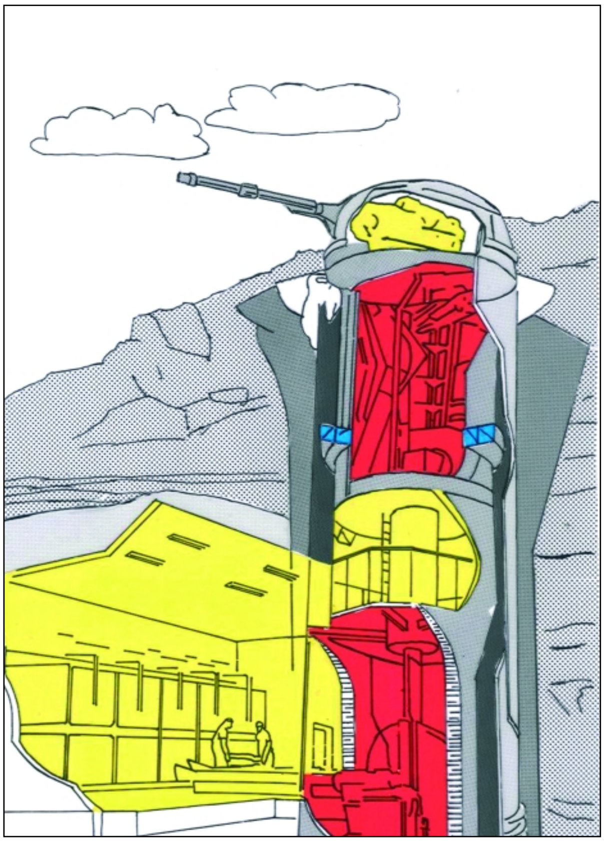


Figure 3.12. Artistic drawing of 120mm coast artillery gun placed underground and a "pop-up" radar station

the Army, Navy and Air forces. The use of underground solutions typically involved facilities as follows:

- For the Army: Head quarters, land fortresses, ammunition stores, workshops for vehicles and guns, communication facilities, close in defence systems.
- For the Navy: Docking facilities, work-shops, oil- and fuel storage, ammunition stores, communication- facilities, radars.
- For the Coastal Defence: Gun-emplacements, torpedo batteries, control room for controlled mine fields, radars.
- For the Air Force: Shelters for aircrafts, air defence positions, work shops, ammunition stores, communication facilities, refuelling systems, liquid oxygen systems.
- For the Civil Defence: Public air shelters and combined underground solutions of various sports facilities and air shelters.

3.8.2 Strengthening of the rock caverns

Rock cover

For an underground defence facility the main protective element is the rock cover. The thickness of the slab of competent rock above a cavern or a tunnel is an important and decisive parameter for the design and layout of such facilities. Empirical and theoretical methods have been developed to calculate required thickness taking various bomb loads and bomb types into account. The rock mass quality influences the bomb's penetration depth.

Portals and entrances leading to a rock cavern constitute normally the weak points in the design of an underground defence facility as these areas may not have a sufficient cover to resist weapon effects. The portals and entrances must be given adequate protection related to the design loads and various types of underground bomb traps etc. have been developed to reach the desired protection level. The typical weapon loads considered in the design of defence facilities are normally caused by the use of:

- Conventional weapons
- Nuclear weapons
- Chemical weapons
- Biological weapons

In addition, sabotage and terrorists actions must be considered while designing underground solutions for defence purposes. However, placing such facilities underground has proved to be beneficial compared to traditional surface solutions, the latter being indeed the most sensitive to hostile actions.

Protection systems

Throughout the years, special protection equipment has been developed in Norway. To day such equipment are available to meet all significant weapons effects. In cooperation with USA and other NATO countries, this equipment is to a great extent tested in large scale. After successful testing, equipment must then be approved by the Norwegian defence authorities. A certificate stating the protection level etc will then follow the equipment.

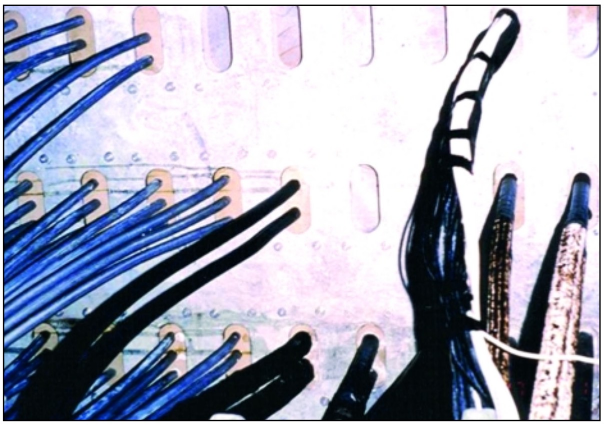


Figure 3.13. Cables entering the blast barrier

A protection system comprises equipment designed for different weapons effects put together.

Blast protection

A variety of different equipments is available such as: blast hatches, blast doors, blast gates and blast valves, all in steel or steel combined with concrete. Further, penetrations are made for cables etc. through the blast barrier. The blast protection equipment put together in the blast barrier in the opening to a rock cavern, create the blast protection system.

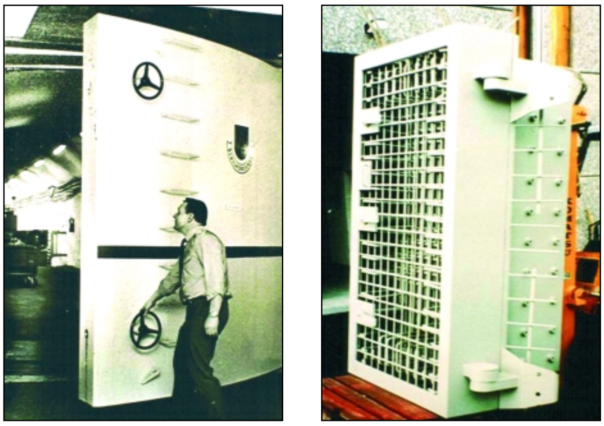


Figure 3.14. Typical blast gate and blast door

Figure 3.15. Gas filters in an underground facility

Ground shock protection
Concerning ground shock, nuclear weapons create the largest threat to a defence facility inside a rock cavern. In principle two kinds of protection of the building and /or sensitive equipment are used:
• Overall protection of the building inside the cavern
• Individual protection of sensitive equipment

The choice of type of equipment will depend on weapon loads and protection level. The overall protection is often used in large facilities designed to resist an induced ground shock. Such facilities usually house a large amount of sensitive computers and/or communication equipment.

Electromagnetic pulse from nuclear weapons (EMP)
The rock cover is normally not sufficient for EMP protection, thus particular EMP-protection has to be included. A building inside a rock cavern is normally of concrete or steel, and the steel reinforcement in the concrete is formed to create a Faraday cage. The spacing of the bars is related to the protection level (20-80 dB damping). A steel building gives normally a very good protection. For the openings in the cage special equipment is available: EMP doors, EMP honeycomb filters for ventilation, EMP penetrations for cables and pipes, EMP locks and EMP arrestors.

Tempest
Tempest is the "nick name" for the enemy's possibility to listen to communications or receive electronic signals. High Tempest protection normally demands steel lining or steel walls. Special treatment for computers and telecommunication equipment is necessary.

Chemical weapons (gas). Radio active fall out.
Chemical weapons are considered as a major threat. All military facilities are therefore equipped with standard gas filters. As for radioactive dust (fall out) the gas filters include dust filter and absolute filters will prevent radioactive dust to enter the inner part of the facility.

3.9 Other rock caverns
Rock caverns are also used for cold stores and freezing stores which can be an economic solution especially because of energy savings. In urban areas several car parking facilities have been built as surface space is highly priced or simply because there is no space elsewhere than underground.

4 GEOTECHNICAL INVESTIGATIONS

4.1 Philosophy for geotechnical investigations

Geotechnical investigations for underground projects are normally divided into two principally different schemes; one such scheme is related to investigations done prior to the construction works, so called ´pre-investigations´ and the other is related to investigations that are executed during the construction phase. The main aim of the pre-investigations is to provide the base data for the design and planning works. Investigations made during construction, for example at the tunnel face aim at providing detailed information as a basis for particular decisions to be taken. Traditionally, the Norwegian tunnelling philosophy allowed a relative large extent of the total investigations to be done during construction. In more recent years, when tunnelling in urban areas and environmental sensitive areas, pre-investigations have been more emphasized. In the following some main aspects of the investigation principles will be described, including also examples of typical application of some main investigation methods.

A research programme has currently been undertaken in Norway, named "Miljø- og samfunnstjenlige tunneler", in English a translation might read as "Tunnels to serve the society". One of the first objectives of this programme was to identify the need of seeking new investigation methods, and to developing technically the existing ones.

The programme concluded: "Pre-investigations shall be done with methods and means to ensure that a geotechnical basis exists sufficient to evaluate the consequences of running a sub-surface project and establish realistic cost estimates". In sub-surface projects it is important that the involved parties have a common understanding of the aims of the pre-investigations and that they are aware of the inevitable risk of not being able to cover all aspects of the geotechnical conditions.

Currently, various methods and means to run pre-investigations exist, and it is quite easy to loose the track and overdo the level of the pre-investigations. Even an extensive pre-investigation schedule may not necessarily improve the confidence of the geotechnical basis, but would indeed increase its costs significantly. A need must be identified on a project specific basis to choose methods and content of the pre-investigations. This could include: prevailing geological conditions, location of the project, geometrical aspects, complexity of the project, design stage and finally the Owners requirement to cost and risk confidence.

Predictability is a key word associated with pre-investigations. In Norway we experienced a trend during the nineties with an increased number of projects facing dramatic cost overruns and also negative publicity. Predictability was not good enough, neither for the cost estimates nor for the consequences of the tunnelling process. This was indeed harming the tunnelling industry as a whole. Insufficient geotechnical investigations may take its share of this development.

The trend is now more towards utilising the technically most developed investigation methods even further. It seems possible to get more information out of the existing methods, and together with a more dedicated investigation program the predictability may again reach the desired level.

Still the designer's responsibility is to identify the appropriate level of investigations, as the process of pre-investigations is inevitably costly and the outcome of the individual investigations may be associated with limitations. A typical philosophy in Norway has been that investigations that were expected to provide very limited probability of construction cost savings were ruled out. It allows a cost and time effective pre-investigation schedule, although it is linked closely to the particular risk sharing principle that is applied in Norwegian tunnelling

Geological mapping will always be the basis for geotechnical design. Normally identification of jointing parameters must supplement ordinary geological mapping.

The geotechnical design has sometimes been made flexible and more investigations and detailed design has been made when an access has been made into the area in concern. This was for example the case for the Svartisen Hydroelectric power scheme near the Arctic Circle in Northern Norway. By the time the 800m long access tunnel reached the planned area for the power house area, a water bearing zone was encountered. Core drilling was then carried out from the tunnel and the detailed location of the power house was based on the findings from these borings.

Design procedure according to Handbook 021
The geotechnical design for underground tunnel projects typically follows the procedure as outlined below, in accordance with the guidelines in Handbook 021 issued by the Public Road Administration.

4.2 Typical investigations at various design stages

4.2.1 Feasibility study
During the first step of a project various alignments and layouts are considered. The investigations are often limited at this stage of the design, and the aim would be to determine the feasibility of a project and how this can be accomplished within reasonable costs. The geotechnical investigations typically consist of and report the outcome of:
- Desk study of existing geological references, maps and publications.
- Study of air-photos
- A field survey

4.2.2 Detailed feasibility design
The recommendations from the feasibility study are further detailed at this stage. The final location for the tunnel is usually selected for further planning. The geotechnical investigations typically employ the following main aspects:
- Registration of soil types and estimation of thickness
- Mapping of rock types
- Measuring of bedding and foliation orientation
- Description and characterisation of jointing
- Mapping of weakness zones
- Evaluation of tunnel entrances
- Geophysical measurements and sounding in areas of low rock cover
- Laboratory testing (rock mechanical parameters, drillability tests etc)

Based on these investigations the final location would be determined and the project cost estimate shall be calculated with an accuracy of ± 25%.

4.2.3 Detailed design
During this stage the chosen project area is evaluated in detail. The geotechnical investigations depend on the local conditions and the extent and content of investigations carried out at the detailed feasibility stage. Supplementary investigations would be undertaken aiming at obtaining additional information of particular geological features, certain areas of a site, etc. The detailed design would typically include the following:
- Core drilling
- Stress measurements
- Geophysical tomography in particular zones
- Perhaps numerical modelling

The cost estimate shall be calculated with an accuracy of ± 10 %

4.2.4 Tender stage
The tunnelling tenders are always supplied with a geotechnical summary report. The geotechnical summary report contains a wrap-up of the geotechnical investigations that have been undertaken for the project. The report shall also include all factual data that has been found during the pre-investigations and describe the geotechnical conditions expected to be encountered during the tunnelling.

