1 INTRODUCTION TO WATER CONTROL IN NORWEGIAN TUNNELLING

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ABSTRACT: The rock mass is a significant barrier in itself. However, it is a discontinuous media, and its hydraulic characteristics may vary widely, from impervious sound rocks to highly conductive zones. Based on environmental sensitivity analysis, water control should be applied to avoid negative impacts caused by tunnelling. A standard procedure in Norwegian tunnelling is to perform pre-grouting of the rock mass to obtain the required tightness. This procedure has been developed from the early metro tunnels in the city of Oslo, through unlined, high pressure water tunnels for hydro power projects, underground oil and gas storage and sub-sea rock tunnels, to the current urban tunnelling.

This introduction lists the various reasons for such water control, and provides an overview over the cost effective tunnelling developed in Norway during the last decades. The other articles in this publication will further detail these aspects.

1 INTRODUCTION AND BACKGROUND

Norway extends some 2100 km from its southern tip to the far north-east corner. Mountains and valleys, deep fjords in the west, and a widely spread population present numerous challenges for infrastructure construction. It has been important to establish a tunnelling technology that enabled the development of a modern infrastructure with such extremes to be overcome. On the other hand, our climate and topography provides a great potential for hydropower development. The hydropower construction required an extensive use of tunnels and underground caverns and contributed to the development of this tunnelling concept. In the 1970'es the oil and gas era in Norway began, and underground facilities were used for transport and storage of hydrocarbon products. Norwegian tunnelling can be characterised by cost effectiveness, flexibility to adapt to changing ground conditions, safe internal environment for the users, and preservation of the external environment.

The following elements have been important for this development:

- pre-grouting of the rock mass to achieve water control,
- utilising the self-supporting capacity of the rock mass,
- establishing drained support structures.

The rock mass itself is often an excellent barrier, having a significant capacity with regard to its tightness characteristics, but owing to its nature, it is not homogenous and its characteristics can vary greatly.

The allowable amount of water inflow to a tunnel is in some cases governed by practical limitations related to the excavation process and pumping capacity, which may result in a draw down of the groundwater level. A commonly used figure in Norwegian sub-sea road tunnels (where the water supply is infinite!) is a maximum inflow to the tunnel of 30 litres per minute and 100 meter, see Blindheim et al. (2001b).

Requirements to the surrounding environment may restrict a draw down to take place. This is applicable in urban areas to avoid settlement of buildings, and where restrictions on groundwater impacts due to environmental protection are required. Projects have been realised where the allowable inflow was in the range of 2-10 litres per minute and 100 meter, see Davik et al. (2001). The primary objective is to employ methods that aim at making the tunnel tight enough for its purpose.

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

2 PURPOSE OF GROUNDWATER CONTROL IN TUNNELLING

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

* Prevent an adverse internal environment. For various reasons tunnels and underground openings are subject to - strict requirements to obtain a safe and dry internal environment. In many cases such requirements do not allow water appearing on internal walls or the roof in the tunnel.
* Prevent unacceptable impact on the external, surrounding environment. Tunnelling introduces the risk of imposing adverse impacts to the surrounding environment by lowering the groundwater table, which may cause settlements of buildings and other surface structures in urban areas and disturb the bio-types, natural lakes and ponds in recreational areas.
* Maintain hydrodynamic containment. The concept of unlined underground openings is used for such purposes as oil and gas storage, cold storage, tunnels and caverns for pressurised air, nuclear waste repository, and other industrialised disposals. “Watertight tunnelling” in this context is to provide a containment to prevent leakage of stored products.

3 NORWEGIAN HYDROGEOLOGICAL CONDITIONS

In Norway the hydrogeological situation is dominated by a high groundwater level. An advantage of this is that it provides a natural hydraulic gradient acting towards underground openings. This allows for the utilisation of unlined underground storage facilities. A disadvantage is the risk that tunnelling activities may disturb the groundwater situation, thus imposing the potential of adverse impact on surface structures and bio-types.
Norwegian rocks are in practical terms impervious with permeability (k) in the range of 10-11 or 10-12 m/sec. Individual joints may have a permeability (k) in the range of 10-5 to 10-6 m/sec. The rock mass is consequently a typical jointed aquifer where water moves in the most permeable discontinuities or in channels along them. The permeability of such rock mass consisting of competent rock and joints may typically be in the range of 10-8 m/sec. This implies that the most conductive zones in the rock mass must be identified and treated. An appropriate solution must be determined to deal with such zones to prevent the tunnel from causing a lowered groundwater table.

CONTROL OF GROUT MIX.

4 BRIEF DESCRIPTION OF THE TUNNELLING

4.1 Groundwater control

During recent years, analyses aiming at identifying the vulnerability and sensitivity of the surroundings have been introduced, particularly focusing on local bio-types in combination with water balance studies, see Grepstad (2001), and also Kveldsvik et al. (2001). Johansen (2001) reports that numerical modelling of groundwater fluctuations has become a valuable tool for such analysis together with traditional empirical formulas. Empirical formulas for analysis of ground settle-

ments and potential damage on surface structures are also available, according to Karlsrud (2001). These are all important aspects to be considered for the determination of the maximum acceptable level of water inflow to the tunnel.

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

Pre-defined grouting criteria will govern the progress of the tunnelling works, as descried in many of the articles in this publication. The tunnel will not be allowed to advance until these criteria have been met, which includes that more than one grouting round may be needed. In areas highly sensitive to groundwater fluctuations probe-drilling and pre-grouting may be executed continuously along with the tunnel advance, e.g. such as every 20 to 30 m and with a specified overlap between each round according to project specific requirements. A pre-grouting round may typically include some 10 to 30 holes, drilled in a specified pattern to create a trumpet shaped barrier in the rock mass, see Figures 1 and 2. The length of grout holes may vary from 15 to 35 m with an overlap of 6 to 10 m between each grouting round, if continuous grouting is required. The pre-grouting scheme must cover the complete 360 degrees of a tunnel and include regulations for control holes and success criteria for the grouting work. Pre-grouting is by far the preferable method to post-grouting. Post-grouting is often an intricate, time consuming and costly process and the result of post-grouting schemes may be rather uncertain and variable.

GROUT PROCESSING AND CONTROL.

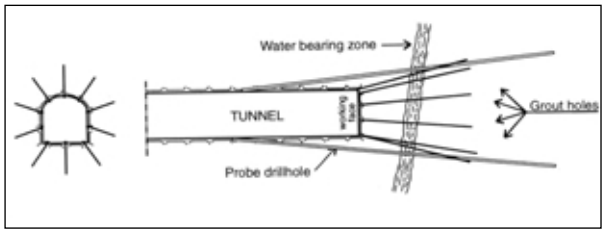


Figure 1. Typical probing and pregrouting setting.

Another typical way of assuring groundwater control is by artificial, pressurised water injection or infiltration in the ground through so called water curtains adjacent to the underground openings. This method is commonly applied for hydrocarbon storage and for unlined air cushion chambers in hydropower schemes, see Broch (2001a). Through such arrangements a groundwater gradient acting towards the opening is maintained as the enhanced groundwater pressure is greater then the internal pressure in the caverns. The stored product is thus provided a hydrodynamically created containment in an unlined storage facility. Water injection may also be applied to restore an accidently reduced groundwater level.

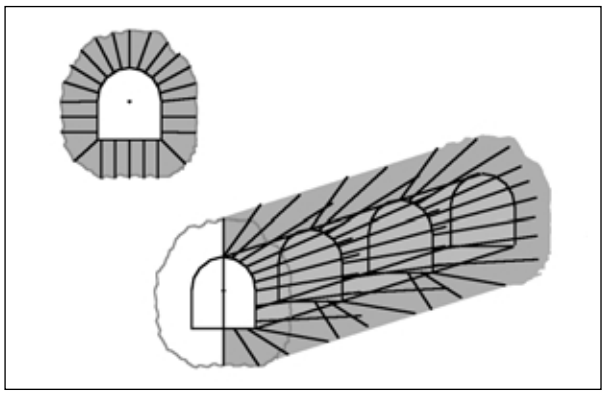


Figure 2. View of pregrouted zone.

The groundwater has seasonal changes as well as cyclic changes over several years. Consequently, it is necessary to define the level of acceptable inflow to a tunnel, the level at which the water balance has to be restored. A new regulation has been proposed in Norway which indicate that a residual flow of more than 5 – 15 % of the mean annual flow from the catchment areas will not be accepted. This is another way of dimensioning the maximum allowable inflow to a tunnel.

The effect of the grouting schemes must be followed-up by an appropriate monitoring program, see Grepstad (2001). Such monitoring may typically include leakage measurements inside the tunnel, water head measurements of the ground water in the rock mass or water level measurements in neighbouring surface wells, dedicated observations holes or lakes/ponds.

4.2 Self-supporting capacity of the rock mass
Most rock mass has a certain self-supporting capacity, although this capacity may vary within a wide range. The fact that there is some "stand-up" time implies that the rock mass for a certain time period is not a dead load, thus it shall not be treated as if it was. An appropriate engineering approach is to take this capacity into account when designing permanent support. Rock strengthening may, however, be needed to secure certain properties/specified capacities, in the same way as is the case for any other construction material. In Norway, permanent rock support consists typically of rock bolts and sprayed concrete. Further descriptions can be found in Grøv (2001).

4.3 Drained tunnel structure
In our tunnelling, the rock mass in combination with the rock support constitutes a drained structure. This means that the support measure installed has not been designed or constructed to take external water pressure. Excessive water must therefore not be allowed to build up behind the rock support measure.

However, even in a tunnel that has been subject to extensive pre-grouting, some seepage may occur. A dry environment, where the surface is free from all visible seepage and damp patches, can be achieved by installing a water protection and drainage system, either locally at wet spots, in larger sections of the tunnel or as a full coverage. A controlled handling of excess water at the tunnel periphery and behind the sprayed concrete lining is required. Excess water is either piped to the water collection system in the tunnel or taken care of by a water protection system. Drainage can be achieved by installing, for example, local collection devices to confine the water and transfer the water via pipes to the drainage system in the tunnel. A number of different solutions have been tested in Norway, Broch (2001b), some are related to tunnels with low traffic volumes, whilst others are applicable for high traffic volumes. Common for these methods is that they do not interact with the rock mass support measures.

Sprayable membranes have also been launched to enable the building of a water drainage structure at the rock surface, or as an interlayer between two subsequent layers of sprayed concrete.

5 COST ASPECTS

Taking standard cost into account, cost comparison figures are available in published articles. According to Garshol (1997), the cost ratio between a concrete lined tunnel and one with fibre reinforced sprayed concrete is approximately 500:225 (excavation costs are 100).

Another article, Aagaard et. al. (1997), demonstrates the cost variations as a function of the actual rock mass conditions. Depending on the rock mass class the cost ratio between concrete lining and sprayed concrete + rock bolts + reinforced ribs varies from 4:1 in poor rock mass to 4:3 in exceptionally poor rock mass conditions. Figure 3 shows the cost comparison for unlined vs. cast-in-place concrete lining. (1 USD is equal to 9 NOK). In adverse rock mass conditions it seems as if the two alternatives are more or less equal from a cost comparison point of view.

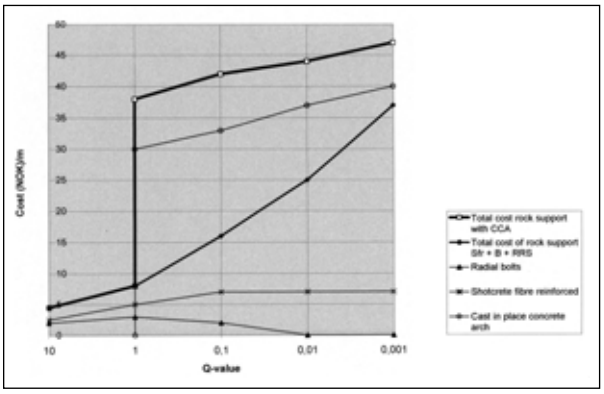


Figure 3. Cost comparsion of applicable method.

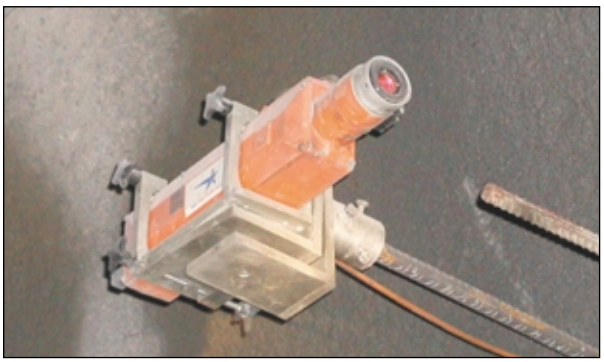
For an exemplified situation the cost aspect can be estimated. For a tunnel size of 60 m2 and in normal rock mass conditions the following cost figures can be derived. The excavation costs are set to 100.

Table 1. Cost comparison (excavation costs are set to 100)

Applied support elements	Cast-in-place concrete lining	Norwegian tunnelling
Temp. rock support*	30 – 80	NA
Rock mass probing/grouting	50 – 150	100 – 250
Perm. rock support at face**	NA	80 – 200
Watertight membrane	20 – 30	NA
Concrete lining (400 mm) ***	180 – 330	NA
TOTAL CONCEPT COST	280 – 590	180 – 450

Notes: * spot bolting and plain concrete sprayed in the arch. ** systematic rock bolting and fibre-reinforced sprayed concrete. *** the cast-in-place concrete liner is applied behind the face.

Applying the above cost approximations, the alternative concept with sprayed concrete and pre-grouting is in the range of 60 to 80 % of the cost of a concept with cast-in-place concrete.



LASER FOR DIRECTION CONTROL.

For a two-shift arrangement, working 10 hours per shift and 5.5 days per week a typical tunnelling progress for a 60 m2 cross section would be in the range of 50 to 60 m per week, including all permanent rock support. Severe grout takes may hamper the progress, although a systematic probing/grouting schedule is normally included in the tunnelling procedure in such a way as to minimise the delay and maintain the high tunnelling advance rate.

PREPARE FOR EMULSION PUMPING.

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2 UNLINED HIGH PRESSURE TUNNELS AND CAVERNS

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ABSTRACT: ***More than 80 unlined pressure shafts and tunnels with maximum water heads varying between 150 and 1000 m are today in operation in Norway. The majority of these were constructed during the years 1960 - 90. The oldest ones have, however, been in operation for 80 years. The development of the general layout for hydropower plants as well as the different design criteria for unlined pressure tunnels are described. Experience from the operation is discussed. Operational experience from ten unlined air cushion surge chambers have been examined with respect to air leakage. Internal air pressure varies from 19 to 78 bars and chamber volumes from 2.000 to 120.000 m³.***

HEAVY DRILLING AT FACE.

