SUSTAINABLE UNDERGROUND CONCEPTS

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NORWEGIAN TUNNELLING SOCIETY

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SUSTAINABLE UNDERGROUND CONCEPTS

Norway is a mountainous country. Topographical features along the western coastline are long fjords cutting into steep and high mountains. The south-eastern and middle part of the country takes on smoother forms; still dominated by mountains and rocky underground. The topography hence creates necessity, opportunities and challenges for rock engineering in the development of the infrastructure of the country. Commended virtues in the engineering sector are competence, ability to find new solutions and conscious approach with regard to environment and costs.

The present publication, number 15 in the English language series from the Norwegian Tunnelling Society NFF, has – as always – the intention of sharing with our colleagues and friends internationally the latest news and experience gained in the use of the underground; this time with focus on sustainable concepts.

Tunnels for roads, railways and hydroelectric power still constitute the dominating part of underground construction. However, underground space for the use of the general public, storage facilities for a variety of products including gas and oil products, treatment plants for water and waste, are all becoming increasingly important these days. Security, safety and strategic aspects are likely to enhance this trend in the future.

The methods, technologies and achievements described in this publication are based on recent or ongoing tunnelling and underground projects. In addition, this publication is also presenting an extract of knowledge gained by executing the development project called "Tunnels serving the society" initiated by national authorities and supported by a number of private sector companies.

NFF expresses thanks to the contributors of this publication. Without their efforts the distribution of Norwegian tunnelling experience would indeed not have been possible.

Oslo, January 2006

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FROM FIRE-SETTING TO FULL-FACE TUNNEL BORING MACHINES

Sylvi Sørensen Blunt
Aslak Ravlo

ABSTRACT
The development of blasting, rock engineering and underground construction is part of history. To remind ourselves of the past and to attract young people to the sector is one of the many duties of the profession. One contribution may be the new underground museum in Lillehammer. Members of NFF played an active role in establishing the facilities. The first stage is now open. Below paper gives an introduction to NFSM (N for Norsk=Norwegian, F for Fjell= Mountain- In the context – Rock, S for Sprengning= Blasting using explosives, M for Museum)

Railbound loader for small cross section

The history of Norwegian rock blasting from the 17th century and up to modern times is on display at the Norwegian Rock Blasting Museum (NFSM) at Hunderfossen in the northern part of Lillehammer 200 km north of Oslo.

Some fifteen years ago a small group of NFF members had the idea that a museum designated to rock blasting and tunnelling ought to be established. They talked, argued and convinced member companies, rough plans and informal agreements were made, a hasty start of the tunnel excavation followed.

Enthusiasm is necessary, more is needed. A separate entity was set up, plans were revised in cooperation with museum professionals, resources for a stepwise construction were made available by member companies, in first hand contractors that handled the tunnels, installed necessary rock support and built the ancillary facilities. It took years. Supporters with political influence, not at least from the labour union and the roads administration were able to release from Government the necessary funds for finalising the first stage exhibition. To update and run a museum requires funds and professionals. The facilities are handed over to the state owned Norwegian Roads Museum (same place); competent development and operation thus safeguarded.

Generations of hard work and large investments in rock engineering projects have put Norway in a forefront position internationally when it comes to rock blasting expertise and the construction of tunnels and caverns. This heritage is on display in a semicircular 240-meter-long tunnel.

Much of what is on display at the museum dates back to the last century. 1950s, a period of reconstruction with new techniques and improved technology started a long period of development that changed society. The countless number of rock engineering projects in Norway continuously resulted in "number ones": tunnel lengths, speed of advance, hard rock boring, support methods, deepest and longest subsea tunnels, high pressure shafts, lake taps, large caverns and widest spans.

The reasons behind these achievements must be sought in needs and enthusiasm combined with the experience from centennials of mining and the construction of railways and roads in a landscape with challenging topographical obstacles. Pioneering work, executed by devoted labour force and staff develops expertise, enhances technology and attracts high standard newcomers eager to perform.

Visitors to the museum can now make an excursion that starts with an insight into the smithy and other tunnel-related workshops. One is then guided into a tunnel where projects and tunnelling activities are

A group of tunnellers inspecting the museum.

demonstrated with the help of sound and TV screens presenting new and old film footage. Additionally, the tunnel allows for authentic display of equipment and machinery. From the early fire-setting methods in the Kongsberg mines (started 1623), through manual shaft raising up to tunnel boring technique.

On display, one will find a tipping wagon, a loader, a boring machine, a miner's lamps and several large units that clearly demonstrate capacity.

The exhibition has displays featuring Bergensbanen, the railway line to Bergen that was opened in 1909, had 184 tunnels, the longest called Gravhalsen of 5311 meter. The Bergen line project had a cost equal to the entire national budget of the time.

From the hydropower sector one will see exhibits of the Glomfjord and the Tokke power stations that were built in the 1950s. Fully hydraulic tunnel rigs introduced in the 1970s increased drilling speed and improved working conditions. One of those rigs can be seen.

The open-air part of the museum displays machinery for open pit mining. A 160 ton dumper-truck donated by Sydvaranger Gruber (iron ore mine in the northern-east end of the country close to the Russian border) is already on display.

The museum of to-day represents a first stage development only. Existing ideas for expansion, both inside and outdoor, requires further stages. Plans include the collection of additional units of heavy machinery. These are available in remote mines, funds for dismantling, transports and installation are not yet sufficiently available. Interesting documents and delicate smaller items needs space in new facilities yet to be constructed.

The Rock Blasting Museum is, as stated above, now operated as an integral part of the Norwegian Road Museum. The Road Museum with a main exhibition building also features a 75-acre open-air museum with historical facilities for the travellers like rest rooms and accommodation, an equipment park, a smithy and more. A number of machines, vehicles and various forging techniques are also demonstrated in an old road station.

During the summer season, visitors may be transported through the museum's park by horse and buggy or on a vintage bus. The museum offers special programmes for school groups, holiday-makers and companies that wish to give their participants, employees and customers a unique experience. The Road Museum is also working for safer road traffic being an adventure centre for the Roads Administration aiming for the "zero-vision" (zero fatalities in traffic). During the summer period your ability as driver may be put to test.

The museum is normally open in the summer period May-September, groups are admitted off-season by appointment. Information will also be found on www.vegmuseum.no

The host municipality Lillehammer is certainly a place not to be forgotten. A friendly town, good climate, interesting places, an El Dorado for outdoor people. From the international airport north of Oslo a railway ride of approximately one hour will take you there.

A truck from mining operation.


DYNO
Dyno Nobel

ENERGY, OIL AND GAS – WHY GO UNDERGROUND?

The first steps of Blaafalli hydropower plant in south-western Norway was constructed during 1954 – 57, with water tunnel systems under ground, penstock and hydropower station above ground. Today, the plant is being upgraded. All parts will be placed underground with the combined effects of increased power production, reduced maintenance cost, environmental and other benefits. The new power plant will be in operation from end of 2006.

From underground reservoirs to underground storage caverns.
Underground caverns for oil and gas storage represent a feasible, safe and well protected alternative to the above ground tank farms. Like at the picture below, the new cavern at Mongstad prepared for low temperature gas storage.

Old penstock at Blaafalli Hydropower Plant

Rock cavern for storage of low temperature gas at Mongstad refinery, north of Bergen

WHY DID THE HYDROPOWER INDUSTRY GO UNDERGROUND?

Einar Broch

ABSTRACT

In Norway more than 99% of a total average annual production of 125 TWh of electric energy is generated from hydropower. 4000 km of tunnels has been excavated for this purpose, and the country has 200 underground powerhouses. Special design concepts have over the years been developed related to this massive use of the underground. One such speciality is the unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. This paper will give a brief introduction to these and other solutions, and explain the advantages of utilizing the underground to its fullest possible extent for hydropower projects.

INTRODUCTION

Topographical and geological conditions in Norway are favourable for the development of hydroelectric energy. The rocks are of Precambrian and Paleozoic age, and although there is a wide variety of rock types, highly metamorphic rocks predominate.

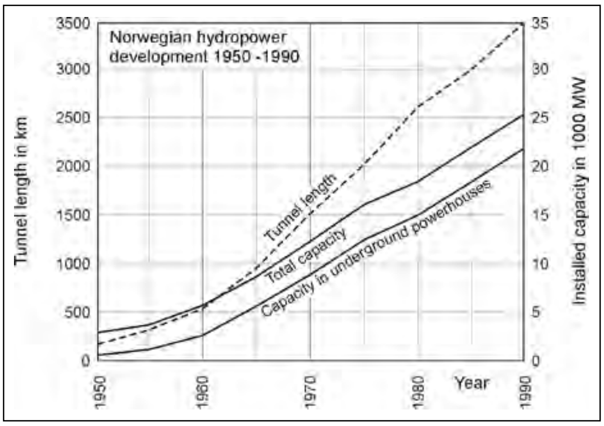

Figure 1. The development of Norwegian hydroelectric power capacity and accumulated length of tunnels excavated for the period 1950 – 1990

More than 99% of a total average annual production of 125 TWh of electric energy is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant. In fact, of the world's 500 underground powerhouses almost one-half, i.e. 200, are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it today has more than 4000 km of tunnels. During the period 1960 - 90 an average of 100 km of tunnels was excavated every year.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today.

Also, special techniques and design concepts have over the years been developed by the hydropower industry. One such Norwegian speciality is the unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. These specialities are described in further detail in Broch (2002).

Most of the Norwegian hydropower tunnels have only 2 - 4% concrete or sprayed concrete lining. Only in a few cases has it been necessary to increase this to 40 - 60%. The low percentage of lining is due not only to favourable tunnelling conditions. It is first and foremost the consequence of a support philosophy which accepts some falling rocks during the operation period of a water tunnel. A reasonable number of rock fragments spread out along the headrace or tailrace tunnel will not disturb the operation of the hydro power station as long as a rock trap is located at the downstream end of the headrace tunnel. Serious collapses or local blockages of the tunnels must, of course, be prevented by local use of heavy support or concrete lining when needed.

EARLY REASONS FOR GOING UNDERGROUND.

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. At that time the traditional design was to bring the water down from the intake reservoir or the downstream end of the headrace tunnel to the powerhouse through a steel penstock. Both the penstock and the powerhouse were above ground structures as shown in Figure 2.

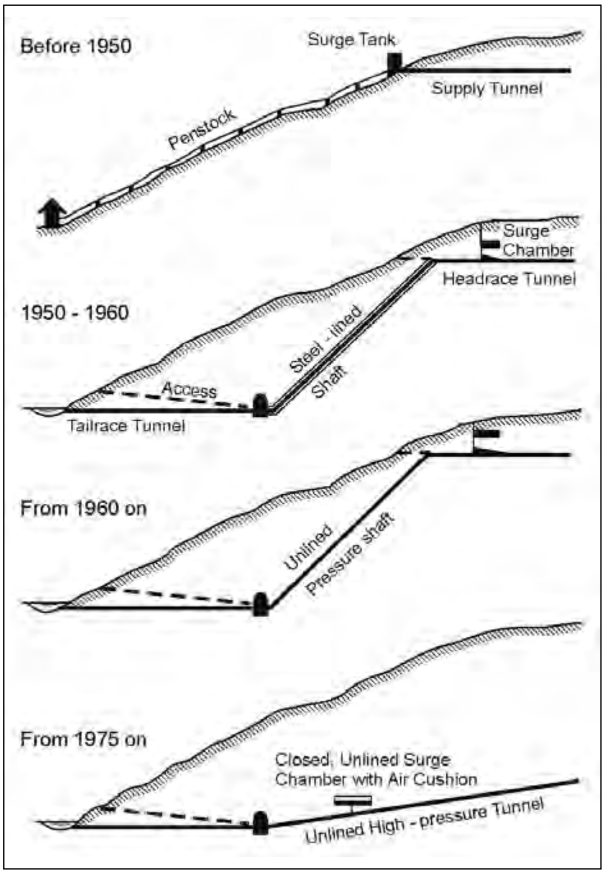

Figure 2. The development of the general lay-out of hydroelectric plants in Norway

With the lack of steel for a penstock, the obvious alternative was to try to bring the water as close to the powerhouse as possible through a tunnel or a shaft. As a result, four Norwegian hydropower stations with unlined pressure shafts were put into operation during the years 1919-21. The water heads varied from 72 to 152 m. One (Skar) was a complete failure due to too low overburden of rock, only 22 m rock cover where the water head was 116 m. One (Toklev) has operated without any problems ever since it was first commissioned. The Svelgen hydropower station, with a water head of 152 m, had some minor leakage during the first filling. A short section of the shaft was lined with concrete and grouted with cement. Since then the shaft has operated without problems. The fourth station, Herlandsfoss, had a 175 m long, horizontal high-pressure tunnel with water head of 136 m. Leakage occurred in an area of low overburden, 35 - 40 m, and the short penstock had to be extended through the whole tunnel to the foot of the inclined pressure shaft. The shaft itself had no leakage. Further details in Broch (1982).

Although three out of four pressure shafts constructed around 1920 were operating successfully after some initial problems had been solved, it took almost 40 years for the record of 152 m of water head in unlined rock at Svelgen to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Until around 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants as demonstrated in Figure 2.

DEVELOPMENT AFTER 1945

Underground powerhouses.

In a few early cases, underground location of a powerhouse was chosen as the only possible option (Bjørkåsen,1921). During and after the Second World War, the underground was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent lowering of the costs, underground location came to be the most economic solution. This also tied in with the development of concrete lined, and later unlined pressure shafts and tunnels, to give the designer a freedom of layout quite independent of the surface topography.

Except for small and mini-hydropower stations, underground location of the powerhouse is now chosen whenever sufficient rock cover is available. Frequently the overall project layout requires the powerhouse to be placed under very deep rock cover where rock stresses may be substantial. This requires an investigation of the stress condition in advance for finding the most favourable orientation of the cavern and the optimum location, orientation and shape of ancillary tunnels and caverns.

In the early powerhouse caverns the rock support of the ceiling was limited to rock bolts. To safeguard against rockfalls, a 25 – 30 cm thick arch of in-situ concrete was placed some distance below the ceiling, see Figure 3.

In poor rock masses, the ceiling was often reinforced by an arch of concrete in contact with the rock. In the latter case a light arch ceiling would be suspended below the roof arch to improve appearance and to intercept any water leakage. The present-day solution prescribes systematic bolting of the rock ceiling immediately after excavation of the top heading, followed by fibre-reinforced sprayed concrete from 70 to 150 mm in thickness, according to rock quality. It is also common practice to install deeply bolted girders for the powerhouse crane right after the excavation of the top heading, see Figure 4.

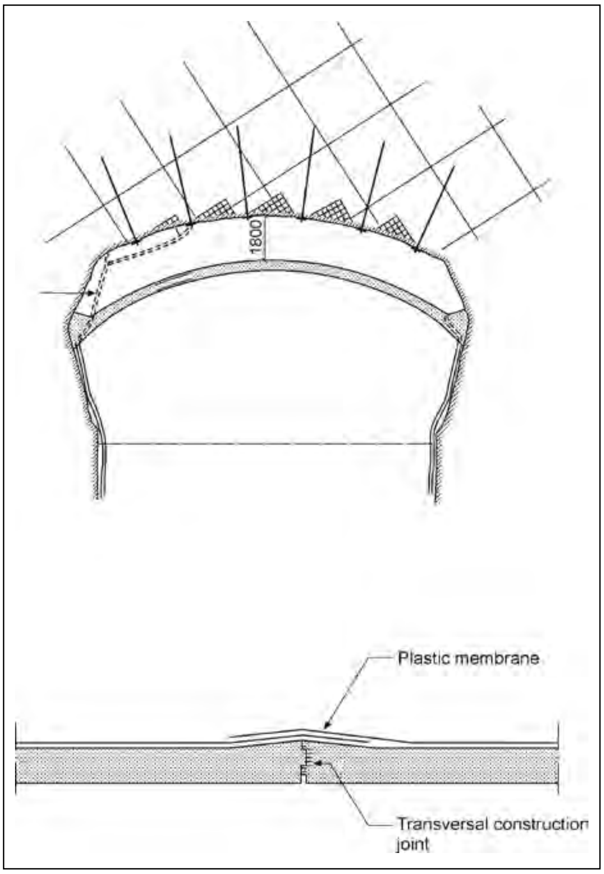

Figure 3. Typical design for a free span concrete arch.

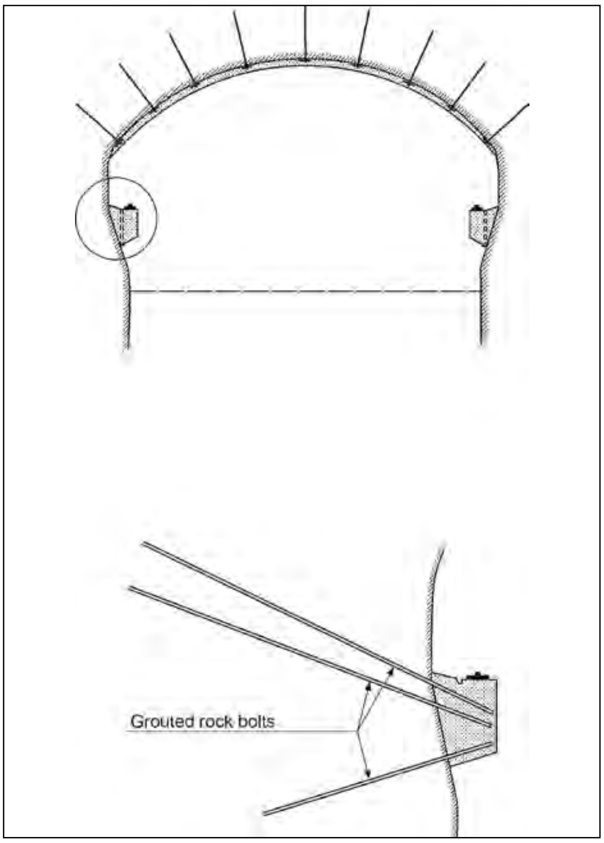

Figure 4. Steelfibre reinforced sprayed concrete arch and rock bolt supported crane beam.

In this way the crane can be installed and be available early for concrete work and installation of spiral cases etc. without having to wait for concrete structures to be built up from the floor level. If needed, the crane girders may be provided additional support later on by columns, cast in place before handling the heaviest installation loads.

In the most common layout the transformer hall is located parallel to the main hall, at a sufficient distance for rock support, and a transport tunnel is utilised as tailwater surge chamber, but other solutions have also been used, see Figure 5.

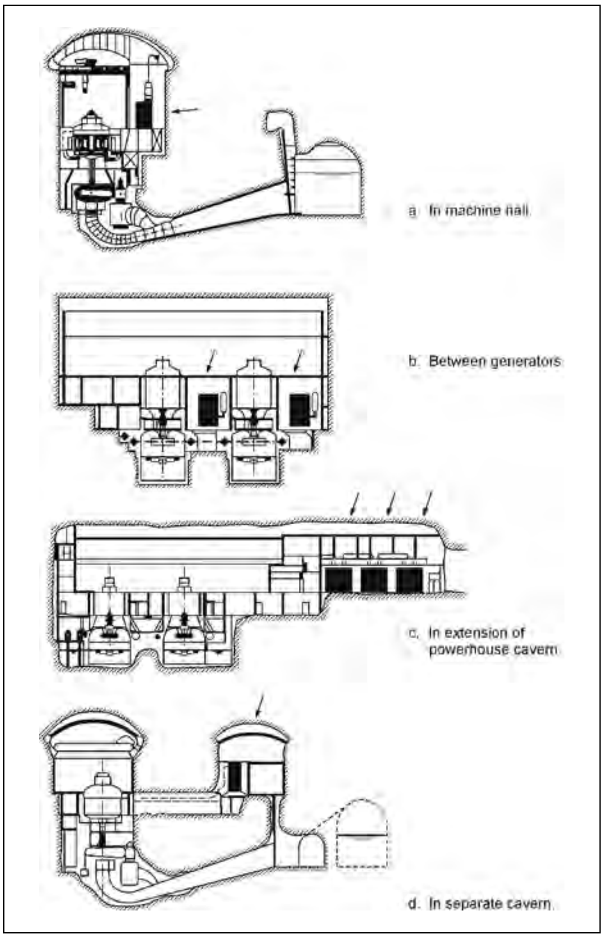

Figure 5. Common transformer locations

Unlined high pressure tunnels and shafts.

When the hydropower industry for safety reasons went underground in the early 1950’s, they brought the steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. During the period 1950-65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed. The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 6 shows, new unlined shafts were constructed in the early 1960’s and since 1965 unlined pressure shafts have been the conventional solution. Today more than 80 unlined high-pressure shafts or tunnels with water heads above 150 m are successfully operating in Norway, the highest head being almost 1000 m. Figure 6 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

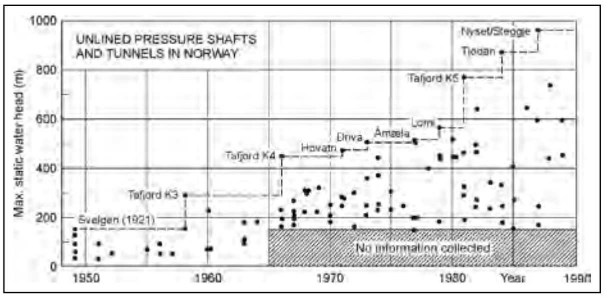

Figure 6. The development of unlined pressure shafts and tunnels in Norway

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is described in detail by Rathe (1975). The bottom sketch in Figure 2 shows how the new design influences the general layout of a hydropower plant. The steeply inclined pressure shaft, normally at 45o, is replaced by a slightly inclined tunnel, 1:10 - 1:15.

Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. This compressed air acts as a cushion to reduce the water hammer effect on the hydraulic machinery and the waterways, and also ensures the stability of the hydraulic system.

In the years before 1970 different “rule of thumbs” were used for the planning and design of unlined pressure shafts in Norway. With new and stronger computers a new design tool was taken into use in 1971-72. This, as well as the “rule of thumbs”, are described in detail in Broch (1982). It is based on the use of computerised Finite Element Models (FEM) and the concept that nowhere along an unlined pressure shaft or tunnel should the internal water pressure exceed the minor principal stress in the surrounding rock mass.

Very briefly, the FEM models are based on plain strain analysis. Horizontal stresses (tectonic plus gravitational) increasing linearly with depth, are applied. Bending forces in the model are avoided by making the valley small in relation to the whole model. If required, clay gouges (crushed zones containing clay) may be introduced.

Whichever method is chosen, a careful evaluation of the topography in the vicinity of the pressure tunnel or shaft is necessary. This is particularly important in non-glaciated, mountainous areas, where streams and creeks have eroded deep and irregular gullies and ravines in the valley sides. The remaining ridges, or so-called noses, between such deep ravines will, to a large extent, be stress relieved. They should therefore be neglected when the necessary overburden for unlined pressure shafts or tunnels is measured. This does not mean that pressure tunnels should not be running under ridges or noses - only that the extra overburden this may give should not be accounted for in the design, unless the stress field is verified through in-situ measurements, see Broch (1984) for further details.

As the permeability of the rock itself normally is negligible, it is the jointing and the faulting of the rock mass, and in particular the type and amount of joint infilling material, that is of importance when an area is being evaluated. Calcite is easily dissolved by cold, acid water, and gouge material like silt and swelling clay are easily eroded. Crossing crushed zones or faults containing these materials should preferably be avoided. If this is not possible, a careful sealing and grouting should be carried out. The grouting is the more important the closer leaking joints are to the powerhouse and access tunnels and the more their directions point towards these. The same is also valid for zones or layers of porous rock or rock that is heavily jointed or broken. A careful mapping of all types of discontinuities in the rock mass is therefore an important part of the planning and design of unlined pressure shafts and tunnels

Hydraulic jacking tests are routinely carried out for unlined high-pressure shafts and tunnels. Such tests are particularly important in rock masses where the general knowledge of the stress situation is not well known or difficult to interpret based on the topographical conditions alone. The tests are normally carried out during the construction of the access tunnel to the powerhouse at the point just before the tunnel is planned to branch off to other parts of the plant, like for instance to the tailwater tunnel or to the tunnel to the bottom part of the pressure shaft.

To make sure that all possible joint sets are tested, holes are normally drilled in three different directions. By the use of Finite Element Models the rock stress situation in the testing area as well as at the bottom of the unlined shaft are estimated. At this stage the relative values of the stresses at the two points are more important than the actual values. During the testing the water pressure in the holes is raised to a level which is 20 to 30 % higher than the water head just upstream of the steel-lining, accounting for the reduced stress level at the testing point. There is no need to carry out a complete hydraulic fracturing test. The crucial question is whether or not the water pressure in the unlined part of the shaft or tunnel is able to open or jack the already existing joints. Hence, it is important making sure that all possible joint sets are tested.

UNDERGROUND HYDROPOWER PLANTS WITH UNLINED WATERWAYS.

To demonstrate the design approach an example of an underground hydropower plant will be shown and briefly described. Figure 7 shows the simplified plan and cross section of a small hydropower plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200 - 600 m.
The figure is to some extent self-explanatory. A critical

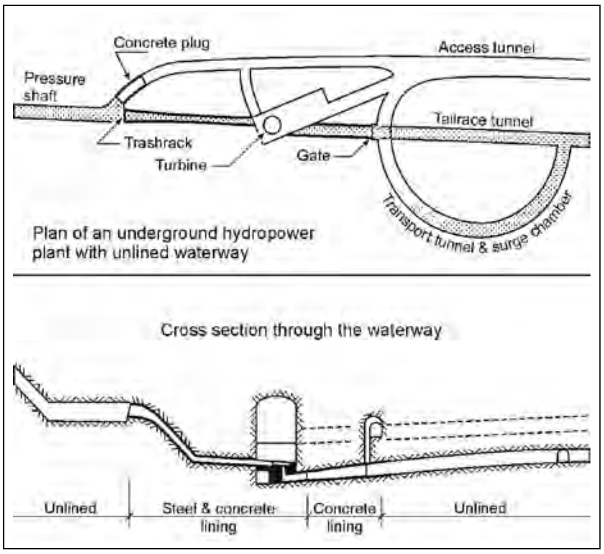

Figure 7. Plan and cross section of an underground hydropower plant with unlined waterways.

point for the location of the powerhouse will normally be where the unlined pressure shaft ends and the steel lining starts. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular
the character and orientation of joints and fissures. Steel lengths in the range of 30-80 m are fairly common
The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10 - 40 m, depending on the water head and geological conditions. As a rough rule of thumb the length of the concrete plug is made 4% of the water head on the plug, which theoretically gives a maximum hydraulic gradient of 25. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnel. Further details about the design of high-pressure concrete plugs can be found in Dahlø et al.(1992) or Broch (1999).

OPERATIONAL EXPERIENCE FROM UNLINED PRESSURE SHAFTS AND TUNNELS.

The oldest unlined pressure shafts have now been in operation for 80 years. None of the pressure shafts and tunnels, with water heads varying between 150 and 1000 m which have been constructed in Norway since 1970, has shown unacceptable leakage. It is thus fair to conclude that the design and construction of unlined high-pressure tunnels and shafts is a well proven technology.
It is normal procedure to fill a shaft in steps or intervals of 10 - 30 hours. During the intervals the water level in the shaft is continuously and accurately monitored by an extra-sensitive manometer. By deducting for the inflow of natural groundwater and the measured leakage through the concrete plug, it is possible to calculate the net leakage out from the unlined pressure tunnel or shaft to the surrounding rock masses. The loss of water from a tunnel is large during the first hours, but decreases rapidly and tends to reach a steady state after 12 to 24 hours, depending on the joint volume that has to be filled.

CONCLUDING REMARKS

Experience from a considerable number of pressure tunnels and shafts have been gathered over a long period of time. These show that, providing certain design rules are followed and certain geological and topographical conditions are avoided, unlined rock masses are able to contain water pressures up to at least 100 bars, equalling 1000 m water head. Air cushions have proven to be an economic alternative to the traditional open surge shaft for a number of hydropower plants. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

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STORAGE OF HYDROCARBON PRODUCTS IN UNLINED ROCK CAVERNS

Eivind Grøv

ABSTRACT

During the last 30-40 years the concept of underground hydrocarbon storage has been implemented in Norway with great success. No negative influence on the environment has been recorded during these years of operation. This is now a proven concept and new storage caverns are being built in connection with Norwegian terminals and processing plants. The concept evolved from the growing hydropower development in the years of industrial growth in the post war Norway. The tunnelling industry established robust and effective tunnelling techniques which are now being applied for underground hydrocarbon storage. The most specific aspects of this concept are related to unlined caverns and the implementation of artificial groundwater to confine the product.

In modern societies there are growing concerns related to the safety and security of our infrastructure system. In addition surface space is becoming a scarce resource placing limitations on urban expansion. The environment needs to be protected and the aesthetics considered. Underground storage of oil and gas has showed an extremely good record in all these important aspects of the modern societies and is thus a popular method for such products.

INTRODUCTION

Storage of hydrocarbon products such as crude oil and liquefied gas is a necessary link in the process of transporting and distributing these products from the oilfields and to refineries and then on to the consumers. Appropriate storage volumes along this distribution line increase the availability of the product and the timing of the supply from pipelines, terminals and refineries. This is certainly in the interests of the consumers. This paper focuses on underground storage of hydrocarbons. It is acknowledged that sub-surface solutions have been utilised for other purposes too, in oil and gas projects, such as shore approaches, pipeline tunnels, slug cathcers etc., but such facilities will not be discussed in this paper.

Hydrocarbon products may be stored in various ways, and aboveground tankfarms have been the most common storage method. However, during the post war era new storage concepts were introduced, in particular for underground, or sub-surface storage. Underground storages included concepts such as:

- Aquifer storage; where hydrocarbons are pumped into porous rock mass formations. In many cases these are formations where hydrocarbon extracting has previously taken place.
- Saltdome storage; where circulating water in deep seated saltdomes creates cavities (leaching) within the saltdome that can be used for hydrocarbon storage. The saltdome behaves in a semiplastic way at great depths thus providing an adequate confinement.
- Mine storage; where appropriately abandoned mine working have been utilised as hydrocarbon storage. However, due to the irregular shape and layout of such openings they may not always be best fit for such storage purposes.
- Rock cavern storage; where the ground conditions fit the purpose and the storage requirement is in the range of 50.000 m3 and greater, rock caverns may be an option for underground storage.
- Depleted oil and gas fields.