4.2.5 Investigations during construction
Geological conditions may be very complex, and it is always an inherent risk that geotechnical investigations performed from the surface have not revealed all details of the rock mass. Further, as described above, the Norwegian tunnelling philosophy is a rather flexible one and certain investigations might serve the projects best being undertaken during the construction stage. The most typical of investigations performed during construction is probe drilling. Further, various geophysical methods have been developed for rock mass prediction ahead of the tunnel face.

Experienced engineering geologists are normally following the tunnelling projects conducting geological mapping underway. This mapping may not serve only as as-built documentation, rather is it also used for forward prediction of the ground conditions. Particularly such mapping is useful when it is used together with geological models and predictions from the pre-investigations and the geological models are questioned and revised according to new findings. Such models are then useful tools for the contractors and owners for production planning, cost adjustments, etc.

4.3 Rock stress and deformation measurements for design and stability control

4.3.1 Introduction
Historically, few specific geological requirements were put forward as prerequisite when planning an underground project. The rock conditions were often fairly good, and the site was in many cases more or less fixed. The design was based on standard engineering geological mapping and experience according to expected and experienced conditions. Hardly any measurements were carried out for stability control. However, as large span excavations became common, particularly in connection with near surface underground sports halls, the need for more accurate stability control measures became more apparent.

Also, experiences from the mining industry showed numerous cases where excavations with extremely large span (60 – 80m and more) were apparently stable without any rock support at all. Research showed that one main reason for this was the existence of sufficient tectonic, horizontal stresses even at shallow depths. Predominant horizontal stresses produce favourable compression and confinement in the excavation crowns, creating stable self supporting structures.

This called for in-situ rock stress measurements preferably before the excavation started, followed by rock stress control and deformation measurements during and after excavation.

This has been carried out in connection with a number of large underground excavations to document the stability of large spans and rock pillars.

Horizontal stresses of geological origin (tectonic stresses) are quite common in Norway, and in many cases the horizontal stresses are higher than the vertical stresses, even at depths greater than 1 000m. The majority of rock stress related problems in Norway actually originates from high horizontal stresses, rather than vertical stress due to the rock overburden. This has been the case in a number of road tunnels and tunnels connected to hydropower development, and high stresses have also caused considerable stability problems in power house caverns. This has again called for rock stress and displacement measurements.

4.3.2 Applied field methods
Rock stress measurements
Principally two different principle methods are employed:
- 2-D ("doorstopper") and 3-D over-coring cells. The principles are shown in Figures 4.1 and 4.2.
- Hydraulic fracturing. The principle is shown in Figure 4.3.

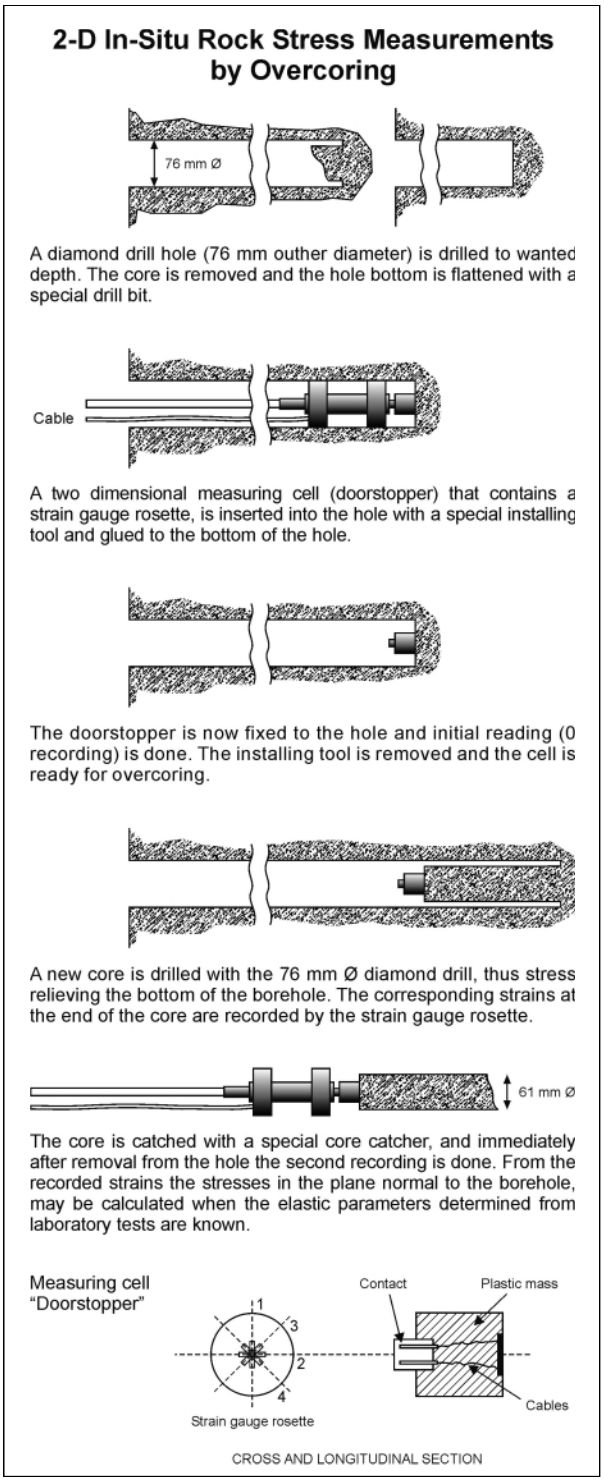


Figure 4.1. Principle for 2-D ("Doorstopper") overcoring method for rock stress measurement

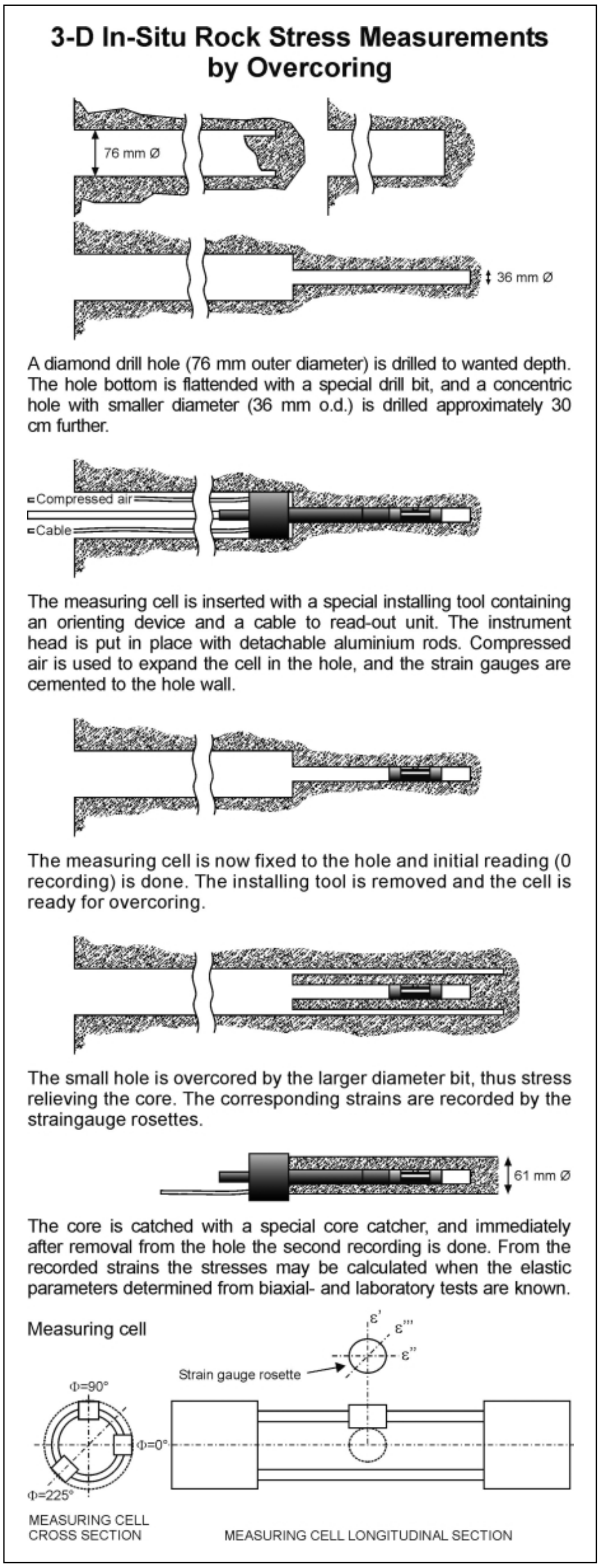


Figure 4.2. Principle 3-D overcoring method for in-situ rock stress measurement

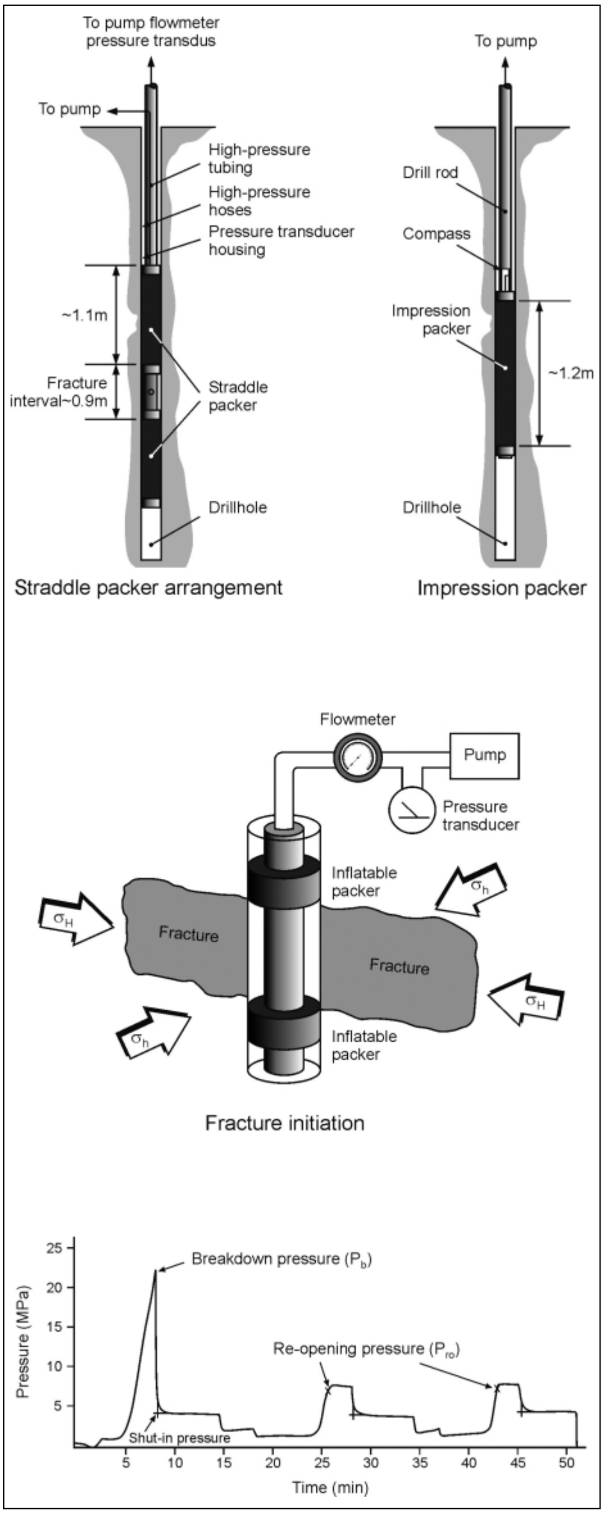


Figure 4.3. Principle for hydraulic fracturing

Deformation measurements

Deformation measurements may principally be carried out in two different ways:

- Borehole extensometer measurements. The principle is shown in Figure 4.4.
- Convergence measurements. The principle is shown in Figure 4.5.