1 INTRODUCTION

Topographical conditions in Norway are especially favourable for the development of hydroelectric energy. More than 99% of a total annual production of 120 TWh of electric energy is generated from hydropower. 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 years 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. Also, special techniques and design concepts have been developed. One such Norwegian speciality is unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. Both concepts utilise the in-situ stresses and the groundwater pressure in the rock masses for confinement. Oil and gas storage in unlined caverns is based on the same concept.

It should be mentioned as a preliminary matter that the rock of Norway is of Precambrian and Paleozoic age. Although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rocks.

2 DEVELOPMENT OF THE GENERAL PLANT LAYOUT

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. 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 . Although three out of these four pressure shafts were operating without problems after some initial problems had been solved, it took almost 40 years for the record of 152 m to be beaten. Before 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants as demonstrated in Figure 1. 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.

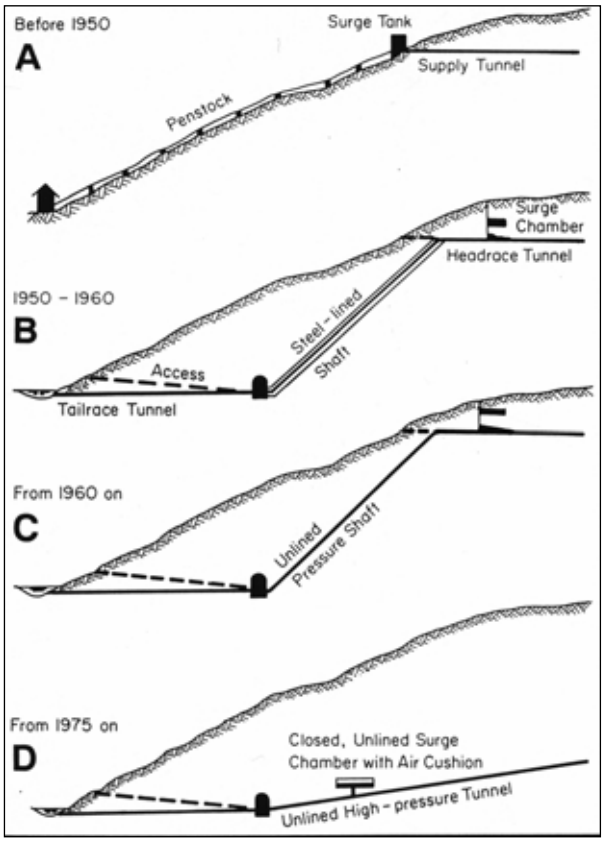

Figure 1. The development of the general layout of hydroelectric plants in Norway.

The new record shaft of 286 m at Tafjord K3, which was put into operation in 1958, gave the industry new confidence in unlined shafts. As Figure 2 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 2 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

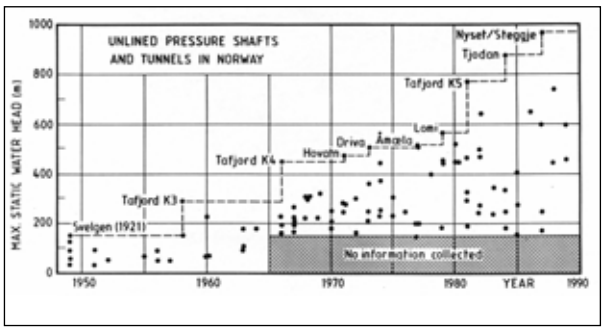

Figure 2. The development of unlined pressure shafts and tunnels in Norway.

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, Rathe (1975). Figure 1 shows how the new design influences the general layout of a hydropower plant. The steeply inclined pressure shaft, normally at 45°, 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.

Impression from tunnel face.

3 DESIGN CHARTS BASED ON FINITE ELEMENT MODELS

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 by Selmer-Olsen (1974) and 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.

In addition to real cases, a number of idealised but typical valley sides have been analysed. One example of an idealised model is shown in Figure 3.
To make the model dimensionless, the static water pressure is expressed as the ratio H/d, where the water head is expressed as a height in the same units as the valley depth (e.g., in meters). The curved lines run through points where the internal water pressure in a shaft equals the minor principal stresses in the surrounding rock mass ($\sigma 3 = H$).

The use of the design charts can be illustrated by an example. Let the bottom of the valley, where the power station is located, be situated 100 m.a.s.l. and the top of the valley side 600 m.a.s.l. This makes d = 500 m. The maximum water level in the intake reservoir is 390m.a.s.l. This makes H = 290 and the H/d ratio = 0.58. At all points inside or below the 0.58 line the minor principal stress in the rock mass exceeds the water pressure in an unlined shaft; hence, no hydraulic splitting should occur. If a factor of safety of 1.2 is introduced, the critical line will be the 1.2 x 0.58 = 0.7 line. As a demonstration, a 45° inclined shaft is placed in this position in Figure 3. A further discussion of how the input data may influence the results is given in Broch (1982).

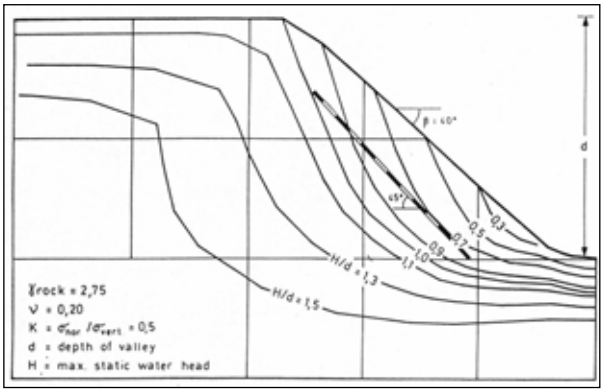

Figure 3. Design chart for unlined pressure shafts based on a finite element model. the curves run through points where the internal water pressure in the shaft equals the minor principle stress in the surrounding rock mass, H = σ_3

4 EVALUATION OF THE TOPOGRAPHY

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.

5 GEOLOGICAL RESTRICTIONS

The FEM-developed design charts are based on the assumption that the rock mass is homogeneous and continuous, an assumption which cannot be absolutely correct even for crystalline rocks like granites and gneisses. However, observations and investigations of stress-induced stability problems such as strain bursts in a large number of tunnels in valley sides clearly indicate that the natural jointing for rock masses has only minor influence on the distribution of the virgin stresses. More important for the stress distribution can faults and weakness zones be as they may cause local redistribution of the stresses, Broch (1982 B).

In rock masses with alternating layers or beds of rocks with different stiffness, the stress situation may be very different from what the idealised FEM models will show. In such cases new models should be established in which the input data for the different types of rock are carefully selected. It is important to keep in mind that in a situation where there is a combination of rocks with varying stiffness, the softer rocks will take less stress than the stiffer. When a high-pressure tunnel passes through such zones of low stress rock masses, they can locally be over-stressed which may result in the opening of joints. This may lead to severe leakage.

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.

6 HYDRAULIC JACKING TESTS

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, see Figure 4.

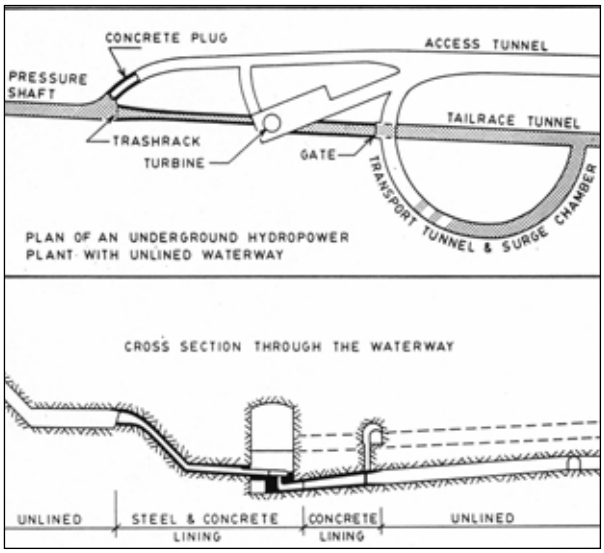

Figure 4. Plan and cross section of an underground hydropower plant with unlined waterways.

To make sure that all possible joint sets are tested, holes are normally drilled in three different directions. By the use of either pre-existing or specially designed 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 the importance for making sure that all possible joint sets are tested.

If the testing shows that jacking of joints may occur, the unlined part of the waterway will have to be put further into the rock. This will normally mean that the whole powerhouse complex is moved further in. A flexible contract which allows for such changes is therefore of vital importance when unlined high pressure shafts and tunnels are planned. Putting the powerhouse complex deeper into the rock adds length to the access tunnel, but not to the waterway.

QUALITY CONTROL.

7 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 4 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. It should be pointed out, however, that when the design charts are used, the dimensioning or critical point for inclined shafts will normally be where the unlined pressure shaft ends and the steel lining starts. This is where the selected $\sigma_3 = H$ line should intersect the waterway. 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. For vertical shafts the critical point will normally the upper elbow.

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).

GROUT PROCESSING AND CONTROL.

8 OPERATIONAL EXPERIENCE FROM UNLINED PRESSURE SHAFTS AND TUNNELS.

The oldest unlined pressure shafts have now been in operation for 80 years. None of the 60 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.

The first filling of a pressure shaft or tunnel should be done in a controlled way. The reason is that during and after the excavation of the tunnel the surrounding rock mass has gradually been drained and may even have been emptied for water. This is shown by the great reduction in the groundwater leakage into tunnels which are commonly observed over time. When the tunnel is later filled with water, the emptied joints and pores are filled too. By carrying out leakage measurements during filling, unforeseen leakage can be detected.
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. Some typical leakage curves are shown in Figure 5. The leakage from a tunnel is large during the first hours, but decreases rapidly and tend to reach a steady state after 12 to 24 hours, depending on the joint volume that has to be filled.

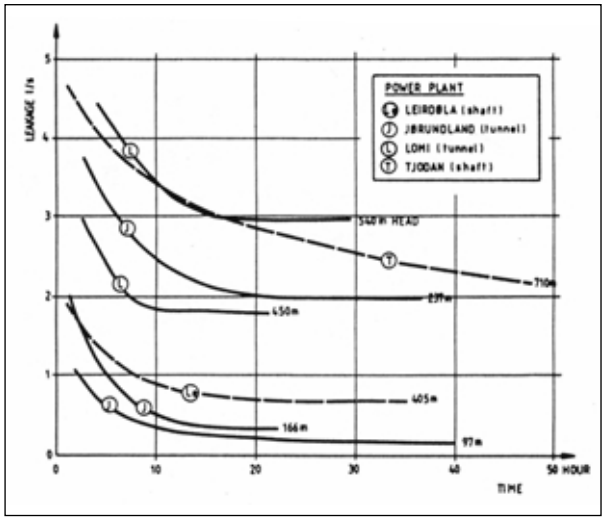

Figure 5. Measured net water leakage out from various unlined high pressure shafts and tunnels, from Palmstrøm (1987)

From some of the pressure tunnels and shafts where leakage measurements have been carried out, an average permeability coefficient of 1 - 10 x 10^{-9} m/s has been calculated. With this very low permeability a leakage of 0.5 - 5 l/s per km has been measured, Palmstrøm (1987).

9 OPERATIONAL EXPERIENCE FROM THE AIR CUSHION SURGE CHAMBERS

As part of a research programme on the air and gas tightness of rock masses, all ten air cushions in Norway have been carefully studied. The results are presented in papers like for instance Goodall et al.(1988), Kjørholt and Broch(1992) and Kjørholt et al.(1992).

Air cushion surge chambers are used as an economical alternative to the traditional open surge shaft for damping of headrace tunnel transients from changes in power plant loading. As illustrated in Figure 6, the air cushion surge chamber is a rock cavern excavated adjacent to the headrace tunnel, in which an air pocket is trapped. The surge chamber is hydraulically connected to the headrace tunnel by a short (<100m) tunnel. This solution gives substantial freedom in the lay-out of the tunnel-system and the siting of the plant. Schemes which have used air cushions have tended to slope the headrace tunnel directly from the reservoir towards the power station as indicated in Figure 2. Air cushions are also favoured where the hydraulic head of the headrace is above ground surface . In such cases, construction of an open surge shaft would require erection of a surge tower.

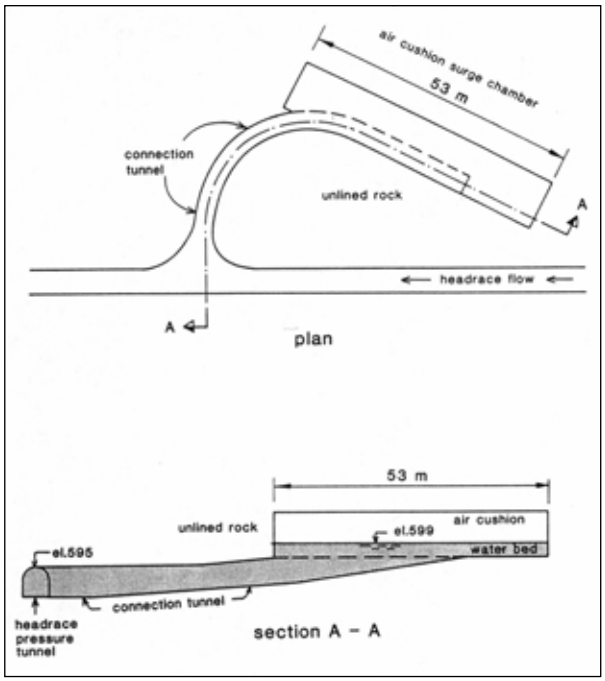

Figure 6. Plan and profile of the Ulset air cushion surge chamber.

Some basic data about the air cushion surge chambers are given in Table 1. Different gneisses are the dominating type of rock. Jointing is in general low to moderate and the estimated permeability is low, not to say very low. It should, however, be kept in mind that the places for these air cushion surge chambers are carefully selected to give as favourable rock mass conditions as possible. Hydraulic jacking tests or rock stress measurements are used when deemed appropriate to verify that the stress conditions are favourable to the concept.

Of all the surge chambers, OSA is situated at the shallowest depth (145 m). The surge chamber was the preferred alternative to a 40 m high surge tower at surface. The Osa site is the first case where excessive air loss was observed. Shortly after commissioning, an air leakage of about 900 Nm^3/h developed. Grouting was undertaken at pressures as high as 4.0 MPa. A total of 36 tons of cement and 5500 l of chemical grout were injected. This grouting program focused on areas where leakage was indicated by water inflow. As a result of the grouting, air loss from the cavern was dramatically reduced from 900 Nm^3/h to 100 Nm^3/h initially and to 70 Nm^3/h in the long term, which is comfortably managed by the compressor plant.

At KVILLDAL air loss from the air cushion surge chamber in the first year of operation was 250 Nm^3/h, but without evidence of concentrated leakage at surface. Because such a loss tested compressor capacity, a water curtain was for the first time installed in connection with an air cushion surge chamber. The water curtain consisted of a fan of about forty-five 50 mm diameter bore holes with a total length of about 2.500 m and a distance between the water curtain and the chamber varying from about 10 to 20 m.