During the post war era in Norway, sub-surface storage of oil and gas became important for strategic purposes. Various options for underground storage were considered but Norway was short of suitable alternatives to rock caverns. During this period utilisation of the underground openings had seen a significant growth in Norway, particularly due to the development in the hydroelectric power sector, where an increased number of projects utilised underground alternatives for waterways, pressurised tunnels and location of hydropower stations and transformer rooms. The Norwegian tunnelling industry developed techniques and methods to improve the efficiency and quality of underground works. A comprehensive experience base was established which became important when the underground

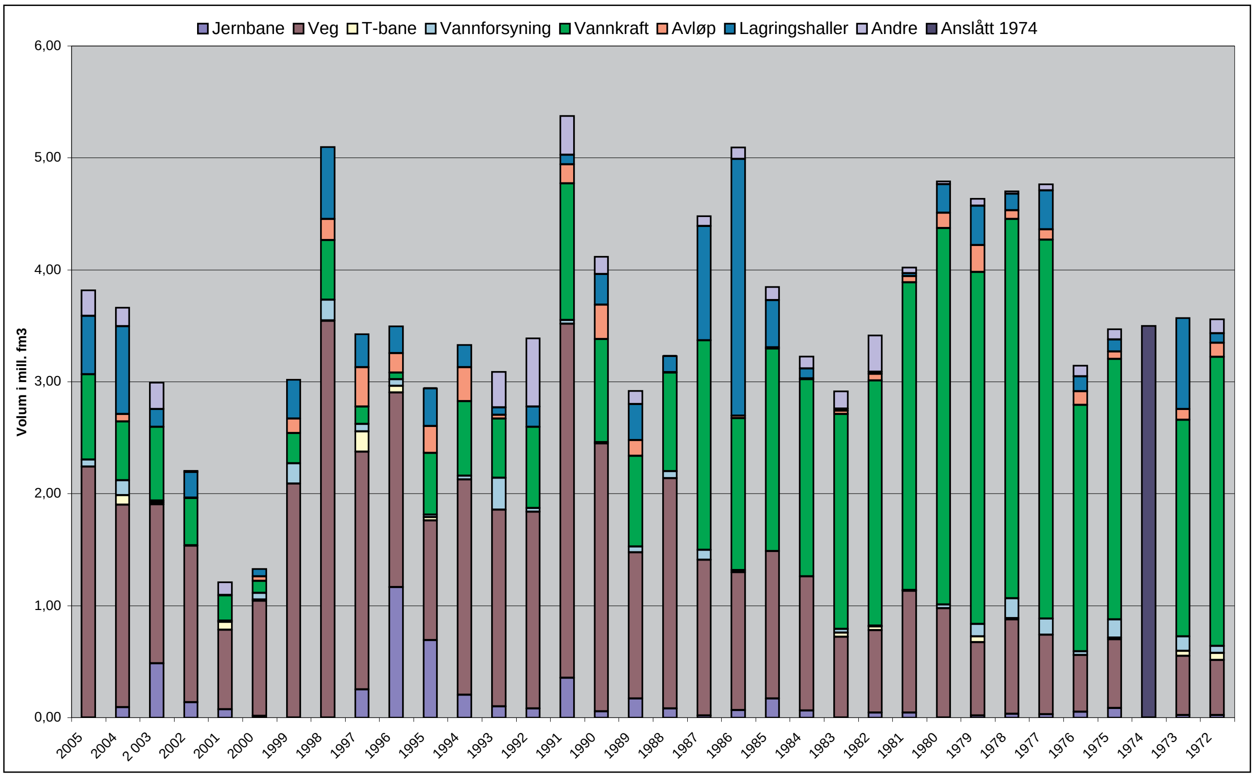

Figure 1. Development of Norwegian underground works (fm3=solid state) [Ref. 1]

storage of hydrocarbons was introduced. In figure 1 it is shown how the development of underground utilisation took place in Norway, shifting the tunnelling industry from hydropower projects to oil and gas storage and then towards the current use for infrastructure purposes.

In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development. Underground oil and gas storages mainly utilise the following capabilities of the rock mass:

- It's impermeable nature.
- It's stress induced confinement
- It's thermal capacity.
- It's selfstanding capacity.

Why to go underground with oil and gas storage in Norway?

In the following this paper presents the rationales and motivations for underground oil and gas storage in Norway, further it presents the development of underground oil and gas storages documented with factual data and case stories as well as presenting the basic principles for the establishing these storage facilities.

ABOVEGROUND STORAGE

The most common way of storing hydrocarbon products has been, and still is in clusters of aboveground steel tanks, as tankfarms. Typically they are found in the near vicinity of airports, close to harbours and ports, in connection to industrialised areas, at electric power plants and of course in the surroundings of refineries and terminals, and finally at natural gas treatment plants.

The close proximity to the production line and the users are the main reasons for such locations. The natural resource of suitable rock mass for underground storage may in many countries be in short supply and the best (and maybe the only) solution may therefore be above ground tankfarms.

The negative elements of such tankfarms are significant, particularly with regards to protecting the environment. In addition aboveground tanks are vulnerable targets to hostile actions such as sabotage and war. Further they are aesthetical undesirable and demand large land tracts of land which can often be utilised in a better way as in many densely populated areas surface space is becoming a scarce resource.

GOING UNDERGROUND

During and shortly after WW II

In Norway, the first underground hydrocarbon stor-

Figure 2. Typical tankfarm

ages were excavated during the Second World War, designed for conventional, selfstanding oiltanks. Later, being located underground was basically for protective purposes during the cold war era. One project of such kind is located at Høvringen, near the city of Trondheim in central Norway, where ESSO is operating underground steel tanks, whilst one other storage is located at Skålevik, and is operated by BP. Following on from these first projects was underground hydrocarbon storage in steel lined rock caverns, designed and built in accordance with for example Swedish fortification standards. This concept implies in brief a steel lining with concrete backfill of the void space between the steel lining and the rock contour. One such project is located in Hommelvik outside Trondheim and is operated by Fina. This project provides the supply of gasoline to the nearby airport. The above described projects were commissioned almost a half a century ago, and are still being in operation.

In the sixties, following experience from the hydroelectric power development, the confidence in unlined tunnels and caverns grew, and the first unlined hydrocarbon storage project was initiated. Concept developments took place in other Scandinavian countries at the same time, however, in Norway unlined pressure shafts had been in use for some time in the hydroelectric power development and the importance of sufficient in-situ rock stress to prevent hydraulic splitting of the rock mass was recognised as an important success criteria. Also the techniques of pre-grouting of the rock mass to stem or reduce water leakage started to be developed during this period. Adding to this, caverns with large cross-sections were already in use as hydropower stations. Thus, the Norwegian tunnelling industry was prepared and technically ready for the new challenge of unlined hydrocarbon storage in rock caverns.

Typically storages facilities during the cold war era were supply storages prepared for war time operation. They were in general owned by the Ministry of Defence but were often operated in peace time by the commerical oil companies.

The Ekeberg Storage

In 1966 construction work commenced on the Ekeberg storage facility located close to Oslo, the Norwegian capitol. The project was designed and constructed as unlined rock storage, and in 1969 oil filling commenced. This storage facility was later expanded to include new storage caverns. The Ekeberg storage introduced a design concept, which in general has been applied for later similar storages in Norway. A storage facility in rock was concluded as being the best solution for fuel storage in the Oslo area, being well secured against acts of war and sabotage. The Ekeberg storage is located adjacent to the Sjursøya Terminal, see figure 3, in the ridged area on the land side of the terminal.

The project was extended with a second stage some ten years after the commissioning of the first stage, when the Ekebergtank entered operation. The Ekebergtank is used for storage of jet fuel and gasoline.

Figure 3. Ekeberg crude oil storage and Sjursøya terminal (city center of Oslo to the left)

Both phases of the Ekeberg storage have been constructed based on unlined cavern storage. In the bottom of each cavern there is a waterbed with water being pumped from the adjacent sea, the Oslofjord. The caverns are situated well below the sea level, with the deepest point at 45m below sea level.

Sture and Mongstad crude oil storages, the caverns are close to 20m wide and 33m high, and this has become a typical cross sectional area for such caverns.

Ammonia storage

Almost at the same time as the Ekeberg project was due

Project	Year of Completion	Main rock type	Width x height, m	Temp. ^{o}C	Pressure MPa	Experience
Kristiansand, Skålevik	1951	Gneis-granite	Ø=32 H=15	40	0,1	No problems reported
Høvringen, Trondheim	1955	Quartsdiorite	Ø=32 H=15	40	0,1	As above
Sola, Stavanger	1960	Mica schist	Ø=15		0,1	Corrosion, decommissoned
Ekeberg I	1969	Granitic gneiss	12x10		0,1	No problems reported
Mongstad	1975	Meta-anorthosite	22x30	7	0,1	Some water leaks
Høvringen, Trondheim	1976	Quartzdiorite	12x15		0,1	Water curtain has been added
Herøya	1977	Limestone	10x15	8	0,1	Leak between caverns
Ekeberg II	1978	Granitic gneiss	15x10	60	0,1	Some blockfalls
Harstad	1981	Mica schist	12x14	7	0,1	No problems reported
Sture	1987/1995	Gneiss	19x33 ~1.000.000m3			
Mongstad	1987	Gneiss	18x33 1.800.000m3			No problems reported

Table 1. Norwegian crude oil storage facilities and refinery caverns for hydrocarbon products

Project	Year of Completion	Main rock type	Storage volume, m^3	Width× height, m	Temp. , ^{o}C	Pressure, MPa	Experience
Herøya	1968	Schistose limestone	50,000 excavated	10×12	6-8	0.8	No leakage, decommissioned
Glomfjord	1986	Gneissic granite	60,000	16×20	- 28 to -33	0.1-0.13, max. 0.2	No leakage

Table 2. Overview of main data for ammonia (NH3) storage [Ref. 4]

The typical size of the rock storage caverns in most recent projects indicates a cross-section of appr. 500m2. In practical terms this means that the caverns cannot be excavated in one blast round, but must be split into a top heading and several benches. As can be seen for the

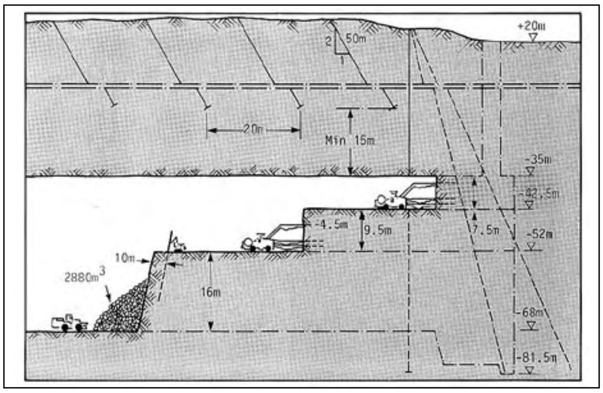

Figure 4. Typical excavation sequence; top heading and lower benches and with artificial groundwater infiltration from surface [Ref. 5]

to be completed, design and construction was ongoing for an unlined, pressurised NH3 (ammonia) storage for Norsk Hydro at Herøya. The project was designed for an operation pressure of 0.8MPa and at normal rock temperature.

Gas storages

In 1976 Norsk Hydro constructed an unlined rock storage for propane at Rafnes (close to Herøya) in Southern Norway. This project included a pressurised storage with an operation pressure of 7 atm at normal rock temperature and with a volume of appr. 100.000 m3.

The storage at Herøya is excavated in Precambrian granitic rock with its roof 90m below the sea level. The rock mass is practically impermeable, but due to jointing and few minor weakness zones, there was a need for grouting. The technique of pre-grouting was applied to prevent leakage of water into the caverns and to control potential fluctuations of the groundwater level. The design criteria required a hydraulic gradient towards the cavern to be greater than 1, a figure which was recognised as being safe, [Ref. 6]. During excavation it was experienced that the water level in some observation wells above the cavern showed a general fall in the groundwater level and it was decided that water infiltration holes needed to be implemented. Piezometers were installed in some of the observation wells to monitor the water gradient immediately above the cavern roof.

The experience during the first projects of this kind was that it was difficult to maintain the surrounding groundwater level without establishing a system for water infiltration of the rock mass. Since these first projects the potential of loosing control of the groundwater has been the governing decision whether or not to install water infiltration systems for groundwater compensation. The correctness of this approach might of course be questioned, and it has, still it is considered as good engineering practice by Norwegian engineers and plant operators to do so. And, as a consequence ´water curtains´ have been included in all of these projects, the extent and layout of the infiltration systems may, however, have varied. In most cases though, the infiltration systems have been installed and entered into operation prior to the excavation work. A very typical attribute of these projects is their shallow location, which necessitated an artificial groundwater infiltration.

Then, in 1986 the first chilled storage facility was constructed in Glomfjord. This included a water infiltration curtain from a gallery above the doughnut shaped cavern. The project was cooled to a temperature of -33oC.

To be able to reach the designed temperature in a chilled storage, different methods have been applied for the freezing process itself. One method that was often used previously was the direct cooling by introducing the product directly into the cavern. However, during the most recent years an improved method has been commonly applied that includes a 2-stage freezing process. Typically air cooling takes place until the 0-isotherm has reached a certain depth in the rock mass, say in the range of 3-5m. This enables necessary inspections to take place inside the cavern allowing qualified personnel to inspect for any defects and instabilities that may exist and rectify these whilst still working in a non-hazardous environment. Then the final cooling stage takes place during storage of the product itself.

Statoil is also involved in the Stenungsund propane caverns project in south-western Sweden. The total volume of 550.000m3 is stored underground at -42ºC.

The latest gas-storage facility commissioned in Norway was the propane cavern at Mongstad in 2003. This cavern was actually the 27th rock cavern excavated at the Mongstad Terminal for underground storage of oil and gas. This latest project was built to replace a cavern that had lost appr. 30% of its storage capacity due to rock falls into the cavern, subsequent water ingress and ice being formed. Despite such a negative one-time event the operator maintained his confidence in this storage concept. Due to the favourable costs associated with underground storage the operator finds advantages with such concepts connected to the operation and maintenance, and also to the safety aspect. There are no reported cases in Norway of negative environmental impact caused by such underground facilities.

This latest extension at the Mongstad refinery is a 60.000m3 underground storage facility for propane. It

Project	Commissioned	Main rock type	Storage volume, m^3	Width× height× length, m	Temp. , ^oC	Pressure, MPa	Experience
Rafnes	1977	Granite	100,000	19×22×256	~ 9	0.65, tested at 0.79	No leakage
Mongstad	1989	Gneiss	3 caverns, total 30,000	13×16×64	6-7	Up to 0.6	No leakage
Mongstad	1999	Gneiss	60,000	21×33×134	- 42	0.15	Reduced capacity
Sture	1999	Gneiss	60,000	21×30×118	- 35	0.1	No information available
Kårstø	2000	Phyllite	2 caverns, total 250,000	Approx. 20×33×190	- 42	0.15	No leakage
Mongstad	2003	Gneiss	60,000	21×33×134	-42(propane) +8 (butane)	0.15	Under construction
Mongstad	2005	Gneiss	90.000	22x33x140	6-7		Construction start 2005
Aukra	2007	Gneiss	63.000/ 180.000	21x33x95 21x33x270	6-7	0,2	Not yet commissioned

*) All with propane; Mongstad 1989 also stores butane and Sture 1999 stores a propane/butane mixture. Mongstad 2005 will be naftalene, Aukra 2007 will be condensate

*Table 3: Overview of main data for petroleum gas storage *) [Ref.4]*

is designed for an operating temperature of -42ºC. The cavern is formed like a ´flat lying bottle´ with a concrete plug being located almost at the bottle-neck and with the pumping arrangement in the far end of the cavern. The maximum height of the cavern is 34m and it has a width of 21m, whilst the length is 124m.

Figure 5. A typical cross-section of an underground oil and gas storage cavern with straight walls

THE CONCEPT OF UNDERGROUND STORAGE

The concept of unlined oil and gas storage in use in Norway follows the main principles and methods as outlined below:

Permeability control and hydraulic containment
The methods for controlling leakage from an unlined underground storage consist mainly of 1) permeability control and 2) hydrodynamic control (or containment). In figure 6 it is schematically shown according to Kjørholt [Ref.2].

By permeability control it is meant that leakage control is achieved by maintaining a specified low permeability of the rock mass. This can be achieved by locating the rock caverns in a rock mass that has natural tightness sufficient to satisfy the specified permeability. However, the rock mass is a discontinuous media and the presence of joints etc. governs its permeability. Permeability control can be preserved by artificially creating an impermeable zone or barrier surrounding the rock caverns by; a) sealing the most permeable discontinuities in the rock mass by grouting; or b) introducing a temperature in the rock mass which freezes free water and filling material in the rock mass; or c) a combination of both methods.

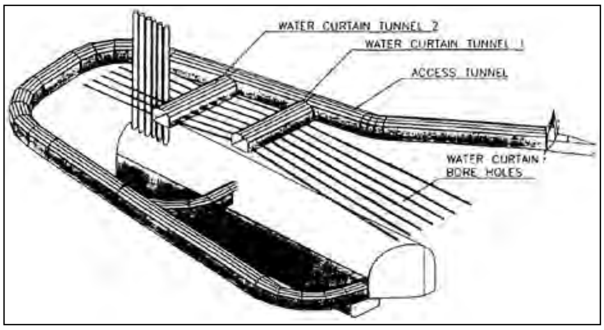

Figure 7. Water infiltration Sture [Ref. 9]

By hydrodynamic control it is meant that there is groundwater present in discontinuities (joints and cracks) in the rock mass and that this groundwater has a static head that exceeds the internal storage pressure. In practical terms it means that there is a positive groundwater gradient towards the storage, or the rock cavern. In general, sufficient groundwater pressure is obtained by a) a deep seated storage location which provides the sufficient natural groundwater pressure, or b) by an additional artificial groundwater such as provided by ´water curtains´ and similar arrangements.

In the invert the crude oil is normally floating on top of a water bed. The water bed could either be fixed or variable, depending on the discharge pump arrangement to be used. An important element in the hydrodynamic confinement is related to the following up of the groundwater level surrounding the storage facility. It would normally be required to install a number of monitoring wells to monitor groundwater levels.

The concept of unlined pressurised storages evolved during the hydroelectric power development that took place in Norway during the 1960's to 1980's. See table 4 below. Both permeability and hydrodynamic control was applied in compressed air storage projects.

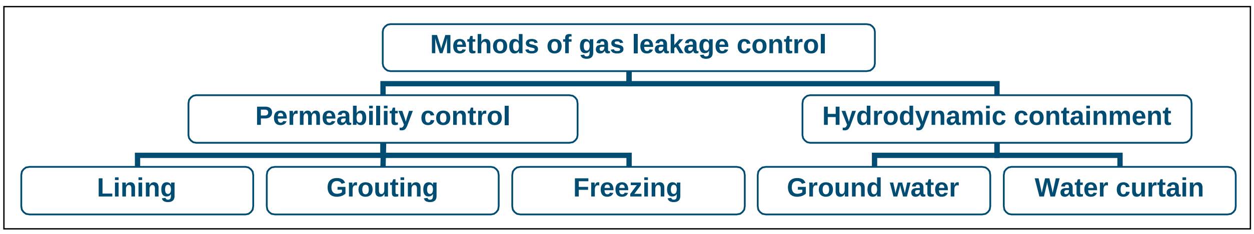

Figure 6. Methods for controlling gas leakage from a pressurised underground storage.

Project	Commis-sioned	Main rock type	Excavated volume, m^3	Cross sec-tion, m^2	Storage pressure, MPa	Head/ cover *)	Experience
Compressed air buffer reservoirs							
Fosdalen	1939	Schistose green-stone	4,000		1.3		Minor leakage
Rausand	1948	Gabbro	2,500		0.8		No initial leak-age
Air cushion surge chambers							
Driva	1973	Banded gneiss	6,600	111	4.2	0.5	No leakage
Jukla	1974	Granitic gneiss	6,200	129	2.4	0.7	No leakage
Oksla	1980	Granitic gneiss	18,100	235	4.4	1.0	<5Nm3/h
Sima	1980	Granitic gneiss	10,500	173	4.8	1.1	<2Nm3/h
Osa	1981	Gneissic granite	12,000	176	1.9	1.3	Extensive grout-ing
Kvilldal	1981	Migmatitic gneiss	120,000	260-370	4.1	0.8	Water infiltr. necessary
Tafjord	1981	Banded gneiss	2,000	130	7.8	1.8	Water infiltr. necessary
Brattset	1982	Phyllite	9,000	89	2.5	1.6	11Nm3/h
Ulset	1985	Mica gneiss	4,800	92	2.8	1.1	No leakage
Torpa	1989	Meta siltstone	14,000	95	4.4	2.0	Water infiltr. necessary

*) Ratio between maximum air cushion pressure expressed as head of water and minimum rock cover

Table 4. Overview of main data for compressed air storage, including air cushion surge chambers [Ref. 4]

According to regulations issued by the Norwegian Fire and Safety Administration (DBE) the natural groundwater or the overpressure resulting from water curtains shall be 2 bar higher (20m water column) than the internal storage pressure, for oil and gas storage facilities, [Ref. 3].

It is possible to combine the two methods of leakage control and apply a combination of both hydrodynamic control and permeability control, this has been termed as a double barrier, by Kjørholt [Ref. 2]. However, and in a rather general manner [Ref. 4]; hydrodynamic control may be the preferable method in situations with a substantial internal storage pressure, whilst permeability control may be preferred in situations with low/atmospheric storage pressure. For storage facilities where the hydrodynamic control has been applied excessive water from the artificial water curtain may be allowed to enter the storage facility and thus the product stored must tolerate the presence of water before it is separated, collected and discharged from the storage.

One typical example of the application of hydrodynamic control is air cushion chambers in hydropower schemes where air is compressed and the ´water curtains´ constitute the containment. On the other hand, a typical example where permeability control is applied is crude oil storage facilities.

The water curtain can be used to balance the migration of the 0-isotherme by applying water with a specific temperature for water infiltration. This concept has been applied at Sture (1999), which is a project that was designed by Skanska Teknik of Sweden [Ref. 9].

A typical example where the two different methods are used in combination as a double barrier would be for LPG storage facilities. The product in LPG storages might hold a temperature of -42 ºC at atmospheric pressure. The surroundings of the storage consists of a saturated rock mass that is frozen to a depth reaching some 10m out from the cavern periphery, thus achieving the desired permeability requirement. A comprehensive pre-grouting scheme is required for the purpose of reducing the amount of water needed for the saturation of the rock mass and consequently reducing the energy required for the freezing process. With the double barrier used in this case the potential for a successful filling and operation of the plant is increased.

Stress induced confinement

A condition for a successful operation of chilled gas storage can be expressed in the following equation, according to [Ref. 10]:

In-situ stress + tensile strength > thermal stress

In situations with a significant internal storage pressure this will contribute on the right hand side of the equation, however in the case of chilled storages the contribution from the internal gas pressure (0,1 – 0,3bar) is negligible.

In the same way, water pressure caused by water curtains or by natural high ground water will act as a reducing factor on the in-situ stress situation, in other words destabilising the equilibrium.

In a system with a pressurised storage cavern, for example such as for LNG storage taking place at ambient temperature a high internal storage pressure would be required. To be able to withstand the internal pressure the in-situ rock stresses must be larger by a factor of safety than the storage pressure. A high in-situ rock stress must be considered as an important part of the containment system. If this condition is not present the internal storage pressure may accidentally lead to hydraulic jacking of the rock mass, resulting in cracking of the rock mass and opening of pathways that enable the stored product to escape from the storage and migrate into the surrounding rock mass, eventually reaching neighbouring tunnels/caverns or the surface. From the hydropower development the Norwegian tunnelling industry experienced the use of unlined pressurised tunnels with almost a 1000m water head. The basis of this design is a minimum stress component that is greater than the water pressure. The analogy goes for pressurised gas storage, namely that the following must be fulfilled:

$\sigma 3 > \sigma_{ip} \times F$ where:

$\sigma 3$ is the minimum stress component.
σ_{ip} is the inner storage pressure in the cavern and F is the factor of safety.

Thermal capacity of the rock mass
In Norway a number of cold storages were actually excavated and in operation before the chilled gas concept was developed. The first of these underground cold storages in unlined rock caverns was commissioned in 1956, with an approximate number of 10 projects in operation. They were constructed with storage capacity in the range of 10-20.000m3. Typically, the temperature in these storages varies between –25 to –30ºC. These cold storages have mainly been built for the purpose of storage of food and consumer products. Ice cream storage is one such utilisation.

From years of experience from the maintenance and repair of these facilities the operators have gained important experience regarding the behaviour of the rock mass in frozen state as well as how the ground reacted upon changes in cooling capacity.

For example, on occasions the freezing element was turned off and the temperature sensors in the rock mass

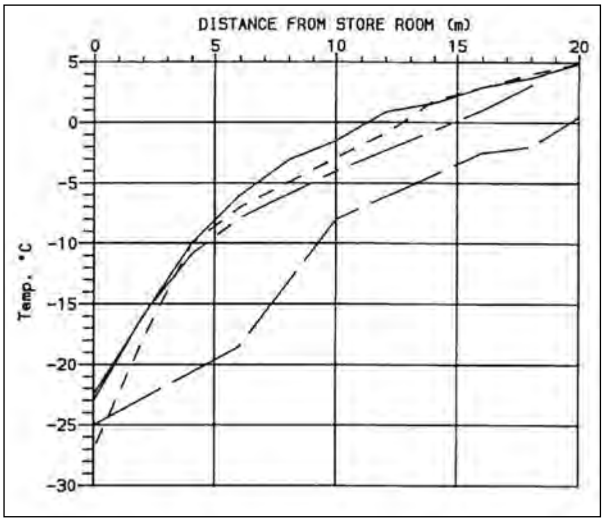

Figure 8. Temperature gradient in rock around a cold storage [Ref. 7]

were followed up to examine the temperature development in the storage caverns and the surrounding rock mass. A normal response to such changing circumstances was a rather slowly increase of the temperature in the rock mass. The 0-isotherme moved in a rather slow speed towards the tunnel periphery, in the same way as it moves slowly outwards whist freezing takes place. The thermal capacity of rock in general implies that the material has a significant capability of maintaining its frozen state, once it has been reached, a factor that influences positively also to the cost aspects of those facilities.

Self standing capacity
Most rock mass have a certain self-supporting capacity, although this capacity may vary within a wide range (Bienawski 1984). An appropriate engineering approach is to take this capacity into account when designing permanent support.

As for any type of underground structure the selection of the site location, orientation and shape of the caverns are important steps preceding the dimensioning and the laying out of the site.

Rock strengthening may, however be needed to secure certain properties/specified capacities, the same way as is the case for any other construction material. The fact that, the rock mass is not a homogenous material should not disqualify the utilization of its self-standing and load bearing capacity. Typically, rock support application in Norwegian oil and gas storage facilities consists mainly of rock bolting and sprayed concrete. The application of cast-in-place concrete lining in such facilities has been limited to concrete plugs and similar structures and is not normally used for rock support. The rock support measures are normally not considered as contributing to the containment, other than indirectly by securing the rock contour and preventing it from loosening.

Furthermore, the Norwegian tunneling concept applies widely a drained concept, meaning that the rock support structure is drained and the water is collected and lead to the drainage system. Thus the rock support is not designed to withstand the full hydrostatic pressure in the rock mass. The experience with large underground caverns was obtained in Norway during the development of hydroelectric power schemes for which purpose a total of 200 underground plants were constructed. Commonly the power-house was located in an underground cavern, typically seized some 15-20 wide, 20-30 m high and tens-hundreds meter long. This experience was useful for the development of underground oil and gas storages.

Various types of monitoring to follow-up the behaviour of the rock mass and the support structures are available for use to document the stability and behaviour of the rock mass.

ADVANTAGES OF UNDERGROUND STORAGE

In the following the main advantages of underground rock storage are described. In brief these are:

- Utilising the variety of parameters of the rock mass.
- Environmentally friendly and preserving.
- Protection during war.
- Cost aspects.
- Operation and maintenance
- Protected from natural catastrophes

It has been documented that the rock mass holds a number of important parameters that are utilised in underground storage of hydrocarbon products. These capacities allow a variety of storage conditions and enable a number of diverse types of products to be stored in unlined rock caverns. With the current knowledge of the mechanical and thermodynamic behaviour of the rock mass the current use of such storage facilities can be said to take place within proven technologies. Future use of underground storages may push these technologies to its limit and thus require improved methods. This will be briefly discussed at the end of this paper.

As far as the environmental aspects are concerned the experience from Norwegian underground storage projects are unreservedly positive. So far product leakage has not been reported in any of these projects indicating clearly that the applied concept and techniques to obtain the required confinement are appropriately proven. For a subsurface solution dedicated systems for collection and handling of various types of spill can be planned thus limiting the spread of any spill. Bringing these storage tanks below the surface allows valuable surface areas to be utilized for other purposes; recreational, cultural and residential. In addition unsightly structures can be hidden away underground.

Crude oil and refined products may in a war-time situation be the subject for hostile actions. The protection against various types of bomb attacks and sabotage are indeed capabilities not widely described and published, but indeed contribute to the overall favourable application of underground storages.

Protection from natural disasters and catastrophes such as earthquakes is a beneficial advantage of underground storage. It has been acknowledged that subsurface structures have several intrinsic advantages in resisting earthquake motions. Experience and calculations show this clearly.

The latest cost figures on construction costs are due in 2004. The total construction cost is in the range of 150 – 310 USD per m3 storage, out of which 50-70% is associated with mechanical and electrical installations.

Shallow locations are indeed a feature that improves the cost advantage of these storages. In figure 9 below a cost comparison of steel lined surface tanks are compared to underground storage caverns, unlined.

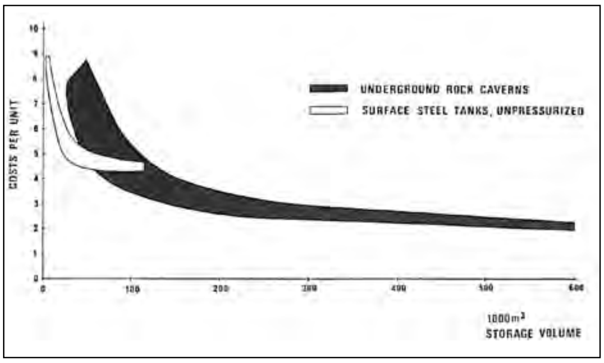

Figure 9. Relative rock cavern/steel tank costs according to Frøise [Ref. 8]

It has been out of our reach to obtain figures on operation and maintenance costs from Norwegian oil and gas storages facilities. The physical isolation of underground structures from the external environmental reduces the detoriation of building components and may result in low maintenance costs for underground structures.

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DEFENCE, CIVIL DEFENCE, STRATEGIC INFRASTRUCTURE AND COMBINED PURPOSES

With the rock mass as extra barrier, the underground alternative is excellent for protection of strategic infrastructure, defence and civil defence facilities. In particular the civil defence facilities might be combined with other activities like here at the Tærudhallen sports facilities.

Entrance of "Tærudhallen", multipurpose caverns for civil defence and sport activities outside Oslo

Interior from "Tærudhallen"

UNDERGROUND TELECOMMUNICATION CENTRES

Jan A. Rygh
Per Bollingmo

ABSTRACT

During the last decades the Norwegian Telecommunication Service has been in a continuous strong development. In the early stages (1960's), the old Post and Telegraph buildings were quickly filled with new equipment, hence additional space had to be made available. In recognition of the strategic importance of the communication systems, safety and security aspects called for special attention. The underground solution very often was preferred and today one will find numerous underground installations over the entire country.

This is a presentation of technical aspects related to the development of the underground alternative for the telecommunication centres.

INTRODUCTION

The extensive development of the civilian Norwegian telecommunication that started some 40 years ago, soon led to lack of suitable areas for new telecommunication buildings.

Underground facilities for hydropower, civil defence, roads and rail had long traditions in the country. Therefore this alternative was frequently selected where suitable rock conditions were available also for telecommunication facilities.
At that time, in the "cold war" period, defence planning and protection against war hazards also supported safe alternatives with a view to the country's total defence concept.

In the following the ruling factors for planning design, construction and use of such facilities are briefly described.

GEOLOGICAL CONSIDERATIONS AND ROCK CONSTRUCTION ASPECTS

2.1 Location of the projects

The telecommunication projects are all located in urban areas, some of these in town centres with limited access possibilities. There were also very limited options for adjustments of location or orientation of caverns in order to obtain favourable rock conditions. Excavation methods, procedures and rock support works are therefore adapted accordingly. (Fig.1)

The rock construction works for the projects followed the practice in Norway by utilizing the rock's strength and bearing capacity. Rock bolts and sprayed concrete were used to safeguard local weak zones, whereas concrete or steel supporting structures rarely had to be used. Smooth blasting to obtain even surfaces with minimum fracturing has been emphasized.

Fig. 1 shows the preparations for the main entrance tunnel to a telecommunication facility in rock.

Construction practice
A flexible design allowed for modifications during the construction period to adapt support methods to the actual rock stability situation. Competent staff, crews and advisers had the authority to modify while concluding related contractual matters.

A normal procedure in the early sixties was to use hand held drilling equipment, fully grouted rock bolts and dry mix sprayed concrete. During the later 40 years drill jumbos have taken over, remote operation of equipment and machinery enhanced efficiency and safety, wet mix and fibre reinforcement are standard and additives allowing thick layers to be applied in one operation have been introduced.