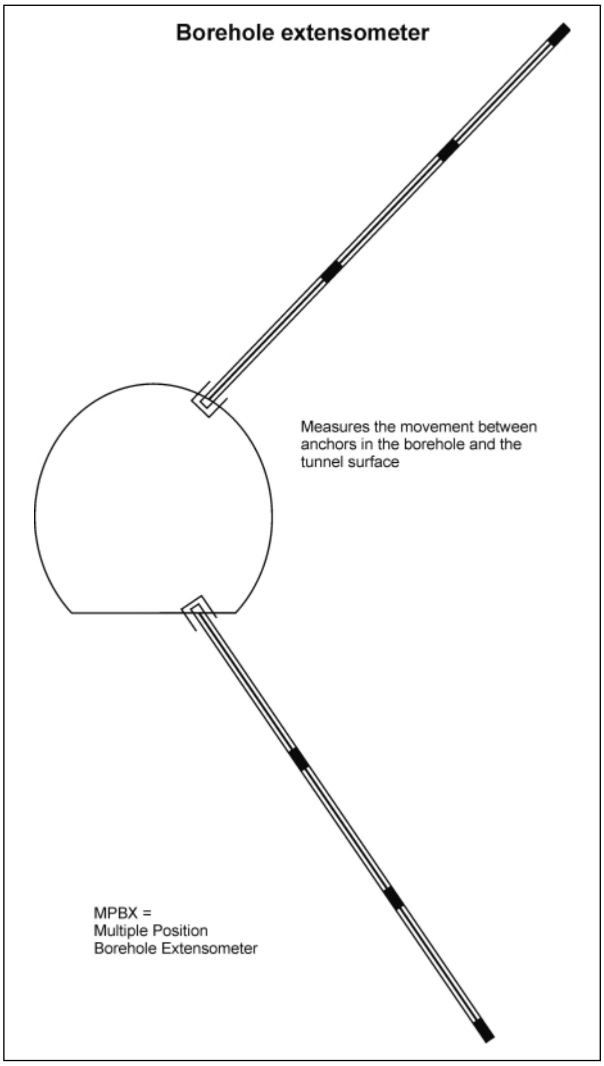


Figure 4.4. Principle for borehole extensometer measurements.

In addition to this, high accuracy surveying instruments may be used for deformation measurements.

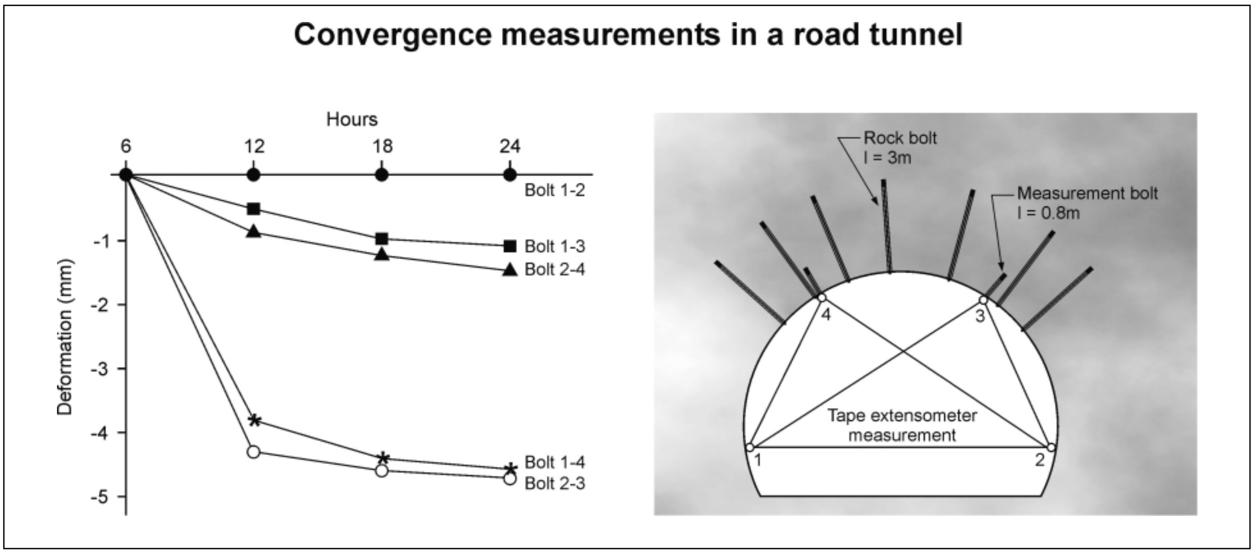


Figure 4.5. Convergence measurements in a tunnel

5 ROCK MASS CLASSIFICATION

5.1 Objectives of rock mass classification systems in Norwegian tunnelling

A typical Norwegian tunnel is characterised by changing ground conditions, varying sections of good rock mass quality and sections of poorer quality. The primary objective of the use of rock mass classification systems is thus to qualify various engineering properties of, or related to, the rock mass. The classification system predominately used in Norwegian tunnelling is the Q-system, whilst others, such as RMR, RMi and GSI are used in few cases.

The output obtained from rock mass classification systems is typically related to:

1. Description of the rock mass expressed as quantified rock mass quality, incorporating the effects of different geological parameters. This enables the comparison of rock mass conditions throughout the site and delineation of regions of the rock mass from 'very good' to 'very poor', thus providing a map of rock mass quality boundaries.
2. Empirical design with support guidelines compatible with tunnel stability and excavation method. Traditionally, this is often seen as the major benefit from the use of rock mass classification systems.
3. Estimates of rock mass properties. Rock mass classification expressed as an overall rock mass quality has been found useful for estimating the in situ modulus of rock mass deformability and the rock mass strength to be used in different types of design calculations.

The classification systems also serve as checklists for geotechnical field- and tunnel mapping and for core logging.

5.2 The Q-method

The Q-system was developed by the Norwegian Geotechnical Institute in the early 1970's (Barton et al. 1974) and has earned recognition around the world. A major update was released in 1993 (Barton and Grimstad 1993), and a new update including a complete users handbook will be released in 2004. The Q-value can be calculated as follows:

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

RQD = degree of jointing (Rock Quality Designation)
J_n = number for joint sets
J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = Stress Reduction Factor

The Q-system contains the experience obtained from more than 1000 case histories from existing tunnels, and an empirically based diagram showing the connection between Q-values and the support used in these cases has been constructed, see Figure 5.1. The diagram also includes Reinforced Ribs of Sprayed Concrete, a design which is partly based on numerical modelling.

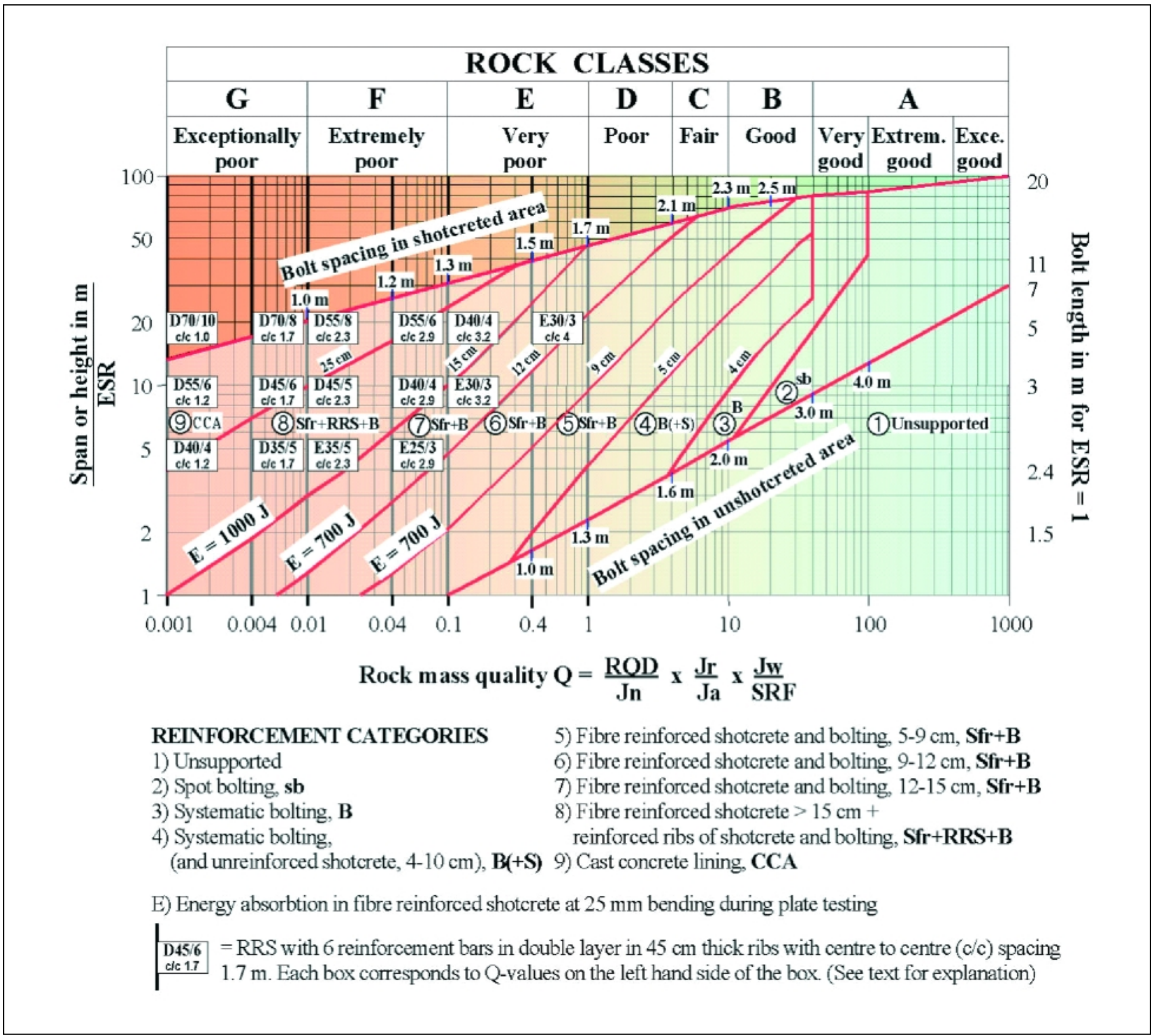


Figure 5.1. Q-values and support guidelines. (Grimstad et al 2002)

5.3 The RMi system

The RMi sytem has been developed primarily for improving the collection and use of geological parameters in rock engineering.

The rock mass index, RMi, is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. The RMi system was first presented by Palmström (1995) and have since been further developed and presented in several papers. It makes use of the uniaxial compressive strength of intact rock (sc) and the reducing effect of the joints penetrating the rock (JP) given as:

1. RMi = σ_c x JP for jointed rock masses. The jointing parameter (JP) is by empirical relations connected to the joint condition factor, jC, and the block volume, Vb. The joint condition, jC, can be estimated by the joint roughness, the joint alteration (similar to Jr and Ja in the Q-system) and the joint size.
2. RMi = σ_c x fs for massive rock having block size larger than approx. 5m^3 (where f_σ > JP). The massivity parameter, f_σ, represents the scale effect of the uniaxial compressive strength (which from intact rock samples to massive rock has a value of approximately $f_\sigma \approx 0.5$).

The connection between the different inputs parameters applied in the RMi is shown in Figure 5.2.

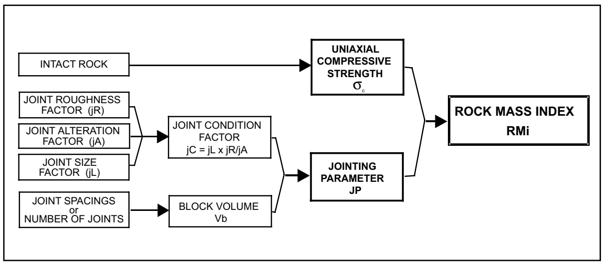


Figure 5.2. The input parameters to RMi (from Palmström, 1996)

The different input parameters can be determined by commonly used measurements and mapping and from empirical relationships presented by Palmström (2000). It requires more calculation than the Q-system and RMR system, but spreadsheets can be used to derive the RMi value.

Based on a characterisation of the rock mass by RMi, combined with geometrical features of the opening and ground factors like rock stresses, different rock engineering issues such as relevant rock support can be estimated using support charts. The charts have been developed from experience of more than 25 different projects and locations. Possible use of the system is shown in Figure 5.3.