The maximum water pressure used in the water curtain is 5.1 MPa which is 1 MPa above the pressure in the chamber. At this pressure the average water flow to the curtain is 32 l/min. Air loss since installation of the water curtain has decreased by more than an order of magnitude and is today practically eliminated.

The TAFJORD air cushion was first commissioned without any leakage preventing measures undertaken. But, even though the leakage at this site was somewhat less than at Kvilldal, the compressors did not have the sufficient capacity. An attempt to grout a major fracture intersecting the cavern did not improve the leakage condition. Fortunately, the Tafjord air cushion is not crucial to the operation of the Pelton system to which it is connected. The power-plant has consequently been able to operate without a surge facility.

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

The TORPA air cushion is the only one where a water curtain was included in the original design. The water curtain consists of 36 bore holes (64 mm diameter), drilled from an excavated gallery 10 m above the cavern roof, see Figure 7. In addition to the water curtain , grouting was undertaken during construction to improve the rock condition.

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

Air leakage through the rock masses are tabulated in Table 1. Also indicated for each site is the ratio of the air cushion pressure and the natural ground water pressure.

PROJECT	CAPACITY MW	YEAR	AIR CUSHION volume , m^3	AIR CUSHION pressure, MPa	PERMEABILITY m^2	AIR LOSS Nm^3/h
Driva	140	1973	7,350	4.0 - 4.2	no data	0
Jukla	35	1974	6,050	0.6 - 2.4	1×10^{-17}	0
Oksla	206	1980	18,000	3.5 - 4.4	3×10^{-18}	5
Sima	500	1980	9,500	3.4 - 4.8	3×10^{-18}	2
Osa	90	1981	12,500	1.8 - 1.9	5×10^{-15}	900/70 *
Kvilldal	1240	1981	110,000	3.7 - 4.1	2×10^{-16}	250/0 **
Tafjord	82	1982	1,950	6.5 - 7.7	3×10^{-16}	150/0 **
Brattset	80	1982	8,900	2.3 - 2.5	2×10^{-17}	11
Ulset	37	1985	4,900	2.3 - 2.8	no data	0
Torpa	150	1989	12,000	3.8 - 4.4	5×10^{-16}	400/0 **

* Before/after grouting ** With/without water curtain in operation

Table 1. Air cushion surge chamber data

Six of the air cushions have a natural air leakage rate that is acceptable. Three air cushions have no air leakage at all through the rock mass. At four air cushions (Osa, Kvilldal, Tafjord and Torpa),the natural leakage rate was too high for a comfortable or economic operation, and remedial work was carried out. These four sites are located in the most permeable rock masses of all ten cushions, and also have ratios between air cushion pressure and natural ground water pressure above 1.0.

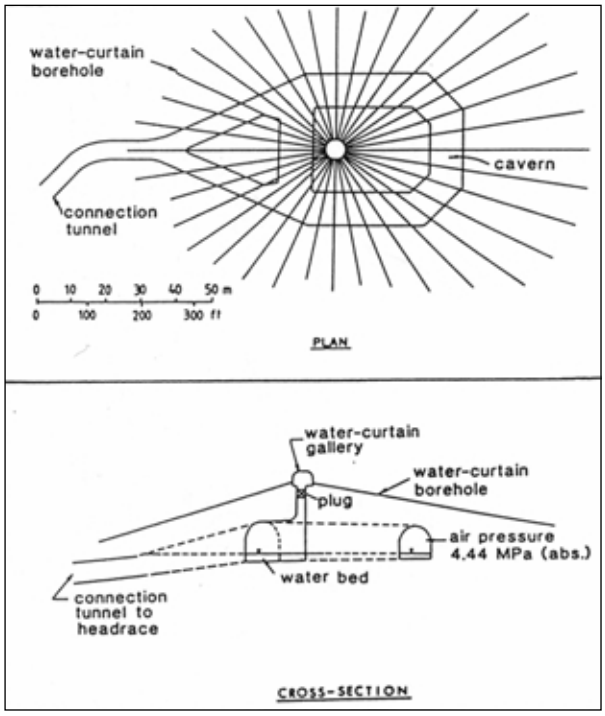

Figure 7. Geometry of Torpa air cushion surge chamber with water curtain.

CONTROL OF GROUT MIX.

10 CONCLUDING REMARKS

Experience from a considerable number of pressure tunnels and shafts have been gathered over a long period of time in Norway. These show that, providing certain design rules are followed and certain geological and topographical conditions avoided, unlined rock masses are able to contain water pressures up to at least 100 bars, equaling 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. Experience shows that the hydraulic design should follow the same principles as for an open surge shaft. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

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

In the future this experience may also be important outside the hydropower industry, for instance in the construction of cheap, unlined storage facilities for different types of gas or liquid under pressure.

Heavy equipment is good for water control in tunneling.

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3 THE WATER BALANCE – DEFINITION AND MONITORING

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ABSTRACT: The phrase "The Water Balance" means that the difference between the precipitation and the combined run off, evapotranspiration and leakage to the tunnel, is the change in the groundwater and surface water storage in the catchment area. The difficult task is to assess how large the leakage volume can be before the area is negatively affected. It is possible to calculate the variations in water storage within the catchment area if reliable run off data exists. The accuracy of the model depends on the data quality and how complex the catchment area is with respect to groundwater storage volume and delayed run off.

The most important parameter used in analytical formulas and models to predict leakage, besides the depth of the tunnel below the groundwater table, is the hydraulic conductivity of the superficial soil and sediments and the deeper rock formations.

An environmental impact assessment should focus on the areas with rich biodiversity important to preserve, as well as indicating areas where rock mass grouting is not essential from an environmental conservation point of view.

It is difficult to predict in detail the influence area of a tunnel. Hence, it is important to closely monitor the leakage from the surrounding rock mass into the tunnel during and after construction, as well as the pore pressure and water levels in lakes and aquifers in vulnerable areas above the tunnel prior to and during construction.

1 INTRODUCTION

"The water balance should not be affected" is a phrase frequently used when the success criteria related to rock mass grouting are mentioned. What does this phrase mean? How is it possible to prove that the goal has been reached and does it really matter?

In sensitive areas leakage monitoring and measurements are important parts of construction supervision. The demand for documentation of adherence to rules and specifications are ever increasing. If strict leakage limits have been set, uncertainties regarding leakage volumes could cause damage to infrastructure or environment above the tunnel or costly delays due to excessive grouting. Though the latter has seldom been the case up to now.

The influence area without mitigation measures, depends on the leakage to the tunnel, the porosity of the rock mass and the sediments, and the availability of water.

The water balance has seasonal changes and also varies from one year to the next. Further the water balance is altered as a consequence of any inflow to the tunnel. Hence "The Water Balance" is just a conception that implies that the water and groundwater level in the catchment area is within the natural range of variation.

What is important is to prevent damage to the natural environment. Upon deciding where to concentrate the grouting efforts a through knowledge is required of: (1) the value, uniqueness and sensitivity of the vegetation in

the area in a local, regional and national context, (2) how drought resistant the vegetation is, and (3) the consequences if inflow to the tunnel occurs.

This article aims at explaining the abovementioned phrase, further describe the various parameters included in the water balance equation and discuss how the parameters vary. Further the article discuss briefly what, how and where to monitor to keep control with the leakage and prevent damage to the natural environment.

2 DEFINITION OF THE TERM "THE WATER BALANCE"

The water balance for longer periods (years) in a defined catchment area is described by the following equation:

$$P = Q + E \qquad (1)$$

Where P is precipitation, Q is run off and E is evapotranspiration. For shorter periods one more term is needed, thus developing the equation:

$$P = Q + E \pm \Delta R \qquad (2)$$

ΔR is the difference in groundwater and surface water storage at the beginning and the end of a given period, Otnes et al. (1978). The above equation can be expressed as follows when there is a leakage L to a tunnel beneath the catchment area:

$$\Delta R = P - Q - E - L \qquad (3)$$

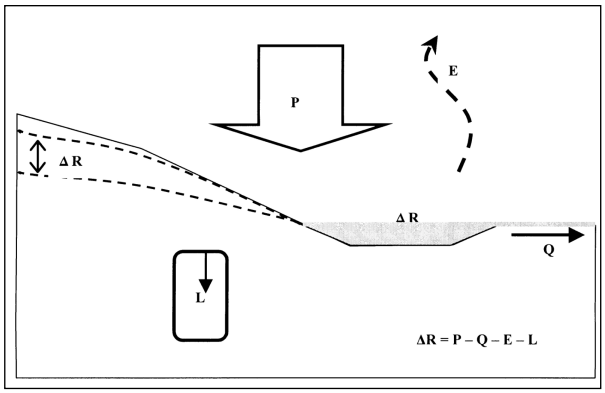

Figure 1. The Water Balance Equation included leakage (L) to the tunnel

The phrase "The Water Balance" means that the difference between the precipitation and the combined run off, evapotranspiration and leakage to the tunnel, is the change in the groundwater and surface water storage in the catchment area. The above equation shows that any leakage to the tunnel will affect the water balance in the catchment. Leakage will affect both the run off from the catchment and the water storage in the catchment in the short or long term depending on the leakage volume. The difficult task is to assess how large the leakage volume can be before the area is negatively affected.

To be able to do this assessment the parameters in the equation (3) must be properly measured (or estimated). Further the kind of impact drainage to the tunnel will have on the natural environment (especially the vegetation) above the tunnel and how drainage will affect the foundation of buildings and other kind of infrastructure must be known. The latter topic has been described by Karlsrud (2001).

3 THE PARAMETERS IN THE WATER BALANCE EQUATION

3.1 The catchment area

To delineate the catchment area is fairly simple. The uncertainty is related to the fact that a deep tunnel can also affect areas outside the catchment area just above the tunnel, if located close to a catchment boundary or if the fracture zones have a gentle dip angle. Hence it is important to have an idea about the actual size of the catchment area or the influence area of the tunnel and the dip angle of the fracture - and fault zones, when estimating the effective catchment area above the tunnel.

3.2 Climatic data (P and E)

Precipitation data *(P)* is usually available from the national meteorological department in most countries. In Norway precipitation has been recorded for more than 100 years on a daily basis throughout the country. Precipitation varies both throughout the year and from one year to the next. Precipitation data is used to estimate magnitude and variation in anticipated run off where run off measurements are not available.

The evapotranspiration *(E)* within the catchment area depends on a number of factors like; topography, vegetation, size of lakes, elevation, temperature, humidity and dominant wind direction. The evapotranspiration varies troughout the year and from one year to another just like temperature and precipitation. Evaporation measurements are conducted at approximately 50 climatic stations in Norway. One of many empirical formulas developed for use in areas far from relevant stations is presented below (Turc's formulae):

$$ET = N / [0{,}90 + (N/I_t)^2]^{1/2} \qquad (4)$$

Where ET is annual evapotranspiration in mm, N is annual precipitation in mm and I_t=300+25t+0,05t^3 where t is annual average temperature in centigrade, Hauger (1978).

3.3 Run off (Q)

In Norway the Norwegian Water Resources and Energy Administration (NVE) publish run off maps for the whole country. These maps show the annual average run off in l/s/km^2. The annual average run off is important for water balance estimations, but even more important is the run off variation trough the year and from one year to the next. The run off in dry years and during dry periods of the year is essential for assessments of how the leakage will affect the natural environment. A possible solution for catchments where run off measurements are not available, is to find a similar catchment with respect to climatic zone, precipitation pattern, size, topography and vegetation where measurements take place. Looking at the run off distribution trough the year in percentage of the annual run off in this catchment will give a fair indication about the run off distribution in the catchment that will be affected by the tunnel. The run off distribution from a catchment area in Southern Norway west of the Oslo Fjord is presented below.

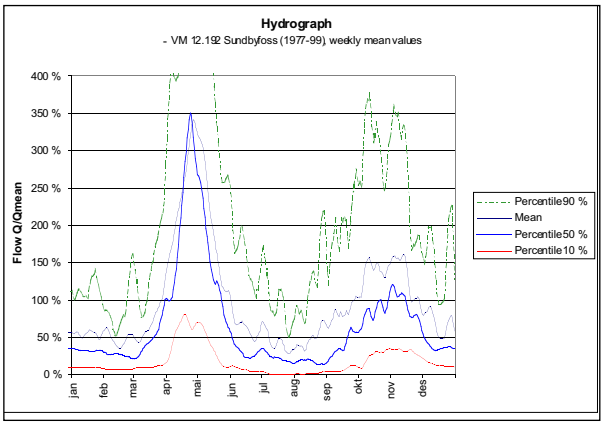

Figure 2. Run off distribution from a catchment area in Southern Norway.

Figure 2 shows that the run off is concentrated to two periods; 1) from the beginning of April to the end of May due to the snow melting period, and from the beginning of October to the end of November. During the period from June to September, the run off is close to 20 % of the mean as shown in figure 2. This period is the critical period of the year when the vegetation is in need of water. The diagram indicates further that almost no run off takes place from mid June to mid September approximately every 10th year, as indicated by the 10% percentile.

3.4 Difference in water storage *(ΔR)*

It is possible to calculate the variations in water storage within the catchment area if reliable run off data exists. The accuracy of the model depends on the data quality and how complex the catchment area is with respect to groundwater storage volume and delayed run off. In a spreadsheet based water balance model, the difference in impact between wet and dry years can be calculated. The model can estimate how much leakage the catchment can accept before it permanently affects the water level in lakes and the groundwater level.

To estimate the size and storage volume of the groundwater reservoirs is quite difficult and is seldom done prior to any tunnelling works commences. Information about groundwater level, depth to bedrock and the soil distribution are also seldom available at an early stage. The storage volume depends on the thickness and porosity of the soil. The porosity of the fracture zone is of minor importance when it comes to storage volume assessments. Further to estimate storage variations is difficult when neither knowing the water level variations nor the porosity or soil thickness. Anyhow, even inaccurate estimates of storage volumes give a fair idea about how draught prone an area will be. Even a rough qualitative assessment of the presence or lack of groundwater storage is valuable. Total porosity and effective porosity (i.a storage volume available for drainage) for some common soils and crystalline rock is presented in table 1. Most rock types in Norway are crystalline rocks, hence the porosity for all practical purposes, is nil outside fracture zones.