To avoid or reduce negative impact to neighbourhood while implementing new facilities the contract specifies limitations to: vibration caused by blasting, water ingress, noise (from ventilation systems etc.), the timing of blasting, working hours, heavy transport and others.

All the telecommunication caverns have been closely monitored regarding blasting vibrations, and together with careful blasting techniques, no serious damage to properties have been experienced. Noise and dust problems have also been considerably reduced. This has been an essential improvement for the acceptance of projects located in urban areas.

Geological conditions and rock support works
The projects have been constructed in various rock types, from relatively weak Silurian schist to good granite and Precambrian gneiss. Engineering geological pre-investigations are normally limited to surface mapping, and inspection of nearby tunnels if available.

The cavern design normally includes an arch height of 1/5 of the width. There have been no serious instabilities of tunnels or caverns. A few weakness zones with crushed rock and swelling clay have been secured with un-reinforced concrete lining. Rock bolting is carried out as spot bolting, normally with fully grouted bolts. Sprayed concrete is usually systematically applied to roof surfaces, and only partly on the walls.

Water ingress to the caverns has been very limited, and no damage to surface constructions from settlements or subsidence due to lowered ground water level has been observed.

In service inspection regarding rock stability and support works
Between the years 1999 to 2003 inspections of all the projects have been performed to ascertain the condition of the rock support works. An evaluation of the present stability situation has been made, and additional support is carried out if required. The experience from these inspections is that all rock surfaces covered with sprayed concrete are completely intact with no additional requirements for support. The surfaces without sprayed concrete had a few blocks which have been considered potentially unstable, and are removed or secured with additional bolting. Minor downfall of small stones is recorded, without any damage to equipment or constructions.

MAJOR CONSIDERATIONS AFTER BLASTING AND EXCAVATION

After blasting and excavation of the rock, the result will be a cavern or systems of caverns with characteristic properties. Fig. 2 shows a typical cavern for an underground telecommunication centre.

The cavern will normally be humid and will have water leakage through the roof, walls and floor. High humidity often occurs when humid outside air (summer) is brought into the cavern and thus leaving condensed water on the cool rock surface. In winter periods, the opposite can happen when warm humid air from the cavern meets a cold surface, especially in the entrance tunnels where the rock overburden often is small and more influenced by the outside temperature.

Fig. 2 Typical cavern, 16 m wide.

The rock temperature inside the cavern will be nearly constant (in Norway 6-8º C). To make this cavern fit for a modern telecommunication facility, the inside climate must be controlled.
To solve this problem and allow for installation of sensitive electronic, electric and mechanical equipment, an inner lining or building inside the cavern is required
These rooms/inner spaces must be dehumidified and ventilated.

Water ingress and condensed water in a rock facility has to be drained, and the drainage/pump system must be absolutely dependable and a clean up system must be installed.

DESIGN CRITERIA

General design criteria
For the peacetime situation, the Norwegian Telecommunication Authorities (NTA) follows national requirements for such facilities.

Special design criteria
For a wartime situation, also including the threat from sabotage, a thorough fortification study was carried out early, and detailed specifications were established (1968).

The basis for these specifications was that a facility inside a rock cavern gives surprisingly good protection against a direct hit, even from large conventional weapons. The openings will be the weak points.

Weapons effects taken into consideration were:
- Conventional weapons, causing air blast (short duration), fragments and ground shock
- Fuel air explosive (FAE) weapons, causing air blast (medium length duration) and induced ground shock.
- Nuclear weapons, causing air blast (long duration), heat- and nuclear radiation, direct and induced ground shock, electromagnetic pulse (EMP) and radiation from fall out. Note: An underground rock facility will be well protected against any kind of radiation. The hazard from fall out radiation will depend on the geometry and length of the entrance tunnel. The amount of food and water supplies in the facility will govern the time of total closure and hence the survivability of the personnel inside.
- Chemical weapons (gas), causing poisoning of unprotected personnel
- Biological weapons (germs, virus etc.), causing sickness of exposed personnel.
- Sabotage

Throughout the years, special protection equipment of high quality has been developed in Norway. Such equipment is available to meet all significant weapon effects and was used in all rock facilities for telecommunication.

CONSTRUCTION PRINCIPLES AND EXAMPLES

The main building
Two main problems were evaluated when the construction material was considered. That was:
- Humidity and
- Fire hazard.

Concrete construction was the natural choice covering both these considerations.

Fig. 3 shows a typical solution

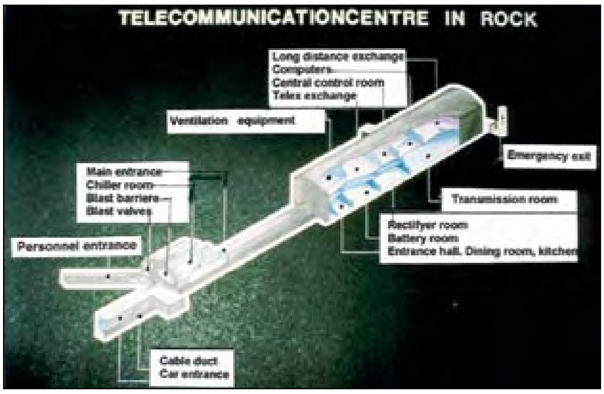

Fig. 3 Typical telecommunication centre in rock
Welding the reinforcement steel bars in the concrete to a "Faraday Cage" solved protection against EMP.

All other materials inside the facility are to the greatest extent non-combustible.
Where water seepage occurred an impervious membrane was installed.

Figs. 4 and 5 show activities from the construction period.

Fig. 4 Erection of the concrete building in the cavern.

Fig. 5 Installation of telecommunication equipment in the concrete building.

Ventilation and humidity

In a telecommunication facility in rock, full control of temperature and humidity (relative humidity) is imperative.

In planning and design it was emphasized that personnel in an underground facility normally are more sensitive to the quality of the air temperature and humidity than in a conventional surface building. Fresh air and good climatic conditions are required, and essential for psychical health and efficiency of the personnel. The relative humidity in all rooms or space is 40-50%, which is considered most comfortable for the personnel, and gives longest lifetime and lowest maintenance cost for equipment and structures.

Since these facilities also shall withstand weapons effects, blast valves protect all openings, and NBC-filters (Nuclear, Biological, Chemical filters) are installed.

Fig.6 Canteen in an underground telecommunication facility.

Fresh air with 40-50% relative humidity, lighting, colours, and good architecture, are vital factors for the well being for both personnel and equipment.

Emergency power supply

A "no break" power supply for the electronic communication system based on accumulator batteries is compulsory.

The main source for emergency power is diesel engine powered generator(s) that provides the power supply to all vital components in the facility. This power supply is sensitive to both ground shock and EMP. Protective measures are therefore included.

Fire prevention

Considerable efforts are used in all stages of planning, design, construction and operation, to prevent fire in these underground telecommunication facilities. Since the escape possibilities are limited, smoke and fire will be even more hazardous than in above ground facilities. The choice of non-combustible construction materials is mentioned before. Escape routes and emergency exits are clearly marked and kept clear of smoke by the ventilation system." Fire barriers" for cable ducts are included. For fire extinguishing, various systems are used depending of the function of the rooms.

OPERATION AND MAINTENANCE

Manuals

A complete set of manuals, tailor made for the specific facility, was elaborated and ready when the project was completed and transferred to the user. These manuals are the basis for a proper operation and maintenance of the complete facility.

Special emphasize are put on the operation in a wartime situation and in case of fire.

Education of employees

Telecommunication facilities in rock, including protection against war hazards are in many ways rather complex.

It was recognized as very important to give technical employees adequate education both in normal procedures for the equipment installed as well as for the protection systems. This included:

- Blast protection
- Ground shock protection
- B-C (biological and chemical) warfare
- EMP-protection
- Fire hazard
- Practical exercise

A course was given to secure the correct use of all kinds of equipment.

COST

Construction cost

The construction costs of underground facilities as compared to above ground structures depend on the actual situation. Aspects to analyse are i.a. access, land availability and costs, rock quality for underground structures, foundation design for above ground etc.

Safety and strategic requirements will frequently favour the underground concept. Also to keep in mind: The cost of the telecommunication equipment to be installed is normally by far more expensive than the construction cost.

Cost of land

Ownership to the underground is disputed. In Norway the owner of the surface land has a restricted right of disposal of the subsurface. The matter has been disputed, for practical use the right has been limited to the owner's normal need and use (construction of buildings, basements, foundation purposes).

Subsurface location of large telecommunication centres saved large and expensive surface areas. This has been cost efficient.

Operating cost
As stated earlier, telecommunication centres in rock demand more sophisticated ventilation (air-conditioning) systems and lighting.
In addition, for the employees working under ground on a regular basis, efforts are made by architectural means as well as light and colours to compensate for an environment without windows. (See fig.6).

Maintenance cost
Telecommunication facilities in rock need less maintenance compared to similar surface buildings. This is mainly because buildings in caverns are not exposed to direct sun, rain, snow and wind.
A main factor is also selection of the right construction materials for protection against water and humidity.

Life cycle cost
Life cycle cost for telecommunication facilities in rock is somewhat lower compared to above ground alternatives due to:
- Low cost of land
- Lower maintenance cost
- Lower operating cost and almost unlimited lifetime.

FINAL REMARKS

The experience gained from a number of underground telecommunication centres shows that the success depends on vital factors such as:
- Planning and design carried out by qualified engineers and architects.
- Well proven technical specifications
- Experienced contractors
- Correct operation and maintenance
- Education and training of the staff

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STRATEGIC INFRASTRUCTURE, DEFENCE, COMBINED PURPOSES.

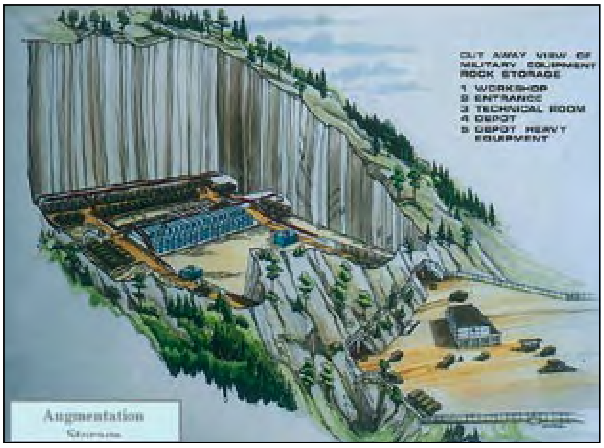
Tom Hermansen

INTRODUCTION

The design principles for underground defence facilities has always been related to the weapon threat, and has mainly focused on the ability to hit (Circular Error Probability – CEP) and the payload (type and amount of explosives) in addition to, for the last century, the effect of NBC-weapons (Nuclear, Biological , Chemical). In the 80's the ability to penetrate was more and more put into focus since the accuracy of the weapons had improved dramatically, this was made evident for the entire world through TV transmissions by CNN during the Gulf-war. One of the lesson learned from this war that was highlighted by the majority of experts, was that you can not hide and that the underground facilities are outdated. However, this is not a balanced statement since there was no distinction between the cut-and-cover facilities and in-rock or deep underground facilities. The accuracy of the weapons seems also to have been exaggerated, even if the TV-transmissions showed an amazing number of direct hits, the number of misfire and failure resulted in a CEP value much more favourable for underground facilities. Further, the ability to penetrate is far away from reaching the deep in-rock facilities.

WHY THE ARMED FORCES SHOULD GO UNDERGROUND

For civilian purposes, underground facilities are primarily tied to financial/commercial aspects. This is also applicable for the Armed forces, but the main reason to go underground is for protection. This protection must be related to the foreseeable threat and operational concept. The promising political development in Europe has changed the Norwegian security challenges dramatically the last 15 years. Given the continuation of this development, there is no direct military threat against Norway in the foreseeable future. Based on this, Norway has changed its concept of operation from an invasion oriented structure to a more alliance dependant concept, i.e. Norway are contributing to the Alliance



with capabilities that are needed for the alliance in order to fulfil its obligation to the participating nations.
The remaining threat on Norwegian soil is the possible act of terrorism, which is more relevant for the civilian society than the military as such. The lack of warning time requires access to protected facilities even in peacetime for all assets that are critical for the society. Hence, the Armed forces will require protection for their mission critical assets. Since most of the armed forces are mobile and designed for out-of area-operations, the need for underground facilities in Norway are limited to C3I (Command,Control,Communication and Information) and logistical assets (Storages, POL-

facilities (Petroleum, Oil and Lubrication), ammunition storages, depots etc.). For other nations, were the invasion threat is still evident, the picture will be completely different and more like our cold war scenario.

Even though Norway has reduced the need for and number of underground facilities, there will still be a significant number of facilities operational. The design criteria for the remaining facilities and possible new ones, will be connected to the relevant threat, which is terrorism and the effects of Weapons of Mass Destruction (WMD), without neglecting the traditional military weapon threats. The following subparagraphs will in general terms describe this criteria and principles.

Design Criteria

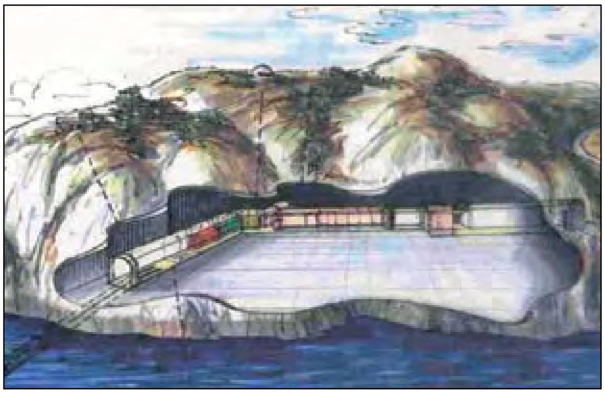
The following criteria have been valid for many years and are considered to be applicable also for the foreseeable future:
• The asset should be difficult to locate,
• if located it should be difficult to identify,
• if identified it should be difficult to hit,
• if hit it should be difficult to destroy, and
• if destroyed it should be difficult for the enemy to make battle damage assessment after the attack in order to determine its state of readiness.
In-rock facilities are the ultimate solution for all this criteria, except the disadvantage with lack of mobility. For the C3I and most of the logistic in peacetime, mobility is not crucial, The availability, when needed, is the driving factor. In this case underground facilities are justified and will be designed according to the following principles.

Design Principles

For the detailed design the following principles should be adhered to:

Overburden
Considerable (20 meter +) overburden of solid rock for the complete facility. That will protect against possible penetrators, reduce the effect of HPM (High Power Microwave) to an acceptable level, eliminate the effect of initial nuclear radiation, and reduce the ground shock considerably.

Camouflage and Blast Traps
All entrances/emergency exits etc should be camouflaged and connected to the main facility by long and dog-legged tunnels with blast traps. The entrances and emergency exits should also have entrance control and close defense firing positions. By camouflaging the



entrances they will be more difficult to detect. But even if they are detected, by constructing the access tunnels in a dog-legged shape, this means that the exact location of the main facility cannot be established.
So far it has not been demonstrated that an air-borne geo-radar will be able to detect such an underground facility with sufficient accuracy. Thus it is very important to make sure that detailed plans of the whole facility showing the main facility in relation to the entrances do not fall into the wrong hands.
During construction false coordinates may be used, and no drawing used on the construction site should show the complete facility, nor should it be possible to "construct" a full site plan from the working drawings.
The excavated materials should be spread in several dumps, used for road and building foundations etc. This will make it impossible to calculate the volume of the facility from measuring the spoil heap.
For the same reason all power lines, communication cables, wave-guides, water and sewage pipes etc should be connected to the main facility using micro-tunnels coming from different directions. This applies also to connections used purely in peacetime. No connection should enter the facility directly through any of the entrances, as they can be traced from the air, and thus will reveal the entrances even if they are properly camouflaged. Micro-tunnels should be used instead of trenches, as a trench is visible from the air for a considerable number of years after it has been backfilled.

Antennas
Antennas and sensors should be spread over a large area quite a distance away from the main facility, and should be connected to the main facility by fiber-optic cables. It is difficult to hide the sites for antennas and sensors, but even if they can be traced, this does not give away the true location of the main facility. The sensors and antennas should be hardened and made able to withstand the designed weapon effects, or have sufficient replacement capability. The antennas should also be located in different places, and all this combined will give the required level of redundancy.

Cooling
If possible, the facility should be cooled by water. By using heat-exchangers and fan-coils, the air supply requirement will be for breathing and power generators only. The power generators may have a separate air supply for combustion, independent of the filtered air for breathing. As 90% of the air supply is normally used for cooling purposes, this drastically reduces the size of the ventilation rooms and equipment (fans and ducts), the number and size of the shock-valves, the power requirement and the thermal signature of the facility.
Underground storage facilities will, however, need additional ventilation during loading and unloading times if transport vehicles driven by combustion engines are used for transport.

Ventilation
The reduced quantity of air is drawn firstly through a stone filter which will give some reduction in any outside blast pressure. Also there is less chance of a complete blockage of the air intake with a stone filter than with a steel pipe intake if they are exposed to weapon effects. After the stone filter, the air then passes an FAE-igniter (Fuel Air Explosives-igniter) before it reaches the blast valves in the blast barrier, and then enters the expansion chamber. The FAE-igniter will set off, by burning or deflagration any FAE near the stone filter, long before it reaches and can damage the main facility.
The exhaust from the diesel generators should be cooled with water and then fed into long micro-tunnels which are terminated in various ways away from the facility. Tests have shown that the exhaust temperature can be brought down to just 2 degrees Centigrade above the water temperature. At the same time the water-cooling also removes all soot and similar types of combustion materials from the exhaust. The combined effect is that there is a significant reduction of both the thermic signature and the noise from the generators, making it very difficult to locate the exhaust exits. The exhaust pipes should preferably end up in rock fills, which can absorb a hit from a warhead without blocking them. It should also be possible to cross-feed the exhaust, in case one pipe gets blocked.

NBC
If required, in addition to the already given protection against blast and radiation the NBC protection will be provided according to the principles of collective protection, i.e. a pressurized Toxic Free Area with access through a Contamination Control Area (CCA).

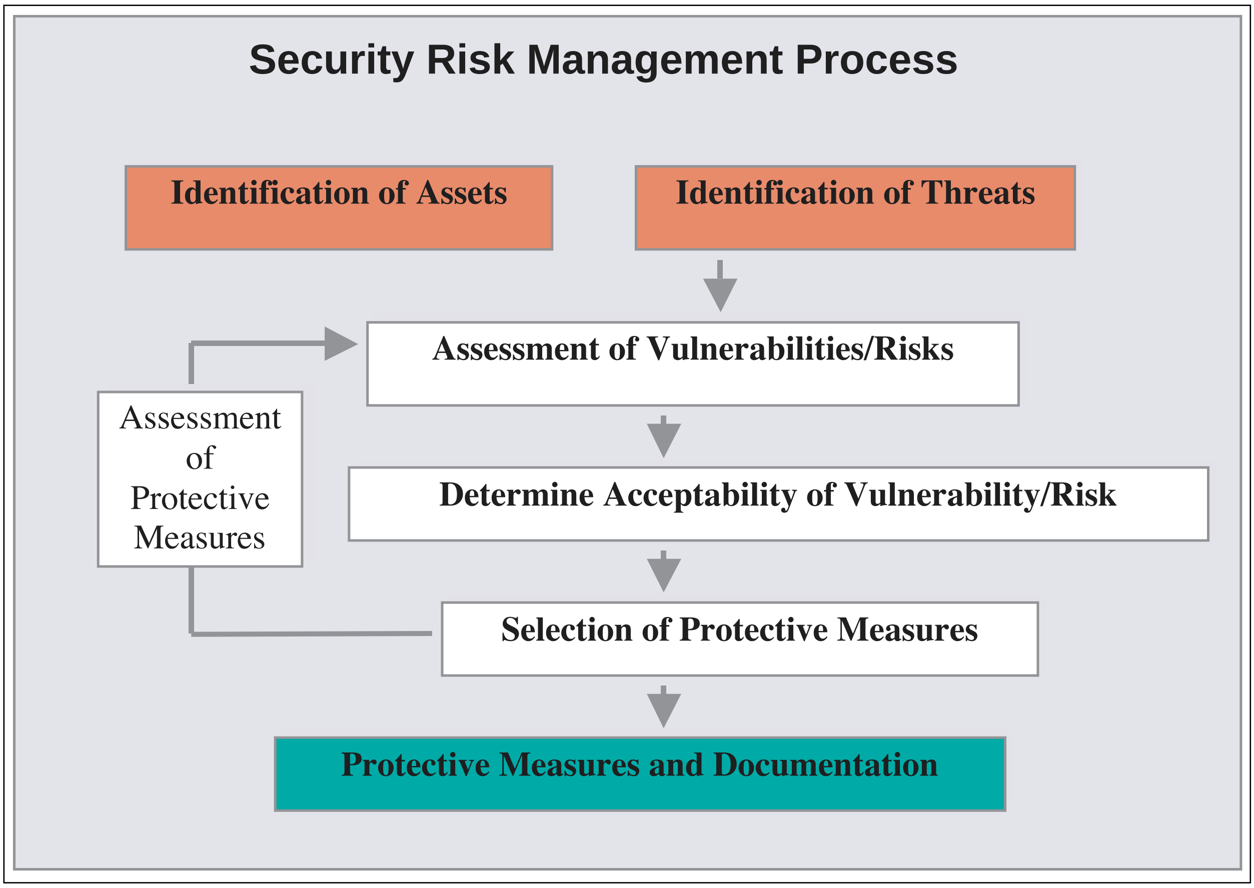
EMP/HPM (Electromagnetic Pulse/High Power Microwave)
The most vulnerable parts of the facility should be given EMP (and HPM) protection. The overburden of 20 meters + will give a considerable attenuation in the relevant HPM wave-bands which reduces the cost of total protection against such weapons.

1.4.8. Ground Shock Protection
The most vulnerable parts of the facility should be given protection against ground shock. One cost-efficient way is to situate all such vulnerable assets like Operations Centre, Fire Control Centre, Communications Centre, Crew accommodation etc in containers of standard ISO-size. By mounting them on coil type shock absorbers, the remaining ground shock inside a container, from any conventional weapon, will be negligible. Full-scale tests have verified this.
By using such containers, most of the technical installation can be done in the factory, greatly reducing the cost of installation, as they arrive at the site fully equipped. They can easily be returned to the factory for major system renovations, or moved to another site, if required (strategic mobility).
In addition, by using containers, the required EMP/HPM-protection could easily be accomplished.

Internal Environment
One very important factor in underground facilities is the internal environment. In-rock facilities are often associated with dark tunnels, water dripping from the ceiling, with maximum humidity and corrosion problems. These problems can in most cases be solved by covering the walls and ceiling with a PVC-lining which is held in place by cables supported by rock anchors. The advantage of this is that it is easy to control the relative humidity. Other advantages are that it is light-colored (white), thus reducing the lighting necessary to give the required levels of illumination, it is also robust, but can easily be repaired if damaged, and is easy to maintain.
The PVC-lining will also be able to withstand rock falls of up to 300 kg falling from 1 meter.

FINAL REMARKS

It is important to emphasise that the presented criteria and principles are guidance for those responsible for planning or managing such facilities. The exact design must be based on the military concept of operation and relevant threat assessment which will be subject to a complete Risk and Vulnerability Assessment in order to focus on the most critical components of the actual asset. This will then include the perimeter defence and access control.



UNDERGROUND FACILITIES FOR WASTEWATER TREATMENT – WHY BUILD THIS TYPE OF PLANTS IN EXCAVATED ROCK CAVERNS?

Per Arne Rønning

ABSTRACT

There are several advantages by placing wastewater installations in rock caverns. If suitable location and adequate rock quality is available this method is a powerful contribution to cost reduction and net positive environmental impact. By utilising the underground, the facilities may be located near to or under city centres or urban areas. Underground facilities can offer steady conditions to the treatment processes and construction work without foundation problems, independent of city traffic and surface structures.

These types of facilities are widely used in Norway, Sweden and Finland. At the South West coast of Norway an inter-municipal wastewater system has been constructed (the IVAR project). The system comprises underground trunk tunnel systems and a wastewater treatment plant in rock caverns. Treated wastewater is discharged through a subsea tunnel and a deep sea outfall. The plant capacity is 300 000 m3 per day. The plant was put into operation in 1992. Extensive experience from thirteen years of operation exists.

FACILITIES FOR WASTEWATER TREATMENT – WHY GO UNDERGROUND?

Location aspects favouring underground solutions.

- An underground facility may offer central location, near consumers, near or under city centres or urban areas resulting in short collection tunnels and pipelines.
- Tunnels and caverns do not occupy valuable land or construction sites.
- Tunnelling does not interfere with surface structures or street traffic.
- Accidental leakages and spill can be contained.
- Most rock masses are excellent construction materials – strong, durable, abundant and cheap.

Operational aspects favouring underground solutions.

- Collection tunnels act as effective retention volumes, that even out flow fluctuations. The treatment plant can be dimensioned to average flows, rather than daily peak discharge, that reduces the size of the plant significantly. In addition, steady flows are most favourable to the treatment process.
- No sunlight, no wind and steady temperature inside the caverns provide limited algae growth in channels and basins, sedimentation is not disturbed by waves from wind. With steady conditions the purification process can be optimised.
- No extra costs of deep (versus shallow) sedimentation basins. Deep basins yield better particle separation.

However:

- A minimum flow is required for self-cleaning of collection tunnels.
- Too long retention times in tunnel may develop hydrogen sulphide.

Design and construction aspects favouring underground solutions.

- Freedom in choice of geometry and dimensions at rock of adequate quality.
- Rock caverns are natural containers. Simple structural elements are needed and no foundation problem exists.
- Construction work can be performed independent of city traffic, surface structures or bad weather.
- Little or no disturbance to neighbours and city life.

However:

- Suitable location and access to tunnel adits are necessary.
- Transport of rock spoil to deposit may cause disturbance during construction.
- Rock caverns offer confined space for construction work.

Environmental impact aspects favouring underground solutions.
- The wastewater treatment plant will be hidden – out of sight with no visible lagoons, tanks or buildings.
- Underground plants are reliable, with well-proven results. Odours, noise and possible leakages can be contained.

However:
- A well-designed ventilation system is required.
- Deposit of rock spoil from excavation is normally an asset, but may cause a problem.

Economic aspects favouring underground solutions.
- The size of the wastewater treatment plant can be reduced due to effective retention volumes in collection and inflow tunnels.
- Central location is economically favourable.
- Investments in land acquisition will be low.
- Costs of interference and repair of existing city facilities will be low.
- Reduced costs of plant maintenance (due to no weathering).
- Plants situated in rock caverns provide long life expectancy.

However:
- Excavation and rock support costs may be on the high side.
- Ventilation and lighting costs will increase compared to a plant in open air.

Basic preconditions for placing wastewater treatment plants in rock caverns.
- Suitable topography.
- Adequate rock quality.
- Designers with proven skills and experience.
- Competent contractors.

AN EXAMPLE:
The Central Wastewater Treatment Plant for the North-Jaeren Region in Norway. (Stavanger area)

GENERAL
In March 1991 the Regional Water, Sewerage and Waste Disposal Company (IVAR) at the South West coast of Norway put its inter-municipal wastewater system, comprising an underground trunk tunnel systems and a wastewater treatment plant in solid rock caverns, into operation. The wastewater system was constructed to meet national and international discharge requirements.

The total construction cost for the entire plant comprising trunk tunnel, wastewater treatment plant, subsea outlet tunnel, emergency overflow tunnel, the comprehensive sludge processing plant, workshop and administration was NOK 420 million (equivalent to USD 60 million) in 1992 currency.

Previously, untreated wastewater was discharged to nearby fiords outlets. The main requirements for a wastewater collection system were to intersect the large number of existing mains and two tunnels discharging into the sea, and transport the wastewater to a planned regional wastewater treatment plant. The existing sewers included both old pipelines containing mixed foul sewage and surface runoff as well as new separate systems.

The wastewater treatment plant is located 10 km outside the city of Stavanger and treats wastewater from five municipalities. Plant capacity is 300 000 m3 per day.

CONCEPT DEVELOPMENT

General
By the time the conceptual design was carried out extensive experience from operation of a trunk tunnel system and a regional treatment plant for the Western Greater Oslo Region in Norway did exist.

The plant concept was carried out along the same lines as for the Oslo plant, however taking in account development and improvements from more than ten years of operation of this plant.

In the project area the bedrock is mainly phyllite that is characterized as an impervious rock.

Transportation system
During the preliminary design stage, the main principle of an efficient transport system was developed. A trunk tunnel located close to the shore line would make it possible to intersect all existing sewers by gravitational flow. Other possibilities were also studied.

An important basis for the choice between alternatives was the ground conditions of the area. In addition, the concept development involved a number of other parameters to be considered:
- Operation and maintenance cost
- Available technology
- Environmental matters
- Time of completion

Based on thorough evaluations and considerations a tunnel excavated in rock was selected for the conceptual design. The design followed a few simple and basic principles:
- The flow should be by gravity, and the intersecting system should consist of only a few main pumping stations. Existing old pumping stations should also be abolished, if possible.

- The system should consist of a tunnel close to the shoreline, and at a level low enough to catch existing main sewers by gravity.
- Self-cleaning effect in the tunnel should be achieved.
- To keep costs at a reasonable level, the tunnel should be situated in solid rock.
- With the selected tunnel dimensions, storage capacity for levelling out the daily variation and peak flows was achieved resulting in reduced size of the following treatment plant that could be designed for average flows, rather than daily peak discharge. Further, steady flow to the treatment plant was achieved, which are favourable to the treatment process.
- The method of excavation based on tunnel boring machines (TBM) had the following advantages:
  - favourable hydraulic gradients and cross-sections
  - short construction time
  - no environmental impact
  - better conditions for making the tunnel watertight
  - favourable cost
- Principally the tunnel should be unlined. Rock support and reinforcement design should be based on "design-as-you-go" principle, using mainly rock bolts and shotcrete.

Wastewater treatment plant.
During the design stages and feasibility studies, aspects such as land requirements, environmental aspects, construction and operational costs were assessed, and a possible site at the Håfjell hill was selected.

From topographical and geological factors and parameters, an optimal location and orientation of caverns and interconnecting tunnel system was found and design parameters as maximum width, length and height were assessed. The maximum span width of caverns is 16 m.

Outfall tunnel, outfall and emergency overflow tunnel.
Location and design of the outfall system in the North Sea was a challenge. From thorough studies and evaluations, location of the outfall for treated wastewater was selected at a distance of 1 600 m off shore, and at depth of 80 m below sea level.

Basically, two different concepts were evaluated:
• Subsea polyethylene pipes anchored to sea bottom.
• Subsea tunnel and the outfall constructed according to the principles of the Norwegian Lake Tap method.

Because of the extremely rough weather conditions in the area the polyethylene pipe concept was rejected, and the subsea tunnel alternative was selected. As a precaution a 600 m long emergency overflow tunnel from the treatment plant to 30 m depth was designed.

CONCEPTUAL LAYOUT AND ARRANGEMENTS.

Transportation system.
The transportation system is shown in Fig. 3.1. An 8.1 km long inlet tunnel to the treatment plant was designed. Wastewater from main city centres is conveyed to the inlet tunnel by two branches of the tunnel system and the main sewer pipelines in the catchment area carry wastewater to a limited number of feeding points to the tunnels. The diameter of the inlet tunnel is 3.5 m and the inclination is 1.1 o/oo.

Fig. 3.1.

At the central treatment plant the wastewater is lifted 20 metres into the screens and grit removal chamber by dry-mounted pumps. Facilities for overflow have been installed for the tunnel system at the inlet to the treatment plant. A ventilation system has been installed to prevent odour discharge in the vicinity of the feeding points and to ensure an acceptable environment for inspection and maintenance crew.

Treatment plant.
Fig. 3.2 shows an artists presentation of the plant layout. The underground area is divided in two areas:
1. Traffic area where all transport and unloading of chemicals and loading of grits and screenings take place. This is obtained by roundabouts for trucks and arrangements with an automatic container handling system.