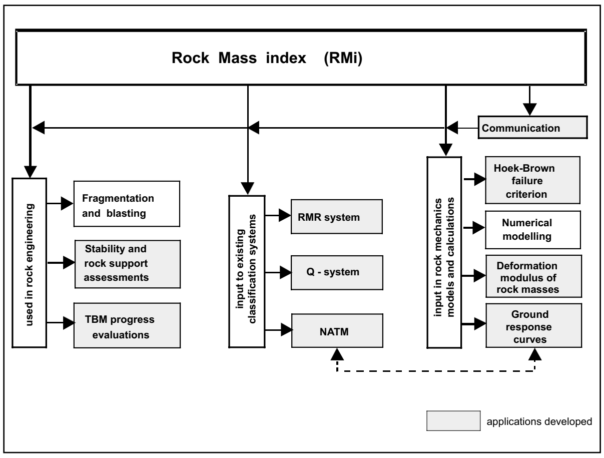


Figure 5.3. Possible applications of RMi (from Palmström, 1996)

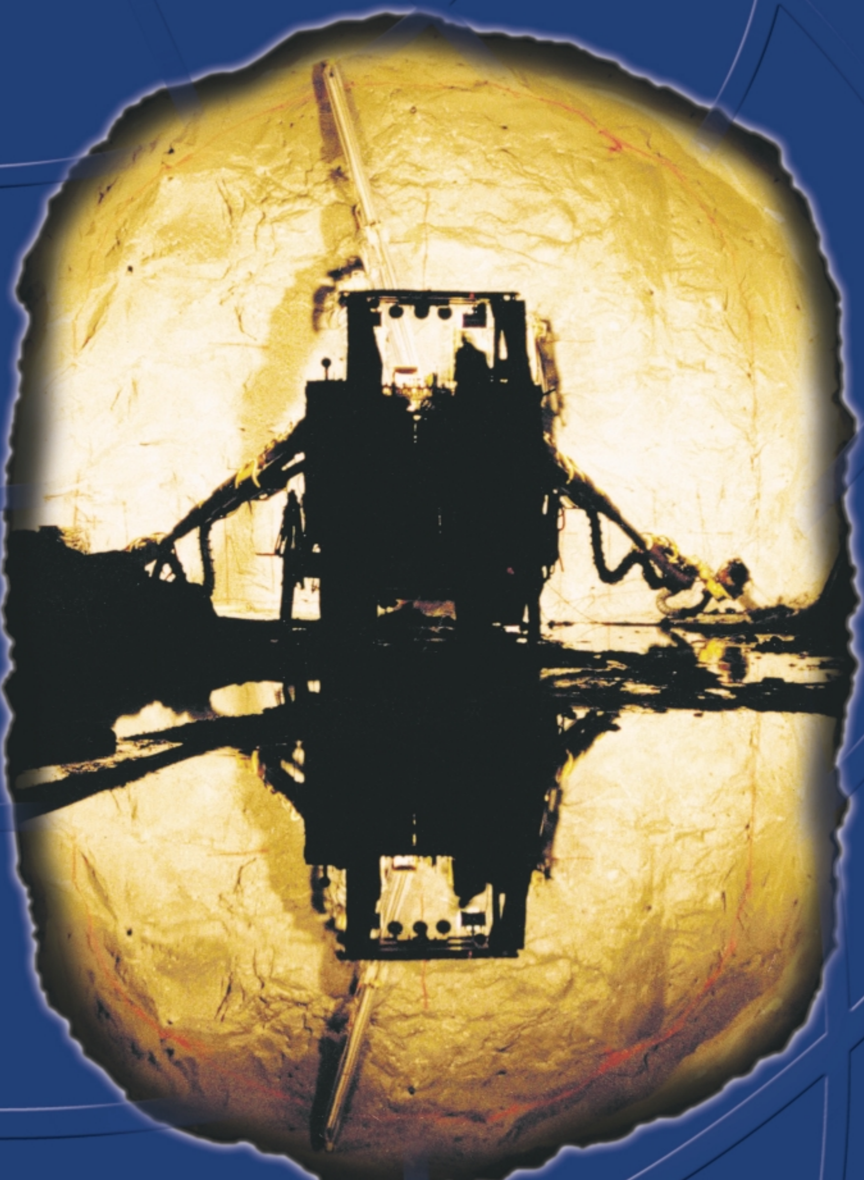
5.4 Experience on the use of rock mass classification systems

A major contribution of the classification systems for use in underground tunnelling is that they provide a way of quantifying the quality and capability of the rock mass, which can be understood in a global context. Their exactness is not of interest in decimals. With careful and respectful application we have learned in Norway that such systems serve as useful tools both in design and construction. Being used in combination with other classification systems and together with; engineering judgement; analytical and numerical analysis, monitoring and observation of the tunnel behaviour, the tunnelling engineer has a powerful toolbox at hand. It is important to keep in mind that the appropriate application of these systems, both during the design and construction phases are when they are being considered as guidelines and recommendations. Site specific considerations and modifications are needed to enable the most appropriate application of any of these classification systems.

It is important to keep in mind that the classification systems have some limitations. The systems characterise the rock mass stability, but not the stability of individual blocks. That means that even in rock mass where the system states good stability, individual blocks may be unstable. Concerning the Q-system it must be stated that most of the case histories are from hard rocks in Scandinavia. In soft rocks, other methods such as deformations measurements and numerical modelling should be used in addition.

The systems are often used for other applications than rock mass stability, such as calculation of different rock mechanical parameters. The results from such calculations should only be considered as rough estimates. However, since it is often difficult to carry out exact field measurements of such parameters, calculations by using the classification systems are often the best assessments the rock engineers can perform.

UNDERGROUND CONSTRUCTION TECHNOLOGY



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6 TUNNEL SUPPORT

6.1 General

The ground conditions in Norway can be said to favour the utilization of tunnelling and underground solutions and the rock support philosophy reflects this situation with a flexible rock support application. However, the rock mass quality may change rapidly, and contractors must therefore be prepared for support of rock mass of poor quality also, having the right measures and procedures at hand to handle these. A cost effective tunnelling philosophy has been developed where the application of well proven, flexible support systems have played an important role (Grøv 2001). In this chapter the most commonly used rock support will be presented.

In Norway, as in many other tunnelling nations, the rock support is usually installed in two stages: the temporary rock support and the permanent rock support. The temporary support shall primarily ensure the provision of a safe working environment during the construction period, and is consequently the contractor's responsibility. The permanent support shall ensure the project requirements on long term durability and in principle be intact during the whole lifetime of the tunnel, which for example is 50 years for road tunnels. The permanent support is thus the client's responsibility. A typical Norwegian speciality, and an important part of the Norwegian tunnelling philosophy, is to combine the temporary and the permanent support allowing the temporary support to later form a part of the permanent support. This implies, however, that the material used in the temporary support must comply with the technical specifications as set forth for the permanent support.

The rock support design is often carried out in co-operation between the contractor and the client. The design of permanent support is usually determined by the client, based on his engineering geological mapping in the tunnel.

Another aspect worthwhile to mention is the application of careful observation of the tunnel during the whole construction period. If needed, convergence pins or extensometers are installed for documentation purposes. Any sign of deformation, cracking of sprayed concrete etc. may indicate that additional support is needed. With a flexible contract system (and support system) based on the use of rock bolts and sprayed concrete as needed, additional support can be installed at any time during the construction period.

6.2 Rock bolts

Rock bolts and sprayed concrete are the basic support means in Norwegian tunnelling. Such rock bolts comprise steel rebar bolts with different types of anchoring methods. Traditionally, rock bolts with end anchoring by an expansion shell have been used for temporary support whilst rock bolts fully embedded by cement grout are used for permanent support. Recently, rock bolts end-anchored by resin capsules have become widely popular in the tunnelling industry and these are now approved for permanent support, having documented 30 years of successful operation. A fundamental investigation of rock bolts is given by Stjern, 1995. One of Stjern's main conclusions is that the fully grouted rebar bolt is considered to be the optimal bolt for hard rock applications, except for ground where large rock stress induced deformations are expected. In particular under shear loading where the fully grouted bolt is superseding the point anchored and the friction bolt by its instantaneous high strength dowel effect.

About 250 000 rock bolts are used in Norway each year, most of them for tunnel support. The long term durability of bolts is an important parameter, and much research has been carried out for example by the Norwegian Public Road Administration and the main producers of bolts. Generally fully grouted rebar bolts have been considered to be the most durable product. These are mainly applied as hot-dip galvanized for corrosion protection, a type of bolt that is generally the standard type in any underground project. The combi-coat corrosion protection has shown to be very effective as it consists of epoxy powder coating additional to hot-dip galvanizing. This reliable corrosion protection has been applied in harsh environments, including exposure to salinity and exhaust fumes in sub-sea road tunnels.

The newest bolt type on the market is the CT-bolt (see Figure 6.1), which consists of a steel rebar bolt with an ordinary expansion shell (for instant support) and an arrangement for injection of cement. The CT-bolts can then first be used as an ordinary end-anchored bolt for temporary support and later on be grouted to a permanent bolt. This bolt is available in the market both as hot-dip galvanized as well as combi-coated.

Figure 6.1. CT-bolt

6.3 Sprayed concrete and reinforced ribs of sprayed concrete (RRS)

6.3.1 Application of sprayed concrete

If the block size is too small to be anchored by rock bolts, sprayed concrete would be the preferable support method. Sprayed concrete was introduced for rock support application in Norwegian tunnels in the 1960's. However, the method did not gain any widespread popularity until 1980. During the last two decades the method has faced a significant improvement and today, sprayed concrete in Norway is performed in accordance with the wet-mix method and as fibre reinforced. General technical specifications and guidelines for use in sprayed concrete have been established by the Norwegian Concrete Association, and published in 1999 as Publication no.7 (revised in 2003, but only in Norwegian).

The Norwegian Public Road Administration initiated in 1995, due to the dramatic increase and the systematic use of sprayed concrete as rock support, a comprehensive project to document Norwegian experience: "Proper use of sprayed concrete in tunnels."

One of the main conclusions from this project was that the production process itself and the application procedure of sprayed concrete could dramatically influence the durability of the final product as well as the support structure. This effect must be given special attention in any process aiming at reaching a highly durable product.

Another output from the project was related to the thickness of the applied sprayed concrete. It was found that a governing parameter for durability seemed to be related to the applied thickness, and a minimum thickness of 60mm has since been required for Norwegian road tunnels. Any major water leakage must of course be drained before application of sprayed concrete commences.

The study focused the following aspects of with respect to sprayed concrete durability:

a) In urban areas tunnel entrance zones are exposed to aggressive loads during the winter, for example icy roads lead to use of de-icing salts. Thus, the entrance zones in 8 main road tunnels, all opened in the period 1970-1992, were studied. Measurements performed on concrete and sprayed concrete linings in the Oslo area showed the depth of deterioration is usually less than 10mm.

b) Deterioration of concrete structures takes place resulting from freezing and thawing processes. Zones exposed to such processes were studied in 22 tunnels opened between 1956 and 1992. Signs of deterioration have only been seen in areas of water bearing rocks with subsequent de-lamination between rock and concrete layer. This de-lamination is predominately occurring in tunnels older than 20 years with thin layers of sprayed concrete (less than 20mm).

c) Tunnels crossing underneath fjords and sounds are exposed to extremely harsh environment; saline water in combination with exhaust fumes. 13 sub-sea tunnels opened in the period 1987 – 1992 were studied. Since 1996 the applied sprayed concrete quality for sub-sea tunnels has been C45 and environmental class Medium Aggressive. The study showed few signs of deterioration in these cases. Investigations of the older applications, however, indicated a change of quality in areas with significant water seepage.

d) The influence on the sprayed concrete by rock spalling and rock burst was included in the program. 17 tunnels from the period 1960-1994 with rock spalling and / or rock burst, were studied. Stress measurements performed in two tunnels showed that the stress level in the fibre reinforced sprayed concrete layer was nearly zero, and the main function of the sprayed concrete is to stabilize rock particles in order to maintain the self-carrying capacity of the rock mass.

6.3.2 Reinforced ribs of sprayed concrete, description and examples

When the Q-value falls below 1, bolting as a support measure may not be adequate on its own, see Figure 5.1. The rock mass between the bolts must be stabilised by means such as sprayed concrete. Reinforced ribs of sprayed concrete is one solution which has become an useful application in adverse rock mass conditions. It consists of a sandwich type construction, based on fibre reinforced (and also plain) sprayed concrete, radial bolts, and rebars (Grøv 2000). Figure 6.2 shows a principal solution of reinforced ribs of sprayed concrete and Figure 6.3 shows an example from a railway station.

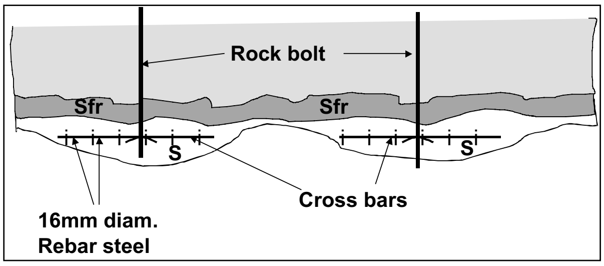

Figure 6.2. Principal sketch of two reinforced ribs of sprayed concrete with one layer of reinforcement (Grimstad et al 2002)

The system has the following advantages:
- Materials to be used are normally available on most construction sites.
- Convenient construction, easy to handle materials, and on-site production.
- Flexible installation and wide span in capacity.
- Cost effective.
- Ductile, allowing rock deformations without imposing load concentration on support.
- Allows tunnel progress shortly after installation.
- Easy to repair and custom design by spraying thicker concrete or adding new ribs.