Material	Total Porosity [%]	Effective Porosity [%]
Clay	45 – 55	1 – 10
Silt	35 – 50	2 – 20
Sand	25 – 40	10 – 30
Gravel	25 – 40	15 – 30
Mixed Sand and Gravel	20 – 35	15 – 25
Moraine	10 – 25	2 – 20
Crystalline Rock with few fissures	< 1	<< 1
Fractured Crystalline Roc	< 1 – 10	<< 1–10

(Modified from Driscoll, 1986 and Fetter, 1994)

Table 1. Total and effective Porosity for some common soils and crystalline rock

3.5 Leakage *(L)*

There are various ways to measure the inflow to a tunnel, but unfortunately such measurements can only take place after the tunnel construction commences. During the design phase the likely leakage values have to be based on analytical formulas and computer models, like the formulae presented by Karlsrud (2001) and the model presented by Johansen (2001).These models and formulas assist in the assessments of the likely leakage value and area of influence before any mitigation measures takes place, and they can further help to indicate the effect of the planned mitigation measures. One of the most important parameters in analytical formulas and models besides the depth of the tunnel below the groundwater table, is the hydraulic conductivity of the superficial soil and sediments and the deeper rock formations. The leakage to a tunnel is proportional to the hydraulic conductivity. In table 1 the porosity of some

soils and crystalline rock was presented. The hydraulic conductivity for the same materials is presented in table 2.

Material	Hydraulic Conductivity [m/s]
Clay	$10^{-8} – 10^{-11}$
Silt	$10^{-6} – 10^{-8}$
Homogenous Sand	$10^{-3} – 10^{-5}$
Homogenous Gravel	$10^{-2} – 10^{-4}$
Moraine	$10^{-3} – 10^{-11}$
Fractured Crystalline Roc	10^{-4} 10^{-8}
Crystalline Rock with few fissures	$< 10^{-11}$

(Modified from Driscoll, 1986 and Fetter, 1994)

Table 2. Anticipated Hydraulic Conductivity Values for some common soils and crystalline rock

The hydraulic conductivity has a wide range for most materials as of a rock mass, it usually changes with a magnitude of 100 to more than 1000 over short distances. The rock mass is a typical jointed aquifer, and hence as shown in table 2, groundwater flow in rock is confined to fissures, fracture – and fault zones for all practical purposes. The following parameters are essential for the hydraulic conductivity for rock mass; rock type, degree of fracturing, the orientation and the continuity of the fractures, the interception between set fractures and rock tension. Hydraulic Conductivity is usually estimated based on water pressure measurements in drilled holes or pumping tests in water wells. The number of tests performed and the homogeneity of the rock mass decides how accurate the formulas and models can predict the leakage volume.

3.6 Influence area

The influence area without mitigation measures, depends on the leakage to the tunnel (mentioned above), the porosity of the rock mass and the sediments and the availability of water. (When it comes to damage on buildings and infrastructure caused by settlement even small leakage's can cause a large area of influence, as described by Karlsrud (2001)).

The hydraulic conductivity is the main geological factor when assessing the influence area and the possible negative impacts on the natural environment caused by a tunnel.

Overlooking damage due to settlements, table 2 shows that the largest area of influence with respect to drainage, is where sand and gravel is underlain by fractured rock. The consequences for the natural environment is not necessarily negative, due to the fact that the vegetation types thriving on sandy soils are often more drought resistant than for the example the vegetation found on bogs.

The reason why tunnels below the highest shoreline have a smaller influence area than tunnels above, is due to the low hydraulic conductivity of marine clay. Groundwater flow in clay is usually lower than in fractured rock, as can be seen in table 2.

4 RESTORED WATER BALANCE

The previous chapters show that the water balance has seasonal changes and also varies from one year to the next. Further equation 3 shows that the water balance is altered as a consequence of any inflow to the tunnel. Hence "The Water Balance" is just a conception that implies that the water and groundwater level in the catchment area is within the natural range of variation.

Consequently it is necessary to define the "acceptable" level of leakage to the tunnel, the level at which "The Water Balance" is restored. There is no uniform accept level for the water balance. The NVE has in a proposed regulation attached to the new Water Resource Law, indicated that they will not accept a residual flow of less than 5 - 15 % of the mean annual flow from the catchment area. The discussion regarding what is "acceptable" flow or not, is not yet finalised. The flow return period and the maximum allowable number of days without run off from the catchment will probably be the deciding factors.

5 VEGETATION MAPPING

What is probably more important than to ensure that "The Water Balance is not affected" is to prevent damage to the natural environment. The parameters in the water balance equation could assist in assessing the amount of water available within a catchment area. The next step is to map the terrestrial (and if applicable the aquatic) vegetation. Upon deciding where to concentrate the grouting efforts a through knowledge of the following is required: (1) the value, uniqueness and sensitivity of the vegetation in the area in a local, regional and national context, (2) how drought resistant the vegetation is, and (3) the consequences if inflow to the tunnel occurs. Drought prone locations are for example springs, bogs and valley bottoms where biotas dependent on a high groundwater table are found. However being dependent on a continuous supply of water, does not necessarily imply that the biota is a rare one and of high value. Valley bottoms usually coincides with lineaments and fracture zones, hence the co-operation between the geologist and the botanist or naturalist is essential in the environmental impact assessment.

An environmental impact assessment should focus on the areas with rich biodiversity important to preserve, as well as indicating areas where rock mass grouting is not essential from an environmental conservation point of view.

6 MONITORING

The great span in the hydraulic conductivity for geological materials and in the factors affecting the water balance indicates that it is difficult to predict in detail the influence area of a tunnel. Hence, it is important to closely monitor the leakage from the surrounding rock mass into the tunnel during and after construction, as well as the pore pressure and water levels in lakes and aquifers in vulnerable areas above the tunnel prior to and during construction.

6.1 Monitoring inside the tunnel

The acceptable leakage requirement will seldom be the same throughout the tunnel, but may vary from one section to the next, based on the sensitivity and vulnerability of the surface infrastructure and the nature. Hence, leakage monitoring and measurements in the tunnel is important also within defined sections. The demand for documentation of adherence to rules and specifications are ever increasing. If strict leakage limitations have been set, uncertainties regarding leakage volumes could cause damage to infrastructure or environment above the tunnel or costly delays due to excessive rock mass grouting.

It is important to keep control with the total amount of water flowing into and out of the tunnel. Water flowing into the tunnel comes from leakage (L) and water used for drilling, washing, etc. (Q_{in}). The flow out of the tunnel (Q_{out}) is measured at the tunnel entrance or the tunnel working face, depending on the inclination. Hence a simplified water budget for the tunnel is:

$$Q_{out} = Q_{in} + L \qquad (5)$$

Usually leakage measurements are performed during weekends and holidays, when the tunnelling activities and the water use in the tunnel is insignificant ($Q_{in} = 0$). The measured value at the entrance (alternatively working face) (Q_{out}) is thus a relevant inflow value. This system is simple and reliable as long as no activities using or adding water take place shortly prior to or during the measuring period. Regular measurements, like every weekend, during the construction period give fairly reliable time series and enable a differentiation of the inflow values section by section.

Water leaving the tunnel in addition to the drainage water is moist in the ventilation system and in the blasted rock. These water volumes are usually small and often insignificant compared to the leakage water, but should not be completely forgotten if the leakage limits are very strict. Drill holes not yet grouted is a significant error source.

Pre-grouting of the rock mass as a measure to reduce the inflow to a tunnel, is more efficient and cost effective than grouting after excavation has taken place. Hence, leakage monitoring at the tunnel working face is important. Probe drilling is often carried out to obtain information of the rock mass condition ahead of the tunnel face, such as leakage, rock mass quality and weakness- and fault zones. Decisions concerning grouting are often based on measurements or tests in these probe holes, like:

- Water pressure tests (Lugeon tests)
- Measurements of the volume leakage water from the exploratory whole(s).

Grouting is then carried out if the Lugeon or leakage value exceeds a predefined limit. If strict leakage limits have been established, exploratory drilling followed by water pressure tests or leakage measurements are also performed after the grouting to ensure that the values now are within the predefined limits.

To verify that leakage limits have been reached is impossible based on the measurements in the probe drilling holes due to the heterogeneity of the rock mass. The measurements at the entrance give an idea during the excavation, but it is also necessary to perform leakage monitoring at regular intervals in the tunnel. In the past these measurements, if performed, have been taken at barriers located at fixed intervals in the drainage ditch where the side

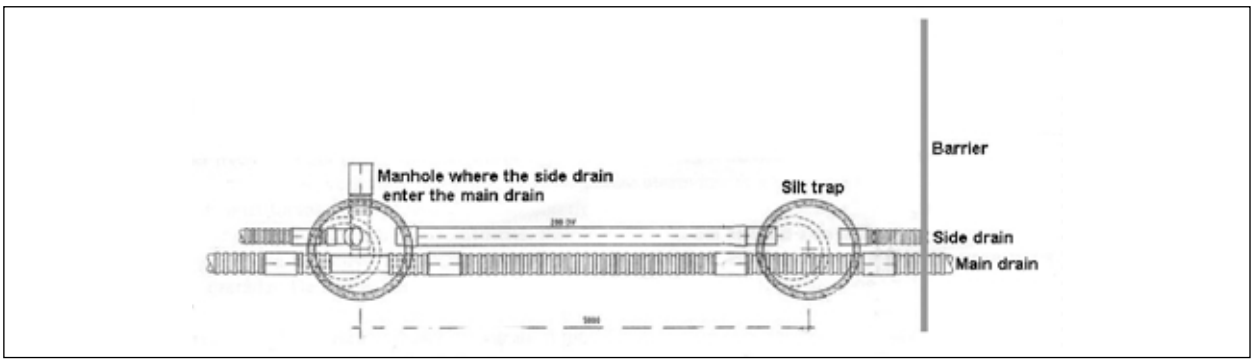

Figure 3. Permanent drainage arrangement with barrier, silt trap, side drain and main drain (NSB Gardermobanen)

drains enter the main drain. Figure 3 shows the principle for the permanent drainage arrangement with barrier, silt trap, side drain and main drain. When this arrangement is in place measurements are fairly easy.

The distance between these measuring locations should be governed by the specific leakage requirement. Sections with strict limits should have shorter distance between the measuring locations than sections with higher limits. Measuring locations should also be established at section boundaries to validate that the leakage limit has been reached within each section.

Figure 4. Permanent side drain entering the main drain (NSB Gardermobanen).

Measurements at barriers are difficult to perform during the initial tunnel construction period. Barriers are usually established late in the completion phase, when the permanent drainage system is installed. Hence, measurements at the barriers start too late to influence the rock mass (pre) grouting program. Having leakage limit boundaries in mind when locating niches needed for other purposes, like temporary silt traps, etc., increases the possibilities of starting "mid-tunnel" measurements at an early stage.

6.2 Monitoring above the tunnel

Monitoring above the tunnel comprises monitoring of water level fluctuations in lakes and groundwater aquifers, pore water pressure in clay formations and if applicable, flow monitoring in rivers and streams. It is important that the water level monitoring above the tunnel commence at an early stage, to ensure that seasonal and annual variations are known prior to the construction phase.

Installations of piezometers for pore pressure measurements in settlement prone areas have been standard procedure in tunnelling projects for years. Groundwater level measurements in rock aquifers and water level measurements in lakes have only lately become part of the standard monitoring procedure in Norway.

Because crystalline rock aquifers are confined to fissures, fractures and fault zones (as shown in table 1), the monitoring wells must communicate / cross these lineaments. The same lineaments are usually the target for the engineering geological investigations like refraction seismic, core drilling or detailed mapping. Hence, the need for groundwater observation locations can be combined with core drilling holes or holes needed for rock mass assessment using radar-, photo- or other common geophysical borehole logging instruments.

Groundwater level monitoring in soil deposits above the tunnel is usually restricted to bogs and other locations were drainage is likely to cause damage or were rare and valuable vegetation is found.

Electrical piezometers and pressure transducers with data loggers monitored via radio or telephone, ensure an instant monitoring and also an effective monitoring procedure including early warning protocols. The Client, the Contractor and if desirable, the public, are then able follow the water level and pore pressure variations on-line.

7 CONCLUSION

Either the main Contractor or the Client has up to now carried out the monitoring in the tunnel, and one or more Consultants have done the above tunnel measurements. May be a separate team should be responsible for all the needed monitoring inside and outside the tunnel. This "external" Environment Consultant should then be responsible for monitoring emission to air and water, vibration, noise, water levels, pore pressure and leakage. The Environmental Consultant would also be the most appropriate neighbourhood contact and the one responsible for reporting to the environmental authorities.

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4 CONTROL OF WATER LEAKAGE WHEN TUNNELLING UNDER URBAN AREAS IN THE OSLO REGION

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ABSTRACT: In urban areas where there are soft clay deposits above bedrock, ground water leakage into rock tunnels can cause severe subsidence and damage to structures. Such conditions are frequently encountered in the Oslo region. The paper describes semi-empirical procedures that have been developed for assessing the consequences of a given leakage level on pore pressures and settlements in clay deposits above bedrock, and how to assess minimum leakage level that can be achieved with present pre-grouting technology. The use of permanent lining and ground water re-charging as means of reducing the potential consequences of water leakage is also discussed.

1 INTRODUCTION

Construction of rock tunnels through urban areas in the Oslo region represent a special challenge due to the frequent presence of soft marine clay deposits above bedrock. Even a relatively small groundwater leakage into a rock tunnel underlying, or close to, such soft clay deposits will very rapidly reduce the pore pressure at the clay/rock interface. This will then initiate a consolidation process in the clay deposit gradually progressing upwards through the deposit as illustrated in Figure1. Such a consolidation process can lead to large settlements of the ground surface and severe damage to buildings, structures and utilities that are not founded directly on bedrock.

The nature of this problem was realised as early as in 1912-16 when the first subway tunnel through downtown Oslo (Holmenkollbanen) was excavated. This 2 km long tunnel caused settlements of up to 30-40 cm of apartment and office buildings along the tunnels, and settlements were observed as far as 500m from the tunnel alignment Holmsen(1953), Karlsrud et. al. (1978) and Kveldsvik et. al (1995).

Although the problem and mechanisms of such leakage-induced settlements (often referred to as subsidence) have been well understood, it has been a long process to find the most viable method for leakage control in such circumstances.

In connection with newer tunnels in the Oslo region one has collected data on leakage into the tunnels, pore pressures at the clay/rock interface, and settlements that have been induced. Combined with data and experiences from grouting and lining of the tunnels, this has given a very valuable contribution to the development of procedures for dealing with this tunnelling challenge.

2 TUNNEL PROJECTS AND MEASURES TO CONTROL LEAKAGE

2.1 General description

Table 1 gives an overview of most rock tunnels in the urban Oslo region which have had potential for causing leakage and settlement problems as dealt with herein. This amounts to a total of about 28 km of traffic tunnels

(subways, railways and roads) with cross section ranging from about 40 to 100 m2, and 36 km of sewerage tunnels with cross section of 10-15 m2.

All the sewerage tunnels were excavated with pre-grouting as the only means of water control. That also applies to about half of the traffic tunnels. The other half of the traffic tunnels were equipped with a concrete lining. Some tunnels or sections of the tunnels have been excavated even without any pre-grouting.