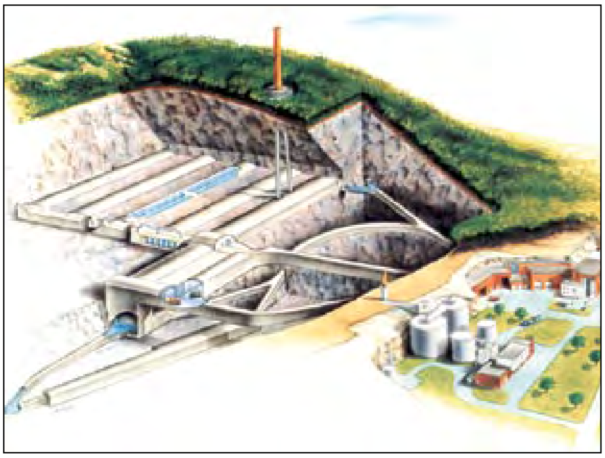


Fig. 3.2.

2. The treatment plant itself with screens and grit chambers located in one common rock cavern, eight parallel settling tanks located in four caverns and one large cavern for heating and ventilation.

A workshop and an administration building are constructed outside the treatment plant in open air.

In order to meet stricter requirements for sludge treatment in the future, a comprehensive sludge processing plant was installed. To avoid interference with the ongoing construction work underground, the sludge processing plant was installed outdoor.

Excavation of caverns and tunnels for the treatment plant gave a rock volume of 300,000 m3. The rock spoil could be utilized as landfill for a new industrial area in the immediate vicinity.

CONSTRUCTION

Transportation system.
The tunnel construction work was carried out under three contracts, one contract for TBM excavation of the trunk tunnel, one contract for the subsea outlet tunnel and one contract for the emergency overflow tunnel.

The rock conditions for TBM excavation were mainly excellent with medium strength and low content of wearing minerals. Minor zones with high content wearing minerals were met. The construction work was completed in 13 months.

The 4.1 km subsea outfall tunnel with a cross section of 20 m2 and inclination varying between 1 and 100 o/oo was excavated by conventional drill and blast technique with limited rock support by rock bolts.

The outfall, at 80 metres below sea level and 1.6 km off shore in the North Sea, was constructed according to the principles of the Norwegian Lake Tap method. It is necessary to establish correct information of rock quality and sea bottom surface details for outfall design and construction. Hence comprehensive exploratory acoustic seismic survey, test drilling and follow-up was required. In this tunnel the rock support consists of approximately 250 scattered rock bolts, 60 m3 sprayed concrete and grouting of the outlet with 140 tons of cement. The construction work was completed in about 15 months.

The 600 m long emergency overflow tunnel was constructed to a depth of 30 m below sea level. The final blast was done in a water filled tunnel.

The wastewater treatment plant.
Approximately 100,000 m3 rock excavation was carried out for the treatment plant. The rock support was performed by systematic rock bolting and sprayed concrete. In areas with electromechanical equipment steel plate drip screens have been installed. Waterways and basins are built using in situ concrete. The concrete and steel work of the treatment plant was a challenge to the contractor, both in respect of the complexity of the work and the tight time schedule.

In total, it took 24 months to complete the civil work, including surface treatment of floors, channels and basins.

OPERATION EXPERIENCE.

The tunnel system and the central treatment plant including outlet and emergency overflow tunnels were commissioned in March 1992.

Inspections have verified that no problems of solid waste sedimentation in the tunnel systems exist. The system has worked satisfactorily since the start of operation. The overall experience with the underground plant at the South West coast of Norway has been good. There has been a minimum use of land above ground and there is full control of discharges to sea and air without any odour problems.

The sprayed concrete has been inspected and is in excellent condition.

By transferring direct wastewater outlets via the tunnel system and the wastewater treatment plant, the water quality in the harbour areas of Stavanger and the nearby fiords has improved significantly. The inter-municipal wastewater system has been a powerful and cost effective contribution to environmental improvements for the entire region.

NEW OSET WATER TREATMENT PLANT FACILITIES SITUATED UNDERGROUND

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ABSTRACT
Establishing water treatment installations in excavated rock caverns underground has become a common practice in Norway. Provided that a suitable location with adequate rock quality for excavation of larger tunnel spans exists, an underground location of such installations has many advantages compared to an aboveground location. Besides the contribution to cost reduction and net positive environmental impact, the underground location has a huge security advantage considering outside threats and possible terrorist attacks as well as protection against contamination. By utilising the underground, the facilities may be located near to the raw water reservoir, near or under city centres or urban areas. Underground facilities can offer steady conditions to the treatment processes and construction work without major foundation problems and independence of surface structures.

These types of facilities are widely used in the Nordic countries of Norway, Sweden and Finland. At the urban border of Oslo City, a new water treatment facility is being constructed in rock in the near vicinity of the existing main treatment plant for Oslo Municipality, also situated underground. The new plant consists of two main caverns with a length of 150 m, and several access tunnels. The span of each cavern is 27 m and the height varying between 16 m and 20 m. The pillar between the caverns is 17 m wide. The blasted solid rock volume is 140.000 m3. Combination of optimisation of large spans, compacting of the underground complex, minimal rock cover and the process plant design in the early pre-construction phases has been one of the major challenges for the project. The new plant will have a capacity of 390 000 m3 of potable water per day. The commissioning date for the new plant is scheduled in May 2008.

INTRODUCTION
Situating water treatment installations underground in excavated rock caverns has become common practice in Norway for water works serving large municipalities or inter-municipal collaboration projects.

Provided that a suitable location and adequate rock quality for excavation of larger tunnel spans exists, an underground location of such installations has many advantages compared to an aboveground location. Besides the contribution to cost reduction and net positive environmental impact, the underground location has a huge security advantage considering outside threats and possible terrorist attacks as well as protection against contamination.

By utilising the underground, the facilities may be located near the raw water reservoir, near or under city centres or urban areas. Underground facilities offer steady conditions to the treatment processes and construction work without major foundation problems and independence of surface structures.

Oslo Municipality has four water treatment plants in operation today. Oslo Municipality, Water and Sewage Works are responsible for the operation. The two major plants, Oset and Skullerud, are both situated underground in excavated rock caverns.

In 2003 it was finally decided to renew and enlarge the water treatment plant at Oset. The consortium AFS – Krüger (consisting of the Norwegian construction company AF Spesialprosjekt AS and the Danish company Krüger A/S) are building the New Oset Water Treatment Plant on behalf of Oslo Municipality. The plant shall be designed and constructed as an EPC contract (turn-key), where the Norwegian consulting company Norconsult AS is the responsible civil and structural engineering consultant.

The New Oset Water Treatment Plant will be Scandinavia's largest water treatment plant and Europe's largest located in excavated caverns, when put into operation in May 2008. It is designed for a production of 390.000 m3 potable water per day; equivalent to 4,5 m3/sec and will deliver water to 85 % of Oslo's inhabitants.

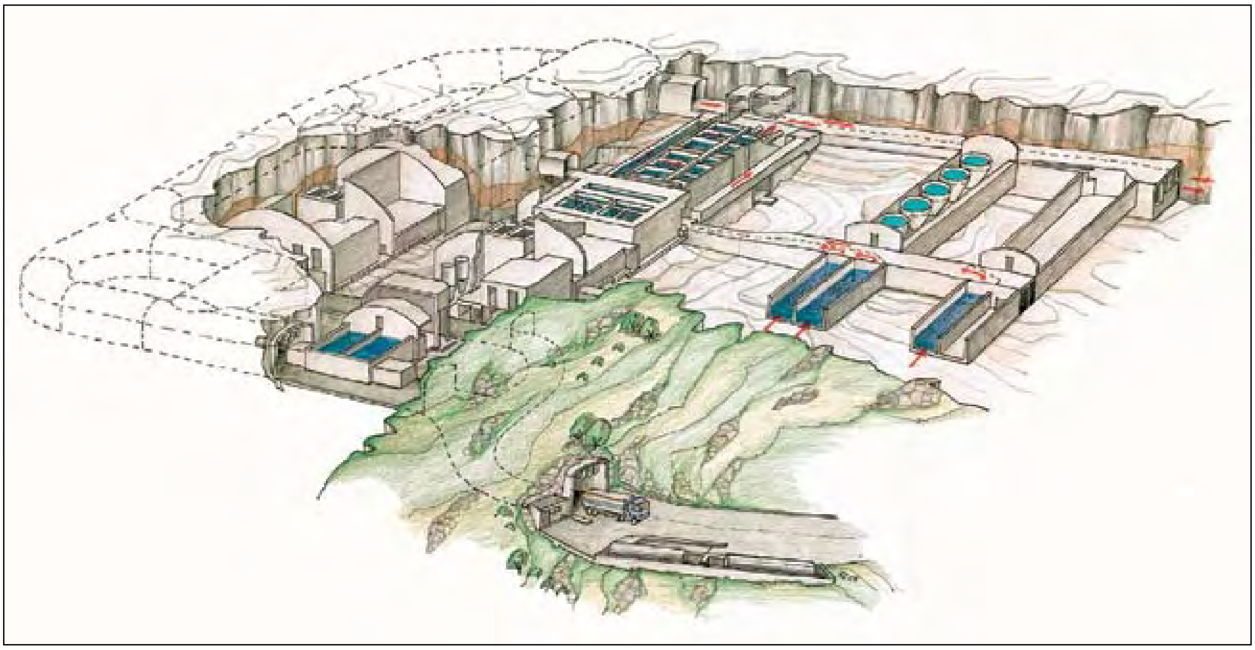

Figure 1: The new plant and its access tunnel portal to the left. Part of the existing plant to the right. Illustration: Krüger A/S

THE EXISTING PLANT

The existing Oset water treatment plant is located in several rock caverns east of the main drinking water reservoir for Oslo Municipality, Lake Maridalsvannet. The construction works started in August 1966 and commissioning of the plant took place in September 1971. The plant covers a total floor area of 28.800 m2.
The existing plant consists of 5 parallel caverns with spans up to 13.2 m and a pillar width between the caverns of 14 m (Figure 1 and 2). The longest caverns are 92 m long with a height of 16 m. The caverns were excavated by drill and blast technique with a top gallery of height 8 m and two subsequent benches of 4 m each. The drilling rigs had 4 drills, which were handled by a working crew of 3 men each shift. The excavation rate was 500 m3 of solid rock each working day. Totally excavated volume was 344,000 m3.
The caverns are lined with concrete arches spanning from abutment to abutment. The concrete arches are dimensioned for minor rock falls.

THE NEW PLANT

Background
The decision to enlarge and renew the water treatment facility at Oset was made by the approval of Oslo Municipality's Water Supply Master Plan in 2003. The New Oset Water Treatment Plant should comply with the new Master Plan's safety requirements by dividing the new plant into two completely separated treatment facilities, which together replaces the old one, but uses some of the old caverns for new treatment purposes.
Oset Water Treatment Plant, which today produces approximately 85% of the potable water consumed in Oslo, has its raw water supply from the nearby Lake Maridalsvannet. The plant shall be upgraded to meet the new and stricter requirement set by European and Norwegian drinking water regulations. That includes the running of the treatment process through two hygienic barriers.

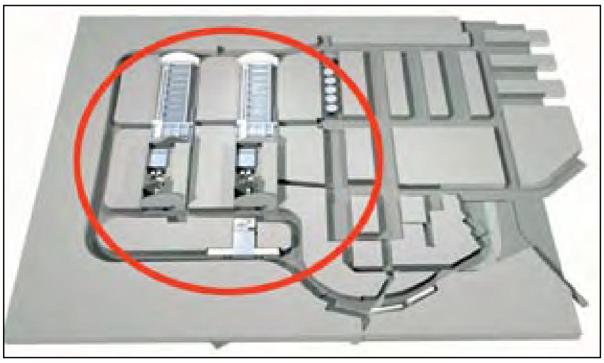

Figure 2: The new plant connected to the existing plant. Illustration: Consortium AFS-Krüger.

WATER TREATMENT AND USE OF ALTERNATIVE SOURCES FOR RAW WATER SUPPLY

The new plant treats the water by use of the Actiflo™ process, which in combination with UV-disinfection gives good hygienic assurance and maximum flexibility with a view to optional raw water sources.
The New Oset Water Treatment Plant shall be completed and ready for commissioning by May 2008. The completion of the project will result in two independent water treatment plants, which have an optimum close

co-localisation in rock with a total treatment capacity of 390,000 m3 water per day. Two minor existing treatment plants in the Oslo area will render superfluous with the commissioning of the new plant.
The two new independent plants will, from a process technically view, be built in a way that allow for replacement of vital components while the plant is running without reducing the production capacity. Both plants shall be able to handle and treat water from both the Maridalsvannet Lake and another possible water source concurrently, and with optional proportion of mixture between the sources.

GEOLOGICAL CONSIDERATIONS AND ROCK ENGINEERING ASPECTS

Both the existing and the new water treatment are situated in the same geological formation. The predominant rock type is a local variation of an alkali-syenite commonly found in the Oslo area. Weakness zones and joint systems are typically steep and have strike in the North-South direction. Other observed joint systems are subhorizontal. Intrusions of diabase and syenite-porphyry appear at regular intervals as lenses in the rock mass often in parallel with main joint system direction.
Dimensions and spans of the underground excavations were governed by the overall desire to make the new treatment plant compact. Requirements given in the contract were that the stability of the existing plant should not deteriorate, and as a guideline it was suggested that the pillar width should not be less than 2/3 of the cavern span and as a minimum requirement not less than 10 m. Large cleaning filters that were to be mounted across the cavern cross-sections were dimensioning the minimum spans. This gave in the first place spans in the magnitude of 30 m. Due to several crosscuts between the caverns it would be cost improving to minimise the distance between the caverns.

The exact location and geometrical layout of the plant was governed by the requirement of best possible cooperation with the existing treatment plant. In order to ensure good access and straight connection axes to the existing plant, the new plant was located just North of the existing and as close as possible towards the hillside surface coming up from the Maridalsvannet raw water reservoir (Figure 3).

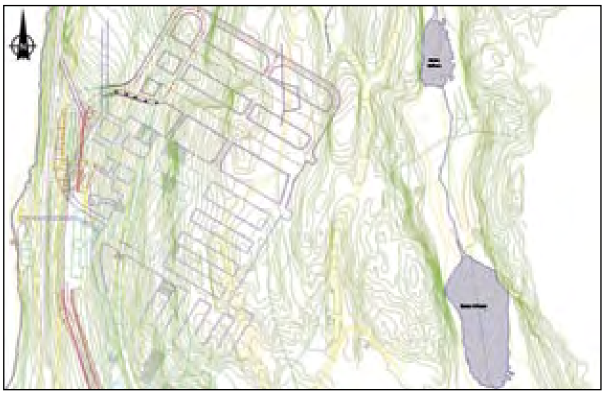

Figure 3: Location of the plant between Maridalsvannet (West) and Småvannene (East). Illustration: Norconsult AS.

Final layout and location was considered based on rock cover, rock stress conditions and intersecting weakness zones (Figure 4). The chosen location was considered favourable with regards to predicted water ingress and possible negative environmental impact on the two small lakes, Småvannene, situated to the East at the top of the hill.
Contractual upper limit for tolerable water ingress to the entire new underground complex was set to 100 litres/min.

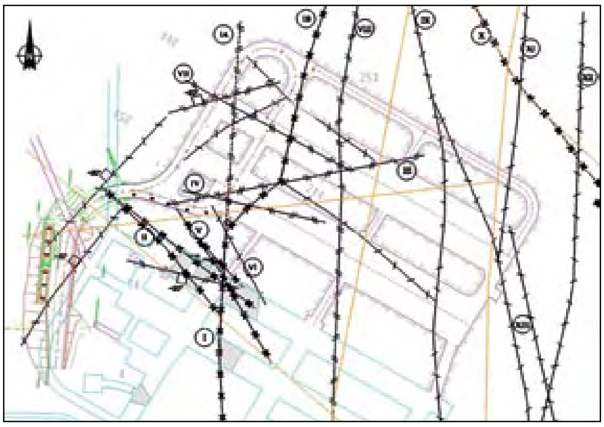

Figure 4: Observed weakness zones and geological structures at the surface prior to excavation. Illustration: Norconsult AS.

CAVERN LAYOUT AND PILLAR WIDTH

In order to minimise the length of crosscut tunnels both numerical and analytical analyses has been utilised to optimise the design. For the numerical analyses the 2-dimensional program Phase2 ™ (Finite Element Method) has been utilised. The model is shown in Figure 5. Analyses have been run with three different rock mass quality parameters and three different pillar widths.
Both the analytical and the numerical analyses showed that a pillar width of 15 m (Figure 6) gives satisfactory global stability for both the existing and new underground facility. These findings applied also for abnormally unfavourable rock stress conditions and rock mass quality poorer than what were expected from the pre-investigations. In the final cavern layout the pillar width has been set to 17 m in order to allow for overbreak in the contour.
The analyses also showed that rock bolt lengths of 4 m in the pillar and 6 m in the crown were sufficient to maintain the stability of the underground opening.

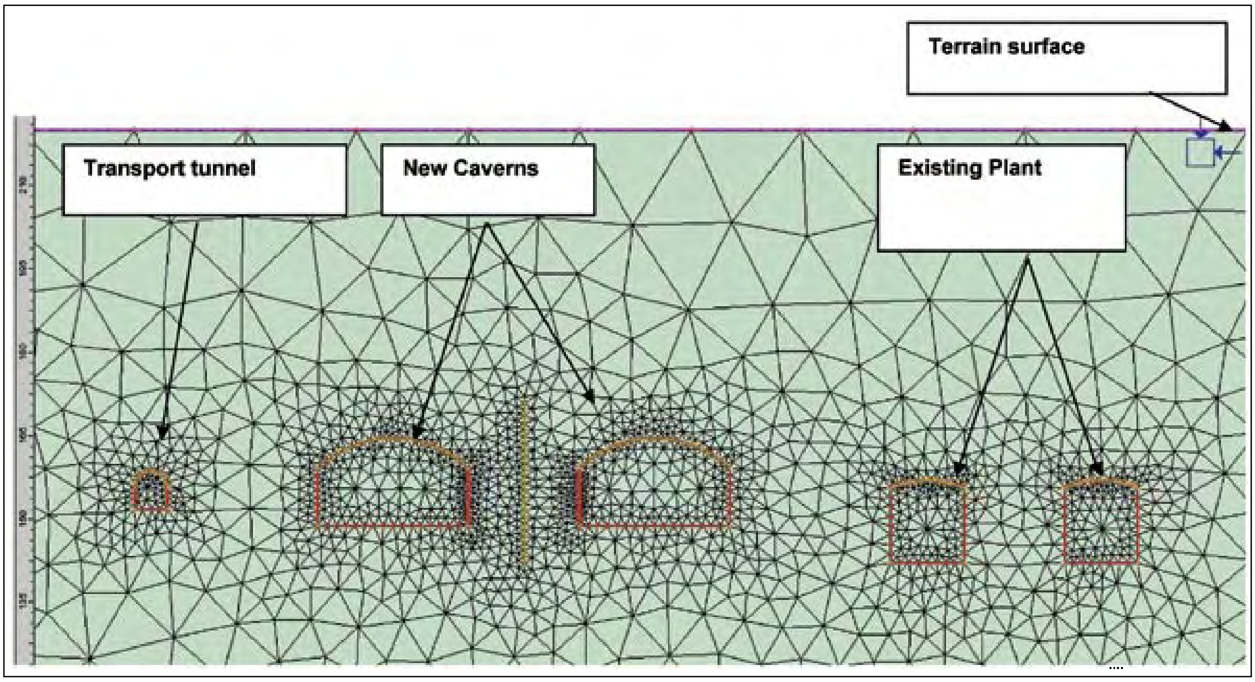

Figure 5: Extract of Finite Element model from Phase2. Illustration: Norconsult AS.

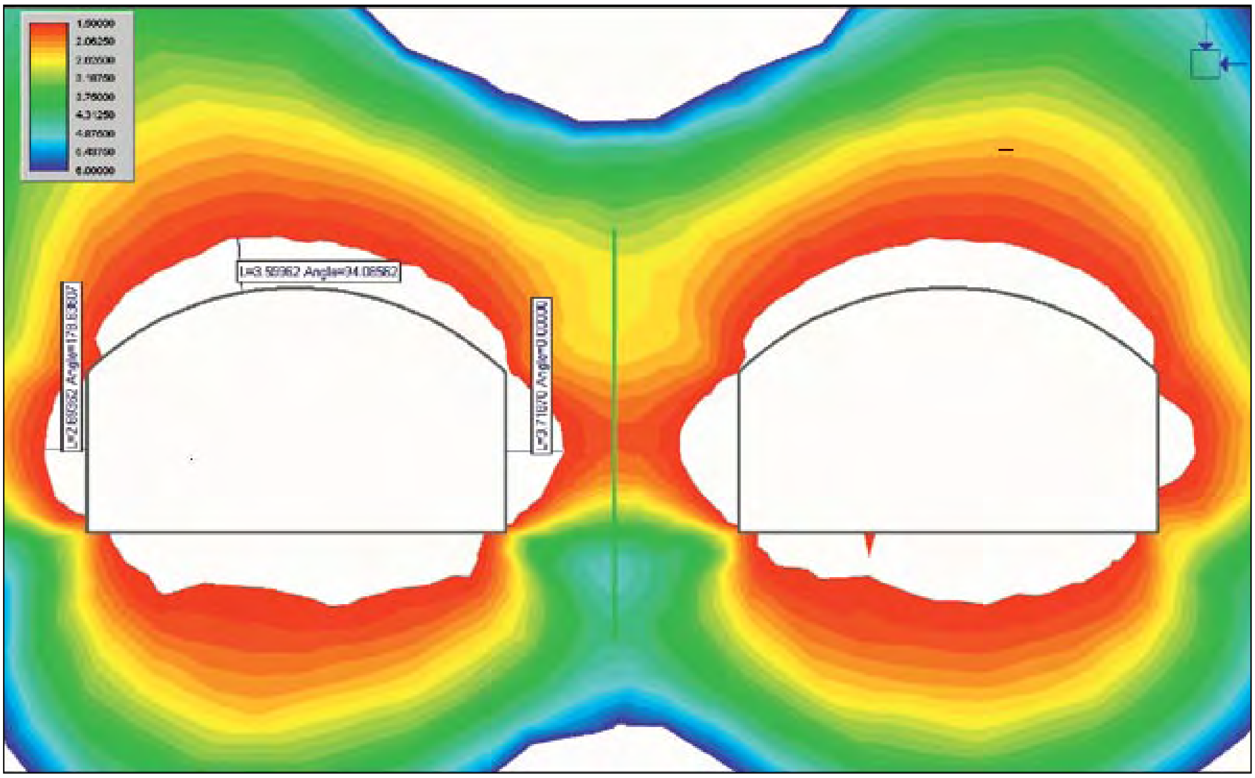

Figure 6: Factor of safety against failure (minimum 1.5) for 15 m pillar width from Phase2. Illustration: Norconsult AS.

GEOLOGICAL FOLLOW-UP DURING CONSTRUCTION

The contractor's experienced engineering geologists performed continuous mapping of geological structures and rock mass conditions in parallel with the excavation progress.

As a requirement from the Client, the mapping was conducted in accordance with the Q-system and the result of the geological mapping is shown in Figure 7.
Dependent upon the actual observed Q-value, rock support was applied in accordance with 5 pre-determined rock support classes (Figure 8).

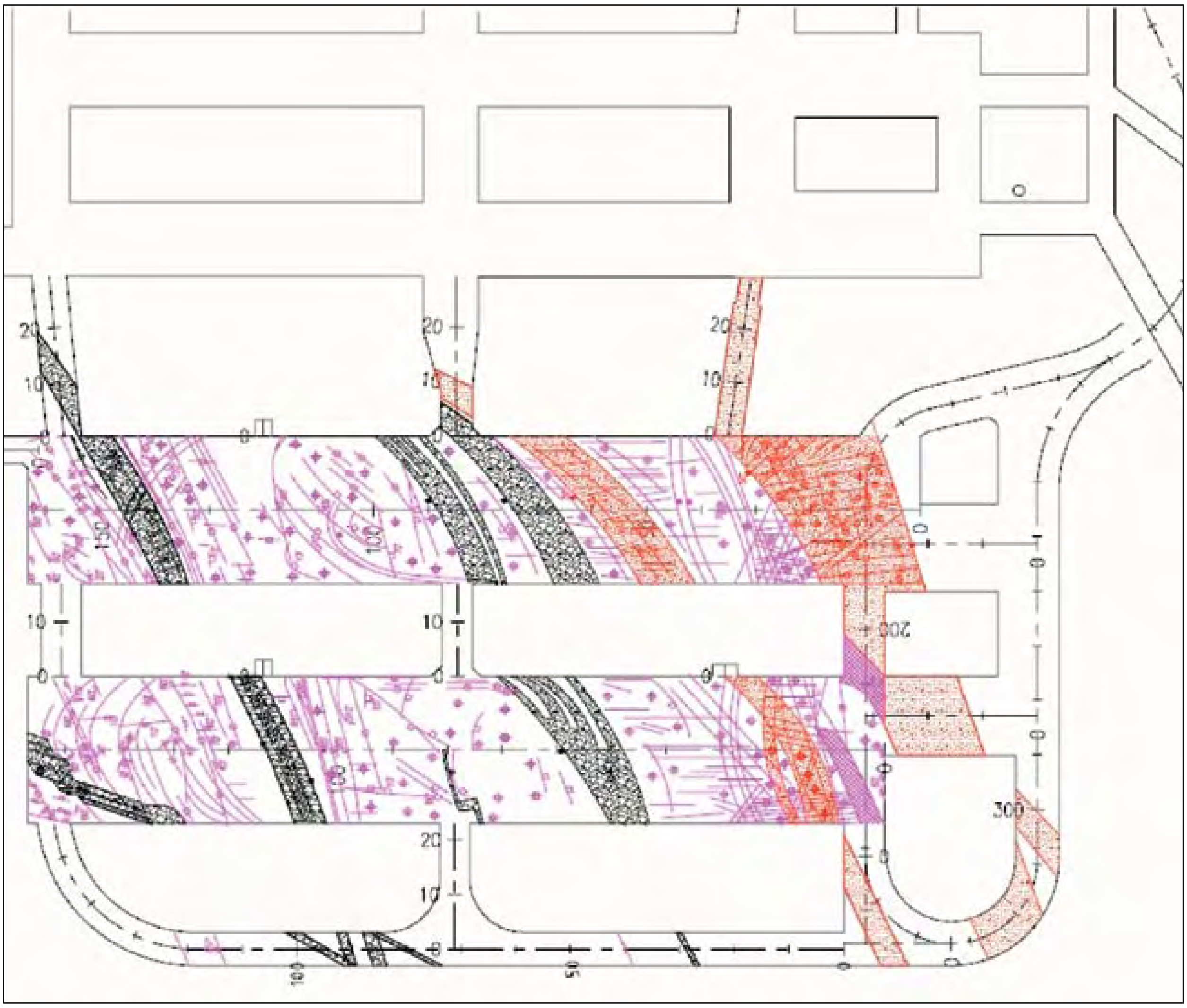

Figure 7: Geological mapping of the new underground facility. Illustration: AF Spesialprosjekt AS/Norconsult AS.

CAVERNS A = 370 - 405 m² Arch length, L = 31 m Wall height, H = 9,3 m

Class	Q-value	Bolt Pattern			Bolt Density		Bolt Length				Sprayed Concrete		RRS
		lm	Crown	Wall	Crown	Wall	Mid crown	Side crown	Pillar wall	Outer wall	Crown	Wall	c-c
SK I	> 40	3,5 m x	3,5 m	3,5 m	0,08	0,08	5 m	4 m	3 m	2,4 m	60 mm	60 mm	-
SK II	10 - 40	2,5 m x	2,5 m	3,0 m	0,16	0,13	5 m	4 m	4 m	3 m	60 mm	60 mm	-
SK III	1 - 10	2,0 m x	2,0 m	2,25 m	0,25	0,22	6 m	5 m	4 m	3 m	100 mm	60 mm	-
SK IV	0,1 - 1	1,5 m x	1,5 m	1,75 m	0,44	0,38	6 m	5 m	4 m	3 m	150 mm	120 mm	1,5 m
SK V	< 0,1	1,0 m x	1,0 m	1,0 m	1,00	1,00	7 m	6 m	5 m	4 m	250 mm	250 mm	1,0 m

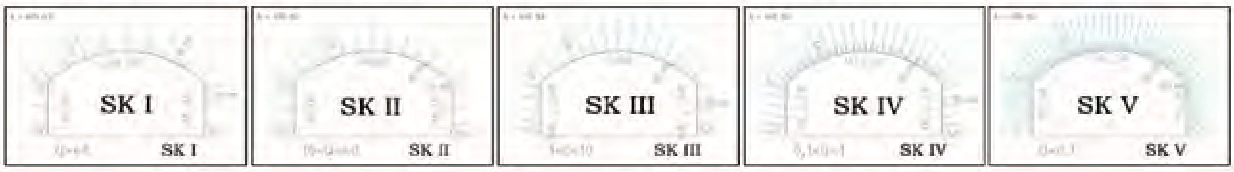

Figure 8: Rock Support Classes for caverns. Illustration: Norconsult AS.

The registered rock mass quality had the following distribution:

Rock Support Class SK I	Q > 40	smaller portions of the transport and access tunnels
Rock Support Class SK II	Q = 10 – 40	ca 70 % of the caverns
Rock Support Class SK III	Q = 1.0 – 10	ca 30 % of the caverns close to weakness zones
Rock Support Class SK IV	Q = 0.1 – 1.0	only occasionally
Rock Support Class SK V	Q < 0.1	not registered

A total of 3,500 m3 of steel fibre reinforced sprayed concrete and 6,000 CT-bolts has been applied as permanent support at the face in crown and walls of the underground facility.

ROCK EXCAVATION

Excavation Methodology

The two caverns have been excavated by drill & blast technique with an approximately 9 m high top gallery. The gallery was sub-divided at the middle of the 27 m wide cross-section with a pilot face one blast round (5 m) ahead of the slash face in a one-way excavation direction from the access tunnel side. The remaining bench was excavated in the same direction as the gallery by means of horizontal drilling and blasting. The caverns were excavated in parallel with the transport tunnel. This methodology gave in fact 5 tunnel faces, which gave sufficient flexibility for the pre-grouting works to go on independently of the rock excavation, as there were always 3 faces to choose between for drilling, mucking out or rock support works.

Control of Blast Induced Vibrations

Strict blasting vibration requirements applied for the performance of blasting operations. The limits set in the contract was a PPV of 25 mm/s for the concrete structures and a PPV of 20 mm/s for the electrical switchboards in the existing treatment plant. The typical rock type in the neighbouring area towards the existing plant was relatively stiff and transmitted the wave propagation induced by blasting well. The existence of unfavourably orientated weakness zones in the pillar rock mass resulted in relative displacement and aggregation of vibrations at certain locations.

The main challenges related to vibration control were therefore blasting close to the existing plant. All blast rounds less than 30 m away from the structures inside the existing plant were performed with a high degree of scattering in detonator numbers in order to reduce the charge per delay interval and use of connector blocks to sub-divide the delays further. Explosives in tube cartridges were commonly used at the most critical locations.

PROJECT IMPLEMENTATION

Process Plant Design and Rock Excavation in Parallel - Experiences And Challenges

Figure 9: Bench excavation by horizontal drilling and blasting. The top gallery is completed.. Photo: C. .F. Wesenberg.

Rock excavation for the underground facilities started up prior to the completion of the process plant design. This situation was especially challenging during the rock engineering design phase when optimisation of the project implied aiming for a compact underground facility with minimised rock excavation volumes at the same time as the process plant design was in the early stages of basic conceptual design.

The experience gained is that the underground excavations could have been slightly larger in order to facilitate more design freedom for the process plant and other disciplines with regards to optimised technical and economical solutions.