For the Bjorøy sub-sea road tunnel outside Bergen in western Norway, the tunnel encountered a zone of loose silty sand over a distance of 25m (Aagaard et al. 1999). The Q-values in the critical section ranged from 0.08 to 0.003. The zone was successfully passed with a design including pre-grouting, pre-bolting (spiling), fibre reinforced sprayed concrete, and radial bolts. The invert was supported with a cast-in-place concrete lining. A monitoring program was established to follow-up potential deformations. The monitoring confirmed a stable situation without long term deformations. Despite the stability of the situation, the tunnel owner, the NPRA, required the rock support to withstand full hydrostatic pressure, which at this particular point was approximately 80m, requiring a concrete cast-in-place lining to be installed.

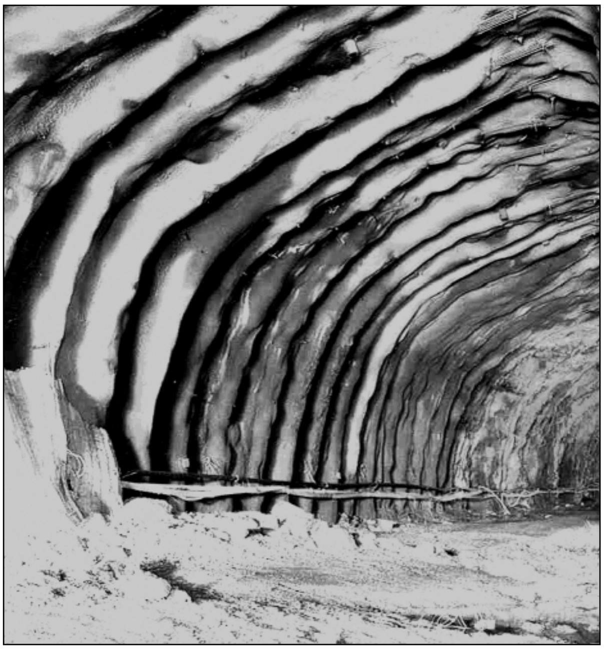

Figure 6.3. Reinforced ribs of sprayed concrete at Nationaltheateret railway station

The Frøya-tunnel outside Trondheim faced a number of weak zones during the course of excavation. Rock support covered a wide range of measures from rock bolts and sprayed concrete to cast-in-place concrete. The effectiveness of various support solutions has been demonstrated by the use of numerical modelling (UDEC) for a relatively wide zone with an average Q-value of 0.0013, i.e. extremely poor rock (Bhasin et al. 1999). In Table 6.1, a summary of the results from numerical modelling is given.

Table 6.1. Results from numerical modelling (UDEC)

Type of Support	Sprayed concrete 250 mm, concrete invert, rock bolts	Reinforced ribs and sprayed concrete (RRS)	Cast-in-place concrete lining (CCA)
Max. Displacement after equlibr.	14.4mm	17.1mm	17.3mm
Max. axial loading on bolts	3.3tons	11.6tons	-
Max. axial load on the structure	1.96MN (crown)	0.88MN (crown)	1.4MN (crown)
Max. joint aperture	3.3mm	3.3mm	3.5mm
Max. shear displacement	10.7mm	10.7mm	11.7mm

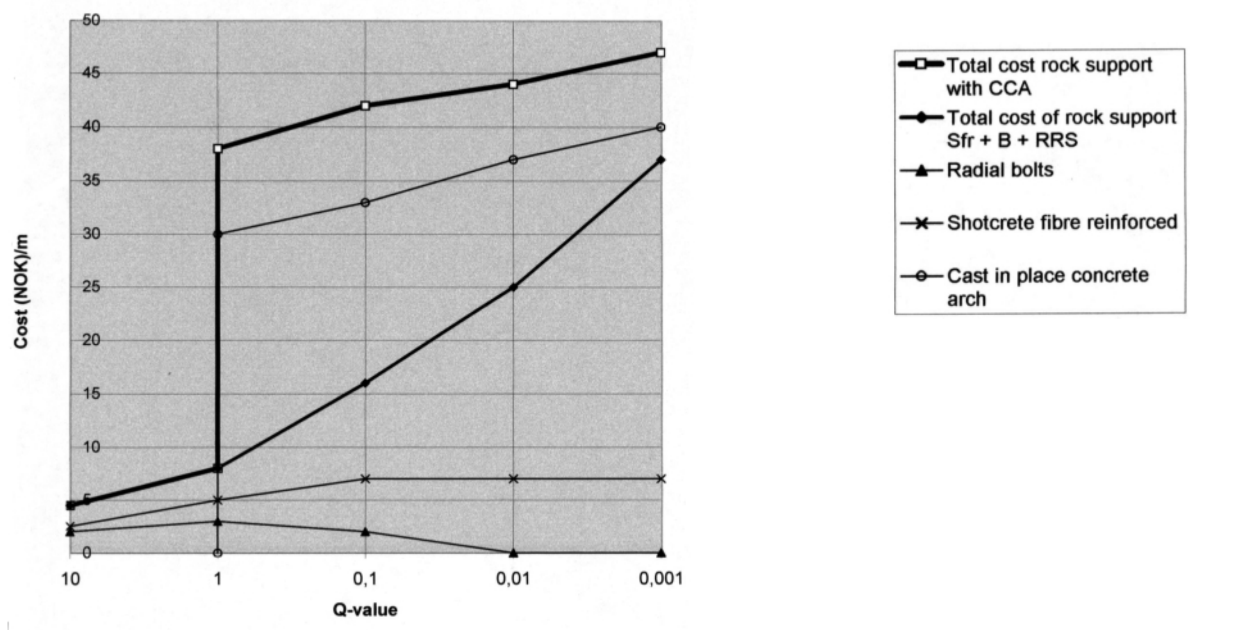

Figure 6.4. Cost comparison of applicable methods

FLAC 3D numerical modelling has also been performed to analyse reinforced ribs for the same reinforcement schemes and rock mass conditions as described above. The results confirm earlier calculations indicating that the two methods (reinforced ribs with sprayed concrete and cast-in-place concrete lining) produced deformations with insignificant difference where a concrete invert was applied in both cases.

A cost comparison of various support measures for adverse rock mass conditions is provided in Figure 6.4, including reinforced ribs and sprayed concrete compared to cast-in-place concrete lining. One US dollar is equivalent to approximately 7 NOK. A comparison of the costs associated with the solution involving reinforced ribs of sprayed concrete with traditional cast-in-place concrete yields that for rock mass classified as 1>Q>0.001, the application involving reinforced ribs is the most cost-effective.

6.3.3 Alkalifree accelerator

Compared to traditional sprayed concrete accelerators, alkali free accelerators offer a final strength increase; by forming a homogeneous and compact concrete matrix without internal tension and micro-cracking. Traditional accelerators produce a loss of 15-50% in potential compressive strength. The various types of alkali-free accelerators on the market can provide the following selection of characteristic properties for sprayed concrete:

- Early strength of 1MPa after 1 hour of curing.
- Final strength reaching as a minimum the same level as without accelerator (some produces 20% increase according to the suppliers).
- Low rebound.
- 300mm thickness sprayed in one operation.
- Low corrosiveness.
- Reduced permeability.

The Norwegian Public Roads Administration has performed a full scale test on a number of alkali-free accelerators currently available in the market (Storås et al. 1999). The results can be summarised as follows:

- No difference in personal dust exposure between the alkali-free and silicate based accelerators.
- Improved early strength development for the alkali-free accelerators compared to water glass.
- Wet conditions at spraying surfaces delay the early strength development for some accelerators.
- The tests indicate a durable, homogenous final product.

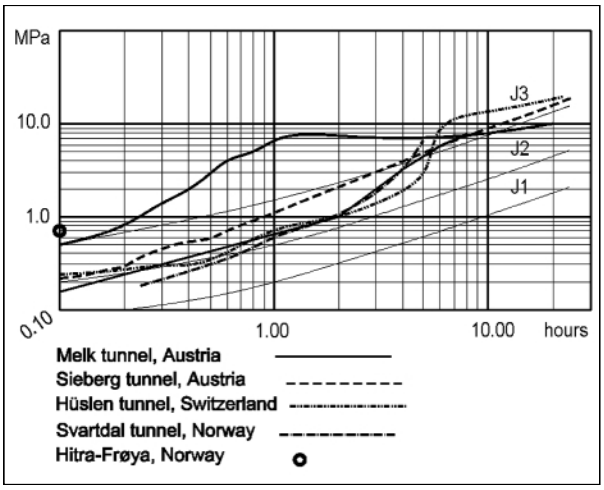

Figure 6.5. Excamples of strength developments using 7% Meyco SA 160

In northern Norway, the construction of the FATIMA-tunnel encountered severe rock conditions forcing the implementation of a cast-in-place concrete lining. A sedimentary rock type consisting of shale, siltstone, schist and sandstone hampered progress. Clay-fillings, slickensided joint surfaces, and a rock consisting of "chips" in general less than 5 cm long (and occasionally up to 20 cm) was common. The rock mass was in general classified as $0.05 < Q < 0.1$ equivalent to "extremely poor" to "very poor" rock mass according to the Q-system.

After a successful testing period, sprayed concrete lining, with a thickness of 200 to 250mm and alkali-free liquid accelerator was established replacing the previous cast-in-place concrete lining. Half the thickness was sprayed immediately after mucking out, the remainder after the next blast round, to allow some deformation to take place. Rock bolts (0.25 bolts per m^2) were installed some 3-5 days after the completion of concrete spraying.

A monitoring program has been implemented consisting of convergence measurements. Almost six months after the support installation, deformations in the magnitude of 1mm/month took place. However, the best indicator of the behaviour of the support measures is the presence of visual cracking or damage on the sprayed concrete, of which there has been none reported.

6.3.4 Drained support structure

Another important aspect of the support system described herein is that the rock mass in combination with the rock support constitutes a drained structure. This means that the support measure installed has not been constructed to withstand external water pressure. Excessive water must therefore not be allowed to build up pressure behind the rock support measure.

However, even in a tunnel that has been subject to an extensive, customized pre-grouting schedule, some seepage may occur. This tunnelling concept includes a controlled handling of excessive water at the tunnel periphery and behind the sprayed concrete lining. Excessive water must either be piped to the water collection system in the tunnel or taken care of by a water protection system. Drainage can be achieved by means of installing, for example, local collection devices to confine the water and transfer it via pipes to the drainage system in the tunnel (Grøv 2001).

Application of sprayed concrete shall not take place in areas of large water inflow until an appropriate drainage has been applied. This restriction has three purposes; to avoid water pressure being built up behind the sprayed concrete, to ensure proper bond between the rock surface and the sprayed concrete, and to avoid dilution of the concrete as water seep through.

6.4 Ground freezing

Ground freezing can be used by civil and mining engineers to stabilize the ground and control the groundwater while negotiating adverse rock mass conditions occurring in weakness zones. The frozen ground provides structural support and excludes transport of groundwater on a temporary basis until the permanent support structures are put in place. Ground freezing is today a controllable process. Modern technology and calculation programs enable accurate predictions of the progress of the freezing, time consumption, power consumption etc. The method preserves the ground water level, is environmentally friendly, and has proved to be the most cost effective method in many cases.

To lower the temperature of the ground, freeze pipes are installed, usually in drilled holes.

The traditional freezing methods are brine freezing and nitrogen freezing. Brine freezing makes use of a conventional freezing plant. The brine is chilled in the freezing plant and circulated in the freezing pipes where it absorbs heat from the ground before it is returned to the plant for re-chilling. Typical brine temperature is between –20°C and –40°C. Nitrogen freezing makes use of liquid nitrogen, which is extracted from air. The freeze pipes are fed with liquid nitrogen, –196 °C at 1 atm. that evaporates in the pipes and are released back to the atmosphere. Frozen ground gets almost impermeable when saturated and frozen. Strength is increasing with decreasing temperature below the freezing point. Design takes into account that frozen soil has visco-plastic behaviour. In the Oslofjord subsea tunnel a section with gravel-like material covering one third of the tunnel cross section at a depth of 130m below sea level was frozen stable and watertight before the tunnel was excavated through the zone by traditional drill and blast technique.