Tunnel	Year excavated	Tunnel length (km)
Holmenkollbanen	1912-16 1926-27	1,4
OTB-transport tunnel to Stortinget subway station	1968-70	0,3
OTB- subway tunnels Domkirken-Stortinget and Stortinget station	1972-75	0,9
NSB- railway tunnel Jernbanetorget-Stortinget	1973-75	0,5
NSB-railway tunnel Abelhaugen- O.K.plass	1973-79	2,0
VEAS sewerage tunnels	1976-82	23
OVK sewerage tunnels	1975-85	13
OTB- subway exchange loop, Stortinget station	1982-85	1,1
E18- road tunnel Festningen	1987-89	1,4
Rv 160 road tunnel Granfosslinjen	1990-92	2,0
Rv 160 road tunnel Bekkestua	1993-94	0,7
E18 Vestbanen intersection road tunnels	1992-94	0,4
Rælingen road tunnel	1995-96	1,4
New National theatre railway station	1996-97	0,8
Rail tunnel Romeriksporten	1995-97	14
Road tunnel Tåsen	1997-98	0,9

Table 1- Overview of tunnels in the Oslo region

The tunnels are generally fairly shallow (20 to 50 m below ground). Most of the tunnels were constructed according to conventional drill and blast techniques through predominately sedimentary rocks including clay shales and nodular limestones and in a few cases alun shale. Within these sedimentary rocks one often finds igneous dikes and intrusions which are generally highly water bearing compared to the base rocks.

The 23 km long VEAS tunnel is the only tunnel in Table 1 which was excavated by tunnel boring machines. In the 1980'ies it was a great challenge to arrange for pre-grouting through the head of the tunnel boring machines used on that project, but it was eventually solved.

2.2 Experience with pre-grouting and observed leakage

For non-grouted tunnels or portions there off, the measured leakage has ranged from about 15 to 80 l/min pr 100m tunnel. Visual observations of leakage in these non-grouted tunnels have shown that the leakage in general is concentrated in fracture zones, fault zones and/or zones of igneous dikes/intrusions in the sedimentary rocks. The inflow can be quite channelled in the worst zones, and appears as a concentrated flow of water like that out of a small hose. Such concentrated leaks have been measured to yield up to 60-80 litre pr.min. for tunnels at moderate depth Karlsrud et. al (1978). In other leakage zones more regular dripping features can be observed.

For the most successfully and systematically pre-grouted tunnels, the measured leakage has been down to 2,5 l/min pr. 100 m for the sewerage tunnels and 6 l/min pr. 100 m for traffic tunnels. There are however, fairly recent examples of larger leakage, actually almost approaching that of non-grouted tunnels.

The leakage (inflow of groundwater) into a rock tunnel can be calculated according to formulae (1) using the rather simplifying assumptions that:

- The tunnel lies in a homogenous media with constant permeability in all directions
- The tunnel is deeply embedded (h/r≥3-4)
- The groundwater table is not influenced by the leakage:

$$Q = \pi k h \frac{2}{\ln(2\frac{h}{r} - 1)} \qquad (1)$$

where, k = hydraulic conductivity (permeabilty) of rock
h = depth below water table
r = equivalent radius of tunnel

If the hydraulic conductivity of the rock closest to the tunnel face has been reduced to a value which is more than factor of about 10 lower than that of the non-grouted rock, the leakage pr unit length of tunnel can be determined from equation (2)

$$Q = \pi k_i h \frac{2}{\ln(\frac{(r_e + t)}{r_e})} \qquad (2)$$

where, k_i = hydraulic conductivity of the grouted rock zone
h = depth below water table
r_e = equivalent radius of the tunnel
t = thickness of grouted zone = r_i - r

Formula (1) and (2) have been used to back calculate the equivalent average hydraulic conductivity of the rock mass with and without grouting. The results are presented in Figures 3 and 4 where the back-calculated hydraulic conductivities are presented in relation to respectively the average normalised grout hole length and the average normalised grout take. These normalised values have been found by dividing the total length of grout holes or total amount of grout used by the internal surface area of the tunnel over the length in question. Note in relation to grout take that one has assumed that 1 kg of cement grout used is equivalent to 1 litre of chemical grout.

For the tunnel stretches where little or no grouting has been undertaken it can see from figures 3 and 4 that the back calculated average hydraulic conductivity of the rock mass typically lies in the range of:

$k = 0.8 \times 10^{-6}$ to 2.0×10^{-5} cm/sec.

Thus, it is clear that there is a significant variability in expected leakage into non-grouted tunnels depending on the nature of the rocks.

The lowest back-calculated hydraulic conductivity of the grouted rock achieved so far lie in the range:

$k_i = 2$ to 6×10^{-7} cm/sec.

This is a factor of 1/25 to 1/100 times the back-calculated permeability of non-grouted rock as given above.

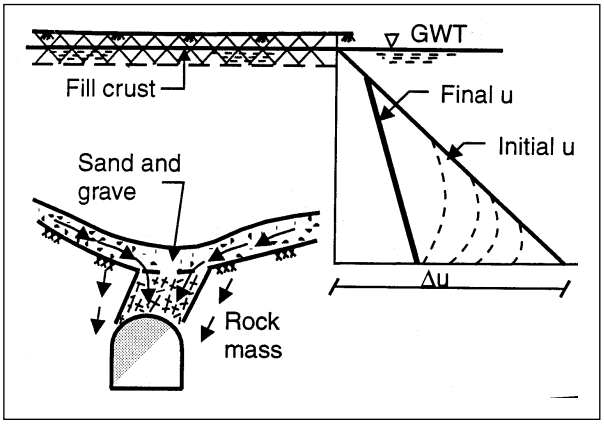

Fig.1 Influence of tunnel leakage on pore pressure in clay deposit

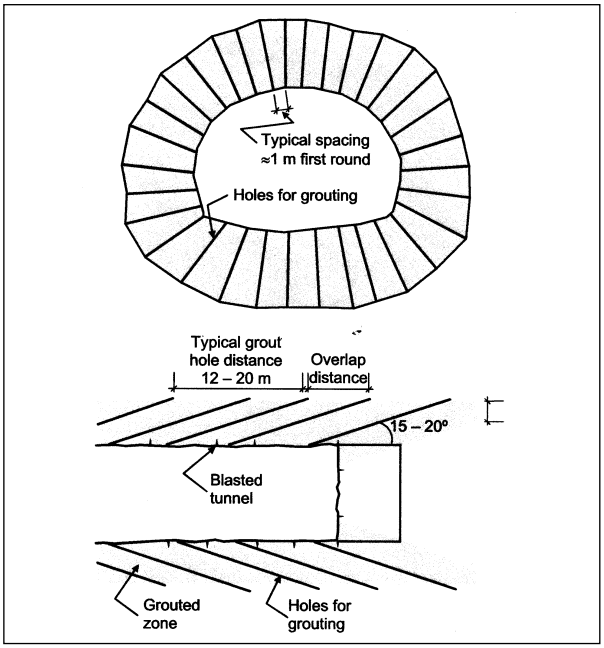

Fig.2 Principle of trumpet pre-grouting

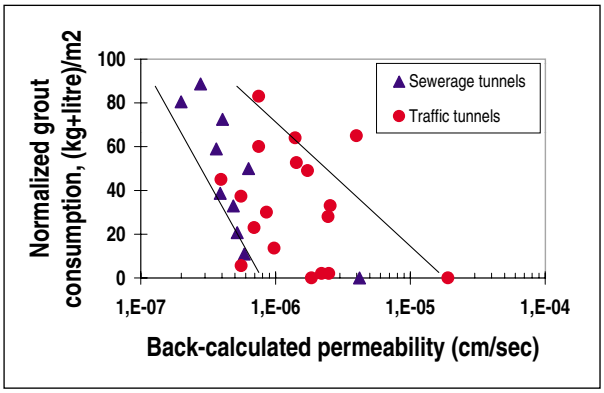

Fig.3 Back-calculated permeability in relation to normalised borehole length

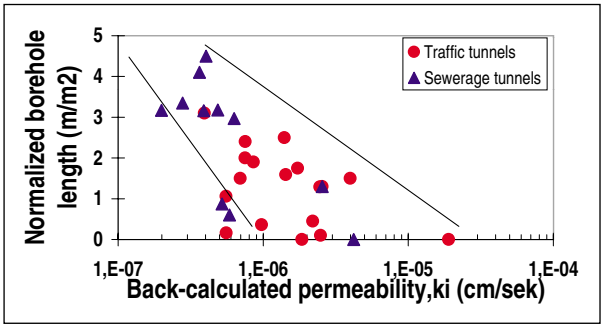

Fig.4 Back-calculated permeability in relation to normalised grout consumption

Figures 3 and 4 show that the amount of grout holes drilled, and the amount of grout used have a significant impact on the achieved reduction in permeability of the grouted rock. Some additional qualitative observations regarding the effectiveness of the grouting are as follows:

- It is a general observation that only 10-15 % of the grout holes drilled show leakage and any significant amount of grout take. This confirms the general impression from observed leakage features in the tunnels referred to earlier that most leakage in rock is concentrated in local channel-like veins. Such channel-like veins may be found at the intersection between joint or fracture systems in the rock, or be a result of joint roughness and relative displacements on the joints/fractures. In addition it may be the result of local erosion or chemical solution of joint filling materials such as calcite, which is frequently found on the joints in the sedimentary rocks in question. To intersect such water-bearing channels with grout holes requires a very dense drilling pattern. As an illustration, there are examples where about 100 grout holes were drilled over a surface area of about 10 m^2 before one hit the water-bearing vein.

• It is also a general observation that use of high grouting pressures (at least 30-40 bars) enhances the grout take and the achieved reduction in permeability. The reason is that use of high grout pressures will cause hydraulic fracturing and thus, give easier contact with the most water-bearing channels even if a grout hole is not in direct contact with such channels. Use of high grout pressures may therefore to some extent allow fewer grout holes to achieve the same result than when more moderate grout pressures (10-30 bars) are used. Use of high grouting pressures involves a potential for the grout to travel very long distances. There are several examples where the grout has reached up to the ground surface and entered into basements, sewerage and water collection pipes. Therefore, one must keep control, and set a limit on, the grout take in each hole.

• Another potential benefit of the use of high grout pressures is that it will improve the "pre-stressing" effect of the grouting on the rock mass and thus, improve the stability of the tunnel and reduce relative joint displacements and tendencies for joints to open up when the tunnel is driven.

• A number of different grout types are available in the market. The type of grout mix most suited must be evaluated on basis of the bedrock conditions and acceptable leakage level. This is of course also a cost issue. The cost of Micro-cements may typically be 3-4 times higher than Standard Portland cements, and the chemical grouts with best properties in terms of control with setting time, penetration, gel strength and synereses (Acrylamid based grouts) may be a factor of 10 times higher than Standard Portland cements.

• There is little doubt that the use of high grouting pressures has enhanced the achieved results with Standard Portland and Micro-cements and thus, reduced the needs for high-permeation chemical grouts. In a tunnel presently under construction a mixture of Standard Portland cement and silica powder has been introduced. This seems to increase the penetration of the grout and give results that may approach what can be achieved with Micro-cements. Note also that both in Norway and Sweden the use of chemical grouts has been more or less banned by public health officials due to the potential health risks.

3 RELATIONSHIP BETWEEN TUNNEL LEAKAGE AND PORE PRESSURES IN CLAY-FILLED DEPRESSIONS

Figure 5 presents a summary plot of maximum pore pressure reduction observed at the clay/rock interface, ΔuR , right above the tunnel axis for all the tunnel projects in Table 1. There is as expected a clear correlation

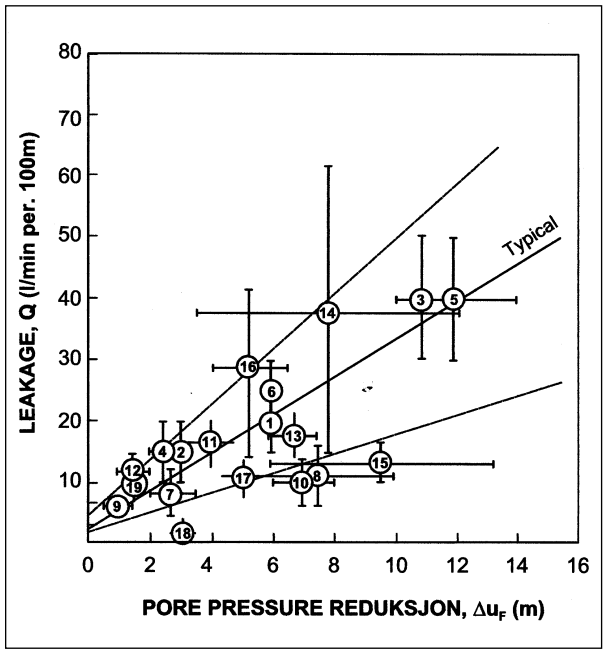

Fig.5 Measured pore pressure reduction at clay/rock interface right above tunnel in relation to leakage

between pore pressure reduction and leakage level. The scatter in the data can be explained by several factors such as:

1) Differences in the local hydrogeological conditions. The most important factors in this context are:

• Variations in the natural hydraulic conductivity of the rock (before any grouting) and especially extent of high conductivity structures and their orientation relative to the tunnel axis and the clay-filled depressions.

• If or to what extent there are coarser sediments (glacio-fluvial sands) in the transition between clay and bedrock.

• The depth and lateral extent of the clay-filled depression

• Topography and other special features that may influence the natural infiltration of groundwater.

2) To what extent the leakage measurements are representative

• There are many practical problems in measuring accurately the amount of groundwater that leaks into a tunnel under construction. It is for instance, necessary to determine the amount of water used for drilling separately.

• For the tunnels in question the leakage has been measured over distances ranging from lengths of typically 200 m to 2 km. The amount of leakage has been averaged over the measured distances, but all visual observations of leakage shows that the leakage varies considerably locally. For instance, it often seems like 80 % or more of the leakage over a tunnel length of several hundred meters is concentrated to a few zones of a few meters length. This means that the averaged leakage over several hundred meters may not be representative, especially if there is close hydraulic contact between a pore pressure measuring point and a local high leakage zone.

The data in Figure 5 shows that for the purpose of avoiding any influence on the pore pressure in clay-filled depressions the leakage must be less than about 2 to 5 l/min pr. 100 m tunnel.

Figure 6 shows the maximum observed pore pressure reduction in relation to distance from the tunnel axis. As expected, the reduction decreases with distance from the tunnel. The influence zone has been up to 5-600 m for the largest leakage levels experienced so far. A more detailed study of the data in Figure 6 has shown that the decrease in Δu_R with distance is fairly independent of the leakage level, and corresponds to about 2m pr. 100 m distance. When this is combined with the data in Figure 5 one finds that the influence distance decrease with leakage level such as typically shown in Figure 7.