Rock Excavation close to Plant in Operation - Experiences and Challenges

The new plant is situated adjacent to the existing plant with a constant distance of 25 m to the closest cavern. The portal area for the access to the new plant was situated less than 10 m above the main existing raw water supply tunnel in full operation. In addition two major weakness zones were intersecting in the same area. At three locations crosscut tunnels were to be excavated into the existing plant in full operation.

Maintaining the operation of the existing plant was one of the Clients main concerns and a stop in, or contamination of the water production causing supply cut-off for the consumers was therefore subject to a hefty fine of 5 MNOK per undesired event. So far the ongoing construction works have not caused any interruption in the water production.

Water Control by Rock Mass Pre-grouting - Experiences

The rock cover above the caverns varies between 20 m near the entrance to 70 m in the inner part of the plant. Systematic rock mass pre-grouting has been performed throughout the entire plant. Only cement-based grouting material has been used.

In the tunnels a standard procedure for pre-grouting involving drilling of grouting fans consisting of 21 m long holes densely spaced around the whole profile with a look-out of 5 m for every third blast (distance 15 m) has been followed.

In the caverns all pre-grouting has been carried out from the top gallery. This caused a large look-out angle in the bottom of the fan in order to reach and cover the rock mass below the lower bench. Due to the large cross-section the fans were also vertically divided into two grouting rig positions enabling the excavation work to start at the first grouted side of the face while grouting was completed at the other.

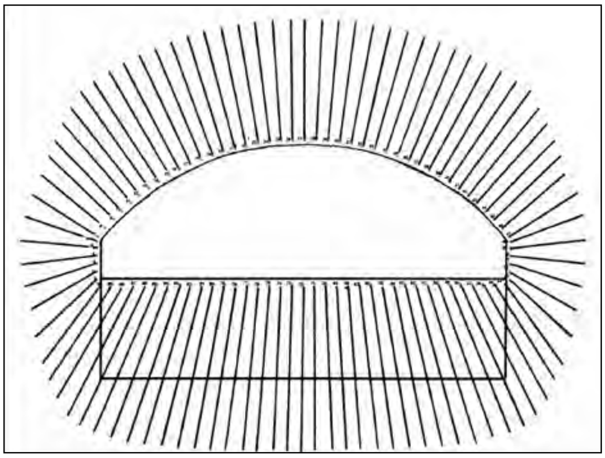

Figure 10: Typical drilling pattern for rock mass grouting fan in caverns. Illustration: AF Spesialprosjekt AS.

The contractor AFS in close dialog with the Client's representative elaborated all grouting mix design. Due to a rock mass containing a high portion of very fine fissures all grouting rounds were started with micro-cement and if pressure build-up was achieved the round was completed with micro-cement. In the case of no pressure build-up after a pre-determined grout take amount was reached, the round was completed with rapid cement and possible use of controlled grout setting by means of accelerator added at the nozzle.

The stop criteria utilised through out the project was a modified GIN-principle criterion with maximum pressures of 40 bars in areas with little rock cover or in the vicinity of the existing plant and 65 bars elsewhere.

In certain zones, often related to lenses of diabase (black zones in Figure 7 and zones marked VIII and IX of Figure 4), water leakage from the face, prior to grouting, was reported as high as 100 litres/min. Control measurement of water inflow to the whole new underground complex after completed excavation shows water ingress of in total 27 litres/min.

Compared to the pre-set requirement of maximum 100 litres/min, the result of the grouting operations is considered to be more than satisfactory. Incidentally, the rock mass grouting works were the only part of the contract that was quantity and unit-price regulated. The total grouting figures amounted to 2,300 tons of cement, which correspond to a grout take of 16 kg per m3 of excavated rock.

Rock Mass Deformation Monitoring - Experiences

In order to document "stable-state" conditions in the caverns, displacements in the crown were measured by means of installed measuring bolts in monitoring stations every 30 m. The exact position of the measuring bolts was measured with the surveying total station.

Now, 11 months after the excavation work was completed and some 16 months after the monitoring stations were installed, "stable-state" conditions has been observed over a 15 months long period. The displacements measured are small and within the range of inaccuracy for this optical monitoring system.

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UNDERGROUND WASTE STORAGE CONCEPTS – FOR PROTECTION OF THE ENVIRONMENT

Waste products represent an increasing problem for the modern society. By utilising the underground space, new opportunities are opened to solve environmental problems.

Going underground the rock mass constitutes an additional barrier preventing leakages and provides protection from natural disasters and human activities, such as war and sabotage. Provided good planning and skilled engineering, underground storage concepts represent safe alternatives for environmental protection.

Environmental Impact Assessment, Environmental Risk Assessments and last but not least, Environmental Monitoring Programmes are important tools to evaluate the long-term safety of underground storage concepts.

In Norway, the underground waste storage concept has been implemented at several locations, such as the rock store caverns at Falconbridge in Kristiansand, Boliden Odda, Stendafjellet, Lonevåg, and the Himdalen repository for low and medium radioactive waste near Oslo.

At the Himdalen repository, low and medium radioactive waste is stored safely in concrete sarcophagus in rock caverns. Environmental Impact Assessments including risk analysis, safety and security studies in a 2000 years perspective were performed to verify and establish a basis for concept and design.

Portal of the Himdalen repository outside Oslo

UNDERGROUND WASTE STORAGE CONCEPTS AT BOLIDEN ODDA AS, NORWAY

Jan K.G.Rohde
Frode Arnesen
Endre Lægreid

ABSTRACT

Waste products called jarosite residue, from the zinc production at Boliden Odda AS, have previously been discharged directly into the sea. Besides activities from other industrial companies, this has created pollution problems in the fjord, and alternative methods for disposal of the residue have been developed. The residue is now stored in rock caverns located 2 km north of the zinc production plant. The first caverns were constructed with a total volume of approx. 70,000 m3, corresponding to one year's waste production. Caverns no. 7 and 9 to 12 are constructed with a volume of approximately 140,000 m3 to double the storage capacity. New concept based on mining technology is adopted and the last two caverns, no. 13 and 14, have a storage volume of 180.000 m3 and 210.000 m3 respectively. Boliden Odda AS is planning to double the zinc production. In this respect, Boliden Odda has carried out an extensive investigation programme to study the environmental impacts due to the increased production. Among these studies are safety studies and assessments of the pollution risk from the rock caverns to the fjord.

INTRODUCTION

Surrounded by high mountains and glaciers, Boliden Odda AS is located in the southern end of Sørfjorden, an arm of the Hardangerfjord in southwest Norway. The company was established in 1924 under the name of "Det Norske Zinkkompani AS". The company has produced electrolytic zinc since 1929. In 2003 the name was changed to Boliden Odda AS.

Besides zinc production, sulphur acid, aluminium fluoride, cadmium, copper residue and lead/tin residues are products from the process. From the metal extraction, a large quantity of residue contaminated with heavy metals called jarosite are produced as waste material. The annual quantity of waste produced is approximately 100.000 m3 of jarosite residue per year. Plans are under preparation to double the annual production of zinc corresponding to a volume of approximately 200.000 m3 of waste production per year.

Previously the waste was suspended in water to a slurry with a dry substance content of 10% and discharged to the fjord as a suspension. In combination with other industrial waste products from the area, the waste has created pollution problems in the inner part of Sørfjorden.

In order to solve the pollution problems of Sørfjorden, Boliden Odda AS has developed alternative methods for waste disposal. Studies for alternative disposal of waste from the zinc production started in 1975. Among several alternatives including solidification, landfill and chemical treatment, waste disposal in rock caverns was found to be the most attractive option.

Photo 1.- Boliden Odda and Sørfjorden

LOCATION AND LAYOUT OF THE PLANT

The site selection and design of the rock caverns were based on geological and hydrogeological evaluations considering the geographical and topographical aspects. From several alternative locations the rock formation Mulen, approx. 2 km north of the factory, was selected.

At Mulen, the rock overburden varies from approximately 200 – 600 m above the caverns. Behind the caverns, the mountains rise with an inclination of 40o up to an elevation of 1000 – 1500 meters above sea level.

Layout of the plant including the transport pipelines is shown in Figure 1. At the industrial area the solid waste is mixed with water to a slurry, which is pumped through approximately 2 km long transport pipelines to the storage caverns. Today, the storage plant consists of an approximately 900 m long access tunnel and altogether 14 rock caverns for waste storage.

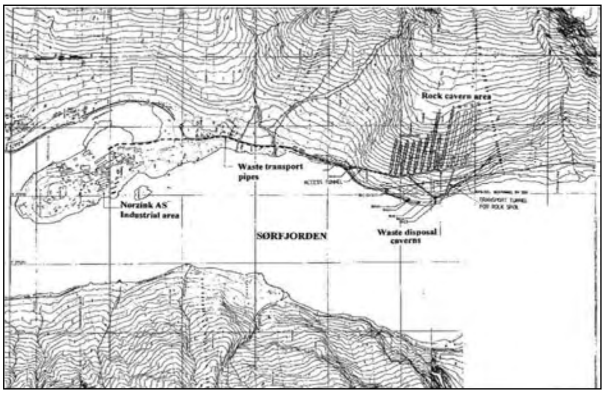

Figure 1. Layout of the Storage Caverns and transport System

First generation of caverns
The first generation caverns (no. 1 – 12) are bottle shaped with various lengths. The rock caverns no. 1 to no. 6 and no. 8 have a total length varying from 211 m to 225 m, width of 17.5 m and maximum height of 23.5 m. The total volume of these caverns varies from approx. 65,000 m3 to 70,000 m3. Caverns no. 7 and 9 to 12 have the double storage capacity, i.e. a length of approximately 400 m, width 17.5 m and maximum height 23.5 m, corresponding to a volume of approx. 140,000 m3. The caverns are excavated by utilising conventional drill and blast method, top heading and benching. The design of the caverns is shown in Figure 2.

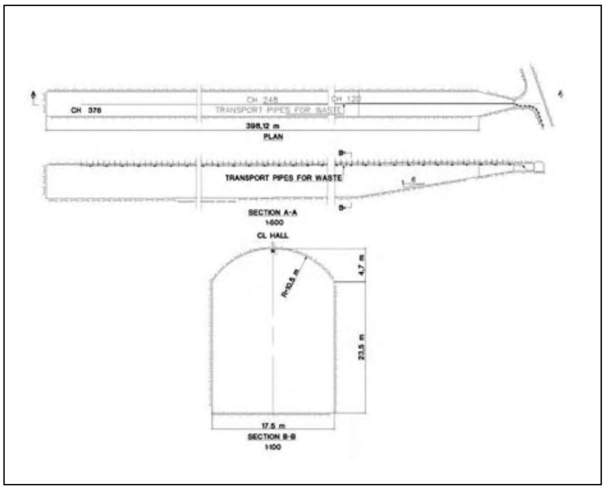

Figure 2. Rock Cavern Design

The caverns are closed with a concrete wall at the outer end. The concrete walls are equipped with an overflow system leading process water for slurry transport and excess water (ground water) in a closed circuit back to the industrial plant and the process.

Second generation of caverns
The second generation of caverns was a result of an engineering process comprising the need of 30% increase of storage capacity, change of rock dumping concept and focus on possible export of crushed rock.

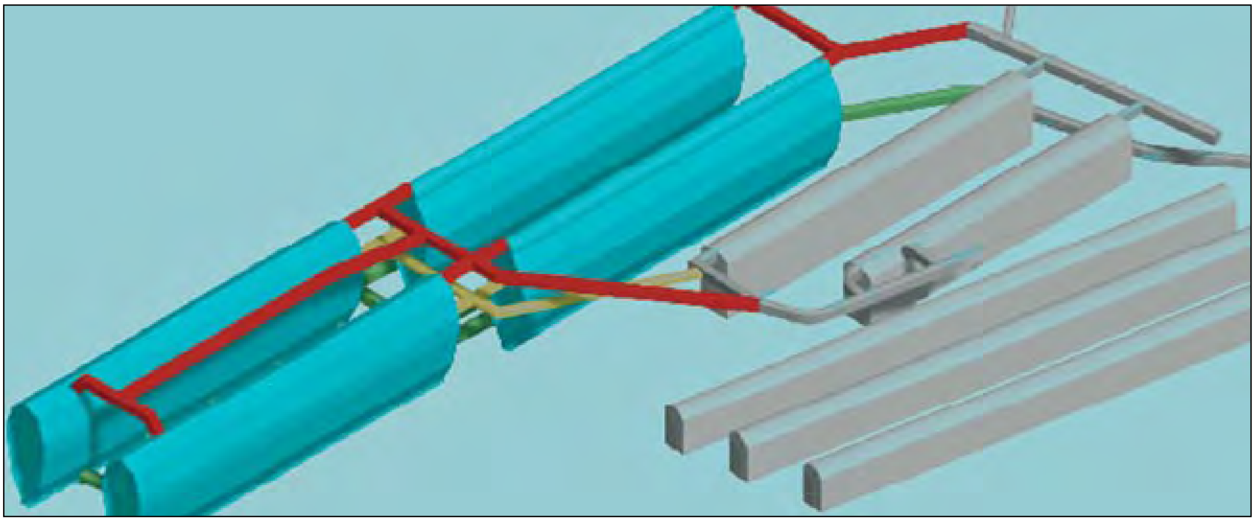

Figure 3 3D illustration of the caverns. Large jumbo caverns (630.000m3) in green colour

Both width and height of the new caverns are increased. Cavern no. 13 is 22m wide, 41m high and 200m long. Cavern 14 is 22m wide, 43m high and 250m long. This gives a storage capacity of 180.000m3 and 210.000m3 respectively.

The new caverns are excavated to a deeper level with access tunnels to the bottom for transport purposes. Adits from access tunnels to the caverns are plugged with concrete.

Need for increased production plus changes in leech technology gave need for increased storage capacity. Mining concept focusing on safety and minimizing support works was adopted. Sublevel benching of up to 80m high and 30 m wide stopes have been proposed.

Excavation of caverns 13 and 14 were carried out by top and mid heading, 10 and 12m high, subsequent systematic rock support of all roof an wall areas, ending with 21 m high bench with no rock support in the walls. The lower bench was excavated from an elevated trapeze-shaped mid section of spoil which was maintained as the excavation moved forwards and subsequently removed as all equipment were pulled out.

After excavation no access to the cavern is allowed. Sealing is achieved by
a 2,5m thick concrete plug, with a PEH – sheeting on the depository side.

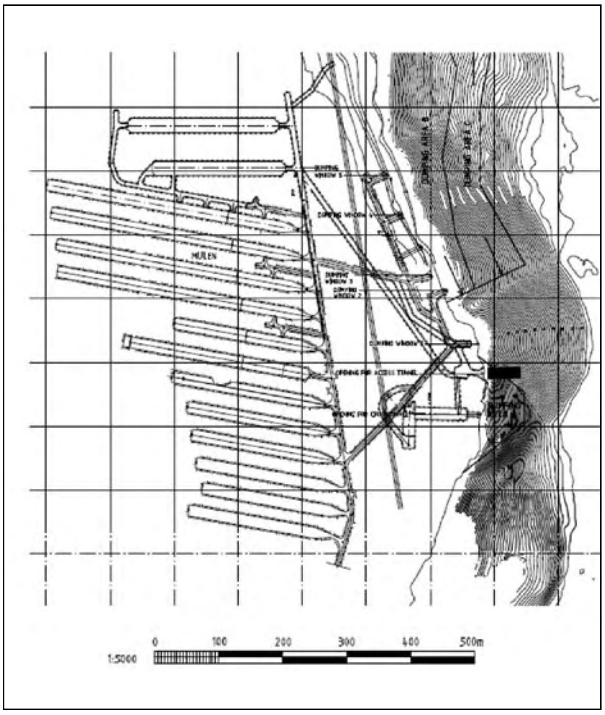

Figure 4 Map showing all excavations: 12 Caverns of 1st generation, cavern 13 an14 of 2nd generation, A ventilation tunnel to the north, a crusherplant/storage at bottom right, and a system of access tunnels between the different levels.

Sarcophagus for Hg barrels
In addition to jarosite residue, 4 caverns are equipped with a sarcophagus for storage of barrels and containers with Hg concentrate. The barrels and container are lined with HDPE membrane, and additionally covered by 30 m of jarosite sediment.

Disposal of rock spoil
Tunnel spoil from the excavation has been utilised for different purposes in the factory areas and within Odda municipality. However, due to a lack of space and little local demand for the rock spoil, most of the rock spoil has been dumped in the fjord which is approximately 130 m deep right outside the caverns. A separate transport tunnel system has been excavated for this purpose.

The rock material is of good quality and there have been studies on utilising or exporting the rock spoil. A rock cavern has been excavated for underground crushing of the material.

GEOLOGY

The bedrock of the Mulen formation is a massive, medium to coarse-grained granitic gneiss with occasional weins of pegmatite, amphibolite and granite. The foliation of the rock strikes in a NW direction dipping steeply towards NE

Besides joint system parallel to the foliation, a joint system occurs with strike in a NE direction and near vertical dip. The different joint systems are illustrated in Figure 3.

Only minor fracture zones occur within the rock formation.

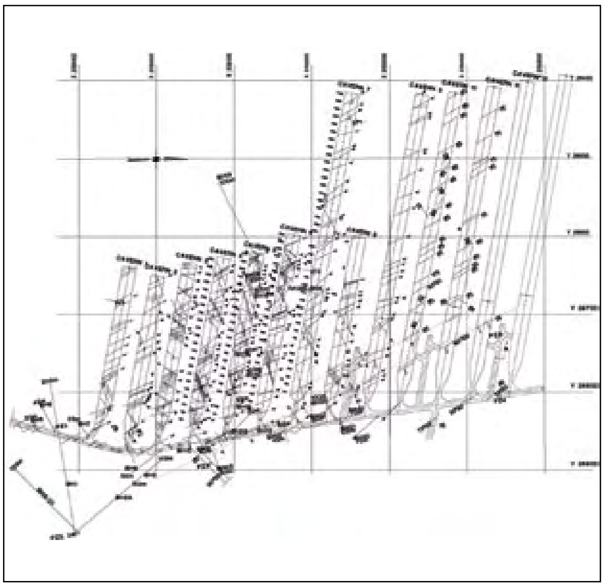

Figure 5 Joints and Fracture Zones

Besides the gravitational stresses, tectonic stresses have been measured. In the deepest parts of the large caverns, high rock stresses with rock burst have been experienced, particularly in the vertical walls.

Q-value range	Percentile on observations
Extremely good (100 < Q < 400)	< 10 %
Very good (40 < Q < 100)	30 %
Good (10 < Q < 40)	>30 %
Medium (4 < Q < 10)	20 %
Poor (1 < Q < 4)	5 %
Very poor (0.1 < Q 1)	< 5 %

Table 1 Rock Mass Quality

The geological conditions as observed in the tunnel system increases in quality when mowing further north and west. Table 1 shows the statistical average of calculated Q values from field observations

INVESTIGATIONS, MODELLING AND MONITORING PROGRAMME

During the initial studies for location of the plant in 1982 coredrillings with Lugeon- and leakage testing were carried out to verify the rock conditions and the hydrological state of the rock formation.

During development and design studies of both concepts, stress measurements, rock mechanical models and analysis were performed.

For the 2nd generation caverns, the programme included stress measurement at 2 sites, Extensometer measurement of wall deformation during excavation of cavern no 13, and a 120m core into an area for future extension of the plant (figure 6).

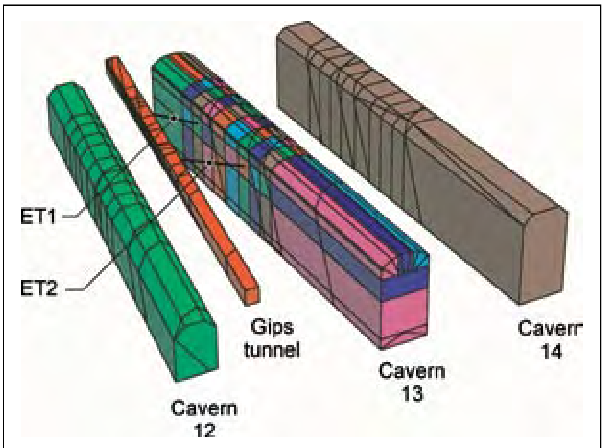

Figure 6 Caverns and tunnels included in 3D analyses. ET1 and ET2 showing location of extentiometers and boreholes for stress measurements and exploration

After an initial rock mechanical study with promising results, a new round of investigations were carried out by the Owner.

Investigation included installation of extensometers in the walls in cavern 13, stress measurement and rock mechanical testing in two locations in service tunnels on each side of cavern 14, and a 120m long core hole drilled into the area for future expansion.
Computer modelling and analysis included comparison of theoretical and measured deformations of walls in cavern 13, studies of stability of caverns 30m wide, 73 m high and 250 m long with different rock stress, and rock quality.

The conclusions were:

- Rock and stress conditions are favourable for large cavern concept ("jumbo caverns") as shown by numerical simulations, based on geotechnical data acquisition & analysis, no. 13-14 caverns excavation experiences, back analysis and monitoring results
- E-W orientation of caverns is suitable; vertical high walls are most critical areas with respect to stability during excavation. The walls tend to get 'rounded' at higher stresses, with higher rock cover
- Global stability of the oval-shaped 30x70 m2 jumbo caverns is reasonable good with 50 m wide pillars between, when having average Q rock quality of 40 and rock cover below 700m. Stability will be reduced with Q 10, without systematic rock reinforcement
- Mining type excavation technique has to be used in jumbo caverns to ensure safety of operators. Operators are not working inside the large caverns but from development tunnels
- Conventional excavation procedure – progressing top down – is also possible when having 'medium' size cross sections (about 900 m2), providing systematic rock support in roof and high walls.

Leakage models are established to analyse the drain pattern around the caverns. For monitoring the water regime, leakages and pollution to the fjord, piezometers, sampling and test wells are established. In addition, measuring weir is installed at the outer end of the access tunnel for leakage measurements.

Water samples are taken regularly from the sampling wells and selected locations at the sea shore for testing, analysis and reporting.

ENVIRONMENTAL IMPACT STUDIES

Boliden Odda AS is preparing plans for doubling the annual zinc production.

As part of the plans, extensive environmental impact studies including assessment of the pollution risk from the rock storage plant have been performed.

The studies include investigations and analysis of the seawater quality along the shore outside the caverns and pollution risk during rock spoil disposal in to the fjord. Furthermore, the studies include evaluation of the potential risk for pipeline fractures and uncontrolled outflow from the transport pipelines as well as the potential risk of pollution leakages from the rock caverns to the sea.

The risk assessment and safety analysis include three different stages:
• The construction stage with a duration of 3-5 years
• Operation stage with duration up to 25 years and
• Post closure of the caverns with a period of up to 1000 years

For identification and selection of scenarios, the following systematic approach for scenario development has been used:
• Identification of potentially disruptive events, features and processes
• Classification of events, features and processes
• Screening of events, features and processes
• Combination of events, features and processes to form scenarios
• Screening of scenarios
• Final set of scenarios

Among the scenarios and events included in the studies are:
• Construction related activities
• Zinc production and process related activities
• Traffic accidents along the transport pipelines
• Rock slides and snow avalanches along the transport pipelines
• Fire accidents and explosion
• War and sabotage
• Sub-sea slides in the rock spoil deposit area
• Long term corrosion of the concrete walls in the caverns
• Long term corrosion of rock support (rock bolts and shotcrete)
• Long term corrosion of minerals along fissures and fracture zones
• Seismic risk and earthquakes
• Climatic changes

To identify and evaluate the potential risks, there has been a systematic assessment of the different scenarios, potential disruptive events, features and processes, reason for the events, probability and consequences of the disruptive events.

SAFETY OF THE ROCK CAVERN STORAGE CONCEPT

For safety verification of the rock storage concept with respect to pollution, extensive geological and hydrogeological investigations are performed. Furthermore, piezometers are installed for evaluation of the ground water table, gradients and the seepage picture around the caverns. Water samples from control wells are analysed regularly in order to monitor the pollution around the caverns and the potential pollution seepage from the caverns towards the sea. Geological and hydrogeological models are established to perform stability analysis and seepage evaluations from the rock store caverns.

To evaluate potential leakages of contaminated water from the caverns, the long-term stability of the following barriers are analysed:

The rock masses between the caverns and the sea
The ground water and gradients around the caverns
The concrete wall at the rock cavern openings
The cut-off trench near the opening of the access tunnel

In the safety analysis of the caverns, the following scenarios are included:

Potential risk of overtopping of the concrete wall due to rock fall and slides inside the caverns
Potential leakages through the rock masses
Potential risk of crack openings in the rock masses due to seismic activities
Long-term corrosion of grout material
Long-term corrosion of the concrete wall in the caverns

Results of the scenario and risk evaluations are described in the following sections.

Scenario 1: Overtopping of concrete wall due to stability problems, rock fall and slides inside the caverns.

The scenario includes overtopping of the concrete wall due waves in stored waste due to rock falls and/or slides inside filled caverns. In this respect, the following causes are assessed:
• Geological features and rock stresses
• Construction activities, rock blasting and cavern excavation in the vicinity
• Corrosion of the rock support
• Earthquakes

In general, the rock masses in the cavern area are of a good quality. By use of the Q-value, the average parameters and rock mass quality of the caverns are as follows:

RQD=100 (massive, no or few joints)
Jn=2-3 (one regular plus random joint sets)
Jr=2-3 (generally rough, undulating joints)
Ja=1-2 (none or minor surface weathering, chlorite and calcite minerals occur at joints)
Jw=1 (none or negligible leakages)
SRF=2 (some rock stress problems, mainly in the cavern walls)

$$Q = \frac{100}{2-3} \cdot \frac{2-3}{1-2} \cdot \frac{1}{2} = \underline{\underline{16-75}}$$

corresponding to good to very good rock quality.

Due to the average good rock quality, the overall probability for stability problems, rock falls and slides in filled caverns are considered to be very small.

The distance between the rock caverns 1 - 12 is 20–24m and the rock quality is good. The maximum quantity of explosives per interval is approximately 135kg (top heading) and 650kg (bench), creating vibrations between 200m/s and 500m/s at detonation. Vibrations of 500m/s may create stability problems if the rock support is insufficient. There have not been any reports of rockslides due to blasting operations so far. Furthermore, based on the rock mass quality, it is not likely that these vibrations will create rock falls/slides, waves and overtopping of the concrete wall in filled caverns.

The overall rock support philosophy of the caverns are to prevent stability problems during construction and filling of the caverns. Thus, the rock support is in any case considered as temporary. The stored waste has a very low pH value and corrosion of the support measures is most likely to occur relatively short after filling of the caverns. Some rock falls or minor slides are thus expected on a long-term basis.

Seismic activities occur in the area. In this respect, a seismic risk analysis has been performed on a 2000 years perspective. Stability analysis during earthquake representing the maximum possible peak ground acceleration (PGA) of magnitude 1,36m/s2 verifies the overall stability of the caverns during earthquakes. However, minor rock falls and/or slides inside the caverns may occur.

Contaminated water overtopping the concrete wall will be caught up by a concrete tub connected to the return pipes and transported back to the industrial plant and cleaning process. Thus consequences by overtopping the concrete wall are not considered to be serious and the associated risk is considered to be low.

Scenario 2: Increased leakages of contaminated water through the rock masses

The water table inside filled caverns is slightly above the seawater level. Furthermore, the caverns will not be plugged in order to prevent increase of the inside water pressure and gradient towards the sea and there has not been identified high concentrations of polluted water along the seashore outside the caverns. Furthermore, piezometer readings indicate a rise in the ground water table between the caverns and the sea, which again indicate the possible existence of a ground water barrier between the caverns and the sea.

Except from a grout curtain between caverns no 7 and 8 plus minor grout curtains to prevent leakages from filled caverns to caverns under excavation (caverns no 9, 10 and 11) there has not been any need for grouting of the surrounding rock masses so far to prevent leakages to the fjord.

In this context, the rock itself is considered to be impervious, and any possible leakage can only occur along joints, fissures or fracture zones. Today, the calculated leakages are very low. The scenario includes potential increased leakages due to corrosion and leaching of minerals along joints, corrosion and leaching of grout materials, crack openings and increased permeability due to earthquakes. Rock modelling and calculations of crack openings due to dynamic loads caused by earthquakes gives no increased seepage to the sea. Based on the rock mass quality, presence of joint minerals and evaluation of potential mineral corrosion, increased leakages of contaminated water due to crack or joint openings are not likely.

Scenario 3: Corrosion of the concrete wall

For evaluation of the potential corrosion of the concrete and reinforcement in the concrete wall, samples, testing and chemical analysis are performed. The chemical environment is considered to be aggressive with respect to steel and concrete corrosion and weathering. The estimated lifetime of the concrete walls varies from 80 to 360 years depending on the weathering process. Due to the potential corrosion of the reinforcement steel, the structural lifetime of the concrete walls is set to 50 years. Inspections and monitoring of the chemical weathering of the concrete walls are required during operation and after closing of the plant. Construction of extra barriers may be required after closing of the plant. In this respect, use of more resistant construction materials should be considered.

CONCLUSION

The rock masses of the Mulen formation are massive and contain few joints. The stability of the caverns is good and there is no risk of unexpected outflow of contaminated water due to stability problems, rock falls or slides even in a long-term perspective including potential earthquake loads. The structural lifetime of the concrete walls is limited and inspection, control and monitoring of the rock cavern disposals will be required also after closing. As an overall conclusion, the risk of unacceptable seepage of polluted water from the caverns is found to be negligible and the rock cavern deposits are considered to be safe.

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UNDERGROUND CONSTRUCTIONS - HOW TO COOPERATE WITH THE ENVIRONMENT

New environmental regulations have enhanced focus on the environmental impacts of underground activities. Environmental impact assessments, risk evaluations and monitoring programmes are important tools in this respect. A programme named "Tunnels for the Citizen" was started in 2000 to improve environmental and urban tunnelling. The programme covers a wide spectre of environmental aspects. The programme was finalised in 2003 and concepts have been developed to construct safe and sustainable tunnels in vulnerable areas.

Nationalteatret Railway Station, underground in the centre of Oslo.

Drilling at tunnel face.

RAILWAY TUNNELS IN URBAN AND ENVIRONMENTALLY VULNERABLE AREAS

Anne Kathrine Kalager
Johan Mykland

ABSTRACT

Great topographic variations which have to be crossed by tunnels to satisfy the demand for better road and rail links. Nowadays, the stronger emphasis on environmental planning is another reason to go underground. Increasing road and railroad traffic has gradually separated the City of Oslo and the suburban areas around the Oslofjord from the sea. By locating major transportation developments underground, this is about to change. However, the different surroundings along the tunnels, with different need for environmental planning and environmental tunnelling have made this a challenge.

HISTORICAL REVIEW OF THE ENVIRONMENTAL TUNNELLING IN THE OSLO AREA

Geology

The geology of the Oslo area is rather complex, with rock types from Precambrian to Permian age. The area is a classic ground in geological research, consisting of Precambrian basement rocks, Cambro-Silurian sedimentary rocks, mostly shales and limestone, and of Permian plutonic rocks and dike rocks. The Caledonian folding and the Permian and younger block faulting made the Oslo graben, with the younger rocks lying in between the older basement. During glacial periods weak zones were eroded, the depressions first being filled by moraine sands and silts, and later being filled up mainly of clay settled on the seabed.

Thus the geology along the tunnels is characterized by rock ridges, alternating with clay filled depressions. The differing rock properties with different needs for support and waterproofing, as well as differing sediment properties with different needs for groundwater control, is therefore a challenge for the geological engineers and constructors.

Holmenkollbanen suburban line

The importance of a waterproof tunnel was dramatically demonstrated during the construction of the underground section of the Holmenkollbanen suburban line starting in 1912. Extensive leakages into the tunnel drained out the surrounding rock and moraine material at the bottom of the clay-filled depressions. This drainage caused lowering of porewater pressures, accompanied by serious settlements of up to 35 cm on buildings founded on clay within 200-400 m from the tunnel. Severe damage was caused to many buildings in the area, especially those which were founded on the steep edges of the clay-filled depressions.