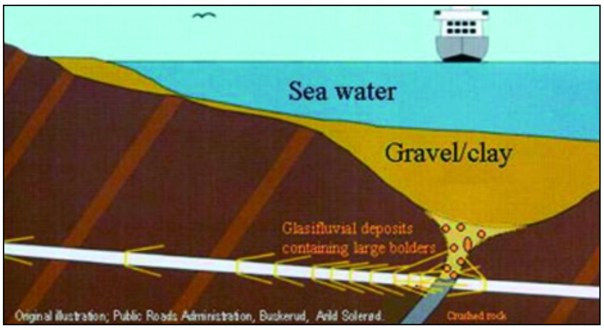

Figure 6.6. Ground freezing in the Oslofjord tunnel

Figure 6.7. Ground freezing in the Festnings tunnel

See Appendix 3 for description of the Oslofjord project and relevant articles on ground freezing.

7 DRILL AND BLAST

7.1 Introduction

In Norwegian tunnelling, application of conventional Drill and Blast technology is characterized as being highly mechanized and requiring a minimum of manpower. Sprayed concrete technology developed during the last 30 years has revolutionised the rock support and constitutes an essential part of the current drill and blast concept.

7.2 State-of-the-art excavation technology

Most civil construction contracts in Norway are unit rate contracts based on Bill of Quantity for relevant support and excavation elements. The contractor has to adapt to rapid changes, and the decision making process is delegated to the tunnel site. The state-of-the-art drill and blast technique in Norway is characterized by the use of modern equipment and highly skilled and independent workers. The shift supervisor and the shift leader together with the client's resident engineer handle the daily tunnelling matters.

In Norway today, most underground excavation takes place using wheel-bound equipment, utilizing flexible and mobile units. Very seldom, and mainly in particular excavations, rail-mounted equipment is used. The benefits of a larger tunnel cross-section allow the faster wheel-bound equipment thus reducing the construction time, is often overcoming the additional costs associated with blasting and supporting a larger cross-section.

Typical standard equipment used during tunnel construction is:

- Computerized drill jumbos. Each drill machine is capable of drilling up to 3m per minute. The computer controlled drilling performance, automatically performs alignment for the next round and surveying of the previous excavated profile.
- Mucking is performed by wheel loader and trucks. Diesel/electric loading equipment is applied for improved environment at the tunnel face.
- Support is performed by highly automated and computerized robots for sprayed concrete whilst drilling for rock bolts is mainly done by the drill jumbo. Scaling is normally performed by use of hydraulic hammers mounted on excavators, but still final scaling by hand-held bars is common.
- Rock mass grouting takes place using computerized units that can deliver grout to several grout holes simultaneously.

The work force in the tunnel has been reduced over the last years and one person will perform various tasks. Typically the following applies for a Norwegian tunnelling crew:

- The shift crew consists of a shift supervisor and maximum three tunnel workers at the face; one driller (the leader), one mechanic and one charger. In addition 1-2 workers for mechanical repair, service facilities and maintenance etc.
- The operator of the sprayed concrete robot could be one of the workers at the face, but in many cases this work is subcontracted to specialised companies.
- The wheel loader for mucking out is normally operated by one of the crew members, while the transportation is subcontracted.
- The number of trucks needed depends on the nature of the project; transport length, cross section etc. The temporary road inside the tunnel is often paved in order to increase the transport speed of the trucks and reduce interference with maintenance work that could hamper the excavation progress.
- Service works such as installation and maintenance of the ventilation duct, the water and air supply, etc, is normally performed by one of the workers from the tunnel crew and an additional operator.
- A chief mechanic and the mechanic working in the tunnel perform heavy maintenance in the tunnel or the workshop. In addition, vendors and suppliers are often contracted to perform maintenance, service and follow up of the equipment.
- The surveyor may not be a full time job, but is often integrated in the work of the QA/ QC department. He will normally also perform some of the quantity surveying.

This specialization has become possible due to the general high education level in Norway, where for instance a driller has to be fully trained in the use of computers and the surveyor has to operate advanced computer software.

7.3 State-of–the-art equipment

The most important equipment related to modern Norwegian drill and blast techniques are the drill jumbo and the robot for sprayed concrete. The need for machines that allows for a higher excavation rate has been driven by the need to reduce the construction time, mainly caused by high indirect cost, or strict demands from the owner.

The jumbo is equipped with a closed cabin for a single operator and it is fully automated. After placing the jumbo at its position close to the tunnel face, it is able to position itself and detect the correct tunnel chainage. When the chainage is detected, the computer retrieves the drill pattern from the pre-programmed tunnel design and starts to drill the complete round, without any interference by the operator. A laser based surveying equipment is installed on the jumbo and during the drilling of the next round, the surface of the previous round is checked to control that the correct cross section is achieved. On one of the onboard data screens the operator may verify if there is any need for corrections to be made. The data is transferred to the QA/ QC department for follow-up. The drill rod length has seen a tendency towards increased length e.g. 20 feet, but today the commonly used length is 18 feet.

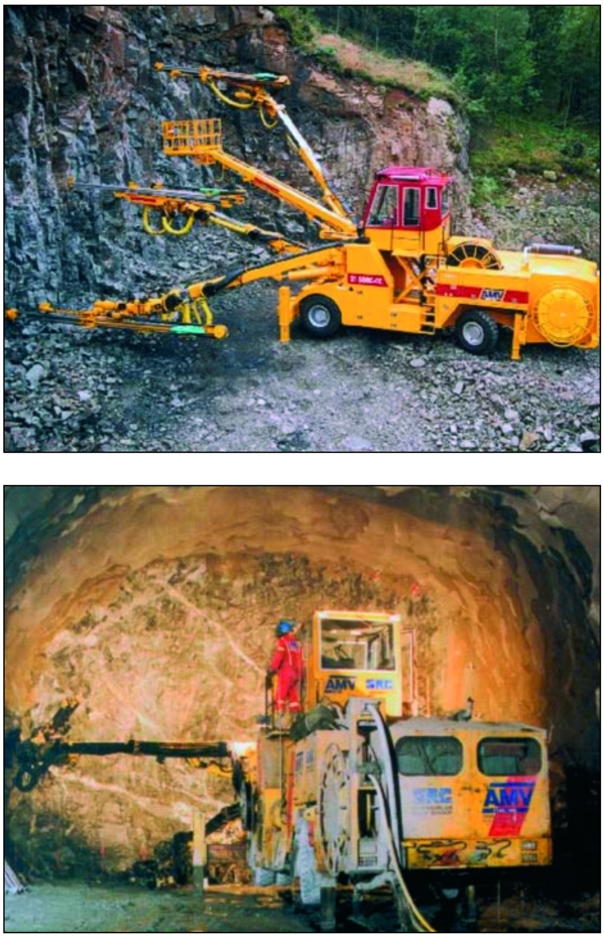

Figure 7.1. Modern drilling jumbos

One single person operates the robot for sprayed concrete and it is equipped with a high capacity concrete pump and accelerator tanks. The theoretical capacity is normally 20m3 per hour and the remote operated arm with the nozzle has a length of approximately 10m. This allows the operator to be placed in a cabin with a good view of the working area, or, if needed he can operate the robot from the tunnel floor. The robot is normally mounted on a chassis from a standard 3.5-7tons truck and will move from site to site as any truck on the road. The ready wet mix is delivered in standard transmixers with a drum capacity of 6-9m^3.

Figure 7.2. Sprayed concrete robot

7.4 Drill pattern and blasting techniques

The drill pattern in Norwegian underground openings is often based on parallel cut in combination with large diameter boreholes. The typical length of a round inhibits the use of other kinds of cuts.

Furthermore, the computerized drilling equipment ensures reduced deviation from the theoretical path of each borehole and hence increased tunnel stability. The increased length of each blast round also reduces the overbreak along the tunnel axis due to reduced inclination of each arm during drilling. A typical successful blast round in competent rock will display most of the drill holes in the tunnel contour.

Typical specifications for drilling and blasting would be:

- Tunnel excavation by 18 feet steels rods which produce a tunnel advance of approximately 4.5 – 5.0m per blast round (10-20 % reduction in length).
- Accuracy of the drilling of the contour holes as follows; collaring within 0.1m measured from theoretical drill pattern, and for alignment deviation, maximum 6% of the depth of the hole.
- The maximum contour hole distance is 0.7m and the distance to the next helper row do not exceed 0.9m.
- The charging of the contour holes and the helper row next to the contour is reduced, using specialised piped explosive or similar measures to reduce the charging, typically the charging would be 0.25 to 0.45kg/m (78% ANFO by weight).

A general requirement to Norwegian tunnels is that no rock should be allowed to protrude inside the theoretical rock contour.

Blasting techniques are developed according to local laws and regulation and the requirements from the major clients. These local conditions are not always easily exported to other markets as the availability and laws vary between countries.

Explosives developed in Norway can be divided into four main components. 1) Explosives used in the contour holes with reduced energy to reduce overbreak and secondary cracking. 2) High-energy explosives in cartridges used to initiate the blast and heavy duty blasting where specially designed blasting is required. 3) Low cost bulk explosives based on standard fertilizers or site-sensitised emulsion. 4) Finally, non-electrical detonators with a highly accurate timing system that enables a wide division of the blast round.

Today’s effect of better accuracy of the drilling process combined with improved quality on explosives and blasting agents lead to less overbreak and secondary cracking of the rock mass. Better controlled fragmentation of the blasted rock is also achieved. Further, these improvements have reduced the vibrations caused by the blasting and finally, they enable a smooth tunnel contour to be made. The contractor and the owner both benefit from these improvements by reduced construction time, better tunnel stability, reduced project costs and safer working conditions.

7.5 Typical blast round for a 100m^2 cross-section

By using modern equipment, it is assumed that around 140 holes are needed per blast round for a 100m^2 cross section, which require a total time for drilling, charging and blasting in the range of 4-5 hours. Ventilation is expected to take 0.5 hour. Before the tunnel face is safe for starting up mucking out, it is expected that 1 hour is needed to do the necessary scaling before loading and hauling can start.

Rock bolting takes approximately 1.5 hours. Application of sprayed concrete is often done at night time, or during standstill in the tunnel excavation, if stand-up time allows it.

A blast round of 4.5m with a 100m^2 cross section produces almost 500m3 (solid rock) inclusive overbreak. With a capacity of 190 to 220m^3 (solid rock) per hour the tunnel face can be cleared within 4 hours. This would require loaders such as Cat 988F, Brøyt X53 or similar equipment and trucks/lorries/semi-trailers capable of carrying 25 to 35tons.

The time needed to perform one complete blast round of 100m^2 cross-section yields a total of approximately 12 hours, giving typical advance rates of approximately 50m per week.

8 TBM TUNNELLING

8.1 Overview

A total length of about 260km of tunnels has been bored by TBMs in Norway, with diameters ranging from 2.3m to 8.5m. These tunnels serve various purposes, for example diversion and transport tunnels for hydro-electric power projects, water supply tunnels, sewer tunnels and road tunnels as well. The tunnel boring machines used in Norway up to today, have all been of the Open Hard Rock type TBM.

Figure 8.1. Open hard rock TBM

The very first full face boring in Norway took place in 1967, executed by The Norwegian Hydro Power Board by boring of a 73m long, 1.0m 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 1.8m and length up to approximately 250m.

In 1972, the 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 2.3m diameter TBM with operators from a German contractor for boring of a 4.3km long sewer tunnel.

Sulitjelma Gruber (a mining company) became the very first owner of a TBM in Scandinavia. In 1974 a 3.15m diameter Robbins TBM was bought for use on the first tunnel section (4.5km) of the main sewer system for Oslo City. Sulitjelma Gruber 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.

See Appendix 8 for further project descriptions and list of references.

8.2 Probe drilling and pre-grouting

On the Western Oslofjord Regional Sewage Project, nearly 40km of tunnels with diameters ranging from 3.0m to 3.5m, 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.

8.3 Prognosis model for TBM boring

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

In 1998 the university issued an updated and more comprehensive report on hard rock tunnel boring. This report (1-98) was prepared by Professor Amund Bruland.

8.4 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 mica schists. 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 8km long, 3.5m diameter transfer tunnel in Glommedal at Ulla Førre Hydro Electric Project. The area contained massive granite and gneiss formations with up to 210MPa unconfined compressive strength. In fact, the rock on this project was so massive, that the NTNU prognosis 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 022m diversion tunnel. The same machine has bored a total of 42km of tunnel on five projects, all in hard rock formations.

8.5 High performance TBM

In 1988 The Robbins Company introduced in cooperation with Statkraft, the High Performance TBM's, using 19 inches (483mm) cutters. The three first HP TBM's ever built, with diameters ranging from 3.5m to 5.0m, bored a total of 34km of tunnels at Trollberget job site, Svartisen Hydro Electric Power Project. (Two other, second hand standard TBM's bored another 23km of tunnels at the Svartisen Project). By this development, the cutters, the TBM machine itself 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 TBM's 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 major improvement is the wedge lock cutter housings that were introduced by Robbins at Trollberget job site, first time ever. This new style cutter housing became the industry standard, leading to improved cutter life and less cutter changes. The development of the

19 inches cutters rated 312kN/cutter was another significant step forward in hard rock tunnel boring.