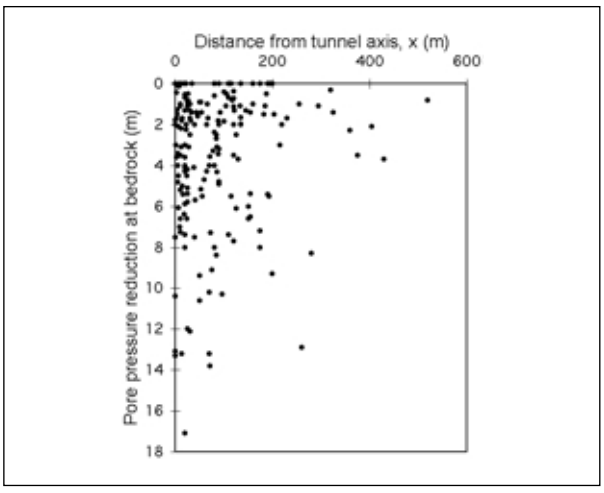

Fig.6 Observed pore pressure reduction in relation to horizontal distance from tunnel

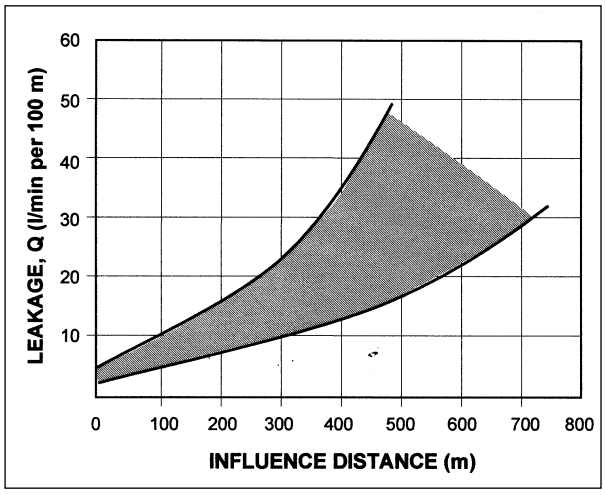

Fig.7 Typical relationship between influence distance and leakage level

4 DETERMINATION OF SETTLEMENT POTENTIAL

For a given pore pressure reduction at the bedrock level, DuR, the upward pore pressure reduction and the accompanying consolidation strains or settlements in the clay deposit can be calculated by rather conventional consolidation theory. The required input is the preconsolidation pressure, compressibility and coefficient of consolidation of the clay as determined form soil sampling and odometer tests on good quality samples. The final equilibrium pore pressure reduction (at time t = ∞) must be defined.

All experiences so far show that the upper natural ground water table in these clay deposits will not be influenced even by very large leakage unless the clay deposits are very shallow or of very limited lateral extent. The reason is that the actual flux of water through the clay deposit even under a large seepage gradient is far less than the natural infiltration from precipitation. The final equilibrium condition will therefore be given by a steady state downward seepage solution. If the permeability of the clay is constant with depth, this means that the equilibrium pore pressure distribution will be linear as depicted in Figure 1.

Figure 8 shows typical final consolidation settlements for Oslo clay condition in relation to Δu_R and the thickness of the clay deposit if the clay is more or less normally consolidated. The clay deposit may however, in some areas be somewhat overconsolidated which can significantly reduce expected settlements.

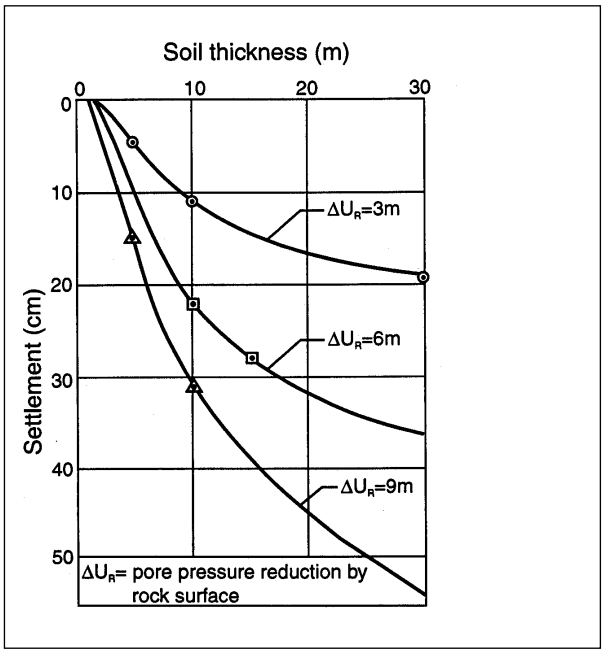

Fig.8 Correlation between settlement, soil thickness and pore pressure reduction for typical Oslo clay

5 EXPERIENCES WITH TUNNEL LINING FOR WATER CONTROL

Some of the traffic tunnels in table 1 have been designed with a permanent watertight concrete lining. A concrete lining is in itself not watertight even if it is equipped with waterstops in the joints. The primary reason is shrinkage cracks and cracks generated by large temperature changes between summer and winter as may occur in Oslo (the extreme difference in temperature between winter and summer may be up to about 300 C in a tunnel).

Observed leakage through concrete linings without any special measures other than waterstops in the joints, has ranged from about 10 to 40 l/min pr. 100 m tunnel. That means, the lining has practically no effects at all on the leakage level.

In the Oslo railway and subway tunnels in Table 1 it was used a reinforced concrete lining. Systematic contact grouting was introduced followed by relatively high pressure grouting in the interface between the lining and the rock to reduce the leakage through the lining Karlsrud et. al (1978). This turned out to be quite successful, and leakage in these tunnels have been measured to less than 1 l/min pr. 100 m tunnel. That is significantly less than what has so far been achieved in only pre-grouted tunnels. Pore pressure measurements showed that with this leakage level the natural pore pressure conditions in the surrounding areas was restored.

In the Festningen road tunnel an un-reinforced concrete lining was chosen with local grouting/sealing of cracks/leaks where and when they would appear. It turned out to be extremely difficult to stop the leakage in this case, primarily because of the continuos cycles of temperature changes causing grouted cracks/joints to reopen or new ones to appear over time. In the Festningen tunnel it has therefore, been necessary to re-grout old and new cracks many times over the years the tunnel has been in operation. Furthermore, the leakage still corresponds to about 6 l/min pr 100 m of this twin tunnel.

In the Vestbanen intersection tunnels constructed after the Festningen tunnel, and connecting into it, the approach of a watertight PVC membrane in between an inner and outer concrete lining, such as commonly used on the continent in Europe was adopted. This gave fairly satisfactory results, but it did not completely prevent leakage. The problem with such a membrane solution is to avoid that it is punctured, and that requires extreme care, control and careful inspection throughout the works. It is also a more expensive solution than a single reinforced lining.

Construction of a conventional concrete lining takes considerable time and interferes considerably with progress of blasting and excavation works if it is carried out close to the tunnel face. In practice, the lining will often trail 100 m or more behind the tunnel face. It must also be taken into account that a tunnel may cause pore pressure reduction in clay-filled depressions within 5-600m distance from such clay-filled depressions. This implies that before a complete watertight tunnel has been established along a stretch of tunnel influencing a specific clay-filled depression, 6 months to a year may elapse. Within such a time period significant pore pressure reduction and subsidence can occur. Thus, also for lined tunnels it is necessary to do pre-grouting and /or combine that with re-charging of groundwater as discussed in the following.

6 USE OF GROUND WATER RECHARGING TO COMPENSATE FOR LEAKAGE

In connection with many of the tunnel projects in Table 1 it has been carried out recharging of groundwater to compensate for the leakage (when the leakage has been above the acceptable level). Recharging by drilling wells directly into the anticipated most water-bearing zones in the bedrock has proved to be the by far most effective way Karlsrud et. al (1978). Re-charging wells have been drilled both from the tunnel and from ground-surface. Experiences show that the re-charging should take place at a distance of at least 20-30 m from the tun-

nel face to limit the possibility for the water to flow more or less directly back into the tunnel. The chances that re-charging is successful also increase if pre-grouting has been carried out and at least to some extent stopped the most extreme leakage.

Re-charging has been planned for and used more or less systematically as a temporary measure for most of the lined tunnels. For some of the non-lined tunnels in Table 1 where sufficiently small permanent leakage has not been achieved by pre-grouting or post-grouting, re-charging has also been applied as a permanent solution. With appropriate construction and maintenance procedures to avoid clogging of the wells, this concept has worked rather well, and some wells have been in operation for up to about 20 years now.

Pumping is always a part of the game.

7 CONCLUDING COMMENTS

Several decades of experiences have given a sound basis for handling the challenging water leakage and subsidence problems associated with excavation of rock tunnels close to areas with soft and compressible clay sediments as commonly found in the Oslo region.

Without measures to reduce leakage, a tunnel may cause pore pressure reduction and subsidence in clay-filled depressions up to a distance of 5-600 m from the tunnel.

The leakage requirements can be set fairly precisely when the compressibility characteristics and settlement potential typically range from about 2 l/min to about 10 l/min pr. 100 m tunnel.

Pre-grouting carried out with maximum efforts can in many cases give an acceptable leakage level. It is however, a continuos challenge to ensure that the owner and contractor maintain the necessary focus on the pre-grouting operations, and do not take short cuts. This means for-instance, to implement a systematic trumpet pre-grouting scheme with a sufficient number of grout holes irrespective of conditions met along the tunnel. It is clearly beneficial to use relatively high grouting pressures, but it is also important to consider what is the best grout to use under the prevailing bedrock conditions. One must also show stamina, and keep on grouting until no water is observed coming out at the tunnel face, also after drilling the holes in the face for the next blast round. In extreme cases this may take several grouting rounds and several weeks time.

Experiences from Oslo show that the cost of systematic pre-grouting a tunnel typically adds 50 –70% to the excavation cost (without any grouting). This cost is still far lower than establishing a permanent watertight lining, which may add another 100-150 % to the excavation cost.

Note also, that even if a cast-in-place watertight lining is planned for, it will not eliminate the need for grouting, as it can easily take 6-12 months for such a lining to be in place, and significant settlements will occur in such a time period. Thus, a lining will always have to be combined with pre-grouting, and possibly also combined with groundwater recharging wells until the lining is in place and have been made sufficiently watertight.

In recent tunnelling projects where there has been uncertainty about the effects of grouting, the tunnel cross-section has been increased to give sufficient room for later lining of the tunnel if that should be found required. This is a reasonable risk reduction approach.

With bored tunnels and a pre-cast segmental lining system one has the potential for a significant reduction in the time laps between driving of the tunnel and when the watertight lining is in place (maybe down to 1-2 weeks). For such tunnels it may be possible to eliminate the need for pre-grouting. Further cost-benefit and technical assessments are planned to see if this is a viable approach to some of the future tunnelling projects in the Oslo region.

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5 URBAN ROAD TUNNELS, – A SUBSURFACE SOLUTION TO A SURFACE PROBLEM

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Helen Andersson
Geoteknisk Spiss-Teknikk AS

ABSTRACT : Planning and constructing tunnels in urban areas requires an environmentally friendly approach. This challenge is the focus of a comprehensive research and development programme, initiated in 1998-99 by the Research Council of Norway. The project "Tunnels for the citizen" includes pre-investigations, environmental concerns and water control techniques. In short, the aim is to make transport tunnels more cost efficient and to minimise the environmental consequences. One activity, called "Grouting strategy for pre-grouting of tunnels", has involved the compilation of grouting experiences from a selection of tunnel projects. Several tunnels constructed between 1995-2001 were studied, all but one in Norway. The selected tunnel projects provided a wide spectre of water control situations in varying conditions. This article presents an update of data and results sustained from these tunnels, as well as some of the conclusions drawn from the study. The aim is to present the challenges and the state-of- the art for tunnelling in urban areas focusing on ground water control. The article will pay special attention to the environmental requirements and subsequently how these have been solved by means of grouting during tunnelling.

1 INTRODUCTION

The Norwegian transport system includes approximately 700 km of road tunnels, 250 km of railway tunnels, and 40 km of metro tunnels. In the late 90's severe problems were encountered during the construction of the Romeriksporten railway tunnel, particularly with respect to ground water control. The question was raised how to plan and construct tunnels in urban areas taking care of the surrounding environment. A comprehensive research and development programme was therefore initiated in 1998-99 financed by Norwegian owners, contractors, consultants and the Research Council of Norway. In short, the aim is to develop and improve Norwegian tunnel technology, to make transport tunnels more cost efficient and to minimise the environmental consequences. After a preliminary investigation to establish what areas were needed to be further studied, the main project was sub-divided as follows:

- Sub project A "Pre-investigations" – different investigation methods are tried out for several projects and the optimal extent of preliminary investigations is sought
- Sub project B "Environmental concerns" – the correlations between tunnel leakage / change of pore pressure / damages are studied, and classifications of accepted limits for tunnel leakage and vulnerability of vegetation / water sources are assembled
- Sub project C " Techniques for ground water control" – cements and procedures used for grouting are studied, both for normal conditions and adapted to difficult conditions and strict sealing demands, as well as natural sealing processes and water infiltration

More information about the objectives and the status for the different activities within each sub project can be found on our web-site www.tunnel.no or in an internal report from the Norwegian Public Roads Administration.

2 STRATEGY FOR PRE-GROUTING OF TUNNELS

Two of the proposed activities within the sub project C were joined, forming a study focusing on "Grouting strategy for pre-grouting of tunnels". This aims to develop grouting procedures both for routine grouting and for grouting adapted to difficult conditions where the routine grouting is inadequate. The objectives for the two activities are described below:

Standard grouting procedure for time efficiency:

- compile experience of both satisfactory and unsatisfactory results from grouting of tunnels under normal conditions (including moderate requirements for ground water control, where a routine time efficient pre-grouting has been used).
- develop a grouting procedure for optimisation of time and quality; i.e. fast progress combined with sealed rock, preferably after only one round of grouting. Methods based on conventional as well as innovative techniques (concerning procedure / grout materials / equipment / organisation etc.) should be studied.

Grouting procedure adapted to difficult conditions and ≤strict water inflow requirements:

- compile experience of both satisfactory and unsatisfactory results from grouting of tunnels under difficult conditions (including strict sealing demands combined with limited rock cover, poor rock quality and / or complicated tunnel design).
- develop sealing methods that are efficient for such difficult conditions. Methods based on conventional as well as innovative techniques (concerning procedure / grout materials / organisational structure etc.) should be studied.

The following scenarios were focused:

- High permeable rock mass and strict water inflow requirements for parts of the tunnel.
- Low permeable rock mass and strict water inflow requirements for parts of the tunnel.
- Zones with varying permeability and possibly demands on critical stability.
- Limited rock cover, possibly with overlying material sensitive to settlements.
- Two parallel tunnels or crossing over / under / close to other rock facilities.

The initial phase of the study thus involved compilation of experience from a selection of tunnel projects. The selection of tunnels was made according to the following criteria:

- Representative modern grouting strategy and range of methods.
- Grouting operations and results well accounted for.
- Relatively strict water inflow requirements.
- Extensive efforts for ground water control.