The effects of drainage in clay were not yet common knowledge and a lot of research was started when the project was halted in 1916. However, when the project started again in 1926 and finally finished in 1928, excessive leakages and serious settlements were still experienced. Later projects have benefited from this early experience, and a large number of tunnels and caverns have been constructed without causing the same harmful effects on the environment and existing buildings.

The Oslo railway tunnel

In the beginning of the 1970 decade, when the Oslo railway tunnel was planned, precise data concerning the physical properties of rocks and soils, and detailed knowledge of rock surface levels, were obtained prior to the final choice of alignment and construction methods. An extensive exploration program was performed at an early stage; this included registrations of previous subsoil investigations, seismic surveys, percussion drilling for the determination of rock surface levels and core drilling for detailed studies of rock quality. Clays were subjected to vane tests and sampling for laboratory analysis. Registration of porewater pressure commenced a few years before construction work began. Initial porewater pressure in clay and the natural water table was surveyed by means of a large number of piezometers and observation wells. Several old houses and historic buildings adjacent to the tunnel alignment were observed before, during and after the construction period. The observation program included settlement levelling, vibration control in conjunction with rock blasting and visual observation of cracks and subsidence. Together with the continual registration of porewater pressures and leakage into the tunnel the observation program provided the necessary data for the accomplishment of safe tunnelling.

In building the Oslo tunnel, the aim was to achieve the highest possible degree of water tightness. To ensure a permanent solution to the drainage problem, the tunnel was provided with a complete reinforced concrete lining, cast in place. Provisional waterproofing for the unlined area between the tunnel face and the lining was achieved by pregrouting. Nevertheless, extensive leakages were experienced in some places. To gain control over porewater pressures a system of artificial recharge wells were put into operation. The reinforced lining allowed the use of high pressure systematic post- grouting at the interface rock-concrete, reducing leakage to about 1 litre /minute per 100 m of tunnel in the 22 m span Nationaltheatret station. This was tight enough to make the groundwater table stable, consequently stopping the settlements and damages on the surrounding buildings and finally taking the artificial recharge wells out of operation.

The extension of Nationaltheatret station in the Oslo railway tunnel
Even if considerable damages on buildings were reported from the construction of the Oslo tunnel, this was in fact the state of the art at that moment. However, combining geological mapping and geological engineering experiences from the above mentioned tunnels and other tunnels built in the period until 1996, made an improved foundation for the planning of the extension of the Nationaltheatret station. A combination of engineering geology experience and already established procedures for site investigations and observation programs were used. However, the damages experienced from the earlier projects mentioned and the growing public opinion against the same damages once again initiated the need for stronger emphasis on environmental planning, both for the construction environment and the surroundings.

Therefore an environmental program was established containing an upper limit of 2 cm settlement on the surrounding buildings when the project was finished. This was the most important premise when the construction started in 1996. During tunnelling, when excessive leakages and temporary drops in the porewater pressure were discovered, artificial recharge wells were put into operation soon enough to prevent the lowering of the groundwater tables in the clay-filled depressions under surrounding buildings. During waterproofing of the reinforced lining, groundwater tables were held at their initial state while the artificial recharge wells gradually were phased out. The extension of the Nationaltheatret station was completed in 1999, and no settlements caused by the tunnelling was registrated on the surrounding buildings.

TO-DAY'S PRACTISE OF THE PLANNING AND THE CONSTRUCTION OF "ENVIRONMENTAL" TUNNELS. AN EXAMPLE

Jong – Asker double track railway tunnels
All the above mentioned experiences were put into the planning of two double track tunnels west of Oslo. Both tunnels cross under either urban areas or vulnerable areas of nature. Tunnelling started in spring 2002 and was finished in spring 2004.

Through an evaluation process based on both economical and safety aspects, a normal profile with theoretical cross-section of 108-112 m2 and a net cross-section of 89.6 – 90.2 m2 was selected. The main reason for selecting a double track tunnel instead of two single track tunnels was for safety, the critical time needed to fill the whole cross-section with smoke in case of fire.

To ensure the best possible groundwater control in and around the new tunnels the same planning procedures were followed as in the extension of the Nationaltheatret station. A geological cross-section along the tunnel alignments had to be established together with the mapping of clay-filled depressions. An observation program was established at an early stage, documenting groundwater balance in different engineering geology settings. Critical cross-sections were selected for more detailed hydro geological mapping, groundwater modelling and water balance estimates. These investigations provided the criteria for tunnel permeability and the grouting strategy focusing on the consequences of groundwater leakage rates, settlement on the surrounding buildings and nature vulnerability.
An environmental program was established with premises and goals to prevent damages on vulnerable nature and on the surrounding buildings. These premises made the suggested leakage limits of 4 l/min (red) - 8l/min (blue) and 16 l/min (green) per 100 m of waterproofed tunnel along the tunnel alignment and a corresponding area of possible groundwater drawdown.

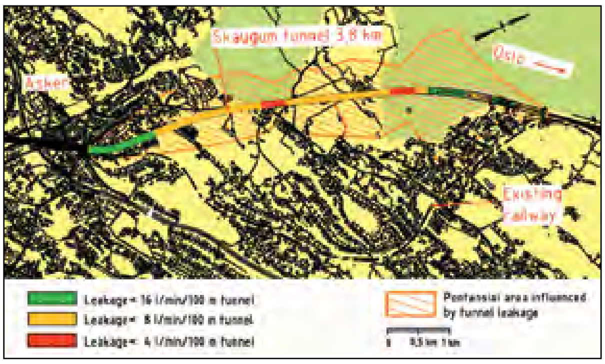

Figure 1: Areas of possible groundwater drawdown corresponding to the suggested leakage rates

To meet the criteria for groundwater control during the construction period an automatic instrumentation system using the Internet was established. This web-site has provided updates on 65 groundwater levels within 24 hours and made it possible to detect groundwater settlements soon enough to give input to the systematic pregrouting procedure and to establish artificial recharge wells soon enough to prevent damages on the surrounding buildings and vulnerable nature. The double track tunnelling between Jong and Asker is now finished. So far the earlier engineering geological experiences together with geological and structural geological mapping and corresponding site investigations have proved the engineering geological conditions to be quite accurate and the results from the observation programs show that neither buildings nor vulnerable nature have suffered notable damages.

EXPERIENCE TO BE CONSIDERED FOR FUTURE TUNNELLING PROJECTS

The experience of environmental damage due to tunnelling can be drawn from above project and from many other tunnelling projects. In remote areas the consequences may have none or little significance. However, in suburban areas around big cities the public opinion has made environmental tunnelling inevitable.

Experience so far shows that a thorough knowledge of ground conditions is of great importance when considering the extent of the surveillance areas and the possible damage on the surroundings. Putting this knowledge into an environmental programme, the acceptance criteria of the surrounding public are established in the planning. By including the environmental programme in the tunnel contracts, different waterproofing limits are established for different environmental settings along the tunnels.

Together with a reasonable time frame for the tunnelling

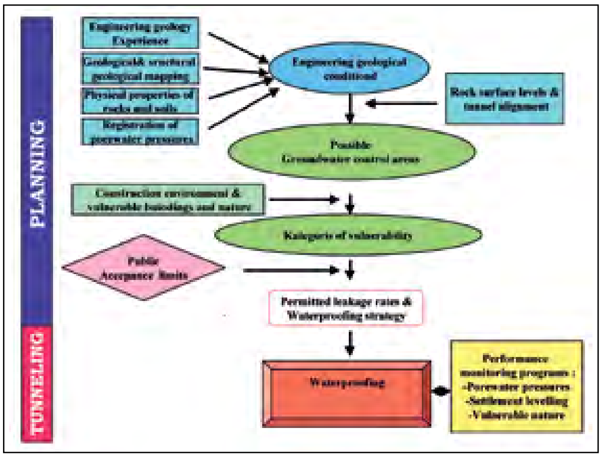

Figure 2: The concept of environmental tunnelling.

contracts and a tailored observation programme overseen by the construction supervisors, this will reduce the damage on the surroundings to a minimum.
By using the knowledge of today it is possible to fulfil the environmental premises both during and after the tunnelling construction.

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UNDERGROUND CONSTRUCTION TECHNOLOGY

NORPLAN

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FROM TUNNELLING IN URBAN AND ENVIRONMENTALLY VULNERABLE AREAS TO SUBSEA STRAIT CROSSINGS

The purpose of modern transportation tunnels is crossing adverse topography or to preserve the surface environment; being untouched nature, cultural landscapes, housing areas or city centres. The surface areas could beneficially be used for other purposes than congested traffic. Tunnels have become a major solution to many cities in Norway.

Sub sea tunnelling represents a Norwegian speciality. So far, 23 sub sea road tunnels are constructed in Norway, two more under construction and several more under planning. The Norwegian fjords are formed by major faults, fracture zones and weak rock formations, thus representing a great challenge for planners and tunnellers.

Asker portal/entrance

Vardø Tunnel at 70^{o} North. The portal with the Arctic Ocean in the background.

Images from Eikesund Connection, 14.8 km highway including the 7.8 km long Eikesund subsea tunnel, a new number one as far as depth is concerned.

Courtesy Svein Skeide, Eikesundsambandet

Opening up for tunnelling.

Support of opening with sprayed concrete.

Exploratory drilling at profile 1614.

Smooth progress.

Exploratory drilling, changing of drill rod.

NEW METHODS FOR TUNNEL INVESTIGATION

Mona Lindstrøm
Alf Kveen

ABSTRACT

A wide range of new methods for tunnel investigation are evaluated in this project. The methods range from satellite and aerial investigations to geophysical (geo-electrical) investigation below the surface. The paper is a summary of project A in the development programme "Tunnels for Citizen"

INVESTIGATION METHODS

New methods, with the potential of locating and investigating zones that may be problematic to tunnel excavation were tested in this project. The main part of these tests was performed by the Geological Survey of Norway. The methods have the potential of providing more detailed information about the relative rock mass quality and water-bearing zones below the surface. Results from both new and traditional methods were evaluated. The tests of different investigation methods in the same area and during tunnel construction provided a direct comparison of the methods, and a precise evaluation of their ability to locate water-bearing zones in the depth.

The main test site was the area above the Lunner tunnel near Gardermoen Airport north of Oslo. The 3.8 km long tunnel is situated below a nature reserve, including a lake. For this reason the requirements for water ingress to parts of the tunnel was set to 10 – 20 litres/minute/100 m (water leakage after pre-grouting). The tunnel opened in 2003. The second test site for the new investigation methods was above the Jong – Asker tunnels, two railway tunnels 2.7 and 3.7 km long, just west of Oslo. Due to risk of settlements which could cause damage in the densely built-up area, the requirements for water ingress was set to between 4 and 16 l/min./100 m.

The aim of investigations for tunnelling is to obtain the information that is necessary to establish the excavation procedures, the design of the appropriate rock support, water sealing and costs in good time before the tunnel construction is under way. The new methods have proven to be useful alternatives and supplements to traditional methods. The methods are user-friendly and the costs are generally lower than for the existing methods. The results and evaluations of the specific methods are summarized below.

Borehole inspection

The optical televiewer (OPTV) is basically a video camera which is lowered into a borehole of 70 – 160 mm in diameter. It provides detailed information about rock type boundaries and orientation and character of structures through a 360º picture of the borehole wall (Figure 1).

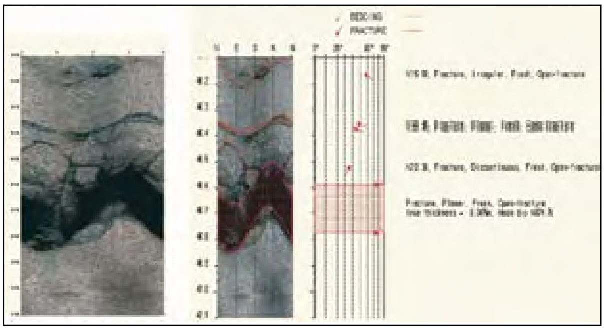

Figure 1 Optical televiewer recording of a borehole, showing a 360º picture of the borehole wall with its structures and an analysis of the individual joints.

Instruments within the OPTV record the frequency, strike and dip and opening of the various structures cutting the borehole, and statistical analysis of the data is presented in diagrams. The OPTV can be used as an alternative to core drilling and logging.

Additional inspection methods are probes which are lowered into boreholes with continuous logging of geophysical parameters which can be interpreted to reflect rock mass quality or potential for water leakage. For example, measurements of changes in temperature and electrical conductivity of the water may indicate open joints with inflow of surface water. Variations in natural gamma radiation may reflect variations in mineralogy (rock type boundaries). Similarly can probes measuring electrical conductivity of the rock mass identify possible weakness zones along the borehole.

Hydraulic test pumping of boreholes is useful for identifying water bearing joints within the borehole, and may be an alternative to Lugeon-testing. The results give the potential for water leakage where the tunnel cuts through these fractures and subsequently help to evaluate the need for pre-grouting.

Two-dimensional (2D) resistivity

Two-dimensional resistivity provides a view of the physical properties of the rock mass below the surface; this has not been possible by using traditional methods of investigation. The resistivity is measured by electrodes attached to cables lying on the surface. By processing the data, an image (2D profile) of the subsurface resistivity is obtained down to a depth of 120 m (Figure 2). However, the best resolution is achieved at depths down to 50 – 70 metres. The results can be interpreted in terms of rock mass condition: high resistivity indicates good quality rocks whereas zones of relative low resistivity may be correlated with jointed rock masses or weakness zones. The interpretation of the results depends on a good geological knowledge of the area from field investigations and other methods.

The tests which were carried out in this project show the excellent potential of 2D resistivity in tunnel investigation. With this method it is possible to locate zones that may cause problems related to stability and inflow of groundwater, and in far greater detail than traditional refraction seismic. The position of the zones relative to the proposed tunnel can be traced, and boreholes for further inspection of the critical zones can be established exactly in order to obtain the maximum amount of relevant information.

Measurements above the Lunner tunnel gave very good results, the profiles show clearly zones which correlated well with mapped structures both on the surface and inside the tunnel during excavation, as well as with borehole logging in the area. In other locations (Jong-Asker) some of the limitations of this method became clear. The lack of distinct results was probably due to both a generally low resistivity in the ground, and a high density of technical installations in the Jong-Asker area. As a rule, this method seems to work well in areas with a generally high resistivity in the ground; above 5000 ohm.

Geophysical survey from helicopter

Geophysical survey from helicopter was carried out over the area where the Lunner tunnel is situated. Magnetic, radiometric, electromagnetic and VLF (very low frequency) electromagnetic data was collected. For all these methods, faults and weakness zones in the bedrock may appear as linear or curvilinear anomalies, also in areas covered with sediments and vegetation. Confirmation of the specific structures is done by field geological and geophysical mapping. The helicopter survey thus provides an efficient method for mapping of regional structures which may influence the tunnel excavation.

Mapping by digital analysis

Digital topographic maps were tested for applicability to register regional geological structures. The digital topographic data is combined with other digital data such as satellite- or aerial photos and maps. By processing these data it is possible to locate lineaments that may represent rock boundaries, weakness zones or faults.

An example from the Oslo region show regional lineaments produced by digital analysis (Figure 3). The lineaments appear clearly on the map, also in areas covered with urban settlement and infrastructure. Thus, this method also provides important information about regional structures which is useful at an early stage in the tunnel planning, especially in densely built-up areas.

Radar interferometry

As part of this project, satellite-based radar interferometry is evaluated as a method to identify and monitor settlements during tunnel construction. Satellite images

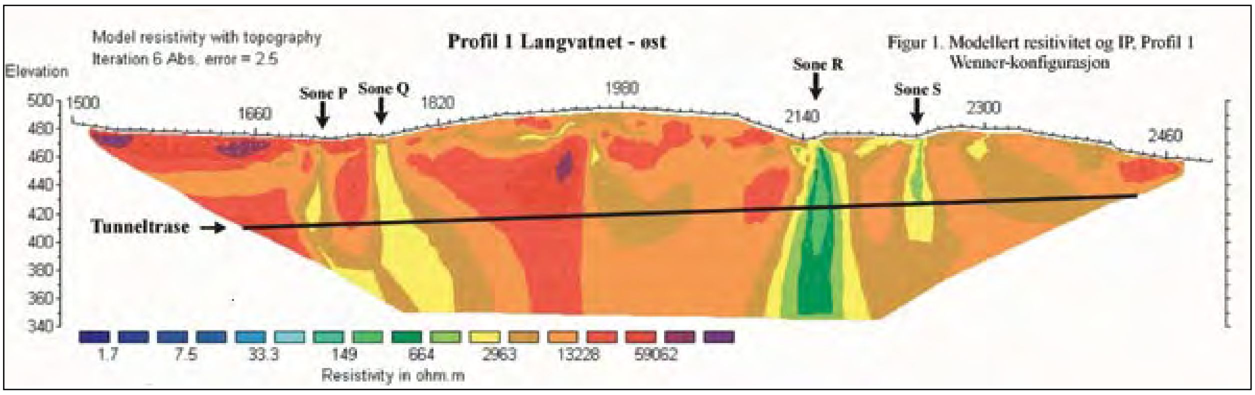

Figure2 Resistivity profile from a section of the Lunner tunnel. Zones of low resistivity are further examined by borehole logging.

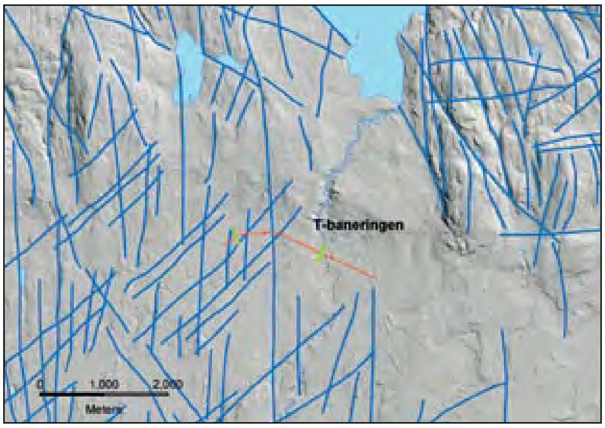

Figure 3 Lineaments produced by digital analysis from the area near the Metro tunnel (T-baneringen) in Oslo.

are available from the period 1992 to 2001 and further on from 2004, and provide very detailed historic data. Vertical displacements down to mm-scale are registered with this method. An example from the area above the Romeriksporten railway tunnel, which suffered significant environmental damage during tunnel construction, was used as an illustration (Figure 4). One of the advantages with this method is that it is possible to monitor a large area in detail, instead of displacement measurements on selected buildings only. The potential for daily or weekly monitoring of an area during a future tunnel excavation is not as good, since data are collected at an interval of 35 days.

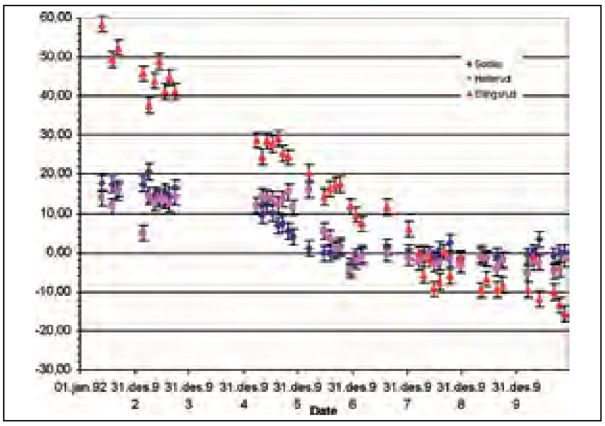

Figure 4 Example of settlements over time of three buildings above the Romeriksporten tunnel. One building, at Ellingsrud (red triangle), shows a natural continuous settlement in the period 1992 to 1999. Two other buildings (Godlia and Hellerud) were unaffected until about 1995 when the tunnel excavation progressed below these houses. For about one year they suffered a settlement of 15 mm until the leakages in the tunnel were under control.

Measuring While Drilling (MWD)

Measuring While Drilling is a relatively new technique to register rock parameters ahead of the tunnel work face during drilling. The instruments are installed on the tunnel drilling machine, and provide automatic registration of selected parameters. These data are then interpreted according to a pre-set scale which is calibrated for the specific project.

Examples are registrations of relative joint frequency of the rock mass and variations in hardness (Figure 5).

The method is still under development and is in use on several new tunnel construction sites. It is a good supplement to engineering geological mapping in the tunnel. MWD also helps to secure the documentation of data from the tunnel excavation, and will improve the communication between work shifts.

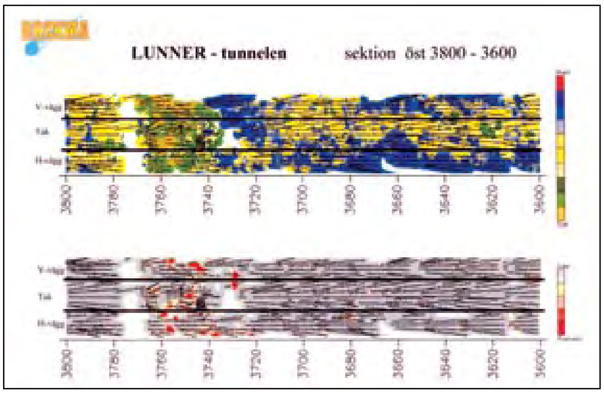

Figure 5 Examples of registrations during tunnel drilling. Relative rock hardness (top), and relative fracturing (bottom) in roof and walls along a 200 m section of the Lunner tunnel.

Refraction seismic modelling

Seismic refraction is widely used in tunnel investigation, especially for sub-sea tunnels. The method measures the seismic velocities in the underground, and the velocities may be interpreted in terms of rock mass quality. Some of the limitations with this method are well known. The interpretation , for example, is based on the assumption that the velocities increase downward – which is most often the case. Furthermore, the method will only provide seismic velocities in the uppermost few metres of the bedrock surface.

The Norwegian Geotechnical Institute carried out refraction seismic modelling to illustrate that the standard interpretation of the available data can be inaccurate. Synthetic seismic models of a rock surface with a sharp depression without a weakness zone below were presented. In the standard interpretation, a vertical weakness zone was positioned below the depression (Figure 6). This is a common interpretation of this type of feature, the ambiguous data leads to interpretations that tend towards the worst case scenario. For a more realistic interpretation, two possible situations could be described from the data. In the future, improvements of the method would be techniques to extract more information from the available data, alternatively more precise information about possible weakness zones may be obtained by using seismic tomography.

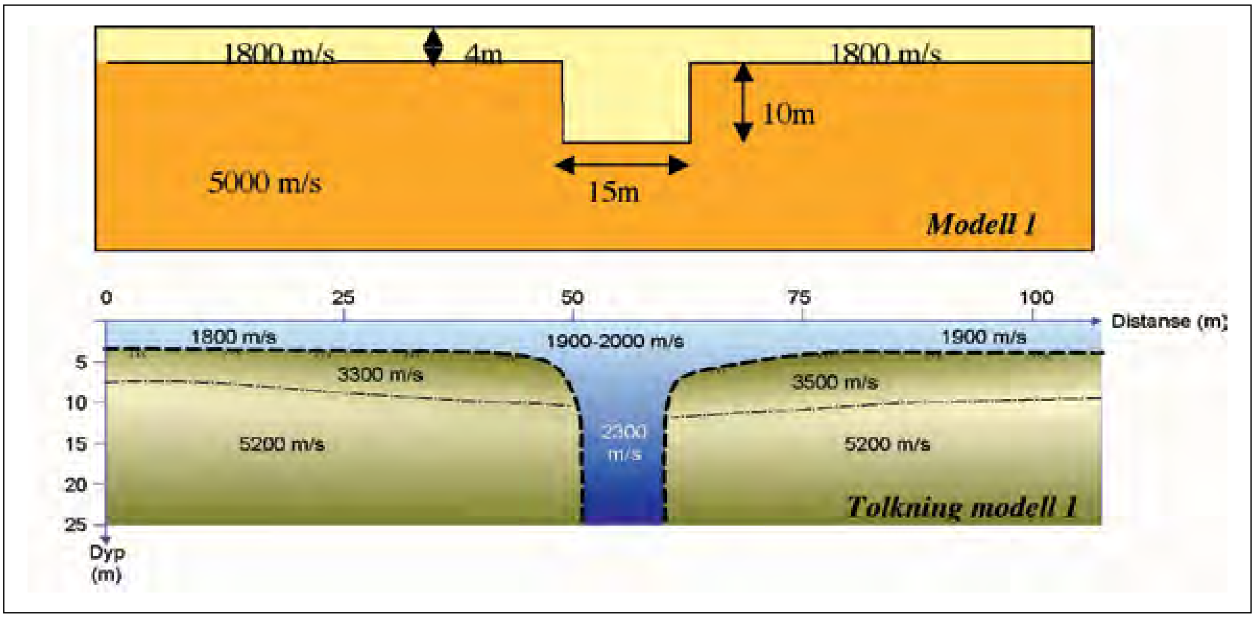

Figure 6 Example of a synthetic model (top) and the interpretation of the model data (bottom) with a major weakness zone located in the depression. Seismic velocities higher than 5000 m/s generally represent good rock mass qualities. Weakness zones usually have velocities lower than 4000 m/s.

ADEQUATE INVESTIGATIONS FOR NORWEGIAN CONDITIONS

21 selected Norwegian tunnel projects have been analysed to work out recommendations on the appropriate amount (cost) of ground investigations for tunnels with today's requirements to the projects. The degree of difficulty in collecting information on the ground quality has been applied in the evaluations performed. In addition, the requirements to the safety of the actual project during construction and use, its influence on the environment, plus risks for encountering unpleasant tunnelling situations determine the project's investigation class.

The definition of investigation class is based on the guidelines in the Norwegian Standard NS-3480 'Geotechnical planning'. Here, the geotechnical project class is defined based on evaluation of a damage consequence class and degree of difficulty of the project. The same principle is used in Eurocode 7, which defines three geotechnical categories.

In summary, the investigation classes which is developed in this project is defined by the following two parameters (Table 1):

- Degree of difficulty. This reflects the engineering geological conditions, extent of weathering, overlying sediment deposits, water or urban settlements, accessibility to perform field observations. The different elements are weighted and given a value reflecting low, moderate or high degree of difficulty. This corresponds to the complexity of the ground in terms of tunnelling and the type and extent of investigation needed.
- Demands to the structure. This parameter reflects stability, possible risks during excavation, possibility to affect or damage the environment, such as vegetation or buildings. The elements are weighted and given a value reflecting low, moderate or high demands during construction and operation.

Table 1 The investigation classes (A, B, C, D) determined from degree of difficulty and demands to the structure. These parameters are deduced from evaluations of various elements (not given here)

Definition of INVESTIGATION CLASS		a. DEGREE OF DIFFICULTY		
		a1. Low	a2. Moderate	a3. High
b. DEMANDS TO THE STRUC-TURE	b1. Low	A	A	B
	b2. Moderate	A	B	C
	b3. High	B	C	D

The analysis of the 21 selected tunnel projects according to this system resulted in the recommended amount of investigation for each investigation class, presented in Figure 7. For a standard Norwegian road tunnel the recommended, appropriate total amount varies between 2 and 10 % of the cost for blasting and mucking out included rig (20 – 30 %). For sub-sea tunnels this value varies between 5 and 15 % plus 2 to 5 % for exploratory drilling ahead of the tunnel working face during construction.

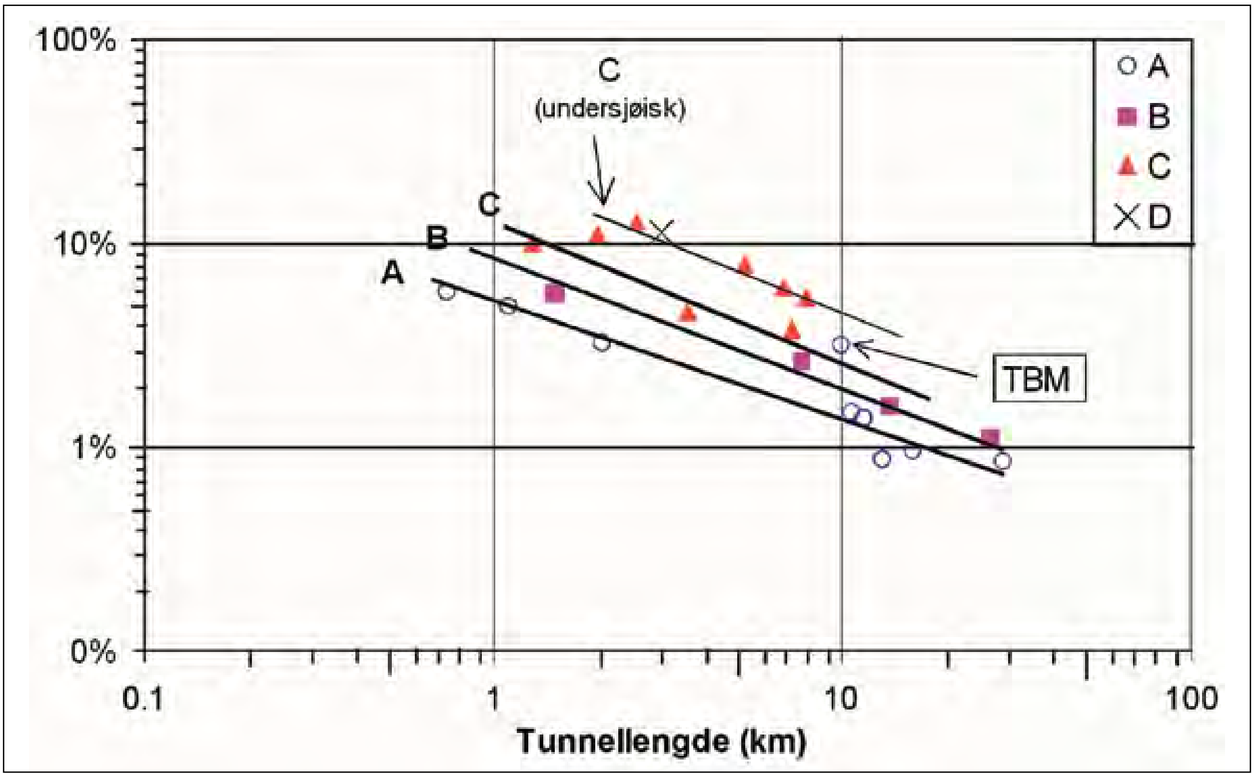

Figure 7 Recommended amount of investigation (costs) for the investigation classes (A, B, C and D), given as percentage of cost for blasting and mucking out included rig, relative to tunnel length.

It is important to stress that ground investigations cannot reveal all structures in the ground; therefore it is always possible to encounter unexpected conditions. The new geophysical investigation methods have made it possible to achieve more detailed information about the ground conditions, which contribute to more precise cost estimates. Well planned and executed investigations increase the knowledge of the ground and thus reduce the probability for unforeseen problems.

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TUNNEL LEAKAGE AND ENVIRONMENTAL ASPECTS

Mona Lindstrøm
Alf Kveen

ABSTRACT
The vulnerability of the environment, especially related to changes in the groundwater table caused by the tunnel construction, is evaluated with the aim to develop methods to quantify accepted levels of leakage into a tunnel. The paper is a summary of project B in the development programme "Tunnels for Citizen"

TUNNEL LEAKAGE AND ENVIRONMENTAL ASPECTS

The background for initiating the research programme was severe tunnel leakages during a specific construction project, the Romeriksporten railway tunnel towards the Oslo Airport Gardermoen The leakages caused damage on the surface, both to vegetation (Figure 1) and to buildings. The aim of this project was to study the effects of groundwater leakage and develop procedures to quantify maximum allowable water inflow to a tunnel based on the possible or acceptable impact on the surface environment. The studies were carried out by the Norwegian Geotechnical Institute, the Norwegian Institute for Nature Research, the Norwegian Centre for Soil and Environmental Research and Norconsult.