8.6 TBM inclined shaft boring

In the 1980's, three pressure shafts for hydropower projects; Sildvik: 45 degree slope x 760m length x 2.53m diameter, Tjodan: 41 degree x 1,250m x 3.2m diameter, Nyset-Steggje: 45 degree x 1,370m x 3.2m diameter, were bored by using open hard rock gripper type TBM's with anti-back-slip system. The rock at Tjodan and Nyset-Steggje consisted of massive granites and granitic gneisses. Shaft boring of long pressure shafts with TBM's proved to be a very good alternative to conventional Drill & Blast technique employing the Alimak-raising.

8.7 Record Performance

In August 1992 Merkraft, a joint venture of Eeg-Henriksen Anlegg AS and AS Veidekke, completed the boring of a 10km transfer tunnel at Meråker Hydro Electric Power Project with a 3.5m 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 by boring 1 029m. Merkraft set outstanding national performance records along the way with the HP TBM working in geology ranging from hard, massive meta-gabbro, with UCS of 300MPa and greywacke and sandstone appearing as mixed face conditions to relatively soft phyllite.

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

8.8 TBM site organization and staff

Norway has long been recognized for its cost efficient tunnelling. Some of the main reasons are probably 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, workshop and cutter repairs. The crew at the face 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 5km long 20m^2 Drill & Blast tunnel and the tunnel intake construction.

8.9 Norwegian TBM contractors abroad

Norwegian contractors with TBM experience have been 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, totalling approx. 75km of tunnels, all in hard rock formations.

See also the publication issued by the Norwegian Soil and Rock Engineering Association, Publication No. 11 on TBM-tunnelling

9 GROUTING

9.1 General

The rock mass is a significant barrier in itself. However, as it is a discontinuous material, its hydraulic characteristics may vary widely, from an impervious medium to a highly conductive zone. As a consequence, for groundwater control, it is normally standard procedure in Norwegian tunnelling to include pre-grouting for the purpose of reducing the hydraulic conductivity. This procedure has developed from the early tunnelling projects in the city of Oslo, through unlined, high pressure water tunnels for hydro power projects, oil and gas storage and sub-sea rock tunnels (where the water supply is infinite!) to the current generation of urban tunnelling. This chapter will briefly list the various reasons for such groundwater control, and provide an overview to the state-of-the-art of pre-grouting technique in use in Norwegian tunnelling projects today.

The primary purpose of a pre-grouting scheme is to establish a zone around the tunnel periphery, where the hydraulic conductivity is reduced. 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 may have the effect of improving the stability situation in the grouted zone. Usually in Norwegian tunnels, grouting serves as the permanent groundwater control.

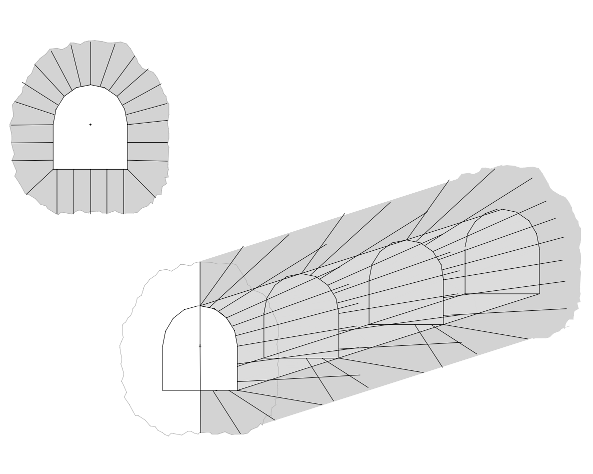

Figure 9.1. View of pre-grouted zone

9.2 Requirements to water tightness

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 associated with strict requirements to obtain a safe and dry internal environment during operation. In many cases the functional requirment for the tunnel do not allow water appearing on internal walls or tunnel crown. Further, pre-grouting shall ensure an acceptable internal environment during the execution of the tunnelling works.

* *Prevent unacceptable impact on the external,* surrounding environment. Tunnelling introduces the risk of imposing adverse impacts to the surrounding environment as the lowering of the groundwater table and/or reduction of the pore pressure in surface sediments may cause settlements of buildings and other surface structures in urban areas; disturb the balance in existing biotypes (flora and fauna, vegetation and wet areas); dry out natural lakes and ponds, springs and steams and wells.

* *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 and repository; and other industrialised disposals. Watertight tunnelling in this context is to provide a containment to prevent leakage of stored products.

The allowable amount of water inflow to the tunnel may in some cases be governed by practical limitations related to the excavation process and pumping capacity. This applies to tunnelling in remote areas without strict regulations on groundwater impacts, and in projects without particular requirements for a dry internal environment. A commonly used figure in Norwegian tunnels is a maximum inflow to the tunnel of 30 litres per minute per 100 metres of tunnel (l/min/m). This requirement is for example the criteria used for sub-sea road tunnels.

Many projects have been realised where the allowable inflow was in the range of 2-10 l/min/100m, to avoid settlements on buildings and impact on natural areas. Such strict requirements may only be successfully obtained by proper planning and thorough investigations, and systematic pre-grouting during excavation.

The primary objective is to employ methods that aim at making the tunnel tight enough for its purpose. The acceptable lekage rates along a tunnel alignment or a cavern system must be decided prior to the excavation phase and take the actual circumstances into account.

Reference is made to Publication no. 12, "Water Control in Norwegian Tunnelling" issued by the Norwegian Tunnelling Society for further details.

Figure 9.2. Grouting before the first blast round.

9.3 Grouting methods

During the last years some 10km of tunnels have been grouted with use of more than 5 000tons of grouting material each year. Almost 100% of the grouting work is systematically pre-grouting with suspensions, i.e. ordinary portland cement or micro cements.

Most of the larger tunnelling projects with strict demands of groundwater control are situated in suburban areas where regards to surroundings of the tunnel are important. Many of the most populated areas in Norway are situated where ground conditions consist of soft marine clay sediments above a hard rock. To prevent reduction of the pore pressure with subsequent settlement of the ground, pre-grouting is necessary in such areas. Other areas with strict demands on water tightness in tunnels can be areas where the preservation of the natural environment is important.

Some experience from recent tunnel projects is summarised in Table 9.1

Figure 9.3 summarizes the projects above in terms of the hydraulic conductivity, Ki, in the grouted zone given that thickness of the grouted zone is 10m. Ki has been calculated from the equation below:

$q = (2 \times \pi \times Ki \times d \times l) / \ln((r_e+t) / r_e)$,

where q = water inflow in the tunnel, Ki = hydraulic conductivity in the grouted zone, d = depth below the ground water level, l = length of the tunnel section, re = tunnel radius and t = thickness of the grouted zone.

The overall conclusions from recent tunnelling projects with respect to grouting are:

- The resulting water inflow has satisfied the contract requirements (or guidelines) under varied and partially very difficult geological conditions.
- Use of high grouting pressure even when the rock cover is small is effective/necessary to seal the rock
- Water/cement ratio of 2.0 – 0.4 has been used with 1.0 – 0.5 as "the regular" w/c-ratio

Table 9.1. Summing up of grouting in recent tunnel projects

Tunnel project	Rock types	Depth (m)	Require-ment (l/min/ 100m)	Result (l/min/ 100m)	Pumping pressure[1] (MPa)	Material[2]	Mass[3] quantity (kg/m2)	Drill hole[3] quantity (m/m2)
Baneheia	Gneiss	20-40	6-12	1.8	5 (8)	Micro cement with micro silica	14	1.1
Storhaug	Phyllite	10-15	3	1.6	3-5 (7)	Micro cement with micro silica	26	1.9
Metro in Oslo	Shales Nodular limestones Igneous intrusions	20-25	7-14	3-8	4-5 (8)	Mostly micro cement with micro silica	67	2.1
Frøya (sub-sea)	Gneisses Plutonic rocks	150 (max below sea level)	30	12	4-6	Standard cement supple-mented with micro cement	NA	NA
Lunner	Syenite Vulcanite Sandstone Conglomer-ate	10-300	10-20	8	5-7	Standard cement and micro cement	36-86	1.2-1.3
Bragernes	Rhombus phorphyry Basalt	10-150	10-30	10	4-7	Standard cement with micro silica	42	0.6
Hagan	Syenite Hornfels	6-60	5-10	4-19	8-10	Mostly standard cement with micro silica	34-103	1.3

1 Typical value(s) and maximum value in brackets.
2 In some projects also Thermax has been used. The purpose of this material is to reduce setting time. Plasticizer is used in all projects. Standard cement is ordinary portland cement.
3 m2 tunnel is the entire tunnel circumference multiplied with full tunnel length

- Adding micro silica to standard cement and micro cement is a success
- Grout intake vary between 14 – 103kg/m^2 (average values for whole tunnels or longer tunnel sections)
- Grouting holes vary between 0.57 – 2.1m/m2 (average values for whole tunnels or longer tunnel sections)
- The variation in quantities is caused mainly by geological conditions, but also by contract requirements and by variation in grouting methods
- Use of materials that allow for regulation of the setting time has proved useful
- Better planning the has resulted in better control of the groundwater
- Increased drilling capacity has been developed
- Automatic grouting stations with high capacity have come into use
- The necessary water tightness has often been reached in one round of grouting at each tunnel face.

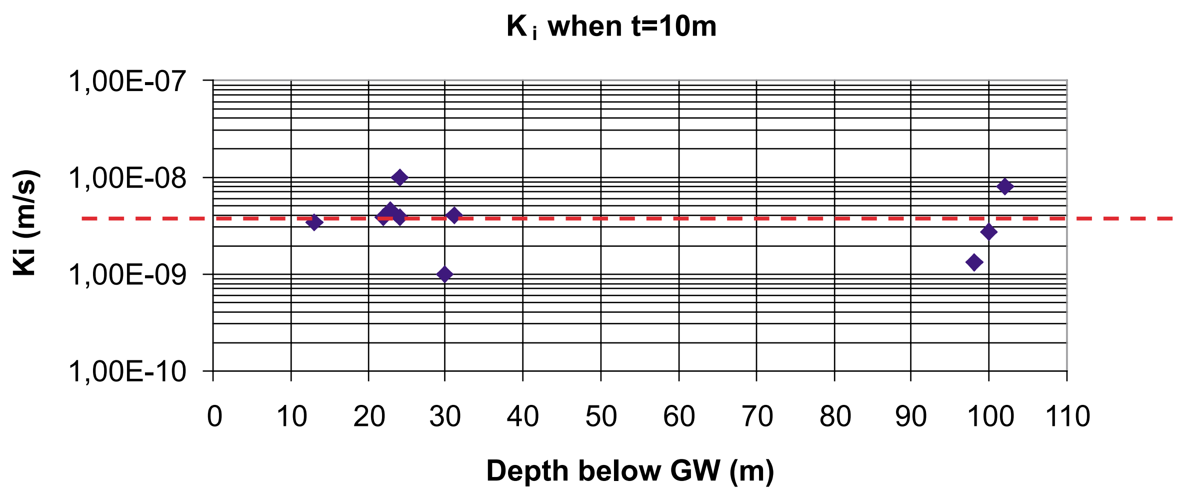

Figure 9.3. Hydraulic conductivity in the grouted zone from recent tunnel projects.

10 TUNNEL LINING FOR WATER AND FROST PROTECTION

The Norwegian tunnelling practice implies a maximum utilisation of the self-standing capacity of the rock mass. This is another important aspect of the tunnelling concept together with that of a drained structure.

As far as the cost aspect is concerned, this approach is clearly favourable. However, the drained support system imposed water and ice problems to become a major uncertainty as regards safety and maintenance in road tunnels. In the 1960's the only measure at hand and tested in Norway was the membrane insulated cast-in-place concrete lining for water and frost protection, which was mainly applied locally. The strict demands on construction and cost factor of this system led to an extensive development and testing of alternative measures to produce cost effective, safe and durable solutions to complement the rock support (Broch et al 2002).

An important element of Norwegian tunnelling is that the stability of the underground structure is taken care of by rock bolts and sprayed concrete. Thus, the installation of cast-in place concrete lining for water and frost protection has been considered as superfluous and basically an unnecessary, additional cost-accelerating element.

Since the 1960's a large number of methods and systems for water and frost protection has been tested including both light weight solutions, PE-foam (polyethylene) as well as various concrete solutions, and some combined solutions. Thorough investigation of the performance record of these systems yielded that some of them did not fulfil the requirements on long-term durability due to the complex loads occurring in road tunnels. Various water and frost protection systems have been installed in railway tunnels too, however, only a few have proved to withstand the complex loads caused by air pressure and suction loads from the traffic.