The tunnel projects that were selected were all road tunnels, excavated in or close to urban areas, and often with limited soil and rock cover or other difficult ground conditions. The projects were carried out between 1995-2001, when the focus on the consequences of possible lowering of the groundwater table, was increasing. Finally, the selected tunnels had a high level of documentation, and it was possible to find data as well as meet with representatives from the involved parties for supplementary information.

In short, the compilation of experience was performed according to the following:

- Existing data and reports for selected tunnel projects were assembled and studied.
- Owner / contractor / consultant were interviewed to get complementary data.
- Preliminary report was compiled and distributed for double-check.
- Final report with concluding discussion was completed.

Fig. 1. Grouting equipment in the Metro ring tunnel (photo: Knut Boge)

In the following, data and results for the selected tunnels will be presented. Further details can be found in a report from the Norwegian Public Roads Administration.

2.1 The Tåsen tunnel, Oslo

The two Tåsen tunnels in Oslo are examples of tunnels with limited rock cover and a varying, sometimes highly permeable rock mass. Mainly sedimentary rocks occur such as clay shale and limestone, interfaced by igneous dykes (e.g. syenite porphyry and dolerite). The maximum water inflow to the tunnel was moderate, 10-20 l/min/100m per tube. In parts of the tunnel, grouting was performed based on the water leakage measurements in probe holes, but in the vicinity of the highly fractured syenite the grouting was systematic. The docu-

mentation of the project was good, including for example a M. Sc. thesis on grouting. A NGI report concerning the pre-grouting scheme, with emphasis on the pore pressure situation and settlements, concluded that systematic pre-grouting was required for a total of 800 m in the two Tåsen tunnels. In some places, however, a substantial decrease of the pore pressure level over the tunnel was observed, - at the most 7m. Permanent water infiltration from wells in the tunnel was chosen to maintain the pore pressure. Key data for the Tåsen tunnel is given in the table below.

Tunnel length	933 m (east tube) – 937 m (west tube)
Tunnel design	Two parallel tunnels and several ramps, cross-section between 65-80 m^2
Excavated	1997-1998
Total inflow	13 l/min/100 m (average in each tube)
Max. allowable inflow	When systematic grouting 10 l/min/100 m, or else 15-20 l/min/100 m
Average cement consumption	478 kg/hole – 24 kg/m hole – 802 kg/m grouted tunnel –26 kg/m^2 grouted tunnel – 870 kg/time

Table 1. Key data from the Tåsen tunnel in Oslo

The water inflow requirements were met in the less difficult areas where a certain leakage from probe holes initiated grouting. In the highly fractured syenite, severe problems were experienced; e.g. drilling difficulties, large grout outflow in the tunnel, limited grout penetration in the rock, and, finally, problems with rock support. During the construction period the systematic grouting programme was modified to include the following:

- leakage from probe holes was chosen instead of water loss measurements as criterion
- the grouting criterion was maintained despite dramatic leakage and pore pressure decrease
- reduced length of grout fan, from 24m (outer) to 10-18m (inner) in difficult areas
- grouting of the face through 10 m holes when large outflow of grout in the tunnel
- an initial grouting pressure of 20-25 bar, was increased to max. 35(-45) bar when low grout take
- larger drill bit diameter (64mm) to reduce drilling problems
- micro cement was tried, but no clear conclusions on the result
- post-grouting with polyurethane reduced the leakage somewhat

2.2 The Svartdal tunnel, Oslo

The Svartdal tunnel in Oslo was included in the study due to the complex tunnel design, limited rock cover (down to 2,5 m) and difficult geological conditions through the Ekeberg fault. The rock mass quality was poor; to the west highly fractured clay shale, then alum shale of very poor quality – ”fragmented rock”. The rate of excavation was very low in these areas, as a result of the need for substantial rock support. After having passed the zone of alum shale, there was gneiss of relatively good quality. Ironically, this is where grouting was required due to much water and uncertainty concerning the foundations of the buildings located over the tunnel.

The sealing demand that was placed on the tunnel by the owner in sensitive areas was very strict, 5 l/min/100 m. The principle for grouting was based on measuring the leakage from probe holes. In the two Svartdal tunnels, systematic pre-grouting was performed for a total of only 260m and the data was evaluated from the original grouting protocols. Key data for the Svartdal tunnel is given in the table.

Tunnel length	1700m(northern tube) – 1450m (southern tube)
Tunnel design	Two parallel tunnels with adjoining ramps, cross-section 65m^2
Excavated	1998-2000
Total inflow measured	150 l/min, *i.e.* 4,3 l/min/100m per tube
Max. allowable inflow	5 l/min/100m
Average cement consumption	1358 kg/hole – 80 kg/m hole – 1719 kg/m grouted tunnel – 50 kg/m^2 grouted tunnel – 978 kg/time

Table 2. Key data from the Svartdal tunnel in Oslo

The final total inflow measured for the Svartdal tunnel, including the critical passage of the Ekeberg fault, showed that the water inflow requirements was fulfilled. Since the leakage was not measured for sections of the tunnel, the location of the leakage is unknown and hence the grouting results cannot be correctly evaluated. The systematic grouting programme was modified to adapt to the local conditions, particularly to very limited rock cover and very poor rock mass quality. The main changes involved the direction and length of the grout holes due to the steep dip of joints and weak zones. Some other factors of interest were:

- leakage from probe holes initiate grouting, but the measurement of leakage from holes in the invert is difficult and may have led to underestimation of grouting need
- blast holes served as control holes to check the results of the ground water control efforts
- drilling difficulties necessitated shorter grout holes than the planned 21 m

2.3 The Storhaug tunnel, Stavanger

Stavanger is located in southwest of Norway, a port on the Atlantic Ocean, and a modern town of (1999 estimate) 112.000 inhabitants. To solve the environmental problems caused by surface road traffic, a 1260 m long tunnel under the city area of Storhaug was constructed. The tunnel is bidirectional and has two lanes and a theoretical cross-section of 50 m^2, according to a T9 profile from the Norwegian Guidelines.

Fig. 2. Grouting works in the Storhaug tunnel

The gradient and the two point of road connection on each side have, in addition to the geology and the soil conditions, been decisive for the choice of horizontal- and vertical curvatur. This has led to an extremely limited rock cover, with a minimum of 4-6 metres between the tunnel roof and the surface.

The tunnel is passing under an old part of the city with buildings from the period from 1900 to 1950. Those are mostly wooden houses on rock and concrete foundations of varying quality. Several of these houses are founded in peat moor area by means of traditional wooden piles whilst some are "floating". The tunnel construction commenced in the beginning of 1999 and it was opened for traffic in May 2001.

The Storhaug tunnel was excavated in a rock mass that consists of different types of phyllites with very low permeability. The leakage restriction was set to 3-10 l/min/100m; the lower value was specified under a peat moor area. In that area, systematic grouting was performed for approximately 165m of the tunnel. A significant project documentation has been made and some key data is given in the table below

Tunnel length	1260m
Tunnel design	One tunnel, cross-section 85m^2
Excavated	1998-2001
Total inflow measured	1,6 l/min/100m
Max. allowable inflow	1250-1550: 3 l/min/100m, 750-900: 10 l/min/100m
Average cement consumption	112 kg/hole – 8 kg/m hole – 1014 kg/m grouted tunnel – 26 kg/m^2 grouted tunnel – 273 kg/time

Table 3. Key data from the Storhaug tunnel in Stavanger

Leakage measurements in the tunnel in the summer '99 showed an average leakage of 1,6 l/min/100m under the peat moor. The satisfactory result was obtained by use of the systematic grouting programme, which involved the following elements:

- water leakage from probe holes and water loss measurements, later water loss measurements left out due to lack of consistent information
- optimal number of grout holes was found to be 62 (including 12 holes in the face) for the 85 m^2 tunnel and the hole length was 14m, with two 3m blasting rounds this resulted in a double cover and a substantial quantity of drilled meters
- micro cement (U12), Grout Aid and SP, low w/c ratio (1,1-0,4, usually 0,9-0,7), grouting pressure varying between 30-50 bar, maximum 70 bar in the sole

2.4 The Bragernes tunnel, Drammen

Drammen is a town situated 30 km south west of Oslo. The Bragernes tunnel in Drammen was excavated through volcanic rock mass, such as basalt and porphyry. The rock cover varied from 10 to 150m, but was on average 100m. The rock mass was regarded as highly permeable and the tunnel was located close to existing underground facilities. The maximum inflow was recommended to 120 l/min over 1200m (10 l/min/100 m), and the water inflow requirements were differentiated over the tunnel between 10-30 l/min/100m. Systematic grouting was performed for more or less the whole tunnel and the objective was to achieve an acceptable sealing result after one grouting round. The grouting work was well documented and data was also evaluated from the original protocols. Key data for the Bragernes tunnel is given in Table 4.

Tunnel length	2310 m
Tunnel design	One tunnel, ventilation and escape tunnels, cross-section: 72-83m^2
Excavated	1999-2001
Total inflow measured	10,1 l/min/100m, *i.e.* 240-1730: 8 l/min/100m and 1730-2540: 25 l/min/100 m
Max. allowable inflow	400-800 and 1700-1900: 30 l/min/100m, 800-1700 and ventilation tunnel: 10 l/min/100m
Average cement consumption	2257 kg/hole – 68 kg/m hole – 1125 kg/m grouted tunnel – 38 kg/m^2 grouted tunnel – 2774 kg/time

Table 4. Key data from the Bragernes tunnel in Drammen

The systematic pre-grouting gave an adequate ground water control above the tunnel, the water leakage was measured to 8 l/min/100m in the critical areas where the criterion was 10 l/min/100m. Active design was used throughout the project, which led to several changes of the grouting programme mostly based on geological factors. For a start, 21 holes with a length of 22m and two 2-3m blasting rounds between each grouting round, changed to only 7 holes of 27m for the same cross-section (72-83 m^2) and 4 blasting rounds of 5m between. The grout take was extreme and the high pressure and volume capacity of the grout pumps were utilised to their full capacity. This facilitated the grouting to be per-

formed in a highly standardised manner, which resulted in time efficiency. The grouting programme was characterized by:

- water leakage from probe holes was described as criterion for grouting, 5 l/min from 1-6 holes, but grouting was performed for more or less the whole tunnel
- notice was to be given if the grout take in a single hole exceeded 1000 kg, later changed to 5000 kg, and after 3000 kg the W/C ratio should be reduced to 0,5.
- amount of grout holes was down to 7 for the 72-83 m^2 tunnel, but in the last part of the tunnel (permeable with low overburden) the amount of holes was increased
- grouting cement (Rapid) and HP, low w/c ratio (1,0-0,5), high grouting pressure (80-90 bar where there was sufficient overburden, 20 bar in the last part of the tunnel)

Crossing the Bjerringdal fault, with highly fractured and permeable rock, necessitated several grouting rounds from the same face and only 1-2 blasting rounds could be executed before the next grouting round. The grouting efforts also included longer holes in the first round in order to block off the water and 90 bar grouting pressure. In this zone, the grout take amounted to at most 60 ton in one grouting round.

2.5 The Baneheia tunnel, Kristiansand

Kristiansand is a city in the southern part of Norway with a population of approximately 65 000 inhabitants (1996).

Fig 3. An overview of the Baneheia tunnel system

In order to solve the traffic problems on the main road (E18) through Kristiansand, it was decided to build two new tunnels to lead the traffic outside the central city area. The tunnels are both 750 meters long also containing two crossings (see fig 2). In the cross area the distance between the passing tunnels has been 1 to 2 meters making it necessary first to construct the tunnel on top and strengthen the rock in between, before construcing the lower tunnel. The tunnels are also constructed below existing rock caverns at a minimum distance of only 2 meters. The cross sections vary from 30 to 70 m^2. In the cross areas the tunnel span could reach 30 meters and the combination of this and low rock cover lead to a concern for unstable rock.

The system of Baneheia tunnels for the E18 in Kristiansand was excavated in a rock mass consisting of low permeable gneiss with dykes of pegmatite. The rock cover is 10-40m, and the tunnel with a very complex design passes only 19m below a very popular recreational area with three small lakes (1.-3. Stampe, see Fig 3). The project attained large interest in media and extra focus was placed on the ground water control of the tunnels, including pre-investigations. The allowable inflow was set to a total of 60 l/min (*i.e.* 2 l/min/100m) below the lakes with either 6 l/min/100m for a 500m stretch or 12 l/min/100m for a single stretch of 100m.
Initially, the grouting was performed based on results from both water leakage observations in probe holes and from water loss measurements. The water loss measurements were left out due to inconsistent information, and early on, systematic pre-grouting was performed. The documentation of the project was available and the data had been studied in two master theses on grouting. Key data for the Baneheia tunnels is given in Table 5.

Tunnel length	3000 m in total
Tunnel design	Two parallel tunnels and several adjoining ramps, cross-section: 44-87 m^2
Excavated	1999-2001
Total inflow measured	1,7 l/min/100m
Max. allowable inflow	60 l/min in total, 6-12 l/min/100m near the lakes Stampene
Average cement consumption	256 kg/hole – 15 kg/m hole – 514 kg/m grouted tunnel – 14 kg/m^2 grouted tunnel – 755 kg/time

Table 5. Key data from the Baneheia tunnel in Kristiansand

The total inflow measured for the Baneheia tunnels, 1,7 l/min/100m, indicated that the sealing demand was fulfilled. The leakage was however not measured for sections of the tunnel since the excavation was descending. Sporadic grouting was found to be insufficient and systematic grouting was performed for 95% of the total length. This led to a standardised grouting procedure with high capacity, well incorporated in the excavation cycle. Following elements in the grouting programme can be highlighted:

- optimal amount of grout holes was found to be 30 (including 4-7 holes in the face) for the 50-80m? tunnel, the hole length was 21-24m, with three 5m blasting rounds this resulted in an overlap of 9m and a large quantity of drilled meters
- objective was to achieve an acceptable sealing result after one grouting round, and the overlap made it possible to blast one round before drilling control holes
- micro cement (U12), Grout Aid and SP, low w/c ratio (0,9-0,7), varying grouting pressure, 50-80 bar maximum, lower close to the Stampene lakes

In a few zones with highly permeable rock, several grouting rounds were necessary from the same face and only 1-2 blasting rounds could be executed before the

next grouting round. The grouting efforts also included reduced length of grout fan, from 17-24m (outer) to 8-15m (inner), and grouting of the fast-hardening Thermax. The distance between the two parallel tunnels was 4-22m and the difference in excavation rate may possibly have made the grouting easier with lower grout consumption in the second tunnel.

Baneheiea. Drilling of grout holes.

3 EXPERIENCE OBTAINED FROM THE PROJECTS

Data and experience from six different tunnel projects excavated during the last five years have been compiled. The tunnel projects presented in this article are all in Norway, and were selected to provide a wide spectre of grouting situations at varying conditions. The rock mass / hydrogeology, and the rock cover varies. The maximum allowable water inflow for these projects differ, as well as the reasons for the restrictions placed on the leakage.