The work involved a study of the correlations between water leakage into tunnels, changes in pore pressure and damage to the environment, both to vegetation and water sources and to urban structures. The acceptable amount of water inflow into a tunnel in a specific area can be determined by studying the correlation between several parameters. These include the water balance in nature, hydraulic conductivity of the rock mass and overlying sediments, the potential for settlements, the vulnerability of the vegetation and grouting procedures.

Numerical modelling
Modelling may be used to simulate the hydrogeological conditions before and after tunnel excavation, and to evaluate the relative importance of the different parameters used in the models. In this way, important informa-

Figure 1 The tarn Puttjern which is situated above the Romeriksporten tunnel. It was nearly drained due to tunnel leakages, and later, due to response from neighbours, restored with grouting and permanent water infiltration (Photo: L. Erikstad).

tion about the groundwater conditions may be obtained in an early stage in the planning process. The hydraulic conductivity of the bedrock in Norway is generally low, with groundwater flowing along joints and weakness zones. The usefulness of numerical flow models will depend on realistic geological and hydrogeological input data and the boundary conditions established for the model.

Several models are available for simulating water flow in jointed rock masses. Two main types were tested in this project to simulate groundwater flow, groundwater drawdown and the effects of sealing the tunnel:

- Two-dimensional models, where the rock mass is modelled as a homogeneous material with average hydraulic conductivity
- Three-dimensional model of water flow in a fracture network, providing a more detailed image of flow in the rock mass.

Experience from 2D modelling

Several example studies were performed to simulate groundwater flow and the effects of tunnel leakage and sealing in a homogeneous rock mass, in order to test the applicability of this type of model. A zone of low hydraulic conductivity around the tunnel in the model represents sealing of the tunnel by cement injection. The study shows that to avoid lowering of the groundwater table of more than a few metres, the leakage must be kept at 1 - 3 litres/ minute/100 metres tunnel. This will require a high degree of sealing effort, corresponding to a very low hydraulic conductivity of the sealed zone.

Experience from 3D modelling

A 3D discrete fracture network model was used to investigate the groundwater flow and to predict water inflow to a tunnel during excavation and after cement injection. The model was built by the Norwegian Geotechnical Institute using the computer program Napsac, which takes into account the heterogeneities existing in the rock mass.

The Lunner tunnel was chosen as a field case because of its part in the research project and the large amount of data available from the field investigations. The numerical model covers an area of 550 m x 550 m along the tunnel (Figure 2). It includes a fault zone which represents the boundary between hornfels and syenite. Joints and faults observed during the field mapping are included in the model, as well as results from borehole investigations and Lugeon-tests. Small-scale joints were statistically modelled, and used to generate a discrete fracture network.

The model provides more accurate results compared with results obtained from 2D models. The limitation of the 3D model is the need for computer power, which in this case has put limits to the size of the area of investigation. Saturated transient and steady state calculations were performed to predict the amount of water leaking into the tunnel. The effects of cement injection of the tunnel were modelled by reducing the transmissivity of joints cross-cutting the tunnel. The results show that a reduction in the leakage rate is observed only after a significant reduction in transmissivity. An extensive injection of the fault zone was shown by comparison to be more effective than a moderate injection along the whole tunnel, although the leakages tend to increase on both sides of the injected section of the tunnel. Before cement injection a leakage rate of 900 l/min./100 m was

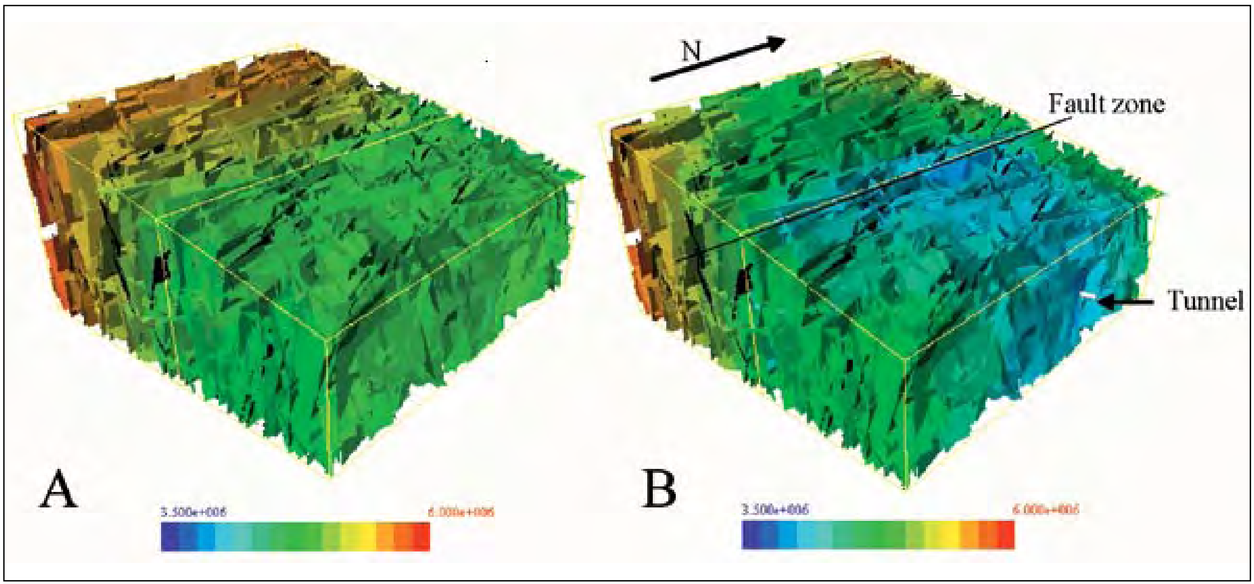

Figure 2 Example of presentation of the steady state pore pressure distribution in a section of the Lunner tunnel. A: The pre-tunnel situation, with geological data and the model fracture network. B: Modelled effects of pore pressure change after tunnel excavation. The water will tend to flow towards low-pressure areas (blue).

predicted in the fault zone, with a significant drawdown of the groundwater table, which would in effect drain the model. Reducing the transmissivity in the fault zone by a factor of 200 will result in a leakage rate of 50 l/min./100 m, and a lowering of the groundwater table of 5 m.

Details of this study are found in Cuisiat et al (2003). The simulations have so far indicated the potential for this type of numerical tool in tunnel planning. Further analyses are needed before this 3D model is ready for use on a major tunnel project.

Tunnelling effects on the groundwater table

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

Accepted leakage in natural landscapes

In order to evaluate the impact a tunnel excavation may have on the environment, it is necessary to assess the value of the surface environment, its sensitivity to a drawdown of the groundwater table and the risks of damage. The areas most vulnerable to damage due to drainage are identified as those having a groundwater table which is directly feeding water-dependent vegetation and surface water. The vulnerability increases with smaller size of the precipitation area. Changes in the groundwater table may also cause disturbance in the chemical balance of surface water due to erosion and oxidation of dried-up sediments, which can lead to a concentration of ions, salts and particles in the body of water. The vulnerability must be evaluated on the basis of practical use of the water source, the biodiversity and the presence of water-dependent vegetation.

A mapping programme with systematic registration of the vegetation above several tunnels with documented leakages was carried out in the course of this project by the Norwegian Institute for Nature Research. The aim was mainly to acquire new information about the relation between damage to vegetation and tunnel leakages. Some effects of drainage are easily identified such as ground settlements and dried-out ponds (Figure 3), but for the most part damage to the vegetation is not evident. It was concluded from the study that this may be due to the fact that the actual damage is insignificant or is healed, that damage to certain species is undetectable due to lack of pre-tunnel investigation or that the scale of detail in the registration is not the appropriate for this type of investigation.

Figure 3 Partly drained lake above the Tokke hydropower tunnel. Seasonal leakages cause significant variations in the water level. (Photo: A. Often).

The areas most sensitive to groundwater drawdown as a result of tunnel excavation are generally quite small compared with the area above the total length of the tunnel. A method is proposed to locate potentially vulnerable areas and to classify the vulnerability of nature elements at the early stages of tunnel planning. From regional mapping, features such as local depressions in the terrain are isolated, as these often contain vulnerable vegetation. The vulnerability of each area identified is then classified by the size of the local catchment's area. The method is well adapted to the most common geological situation in Norway, with a relatively thin layer of soil lying on top of crystalline magmatic or metamorphic rocks. Local depressions in the terrain usually coincides with lineaments such as weakness zones in the bedrock, and this combined information is of importance both in finally establishing the tunnel route, decision of the excavation procedures and in the planning of sealing measures to protect the most sensitive areas.

The evaluation of acceptable changes on the surface environment involves a definition of the value of sensitive vegetation or surface elements along the tunnel route. In addition to economic value, the (non-monetary) values can be classified in terms of: 1) Nature, including biodiversity, 2) Recreational, and 3) Importance to local communities. The value of each element is graded according to a pre-set scale, for example according to local, regional or international interest, or high to low value.

The accepted impact on the surface is not determined by the leakage rate, as a relatively low leakage may cause severe damage in more sensitive areas. The accepted consequences will be defined by the value of the area, for example a high value implies a low acceptance level. The level of acceptance for each area may be converted into a maximum allowable leakage rate along the respective sections of the tunnel.

Procedure to determine accepted leakage rate in sensitive landscapes

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

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

Accepted leakage in urban areas

The requirements on maximum water inflow into tunnels in urban areas are related to possible soil settlements which may cause damage to buildings and other surface structures. Experiences from Norway, collected by the Norwegian Geotechnical Institute, show that the risk of damage is highest in areas where the building foundations are placed on soft marine clay deposits. Groundwater leaking into a rock tunnel can cause significant reduction in the pore pressure at the clay/rock-interface, which leads to consolidation processes in the clay and subsequent settlements. This situation with marine clay deposited on bedrock is found in the Oslo region, which represents the most heavily populated area in Norway. Data from measurements on pore pressure reduction at the clay/rock-interface and leakage rates are compiled from a number of rock tunnels excavated in the region; for roads, railway, metro and sewers (Figure 4).

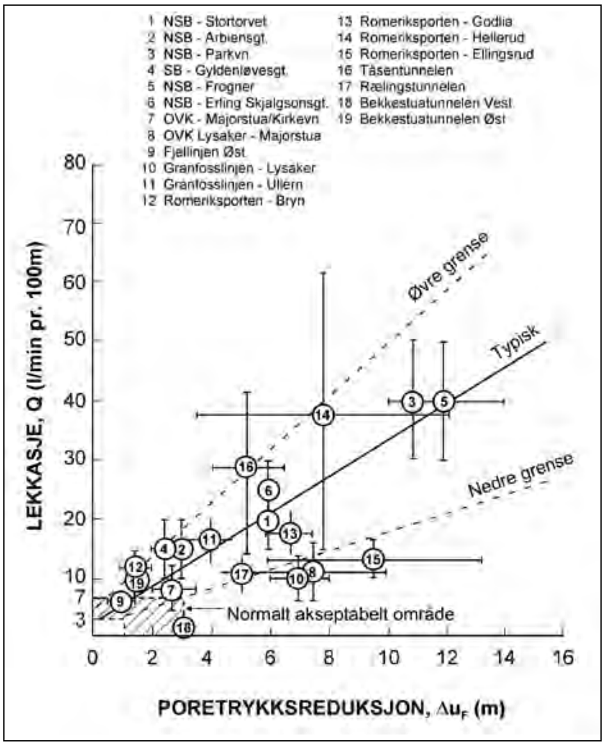

Figure 4 Correlation between pore pressure reduction ΔuF (m) above existing tunnels, and leakage rate, Q (in l/min./100 m). The data forms the basis for a recommended acceptance level for leakage in this type of rock/clay-underground, shown by a characteristic area (hatched).

From the data in Figure 4 it is possible to predict the pore pressure reduction in clay deposits caused by a tunnel excavation. The data forms the basis for the recommended procedure to establish maximum allowable water inflow into a tunnel. The data indicate that an acceptable limit to the leakage rate should be 3 - 7 litres/minute/ 100 metres tunnel, which corresponds to a pore pressure reduction of 1 - 3 metres (Figure 4).

The study shows that systematic grouting is necessary to fulfil strict leakage requirements. There is a clear correlation between the grouting procedures used in the tunnels, the amount of grout cement used, length of the boreholes and the resulting hydraulic conductivity in the rock above the tunnels. Recently excavated tunnels generally show better results in terms of fulfilled requirements, mainly due to improved grouting techniques and materials.

The potential for consolidation settlements in the sediments in relation to pore pressure reduction can be determined from soil sampling and laboratory analyses. Clay deposits generally contain small amounts of water and the groundwater table will not be influenced significantly by leakage. Drawdown of the groundwater table is shown to occur mainly in areas where the clay deposits have a limited extent or where the clay deposits are shallow.

Procedure to determine accepted leakage rate in urban areas

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

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

Subsea access tunnel to the oil and gas processing plant at the Melkøya isle off Hammerfest. Courtesy VS-Group.

TECHNIQUES FOR GROUNDWATER CONTROL

Mona Lindstrøm
Alf Kveen

ABSTRACT

A specific grouting strategy utilizing thick cement grout is developed. It is a result of evaluation of grouting performances in several recently built tunnels, and has proven to be efficient and give better control on the amount of water draining into a tunnel. The paper is a summary of project C in the development programme "Tunnels for Citizen"

TECHNIQUES FOR GROUNDWATER CONTROL

Techniques for groundwater control are studied in this project, with the aim of improving conventional techniques and development of new methods. Safe and efficient methods for sealing are of special importance in tunnels with strict requirements for water ingress. The best method by far for sealing rock mass to reduce groundwater ingress to tunnels, is cement grouting ahead of the tunnel work face (pre-grouting). The project activities included studies of grouting strategies for pre-grouting of tunnels, procedures for both systematic grouting and grouting adapted to difficult geological conditions and complex tunnel design, as well as procedures for time efficient grouting. In addition, the project evaluated natural sealing processes and water infiltration.

As a result, a specific grouting strategy is systematized, based on tests performed on site during tunnel excavation, laboratory tests and a compilation and evaluation of grouting performances in several recently built tunnels. This grouting strategy gives good control on the amount of water ingress to the tunnel after grouting.

A major part of the work in this project was monitoring of grouting procedures and practical tests of grouting strategies in ongoing tunnel construction projects: T-baneringen (Metro tunnel in Oslo), the Jong-Asker railway tunnels and the road tunnels Hagan and Lunner just north of Oslo.

LABORATORY TESTING OF GROUT CEMENTS

Laboratory tests were performed for the documentation of the properties of cement types used in tunnel grouting, and an evaluation of the usefulness of performing laboratory tests on these materials. Testing of cements was performed by conventional laboratory methods, using the actual cement types used in the grouting of the Metro tunnel. In summary, the tests showed that water/cement-ratios (w/c-ratios) higher than 1.0 result in too long hardening time, the control of the actual w/c-ratio is best done by measurements of density, and only fresh cements (newly produced) should be used. Test results gave no indications that the temperature of the cement during grouting is of importance, but the temperature of the injected rock may inflict on the hardening process.

GROUTING STRATEGIES

The project group carried out a test programme in cooperation with the builder and contractor of the Metro tunnel in Oslo. The aim was to confirm the grouting technique that most efficiently provides sufficient sealing of the rock mass.

The Metro tunnel is 1240 m long and part of the new Metro Ring system in Oslo. The rock overburden of the tunnel is between 5 and 25 metres, and with a cover (0-20 m) of sediments (clay). Due to risk of settlements in the clay causing damage to buildings, the requirements for water ingress to the tunnel after grouting was between 7 and 14 l/min./100 m, and grouting was performed for most of the tunnel. The tunnel was completed in 2002. Tests of materials and grouting methods were carried out along different sections of the tunnel construction site adjusted to the rock mass quality, rock cover and demands for water inflow (see Figure 1).

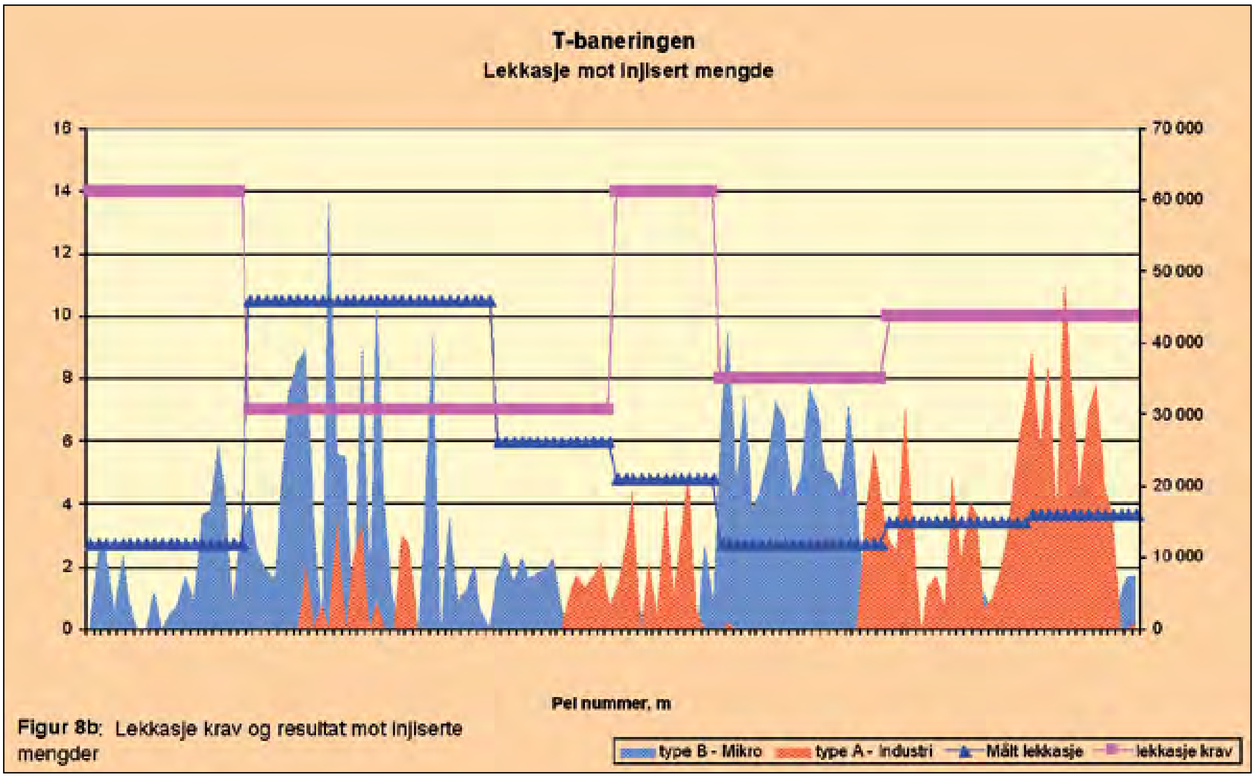

Figure 1 Example of results of the grouting of the Metro tunnel: water leakage and amount of grout cement along the tunnel route. Horizontal lines: red= pre-set requirements for leakage (in l/min./100 m), blue= measured water ingress after grouting. Amount of cement grout used: blue= microcement, red= standard cement.

Some of the tests and results are listed below:

- An evaluation of grouting procedures proved that systematic pre-grouting of the tunnel is more efficient than sporadic grouting. Sporadic grouting involves adjustment of the procedure based on results from measurements of water leakage in probe holes, and the risk is that water will seep towards non-grouted sections of the tunnel.
- A water/cement-ratio lower than 1.0 is necessary to provide rapid and sufficient sealing, the ratio may be as low as 0.5. This was also confirmed by the laboratory experiments. Silica additives to the grout mix and improved pumping capacity make injection of this thick cement grout possible.
- A low water/cement-ratio requires high pressure pumping, up to 10 MPa, which 'kick' the grout into the finer joints and assures that the rock mass close to the tunnel is sealed.
- Different types of cement grouts were used under similar rock mass conditions and requirements for water ingress. In this particular tunnel, no difference in the results from standard cement and microcement was recorded.
- Tests were performed of optimum grouting time consumed compared with the total excavation time and the result of grouting, by adjusting the work procedures. The most time efficient procedure would be a fast construction progress, with the criteria for water ingress fulfilled from a single round of grouting. The adjustments proved very efficient when working through good rock mass quality and not so strict sealing requirements.

The requirements for water ingress to the tunnel as a whole were fulfilled, with an average of 4.3 l/min./100 m. The exception was a specific 50 m wide weakness zone, where water leakage is up to 8 l/min./100 m. As a result, water- and frost protection measures are reduced for parts of the tunnel.

DOCUMENTATION OF GROUTING PROCEDURES: MAPPING OF EXPERIENCE

A selection of eight newly built tunnels (7 in Norway, one in Sweden) is examined with regard to experience from grouting of the tunnels. The mapping procedure involved interviews with on-site personnel representing owners, consultants and contractors. The selection criteria for these tunnels were strict requirements for water inflow (2-20 l/min./100 m), and carefully planned and well documented grouting strategies. The experience with different types of rock, the grouting strategies under various conditions, equipment, materials, performance and final results are mapped and compared. The detailed results from the mapping are listed in tables which have proven to be of significant

Figure 2 The Tanum tunnel (Jong-Asker railway). Markings of injection holes in the tunnel work face.

importance when planning the grouting strategy for new tunnel construction projects.

The general conclusions from the experiences are comparable to the experiences from the Metro tunnel. These conclusions form the basis of the recommended pre-grouting procedure ('Active grouting', see below).

NATURAL SEALING PROCESSES

In Norwegian tunnels, water ingress tends to decrease with time. In most cases this could only be caused by natural sealing. Laboratory experiments were carried out by Aquateam with the aim of tracing the sealing mechanism and to find out if this mechanism can be put to practical use in tunnels. Water samples from a selection of tunnels were analysed; sub-sea tunnels and one land tunnel. The main results show that the water leaking into the tunnels is rich in particles of iron, and contain lesser amounts of calcium, barium and manganese.

Laboratory testing showed that oxygen injected to sand columns containing Fe^{2+} caused oxidation to Fe^{3+} and subsequent deposition of the iron, the rate depending on the particle size within the column. The results of these tests are interesting, but further tests are needed to find out if water leakages can be reduced by accelerating natural sealing processes.

WATER INFILTRATION

Groundwater infiltration is used to control the pore pressure temporarily during excavation of tunnels. The reduction of pore pressure in sediments due to water leakage into a tunnel may cause settlement and significant damage to the surface areas and to buildings. In this project, experience from water infiltration over 20-30 years is compiled in a report from the Norwegian Geotechnical Institute.

The conclusions from the evaluation are that infiltration holes must be placed in bedrock, and established as result of good knowledge of the hydraulic conductivity of the rock mass and the nature of sediment deposits. Wells placed in sediments are unpredictable and have occasionally caused severe problems due to erosion. Furthermore, water infiltration should be used as a temporary measure during construction only. All of the permanent installations that are in use today were not planned as such, and had to be kept in function due to insufficient sealing of the tunnel or underground structure. These installations do not guarantee that the pore pressure is maintained and are costly due to the unforeseen, long term, 'indefinite', operation and maintenance.

PRE-GROUTING TECHNIQUES

Theoretical and empirical background for high-pressure grouting

A theoretical and empirical background for high-pressure grouting is assembled for this project by N. Barton and Associates. The report describes the problems, and some solutions, concerning pre-injection in jointed and faulted rock masses ahead of tunnels. The application of very high pressure pre-injection for sealing and improving the stability of tunnels has focussed attention on the need for quantitative explanations of grout take volumes and on the effects on the rock mass, of the 5 to 10 MPa injection pressures. The report provides explanations of joint properties and change when the rock mass is subjected to high-pressure grouting of thick cement grout.

In conclusion, there is practical evidence and empirically-derived support for increased seismic velocities, increased deformation modulus, reduced deformation, reduced permeability, and reduced tunnel support needs as a result of successful high-pressure pre-injection with stable cement grout.

Rock mass quality

The quality of the rock mass will determine the choice of grouting strategy. The rock masses that are most common in the bedrock of Norway are here divided into four groups (A, B, C, D) based on experiences from engineering geology and their general properties during grouting. For each group, the number of injection holes, length of holes, water/cement-ratios and cement types are recommended, as summarized below.

A. Open joints with little or no clay filling, found most frequently in sandstone, quartzite, syenite, granite. The hydraulic conductivity of the rock mass is relatively high, and the rock mass has low resistance to grouting. In general, few and long injection holes are used, and low water/cement-ratios.

B. Jointed rock mass with joints partly filled to produce local channels. This situation is frequently found in gneisses, which represent a major part of the Norwegian bedrock. The joints are typically filled with clay minerals. The rock mass is less easy to grout, and the injection strategy must be adjusted to the local rock mass condition. The procedure is generally an initial high w/c-ratio (c. 1.0) and relatively low injection pressures, towards a final low w/c-ratio and high pressure pumping (up to 10 MPa). The high injection pressure is important in order to establish communication between joints.

C. Dense, plastic rock masses represented by metamorphic rocks such as mica schist, phyllite and greenschist. Joints are typically filled with clay, and have small

Figure 3 Grouting in the Hagan tunnel syenite

channels occurring on narrow joints. The hydraulic conductivity of the rock mass is low, and grouting may be difficult. In order to grout the small and scattered channels, many short injection holes are needed. An initial high w/c-ratio (c. 0.9) and microcement is used, with a final lower ratio and high pressure pumping.
D. Rock masses with extremely open joints as a result of fault zones or karst. Tunnelling in these rock masses will require extraordinary efforts.

'Active grouting'
A recommended procedure for pre-grouting is presented, based on the results from this project combined with the compilation of well documented and successful grouting results in various rock masses. The foundation for the application of this grouting strategy is an understanding of rock mass quality, grouting pressures, cement properties, the amount of cement and cement additives used, the geometry of the grouting fan and the number of injection holes in the tunnel work face.

- Low water/cement-ratio combined with high pressure pumping is the main condition for successful grouting. The w/c-ratio should be as low as practically possible (down to 0.5) in order to obtain a marked pressure loss away from the tunnel. This assures that the cement is concentrated close to the tunnel. The pressure build-up in each separate hole must be constantly surveilled, with adjustments of the w/c-ratio based on the observations. The pressure build-up should increase steadily to allow a smooth cement inflow, and with a final injection pressure as high as is possible (10 MPa is the maximum capacity of most of today's pumps).
- The type of cement is adjusted to local geological conditions; both standard cement and the fine-grained microcement are used. Additives of superplasticizers and silica increase the flow stability of the cement grout. It should be noted that with the cement grout mix and the pump capacity for handling dense masses, there is no longer need for the highly toxic chemical grouts.
- The tunnel is to be grouted from the sole and upward. In this way, the water is pushed up and away from the tunnel. The grouting is most efficient when using many holes and as long holes as possible. The holes must be placed both circling the profile and in the centre of the work face. The geometry and the length of the holes in the grouting fan must be adjusted to the local conditions. An initial situation with many holes is recommended, with an adjustment based on observations and adaptations to the rock mass condition on the site. Use of modern injection rigs with two or more separate grouting lines increases the efficiency.
- Systematic grouting where the leakage requirements are strict has proven to give the best results. Where the rock mass quality is favourable, the grouting procedure may be adjusted to a more time efficient 'factory' performance.
- A successful sealing, with little or no water leakage into the tunnel will reduce the need for water- and frost protection and rock support measures.
- Continuous supervision, control and adjustment of the grouting procedure as well as an experienced professional crew is the key to a successful result.

In conclusion: It is possible to build technically very complicated tunnels with total control on the groundwater by using systematic cement-based grouting ahead of the tunnel working face, based on the principles of active grouting. This is of particular importance in areas where the tunnel construction is not to cause unwanted environmental consequences.

NORWEGIAN SUB SEA TUNNELLING

Olav Torgeir Blindheim
Eivind Grøv
Bjørn Nilsen

ABSRACT

The Norwegian coast line is governed by deep fjords typically cutting into the land mass creating infrastructure barriers and with small, scattered populations. These barriers were traditionally overcome with ferry connections and in Norway we have had an extensive network consisting of a large number of ferries being vital elements in the public service. However, such ferry connections are sensitive to harsh weather, are expensive in operation and slow. The modern society has demands on quicker and more reliable services, particularly to allow new enterprises to grow at these remote places. In this context sub sea tunnels have played an important role in providing and establishing consistent fixed links between remote areas and the main land infrastructure network.

Important issues concerning investigation, planning, design and construction are described, and important lessons learned from these projects are discussed.

The concept is now spreading; the Faroe Islands and Iceland have both got their first sub sea tunnels. Similar sub sea road tunnels are under elaboration on other Atlantic islands; Greenland, Orkney and Shetland, and on Åland in the Baltic Sea. A sub sea tunnel connecting the island of Sareemaa to the mainland of Estonia is also being considered.

INTRODUCTION

In Norway, 23 sub sea road tunnels have been built since the Vardø tunnel was officially opened in 1983, representing a total length of more than 100 km. In addition eight sub sea tunnels have been built for the oil industry as shore approaches and pipeline tunnels, and another eight for water supply and sewerage. All these tunnels are excavated entirely in bedrock by drilling and blasting (no submerged culverts), with a strong reliance on probe drilling and pre-grouting, and with drained rock support structures.

The Bømlafjord tunnel is presently the longest at 7.9km. The Hitra tunnel is so far the deepest at 264m below sea level; the Eiksundet tunnel under construction will reach 287m depth. These tunnels have successfully replaced many congested ferries on the stem roads and connected island communities to the mainland. In total, this represents no less than a new era in coastal communication and development. A record breaking 24km long sub sea road tunnel below a wide open fjord exposed to hard weather is at the planning stage. Another project has been considered reaching depths below sea level down to 400 m.

Figure 1. Norwegian sub sea road tunnels

COMPLETED NORWEGIAN PROJECTS

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

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

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

The Public Roads Administration approves the plans, including requirements to maximum gradient, minimum rock cover, safety equipment etc to ensure a consistent approach to safety and to adapt the standard to the expected traffic volume. Guideline regulations are implemented to keep costs at a reasonable level. If cost levels are allowed to escalate, it would mean that fewer connections could be established.
Norwegian road tunnels are classified in six classes labelled from 'A' to 'F' according to tunnel length and the Annual Average Daily Traffic (AADT), see Figure 4. For AADT below 7500, a tunnel width of 8.5m is required for two lanes; for higher traffic volumes the width has to be increased to 9.5m. The tunnels frequently have 3 lanes in the steep slopes (see Figure 4) to accommodate the uphill traffic.

The required installations are also related to the tunnel class. Depending on length and traffic volume, this may include:

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

Figure 2. Excavation for the portal of the Nappstraumen sub sea road tunnel.

Figure 3. The 3 lane Oslofjord tunnel with artistic light effects for driver's comfort.

The design fire load is 5MW for AADT(10) <10,000 and 20MW for AADT(10) > 10,000. These figures are under revision in view of recent fire tests showing very high effects of ordinary truck loads.

Pumping of remaining water inflow is always included. A water sump is located at the low point with capacity to store at least 24 hours of allowed inflow (typically 300 litres/min per km or less).