As a result, the concept of self-standing, independent systems penetrated the tunnelling industry, and one such system was the pre-cast concrete segment lining which was introduced for use in tunnels with high traffic density in the 1990's. Inner lining of pre-cast segments could be applied as insulated as well as not insulated, depending on the actual frost load in the area of concern.The inner lining does not allow water to pass the lining and enter the carriage-way, consequently, the use of water tight membranes has been enforced. Various modifications on the pre-cast concrete segment lining were tested, including sandwich solutions, lightweight concrete segments etc. to provide an inner lining which is a stable installation, water-tight, durable and frost insulated. Figure 10.1 shows a typical cross-section of an inner lining of pre-cast concrete segments.

This pre-cast concrete segment lining is a free standing lining. It must be emphasised that the segment lining do not carry any load imposed by the rock mass. The stability of the rock mass is solely taken care of by the rock mass itself supplemented by appropriate rock support. The pre-cast concrete segment lining is fixed to the surrounding rock mass by fixation bolts anchored in stable rock.

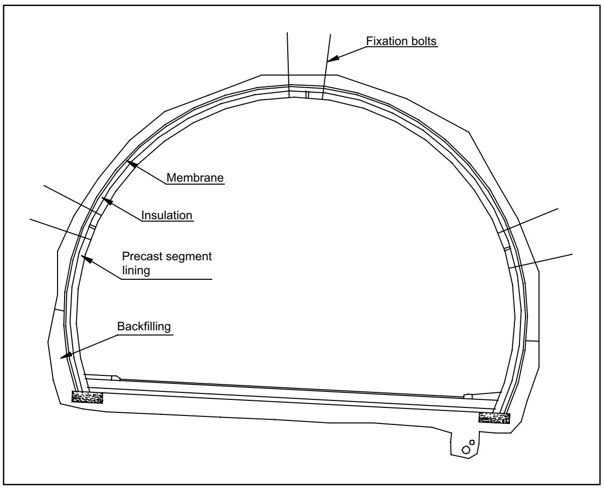

Figure 10.1. Pre-cast concrete segment lining

In other underground openings, with less critical loading situations, other concepts have been found applicable. The Giertsen system is one such solution consisting of steel frames and PVC sheet. This system has been widely used in underground facilities such as storage facilities, defence facilities, etc. See example in Figure 10.2.

Figure 10.2. The Giertsen system installed in a defence facility

11 COST-EFFECTIVE TUNNELLING

Norwegian management style is democratic and informal. This also goes for the tunnelling industry, where responsibility and authority are both delegated to the person best positioned to handle the situation. Decision-making is encouraged at all levels in the project hierarchy, and an erroneous decision will not automatically impose a punishment for the decision maker. Therefore, engaged professionals dare to try out new techniques to overcome the sort of unplanned challenging situations that are normally encountered in tunnelling. This leads to quick decision making, often at the tunnel face itself, essential in unstable rock conditions as well as where significant water problems have been encountered. This approach has been experienced as the fastest empirical learning method, leading to well-trained, self confident and motivated tunnelling teams. The optimisation of working methods has lead to reduction of the number of people working at the tunnel face, and they have now become competent and skilled in more than one speciality.

In Norway the basic crew employed at a tunnel face for a normal sized tunnel consists of three skilled persons, covering all production activities. This includes: mark up, drilling, charging, blasting, scaling and temporary support. For single face sites the basic crew is also involved, fully or partly, in operations like mucking and permanent support. If the site opens for alternating faces, permanent support and mucking and transport is solved by separate crews or these operations are sub-contracted. It may be extra manning for installation and maintenance of utilities. The different working operations are run continuously, with flexible stops to fit the effective production, and not according to fixed change hours between operations. This gives tidy, well-organised and therefore safe working conditions.

Combined with modern equipment, this approach ensures the best possible productivity under any given geological condition. In Norway, hard and competent rock is common, but mixed with all sorts of weakness zones, high-pressure water, extreme rock stresses and other challenges. These circumstances have provided useful experience for handling the tunnelling challenges met when working in other countries, with different and often younger geology. Under such conditions, it is generally necessary to tie up the production to a limited number of pre-defined procedures for excavation and support. The temporary rock support is then determined just prior to blasting of each round, in order to avoid delays waiting for decision – making, as time is often critical at an unstable tunnel face. A close follow-up from engineering geologists is necessary to develop and adjust procedures as necessary.

Every project undertaken in foreign markets is physically unique and also by way of organisation. The optimum mixing of local workforce and expatriates has therefore to be founded on an empirical basis. It is also a matter of on the job training for all parties, leading to a continuous optimisation and cost cutting process both at direct production and management levels. A flexible model is therefore the answer to this challenge, via an open-minded and respectful attitude building on combined resources and competence.

It is important to keep, or get back to, the problem solving at site instead of in the courtrooms. Norwegian tunnelling has always been associated with the capability to solve problems at the site, due to highly qualified professionals and tunnellers. An additional tool to assist in achieving this may be the use of advisory ‘reference groups’.

Another key-issue is the co-operation by the parties at site. 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.

Another important aspect that enables a cost effective tunnelling concept in Norway is related to the methodology’s adaptability to the actual ground conditions, and the acknowledgment of the self-standing capability of the rock mass. By careful following-up of the encountered ground conditions by mapping and classification of the rock mass, support measures that are best fit to the ground conditions are used. This involves rock support by means of rock bolts and sprayed concrete as the prime means, spiling, support ribs and cast-in-place concrete may then only be used in adverse rock mass conditions.

Installation of permanent rock support takes place as close to the tunnel face as practically possible and advisable for the utilisation of the resources at the site. Applying permanent support measures as temporary support enable the latter to be integrated and converted to permanents rock support. This allows a significant reduction in construction time, as well as cost savings.

A careful consideration of the most cost consuming elements in the tunnelling process has identified such elements as cast-in-place concrete lining and water handling as elements to be focused. This leads to using rock bolts and sprayed concrete in stead of cast-in-place concrete wherever possible as well as developing rock mass pre-grouting schemes that cuts the water leakage effectively, both in time and materials.

12 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 addressed 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, who coined the term NoTCoS – the Norwegian Tunnelling Contract System. In Figure12.1 it is illustrated how this risk allocation produces the lowest cost possible in average for a number of projects.
Some main characteristics of Unit Price Contracts:
The typical unit price contract in Norway is characterised by the following:

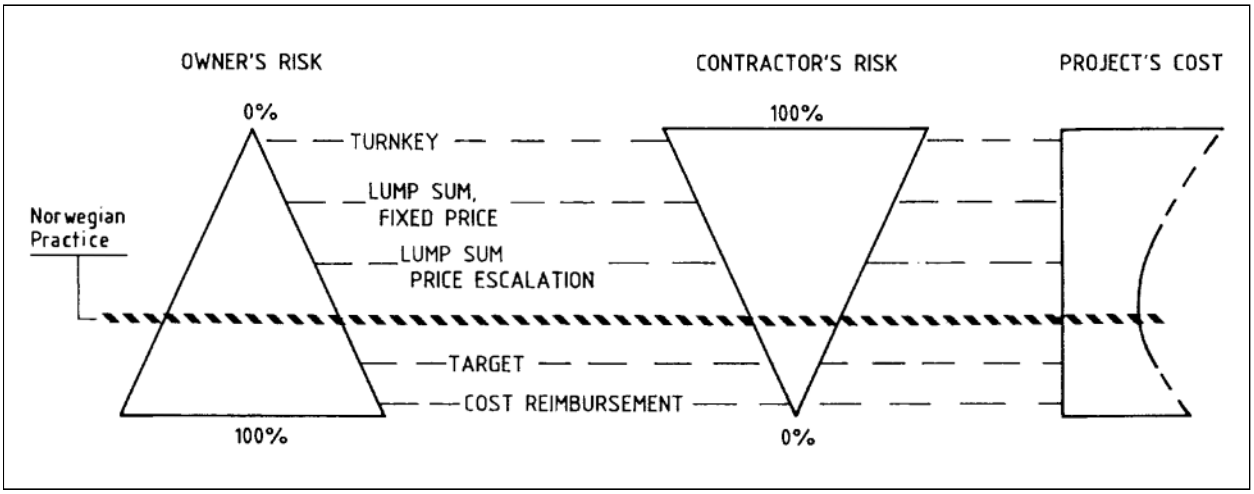

Figure 12.1. Risk allocation principles (Kleivan)

- 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.
- 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.

13 HEALTH AND SAFETY IN NORWEGIAN TUNNELLING

13.1 Background

Norwegian tunnelling is heavily mechanised, with extensive use of self-moving units, not only for drilling of blast, probe and grout holes, but also for charging, grouting, shotcreting, as well as for muck loading and transport. This means that a lot of the heavy manual work that used to characterise tunnelling is gone.

The typical Norwegian tunnel worker is multi-skilled, highly motivated and well paid. The face crews are organised as autonomous work groups, lead by qualified shift bosses. The crew even negotiates with the contractor their own bonus connected to production rate, for excavation as well as the other work activities. The foremen's tasks become one more of support to the face crew to help that all function efficiently, rather than traditional supervision.

This type of organisation is highly geared towards production. The experience the last years show that this actually can be combined with high emphasis on safety as well.

13.2 Laws and regulations

13.2.1 Work Environment law in 1977

The Work Environment law of 1977 set some of the modern principles of safety into action. The responsibility was no longer only on the individual worker to behave in a safe manner, the employer now got the responsibility to arrange for work to be done in a safe manner. If the employer allowed or ignored unsafe practises, he could be held responsible. Everyone involved, with the possibility to observe and influence how the work was carried out had a responsibility to act (at least warn) if he found it necessary, independent on position.

Naturally, the focus was mostly on safety, in the sense of preventing accidents, related to items that could easily be observed. In overall, the contractors had the heaviest responsibility. Especially the number of fatal accidents started to decline. The other aspects of the work environment, i.e. the chemical and physical factors that could negatively influence the long-term health got less attention.

13.2.2 Owners' responsibility regulation of 1995

This regulation puts the overall responsibility to the owner, basically to provide the framework for healthy and safe working conditions. The owners have to include requirements to a Health & Safety Plan in bidding documents and to ensure that it is implemented.

As a result, the quality of the Health & Safety Plan, and the safety record became important criteria's in the qualification process for the contractors. Out of necessity, the contractors put more effort into developing safer systems, but also out of realising that doing the work right the first time and in a safe manner, is indeed very economical.

13.3 Challenges to the modern H&S work

It is clear that the high mechanisation relieves previous burdens of heavy manual work. To take a chance once in a while is no longer tolerated. The overall safety has improved, and accident rates reduced from being above the industry average, is now on a low level compared to before. The construction division of one of the major contractors, reported in 2002 an overall rate of H = 2.5 (H = number of accidents causing absence from work per million work hours). It is not unusual anymore that even large projects may achieve a rate of H = 0.

It is of course a challenge when the target has been moved to 'zero accidents'. However, it has proven equally, if not more, difficult to improve the health aspects of the work environment in a similar manner. The heavily mechanised methods introduce other hazards to the workers, such as body vibration, draft, air pollution from blasting fumes and diesel exhaust, risk of exposure to toxic chemicals, etc.

13.4 Development projects

Accordingly, the last few years have seen a series of development projects, with broad participation from the main owners (including public owners), the contractors and manufacturers of different kind of equipment. By extensive full scale testing and observations at the tunnel face, one is able to better understand the involved factors, and how the negative exposures or hazards can be reduced or even eliminated. Invariably, this knowledge has lead to a combined effect; the work gets both safer and more effective.

Several of the projects have been initiated or supported by the Norwegian Tunnelling Society and detailed descriptions can be found in Publication No 13 in this series.

Below is listed some of the projects that have been or are under development lately:

- Ventilation methods: improved ventilation regulated by sensors, giving sufficient air volume in critical periods, saving power in less critical.
- Automatic charging: by emulsion explosives, facilitating reduced cycle time, even if workers are no longer allowed in front of the drilling jumbo during drilling.
- Diesel underground: the implementation by economic incentives and regulations of higher quality diesel, possible use of early installed permanent power where applicable.
- Sprayed concrete: work environment by using alkali free accelerators, investigation of exposure to dust, etc, use of protective equipment, improved operator's cabins.
- 'Rear zone ('Behind the face') activities: how to create acceptable work environment for activities further out in the tunnel during excavation, for road-works, installation etc.
- Electronic access monitoring: automatic monitoring of personnel traffic in and out of the tunnel, allowing full overview over how many are present in which zones, readily available in case of emergencies.
- Safety container: development of an industry standard for safety containers, to ensure good functionality, and relevant requirements in bidding documents.