The different requirements are gathered in a standard issued by the Norwegian Public Roads Administration (Norwegian Public Roads Administration 2002a). The standard is based on the experience gained from almost

No	Project	Year completed	Main rock types	Cross section, m^2	Total length, km	Minimum rock cover, m	Lowest level, m below sea
1	Vardø	1981	Shale, sandstone	53	2.6	28	88
2	Ellingsøy	1987	Gneiss	68	3.5	42	140
3	Valderøy	1987	Gneiss	68	4.2	34	145
4	Kvalsund	1988	Gneiss	43	1.6	23	56
5	Godøy	1989	Gneiss	52	3.8	33	153
6	Hvaler	1989	Gneiss	45	3.8	35	121
7	Flekkerøy	1989	Gneiss	46	2.3	29	101
8	Nappstraumen	1990	Gneiss	55	1.8	27	60
9	Fannefjord	1991	Gneiss	54	2.7	28	100
10	Maursund	1991	Gneiss	43	2.3	20	92
11	Byfjord	1992	Phyllite	70	5.8	34	223
12	Mastrafjord	1992	Gneiss	70	4.4	40	132
13	Freifjord	1992	Gneiss	70	5.2	30	132
14	Hitra	1994	Gneiss	70	5.6	38	264
15	Tromsøysund	1994	Gneiss	60 a)	3.4	45	101
16	Bjorøy	1996	Gneiss	53	2.0	35	85
17	Slöverfjord	1997	Gneiss	55	3.3	40	100
18	North Cape	1999	Shale, sandstone	50	6.8	49	212
19	Oslofjord	2000	Gneiss	79	7.2	32 b)	134
20	Frøya	2000	Gneiss	52	5.2	41	164
21	Ibestad	2000	Micaschist, granite	46	3.4	30	125
22	Bømlafjord	2000	Greenstone, gneiss and phyllite	74	7.9	35	260
23	Skatestraumen	2002	Gneiss	52	1.9	40	80
	Halsnøy	2008	Gneiss	50	4.1	45	135
	Eiksundet	2007	Gneiss, gabbro, limestone	71	7.8	50	287

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

Table 1: Norwegian sub sea road tunnels completed or under construction

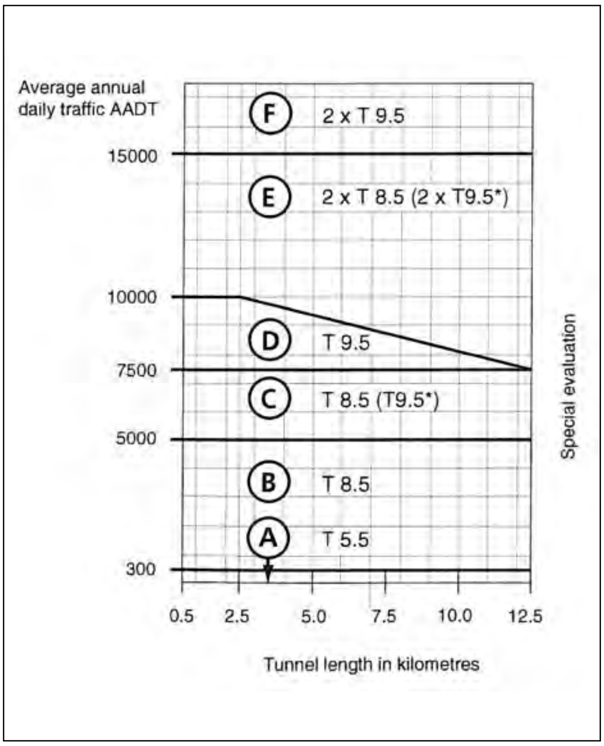

Figure 4. Tunnel Classification according to the Norwegian Public Roads Authorities (2002a). "T" refers to tunnel width in metres.

a thousand km of road tunnels, including also sub sea road tunnels. Any deviation from the specifications in the standard must be approved by the Directorate of Public Roads.

Present cost levels for sub sea road tunnels, including installations, vary between USD 6,000 and USD 10,000 per meter tunnel, depending on whether it is two or three lanes (the latter often used in the up-slopes). All tunnels so far, except one, have one tube.

Contract types have, with one exception, been the traditional Norwegian unit rate contract (Blindheim & Grøv, 2003). This includes payment according to experienced quantities of excavation, rock support, probe drilling and grouting and other waterproofing, which takes care of most of geological risks with respect to variations

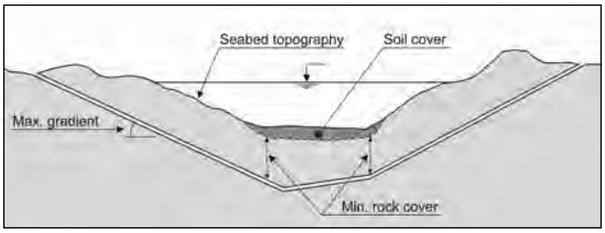

Figure 5. Typical section of sub sea tunnel.

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

a) No information

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

in rock mass quality. Notably, the contract also includes 'standard capacities' which allows automatic adjustment of construction time according to the experienced quantities. This provides for a risk sharing between the owner and the contractor which is especially suitable for sub sea tunnels. The owner maintains the risk for any 'surprises'; after all he has decided the extent of the site investigations.

In addition to the road tunnels, several sub sea tunnels have been built by the oil industry for oil and gas pipelines, and some for water supply and sewerage. Key data for some of the most significant are shown in Table 2.

BASIC PRINCIPLES AND LESSONS LEARNED FROM NORWEGIAN SUB SEA TUNNELS

Site investigation strategy

Besides normal geological surveys on both sides of the fjord, and on any adjacent islands, the site investigations rely heavily on seismics in the first stages. Acoustic profiling will first cover a large area to determine the most suitable corridor, then extensive refraction seismics to select the best alignment and to provide information about soil deposits above the bedrock and about weakness (low velocity) zones in the bedrock. If possible, directional core drilling as illustrated in Figure 6 is used from shore to the critical deepest points of the alignment, which typically also could be the location of major fault zones. Core drilling from drilling ships has been applied in a few cases, if other methods were not feasible, or the results in doubt. Such drilling is seldom cost effective and not always conclusive; if feasible it may be better to plan for more directional core drilling.

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

The established practice of site investigations has proven to be reliable, but exceptions have occurred. In the Oslofjord tunnel, despite of an extensive program of seismics, directional core drilling, hole-to-bottom seismics and seismic tomographic interpretations, the glacial erosion along a known depression along the bottom of the fjord proved to be much deeper than interpreted and left the tunnel without rock cover over a short section. This was detected by probe drilling during construction; a by-pass tunnel was prepared to allow continued tunnelling under the fjord. The soil filled section was frozen (at 120m water pressure) and excavated through (Backer & Blindheim, 1999). Figure 7 demonstrates that if the core hole had been placed above the tunnel alignment, not within the cross section, the eroded channel could have been avoided.

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

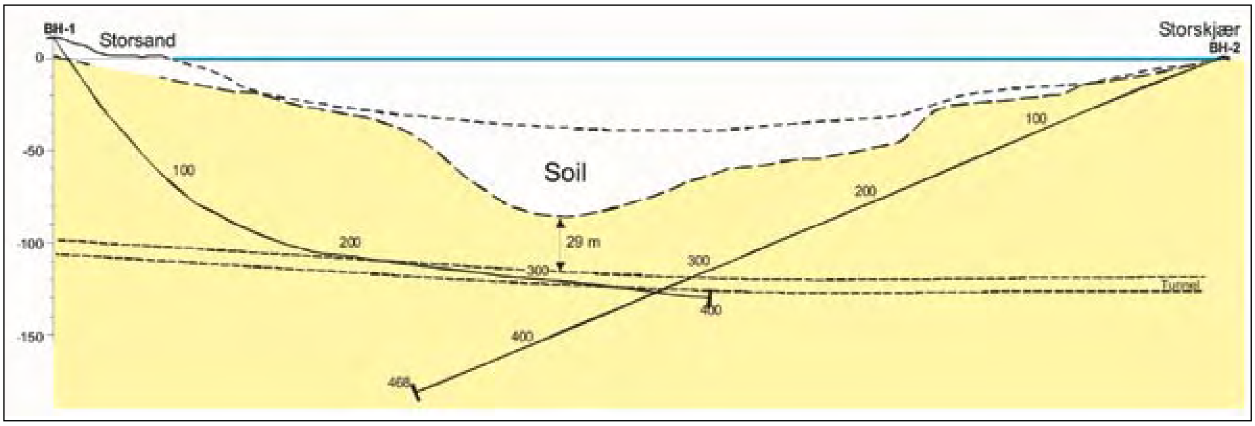

Figure 6. Conventional and directional core drilling applied for investigation of critical part of the Oslofjord tunnel (based on Palmstrøm et al. 2003).

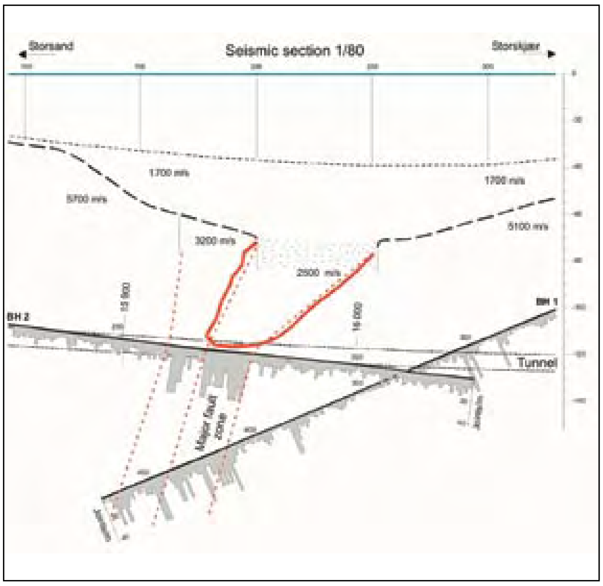

Figure 7. The eroded channel at the deepest point of the Oslofjord tunnel (based on Palmstrøm et al. 2003).

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

The requirement to rock cover has basically been the same for all the road tunnels built until 2002, i.e. minimum 50m. A rock cover of less than 50m could be accepted when detailed site investigations demonstrated fair rock mass conditions (taking into account the typical occurrence of fault zones at the deepest point). This is left much open to interpretation, and rock cover less than 20m has been used, but then typically restricted to shallow waters and for good rock conditions. In some cases, the economic feasibility of a low traffic tunnel project depends on the minimum rock cover being cut to a safe minimum (Blindheim & Nilsen, 2001).

Basically, the rock cover can be looked upon as including an rock mass arch of sufficient bearing capacity (considering the water pressure), a margin for undetected variation ('surprises'), and a margin for 'reaction time' should a fallout occur. The latter proved useful in the Ellingsøy tunnel (Olsen & Blindheim, 1989), where a cave-in started in a blasting round through a fault zone and developed upwards at a rate of 1m/h. It stopped however after 10m. Due to such incidents, and a couple of other 'surprises', the Norwegian Public Roads Administration now, unless fair rock mass conditions have been proved, insists on a minimum rock cover of 50m. Smaller cover has to be approved by the Directorate of Public Roads, and is checked by independent review.

As always in tunnelling, much effort is put into avoiding 'surprises'. Many so-called unexpected geological conditions are indeed foreseeable. But they may be more difficult to check out due to the sub sea conditions. A certain remaining risk has to be considered, even after significant and relevant site investigations. This is why risk control during planning and construction becomes important. For the Frøya tunnel, an external team of experts provided an independent risk assessment (Nilsen et al, 1999). This is now recommended for all sub sea tunnels.

Continuity in planning and investigation should always be aimed at to ensure that interpretations from early phases are brought forward to the detailed design and construction phases.

Excavation, probing, grouting and rock support
All sub sea tunnels in Norway have been excavated by D&B, as illustrated in Figure 8. This method provides great flexibility and adaptability to varying rock mass conditions and is cost effective. The 6.8km North Cape tunnel was considered for TBM, but the risks connected to the potential water inflow were considered too large. In hindsight, this would not have been critical, as the

Figure 8. Drill and blast excavation in difficult rock mass conditions in the Frøya tunnel. Extensive shotcreting and concrete lining were required.

main problem proved to be thinly bedded rock causing stability problems in the D&B drives, which would likely have been less in a TBM drive.

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

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

All rock support structures are drained, whether they are made of cast-in-place concrete (mostly horseshoe) lining, sprayed concrete ribs (see Figure 9) or sprayed concrete. Sprayed concrete is dominantly applied as wet mix steel fibre reinforced. Extensive testing demonstrates that, if the thickness of the sprayed concrete is above a minimum of 60-70mm, and the concrete quality is good (C45), corrosion of steel fibres is not a problem. The use of sprayed concrete has increased over the years from 0.7-1.0m3/metre tunnel to about 1.5-2.0m3/

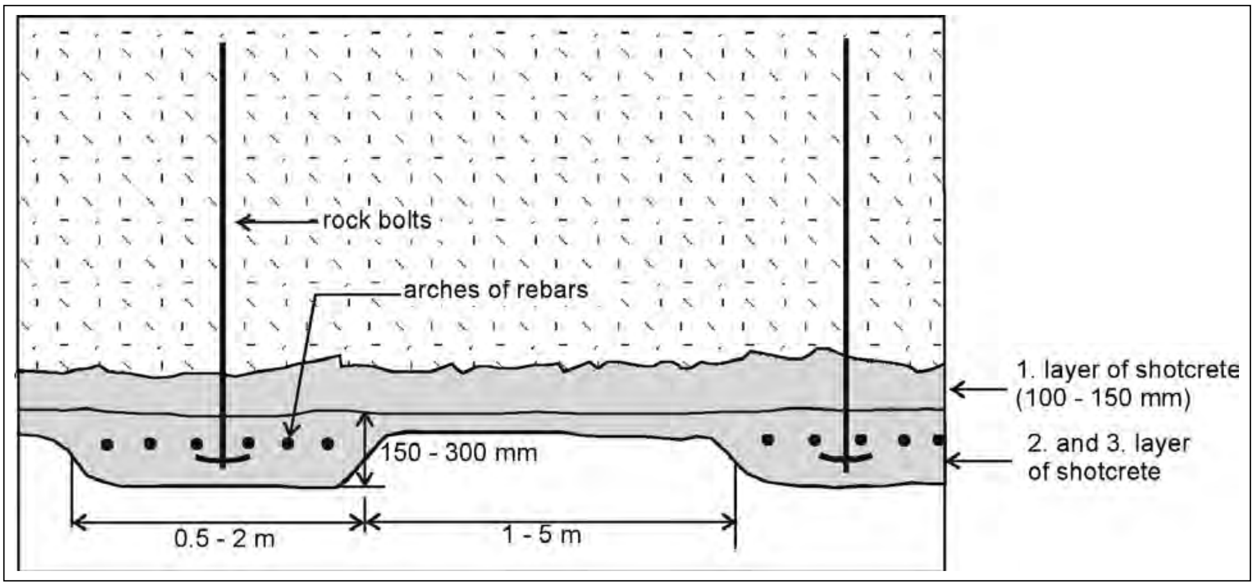

Figure 9. Typical design of shotcrete ribs used at Frøya and other Norwegian sub sea tunnels.

metre tunnel (Norwegian Public Roads Administration, 2002b). This reflects the increased demands to detailed stability and reduced maintenance.

Rock bolts have extensive corrosion protection. In the Eiksundet tunnel, the multiple corrosion protection provided by the CT-bolt, by hot-dip galvanising, epoxy coating and cement grouting applied on both sides of a plastic sleeve, provides excellent corrosion protection on the sub sea sections. For the different tunnels, the average number of rock bolts has varied from 1.5 to 7 bolts/metre tunnel.

Experience from the deepest tunnels (>250m)
The three deepest tunnels, Hitra 264m, Bømlafjord 260m and Eiksundet 287m (under construction) have not experienced any special problems. Grouting against water pressures of 2~3MPa can be efficiently achieved with modern packers, pumps and grouting materials. Grouting pressures up to10MPa are today quite common with modern grouting rigs as shown in Figure 10.

Figure 10. Andersen Mek. Verksted high pressure grouting rig.

Operational experience
Water ingress at time of opening has varied from 20 to 460 litres/min per km, depending on the conditions. The normal target upper level is 300 litres/min per km, which is achieved by pre-grouting. The remaining water seepage has stayed constant or reduced by self-sealing (up to 50% reduction) in all tunnels.

A curious problem in some tunnels is the development of algae in the drainage water and the pump sumps. The algae population seems to expand to a certain level, collapse and then expand again. If exposed onto the driveway, which shall not normally happen, the algae make the asphalt slippery.

A number of installations have to be replaced periodically (‘re-investment’), including pumps, drainage pipes, electrical installations and water/frost shielding. The tunnel environment is normally quite corrosive, in sub sea tunnels in-leaking saline water makes it even harsher. The recommended and experienced lifespan for installations may vary from 15 to 40 years. Steel quality shall be corrosion resistant. Typical costs for re-investment, maintenance and operation for several tunnels have been 65-130 USD/metre per year, or 1-1.5% annually of the initial investment.

Costs for electricity amounts to 25-50% of the annual maintenance and operation cost, with ventilation taking the highest share.

For the water & frost shielding, several solutions have been tried:
- Corrugated aluminium sheets with rock wool insulation;
- Polyethylene foam with sprayed concrete fire protection;
- GPR sandwich segments;
- Aluminium or steel panels with insulation;
- Pre-cast concrete segments.

Some of the aluminium sheets have shown too little resistance to corrosion and to the air pressure impact loads from passing trucks.

Accidents
Statistics related to accidents and road tunnel safety is available from an extensive study performed in 1997 including almost 600 road tunnels (Directorate of Public Roads, 2002). The tunnels were divided in zones: 50m outside and inside the portal, the next 100m and the middle part of the tunnel. A special study was performed for 17 sub sea tunnels opened before 1996. Of these 9 were longer than 3.5km and all had an AADT <5000.

The mean accident rate for the 17 sub sea tunnels was found to be 0.09 accidents per million vehicle-km per year, which is comparable to the accident rate in the middle zone in the 1997 study. This rate is lower than on open roads in Norway. In the tunnels with 3 lanes (an extra lane in the steep slopes), the accident rate was 0.07. The accident rate for the Tromsøysund tunnel with two tubes was as low as 0.05. The tunnels with a slope of 9-10% had an average rate of 0.18, whereas in tunnels with slope 8-8.5%, the rate was 0.06. The highest accident rate of 0.45 was recorded for the Hvaler tunnel. There is a pronounced overweight of young drivers in the accidents.

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

FUTURE DEVELOPMENTS

In the Nordic countries many more tunnels of the same kind as described in this paper will be built in the future, providing practical and cost efficient solutions to replace ferry connections. In Norway this includes a very challenging project across Boknafjorden in Southeast Norway (Rogfast), where the possibility of a 24 km sub sea road tunnel at depth down to about 400 m below sea level is looked into. The expected AADT is 3,000-4,000. This project emphasises the need to develop safe but economically realistic solutions. Several alternative solutions concerning design are being looked into, including one versus two tubes. In the planning process, the experience from construction and operation of the 25km long Lærdal road tunnel undoubtedly will be of great value.

An even deeper tunnel, although not as long as Rogfast, was under planning some 10 years ago, at Hareid-Sula Northeast of the Eiksundet tunnel presently under construction. This tunnel was intended to reach 630m depth, with a length of 16.8km. The tunnel was considered technically feasible, but did not reach the priority list for the main stem road along the Northwest coast for economical and political reasons (Beitnes, 1994).

Sub sea road tunnels have been elaborated for inter island connections to replace ferries in both the Shetland Islands and the Orkneys. A connection between Orkney Islands to the mainland of Scotland, that involves an 11-12km long tunnel, is also an idea being looked into. In Greenland a private initiative has been taken to develop a new airfield at Nuuk with necessary infrastructure including a sub sea road tunnel. Åland, a group of 6500 islands in the Baltic sea, has numerous ferry connections and efforts have been undertaken towards sub sea tunnelling as an alternative for improved infrastructure. These projects have in common that the technology has yet to be accepted as none of the countries have their own experience yet with sub sea tunnels. The Norwegian concept as adapted in Iceland and the Faroe Islands is considered feasible as the traffic density and the bedrock conditions are comparable to those in Norway.

The proposed EU tunnel safety directives presently under finalisation will have consequences both for existing and future sub sea tunnels. In particular this relates to maximum slope, where 5% most likely will be a restriction for all tunnels in the Trans European Road Network (TERN). For TERN tunnels with AADT >4000, there are likely to be requirements to evacuation for each 500m, radio and ventilation for tunnels >1km, control centre and video surveillance for tunnels >3km, besides requirements to emergency lay-bys, telephone, fire fighting etc.

For already constructed tunnels, it shall be assessed how the new requirements best can be implemented. New structural requirements, such as the slope, do not apply to existing tunnels, but it shall be documented how similar safety can be achieved.

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Twin track rail tunnel Asker –Jong for high speed service in the Oslo area. Courtesy: Jernbaneverket (National Rail Administration).

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THE STAD NAVAL TUNNEL PROTECTION OF SHIPS IN STORMY WEATHER

Eystein Grimstad

ABSTRACT

The Sea outside the Stad Peninsula in western Norway is dangerous, and is known for having the highest wind velocities along the Norwegian coastline. The paper introduces the plans for a tunnel project to reduce sea hazards for small and medium sized vessels

BACKGROUND AND LOCATION OF THE TUNNEL

The sea outside the Stad Peninsula in western Norway is dangerous, and is known for having the highest wind velocities along the Norwegian coastline. During the years 1970-1990 eight ships were wrecked, and ten fatal accidents with loss of lives occurred. Making a tunnel through a narrow neck of land between two fjords about 25 km inside the tip of the peninsula will improve the conditions considerably. The Norwegian Parliament decided to allocate funds for a thorough investigation of a naval tunnel in 1999. The counties Sogn & Fjordane

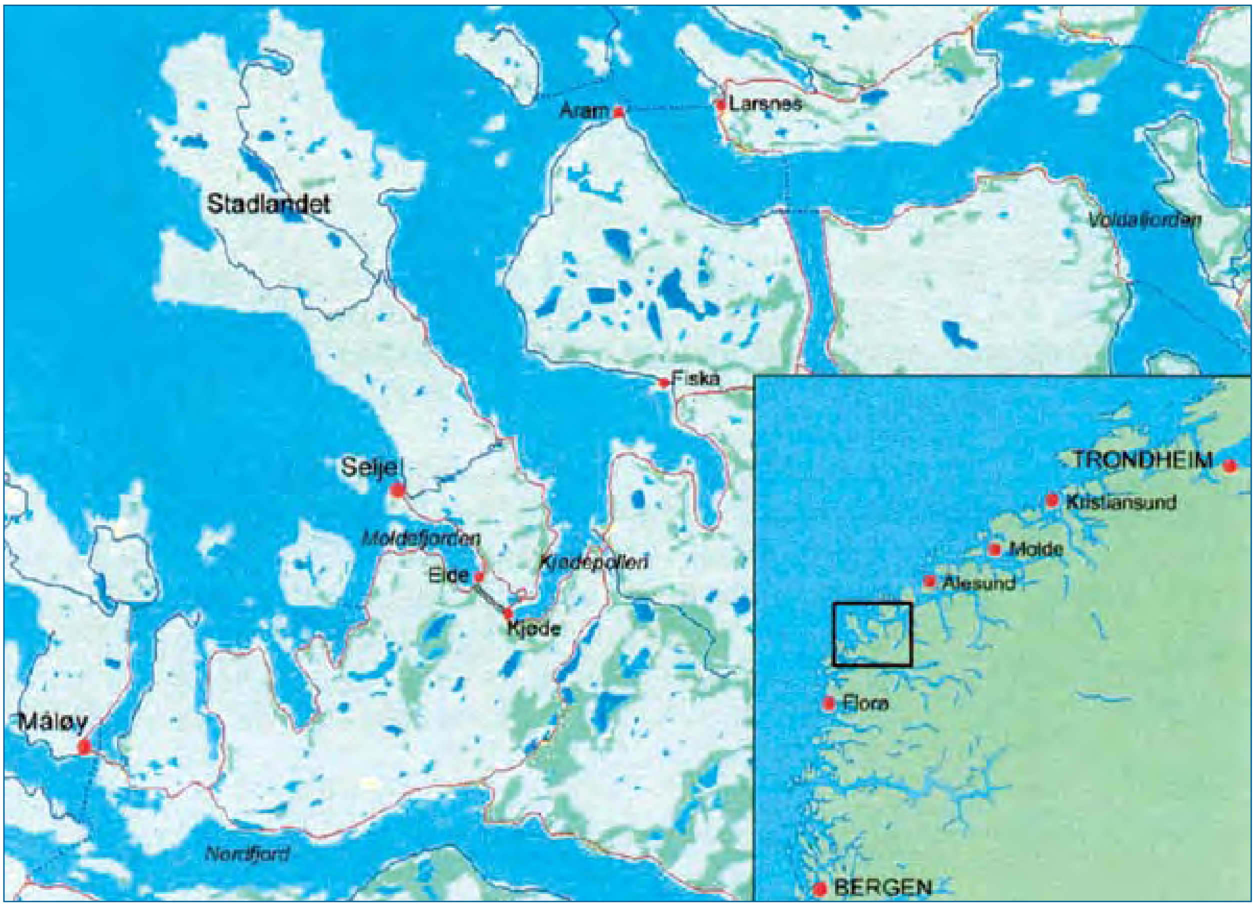

Figure 1. Map of the area and location of the tunnel at the Stad Peninsula

and Møre & Romsdal, where the proposed tunnel is located, contributed to the funds with 25%.

The National Coastal Administration, which is a part of the Ministry of Fisheries and Coastal Affairs created a project, which involved many disciplines and companies. The following disciplines and companies were involved: Rock and soil engineering: Norwegian Geotechnical Institute (NGI), Traffic analyses and cost analyses: Asplan Viak AS, Maritime conditions: Det NorskeVeritas (DNV) and The National Coastal Administration, Safety: Det NorskeVeritas (DNV), Environmental conditions: Norwegian Institute for Water Research (NIVA) and Norwegian Institute for Air Research (NILU), Hydraulics: The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology (SINTEF), Construction: Instanes AS, Ventilation: Public Roads Administration, County Sogn & Fjordane.

ADVANTAGES OF THE PROPOSED TUNNEL

Sheltered location during stormy weather will make small and medium sized ships able to enter the tunnel at all weather conditions. Wave heights, currents, tidal water, effects on the environment in both fjords during and after tunnelling etc. have been explored as a part of the project. Also cost effectiveness, traffic density, saving of lives and property by avoiding the open sea in poor weather and safety during crossing the tunnel including fire protection has been investigated. The tunnel will shorten the transport length for goods and make it possible to create a regular and reliable communication line for express boats between the local communities. The shortest distance between the fjords is about 1900m. Because of the deep rock cuttings at the entrances the planned tunnel will be reduced to about 1795m length. The highest rock overburden along the tunnel will be about 340m above the tunnel crown.

SIZE AND SHAPE OF THE TUNNEL

Making a wide and high tunnel has to be balanced against economic limits of the project. Evacuation routs in case of fire or other accidents need space, as well as mechanical ventilation and fenders for stabilisation of the ships during sailing through the tunnel. In order to take ships up to 23m in width the tunnel has to be 27m wide after excavation, before any installations. The minimum water depth below low tide is 12m. The maximum difference between low and high tide is 3m. The height of the tunnel crown above average sea level is 25m. The total excavated height of the tunnel will be 38m as indicated in Figure 2. The crown will be shaped

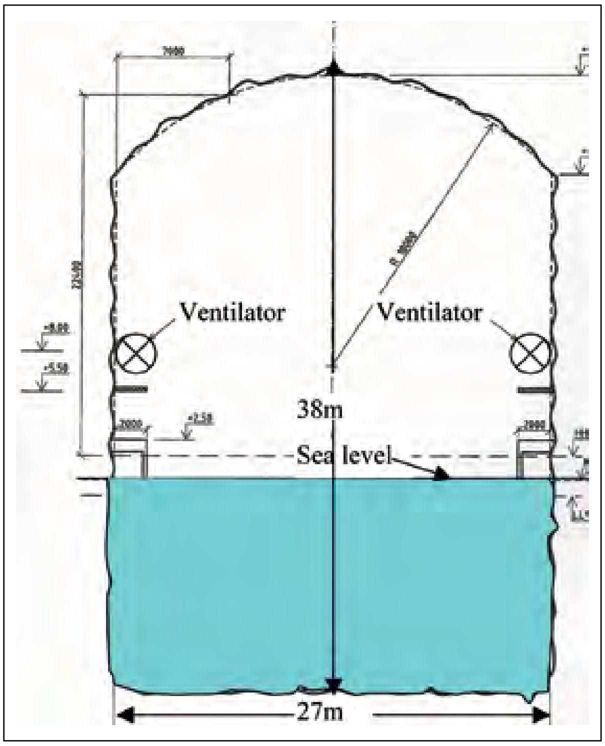

Figure 2. Cross section of the Stad Naval Tunnel.

as an arch with a planned radius of 18m. The cross section of the tunnel will be 980m2. Both entrances will be widened outwards to the fjords up to a width of about 65m at the original shore line. Ship barriers with fender beams will continue from the cuttings 20 to 30m outside the original shore line in order to prevent ships to run aground.

GEOLOGY AND ESTIMATED ROCK MASS QUALITY

Geological mapping of rock types, weakness zones, joint directions, joint characters, weathering etc. was carried out from the terrain surface. Further investigation was done by refraction seismic in areas on land covered by sediments. Core drillings were performed in areas where heavy weathering or weakness zones were encountered at the surface. Refraction seismic, boomer and echo recorder together with test drilling was carried out in the sea bottom from a survey boat in the fjords on both sides of the tunnel. Some moderate weakness zones, covered by soil were found at both ends of the tunnel. Some laboratory tests were carried out in order to find mechanical parameters and the content of different minerals for possible application of the excavated rock mass. The volume of excavated rock from the tunnel will be about 1.8 mill. m3. In addition excavated rock from the cuttings and other areas add another 1.0 mill. m3. According to the mechanical properties most of the rock may be used as aggregate for roads and railways. If excavated in large blocks, a large part of the rock may also be used for protection against erosion from waves or rivers.

The main rock type expected in the naval tunnel is light grey, banded gneiss which contains 23-33% quartz, 51-61% feldspar, 9-13% mica and 0.5-4.5% epidote. Up to 32% mica, and up to 13.5% amphibole was observed in layers of mica schist/mica gneiss. Locally it was up to 48% quartz in the gneiss. At two places eclogite was found. The border between eclogite and gneiss is heavily weathered, and appears as rusty soil near the terrain surface. Core drillings show shallow weathering (some tens of meters). The weathering is not expected to reach down to the tunnel.

The rock mass is dominated by foliation joints and two other joint sets. Along most of the tunnel at least one of the joint sets will have an acute angle to the tunnel axis which may affect the stability. Except the weathering no large real poor weakness zones are found along the tunnel. From observations at the surface and from core logs the rock mass quality is mapped using the Q-system. In limited areas extremely poor rock ($Q > 0.01$) is found. Between 3 and 6% of the tunnel length is expected to have Q-values lower than 1 (very poor rock mass). About 45% of the tunnel is expected to be in the category good ($10<Q<40$), and 35% is expected in the category poor to fair ($1<Q<10$).

ESTIMATED ROCK SUPPORT

Observations from the bore holes indicate rather low rates of leakage at the tunnel level, except near the ends of the tunnel which are close to the surface. Therefore little systematic grouting is expected during the excavation of the tunnel. According to Norwegian traditions and experiences the rock support is expected to consist mainly of rock bolts and fibre reinforced sprayed concrete. The rock support is estimated for three cases: The worst case, the most probable case and the best case. The largest uncertainties are related to the weathering. In case the weathering is going down to the tunnel, cast concrete lining is likely to be used in combination with application of spiling bolts. Table 1 shows the estimated rock support in the three cases. A large part of CCA may be replaced by RRS if narrow zones of weak rock are excavated.

The total cost of the Stad Naval Tunnel (excavation, rock support and access roads included) is estimated to 347mill. NOK » 45mill. EURO in the year 2000. When all other costs (client's administration, fender beams, evacuation routes, ventilation, illumination, warning systems, other electrical installations, contractor's general costs etc.) are added the total cost was estimated to be 703mill. NOK » 90mill. EURO or 105 USD in the year 2000.

Type of support	CCA (m^3)	RRS (no.)	S(fr) (m^3)	S (m^3)	Bolts (no.)	Spiling bolts (no.)
Best case	329	13	7500	5500	21100	750
Most probable	1225	22	9000	5900	23300	1340
Worst case	3950	34	11500	6100	26200	2900

Table 1. Total amount of rock support for three levels of rock mass qualities. CCA = cast concrete lining, RRS = reinforced ribs of shotcrete, S(fr) = fibre reinforced shotcrete, S= plain shotcrete, B = rock bolts.

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