In parallel with the increasing focus on possible consequences of to the excavation of tunnels in urban areas, a rapid development of systematic pre-grouting has been evident over the last few years. This study clearly showed that even for tunnels with moderate water inflow requirements, sporadic grouting efforts based on leakage from probe holes were insufficient. The use of a standardised systematic grouting schedule throughout the tunnel, was pointed out as most advantageous for the excavation cycle and the inflow of water. The development of a well incorporated grouting procedure is characterised by the following elements:

- increased capacity and precision of the drilling helps provide the large quantity of drilled meters needed for the new standard of optimal number of grout holes
- enhanced use of superplasticizers and silica additives have increased the penetrability and pumpability for both grouting and micro cements
- better penetrability has made it possible to use low w/c ratios, which in turn has improved the quality of the grout, and increased pumping capacity in dry cement
- the use of 2(-3) complete grouting lines (pump, activator, and mixer), better capacity for transport, weighing, mixing and pumping
- the use of higher grouting pressure in several projects, as high as 90 bar, resulting in better penetrability and grouting capacity
- experience especially from the Storhaug tunnel proves that the grouting efforts have a major effect on the rock mass stability, reducing the need for rock support

Further development of the elements described above, may provide a possibility to limit the number of grout types and mix designs at the site for routine grouting. This can make the grouting process more time efficient and in itself contribute to increased capacity. The objective being to achieve acceptable results through a single grouting round.

As for the grouting adapted to difficult conditions and strict sealing demands where the routine grouting is inadequate, the interim report describes several interesting situations. Four of the projects have parallel tunnels, and it may be an advantage that one tunnel face is always some tens of meters ahead of the other, not only for stability reasons, but also for the grouting. In cases with limited rock cover, the arrangement of the boreholes and the grouting pressure are of course adapted. In rock of poor quality, or for tunnels with very strict sealing demands, the grout holes tend to be shorter.

Details such as the unsolved problems with leakage around packers or through bolts (and a plan for how to deal with this) were also discussed in the study. Finally, the study placed some attention to general aspects like quality control, and there is a great potential for improvement of factors like; checking the rheology of the cement grout, measuring of water leakage in the tunnel,

Baneheiea. At face, charging.

Baneheiea. Ready for grouting.

and evaluating the final water inflow results as well as the demands.

4 CONCLUSIONS

The experience gained during the construction of the Romeriksporten railway tunnel which included a significant draw-down of the ground water in the area, initiated a strict focus on the use of systematic pre-grouting. This study has shown that during the last 5 years a number of road tunnels have been constructed with a successful use of such grouting techniques.

The project “Tunnels for the citizen” will be completed by the end of 2003. In view of the increased focus on the need for watertight tunnels in urban areas and environmentally friendly tunnelling concepts, the timing for performing this study was appropriate. The ground water control strategies and range of methods for modern pre-grouting of tunnels have been established. Now, some of these experiences are being verified in the Metro Ring tunnel in Oslo and the results of this will be useful in future projects, Tunnels & Tunnelling (2001)

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6 DESIGN PRINCIPLES AND CONSTRUCTION METHODS FOR WATER CONTROL IN SUBSEA ROAD TUNNELS IN ROCK

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ABSTRACT: This paper describes the control of water in subsea road tunnels in rock in Norway. The occurrence of water is difficult to predict accurately, so it is important to be prepared for large variations, both with respect to location and volume. Probe drilling ahead of the face is mandatory. The needed tightness can be achieved by pre-grouting. All structures, also concrete lining, are drained. As no water is allowed to drip on the driveway, shielding by frost insulated drip protection is installed in all sections with remaining seepage. The drainage water is collected in a drainage sump at the lowest point and pumped out. Experience shows that in most tunnels the seepage is decreasing with time.

1 INTRODUCTION

Fig. 1. The completed Oslofjord tunnel with artistic light effects for driver's comfort. No water seepage visible due to cost-effective pre-grouting and water shielding.

1.1 Characteristics of water problems and control

The proverb "Prevention is better than cure" fits very well on water control in tunnels, and especially so for subsea tunnels. Despite the recent developments in site investigation techniques, it is very difficult to predict the location and extent of water problems. In all phases of planning, design, tendering and construction of subsea tunnels this must be taken into account.

Although it is fully possible to reduce the amount of water seepage to a target level by pre-grouting, the budget, time schedule and contract must account for the potentially large variations in the amount of work needed to control the water during construction.

The seepage of salt water represents a challenge with respect to durability of materials and structures. However, the experience from the 22 subsea road tunnels built in Norway over the last 20 years proves that these problems can be solved.

1.2 Completed tunnels

The experience base from the planning, design and construction of these tunnels is extensive. As Table 1 demonstrates, the experience includes both deep and long tunnels; reaching as deep as 264m below sea level and with length up to 7.9km. The lessons learned, both positive and negative, have been communicated throughout the tunnelling community in Norway. The Construction Division in the Public Roads Administration authorises all plans and checks all tender documents, in order to ensure consistent quality and cost effective solutions, and to promote a steady development of the employed methods. Typical construction time is 2-3 years or less.

The construction of subsea road tunnels dates back to the early 1980-ies. Actually, the first subsea tunnels were two tunnels completed in 1976 and 1977, the first for a gas pipeline with a deepest point of 253m below sea level, the second for water transfer. From 1983 to 1995, eight subsea tunnels for oil/gas pipelines were completed with lengths from 0.4 to 4.7km, cross sections 20- 66m2, and deepest point from 93-260m below sea level, Palmstrøm & Naas, (1993). The experience from these tunnels contributed significantly to the development of the design principles and construction methods.

Tunnel	Year of opening	Length, m	Largest depth, m	Water ingress at opening, l/min·km	Grouting, kg/m	Water & frost protection, m^2/m
Vardø	1983	2 892	88	460	25	16
Ellingsøy	1987	3 520	140	310	10	14
Valderøy	1987	4 222	145			
Kvalsund	1988	1 650	56	200	0	9
Godøy	1989	3 844	153	300	430	7
Flekkerøy	1989	2 327	101	100	50	6
Hvaler	1989	3 751	120	100	15	6
Nappstraum	1990	1 780	60	200	40	8
Fannefjord	1991	2 743	100	140	15	7
Maursund	1991	2 122	92	210	5	4
Byfjord	1992	5 875	223	100	10	8
Mastrafjord	1992	4 424	132	25	5	8
Freifjord	1992	5 086	132	75	15	13
Hitra	1994	5 645	264	60	20	14
Tromsøysund	1994	3 376	101	265	30	18
Bjorøy	1996	2 000	85	400	665	13
Sløverfjord	1997	3 200	100	150	0	13
Nordkapp	1999	6 826	212	65	15	4
Oslofjord	2000	7 252	134	250	360	18
Frøya	2000	5 305	164	105	200	17
Ibestad	2000	3 398	112	110	15	2
Bømlafjord	2000	7 900	260	70	30	8

Table 1 Key data for completed road tunnels (after Melbye & Øvstedal, 2001)

2 DESIGN PRINCIPLES

2.1 Requirements to water tightness

The Public Roads Administration's Handbook No. 021 on Road Tunnels (1992) does not set a specific requirement to water tightness in subsea road tunnels. However, according to practise, the basic target for water tightness is that the overall remaining seepage shall be no more than 300 litres/min/km. This level is chosen based on an optimisation of the cost of grouting versus the cost of pumping facilities, i.e. pumping capacity, pumping energy, the size of the safety buffer reservoir etc.

Experience demonstrates that this target in most cases can be reached with a reasonable effort of pre-grouting. Lower water seepage can also be achieved, but this is normally not economical due to increase of grouting costs. It is emphasised that this design philosophy sharply contrasts designs for which minimisation of water seepage is the target, with resulting large costs. On one hand the sea provides an unlimited supply of water, which may be a hazard, on the other hand one does not have to prevent minute seepage, as may be the case e.g. below settlement sensitive soils under cities.

From the user's point of view, the tunnels appear watertight. As will be described, this is achieved by installation of water shielding systems. The requirement of not allowing any dripping on the final roadway gives comfortable and safe driving conditions. From the owner's point of view, it reduces driveway maintenance.

2.2 Drained structures

The tunnels are designed as drained structures. Where the natural seepage is low, the rock mass is left untreated. Where the seepage is too high to be acceptable for the overall requirement, grouting of the rock mass all around the tunnel contour is done. Accordingly, in the grouted zone, the water pressure in the joints will be gradually reduced until zero pressure at the contour. See illustration in Fig. 2.

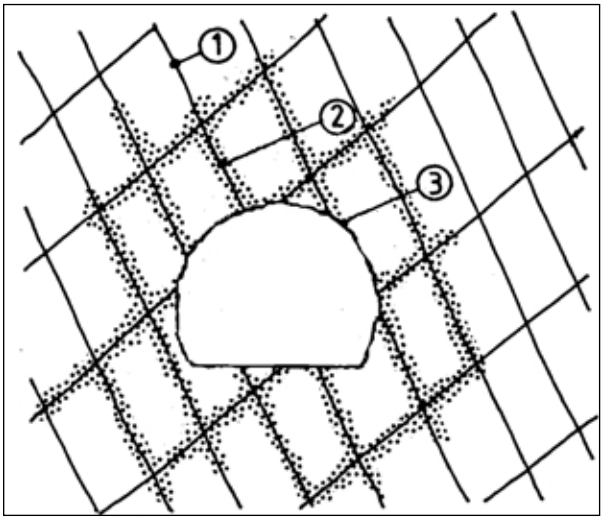

Fig. 2. Zone of grouted rock mass around a tunnel
1. Jointed rock mass with full water pressure
2. Pre-grouted zone, grout in joints
3. Tunnel contour

If a concrete lining is used for stability support in weakness zones, it is normally not designed for full water pressure. If significant water inflow is present in such sections, it is ensured that the lining is drained by installing flexible drainage channels before casting or by drilling holes afterwards. This increases the need for water shielding (see next section) but keeps the cost down. This simple approach has its background in that concrete lining for stability purposes usually is needed in less than 10% of the tunnel, typically for 2-5%.

Rock support by sprayed concrete is also drained either by pipes or sprayed-in channels, to avoid or reduce seepage through the concrete, and to prevent pressure build-up.

2.3 Shielding of remaining seepage
The most used method for drip protection is to install sheets of polyethylene foam (with a layer of mesh reinforced sprayed concrete for fire protection) in the roof, and concrete segments in the wall, as exemplified in Fig. 3. Note that the drip protection has to provide frost insulation, as in the winter frost may extend long distances into the tunnel from each portal.

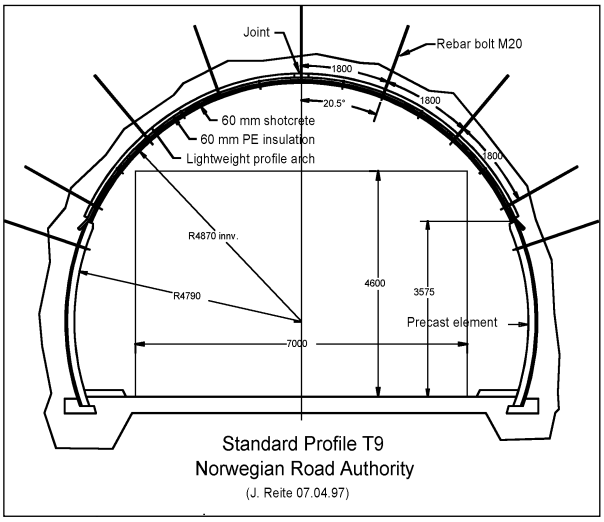

Fig. 3. Shielding of remaining seepage by polyethylene foam (covered by sprayed concrete) in the roof and precast concrete segments in the walls

The typical consumption of drip protection varies a lot. In the recently completed Oslofjord tunnel, a total of 18m^2 drip protection was installed per metre tunnel. This constitutes about 80% of the tunnel contour.

2.3 Drainage and pumping
Crushed rock (15-30mm) is used for drainage in the trenches and below the driveway. One or two 150-200mm PVC pipes are installed in trenches on one or both sides. At the lowest point the pump station has a buffer reservoir with minimum 24 hours capacity in case of power loss. One stage pumping is applied up to 200m lifting height (usually in 160mm PE pipes) to the portal or to a (grouted and re-drilled) hole from the surface near the shore.

3 GEOLOGICAL CONDITIONS AND SITE INVESTIGATIONS

3.1 Rock types
The Norwegian subsea road tunnels have been built in Precambrian and Palaeozoic rocks. These include gneiss, greenstone, phyllite, sandstone, shale, intrusives etc. As below land, it is a common experience that rocks like greenstone or phyllite with darker mineral, e.g. amphibole or biotite, often have less seepage than lighter coloured rocks, presumably because they are less brittle and less prone to develop open joints. There are always exceptions to such 'rules', as the previous exposure to tectonic movements has a strong influence on the water bearing capacity of the rock mass.

3.2 Weakness zones
As the location of fjords and straits often are defined by major faults, a significant portion of a subsea tunnel must be expected to cross such weakness zones in the bedrock. This is often at the deepest point, where the rock cover is lowest and the water pressure highest.

It is a common experience that the central part of a weakness zone may not cause large inflow, as it often contains crushed rock and clay gouge. However, the side-rock of the weakness zone may frequently carry a lot of water in open joints, especially in joint sets opened by tensional tectonic movements. In such zones, the combination of poor stability and presence of water presents a high risk of collapse.

The occurrence of high water inflows in tensional joint sets can be pronounced for tunnels in or close to graben systems, or in tunnels located at the outer part of the coast, close to the major faulting along the continental shelf. In such cases, joints with apertures of 2-5cm have been encountered and pre-grouted. However, most joints have apertures of 1-5mm and require less grouting.

3.4 Site investigations
Site investigations for subsea tunnels rely heavily on reflection and refraction seismics. In some cases, hole-to-bottom seismics has been performed and used to pre-

pare seismic tomographic profiles. Weakness zones with low velocities (less than 2500-3000m/s) may carry a lot of water. This is in contrast to velocities of 3500-4500m/s in blocky rock and 5000-6000m/s in more massive rock. Although the seismic velocity can give an indication about the degree of jointing, and thereby about potential inflow zones, it is difficult to predict water inflow from such investigations.

Other geophysical methods have been tried, e.g. geoelectric measurements (resistivity and induced polarity) on weakness zones if and where they run ashore. The success is limited, both due to practical limitations and since the observations are difficult to interpret because of salt water content in the zones below the upper marine level.

Directional core drilling is frequently used for checking of rock cover at critical points, and may give some information if water testing is performed. The reach and efficiency of this method is improving thus increasing its usefulness, but the boreholes normally cover only a part of the tunnel alignment.

Experience from similar geological formations can sometimes be misleading, because the tectonic setting may be different locally, even if the overall situation appears similar. It is therefore important to be prepared for large variations, no matter what the site investigations may indicate.

Accordingly, the most important site investigation is the one taking place in front of the tunnel face, in form of probe drilling, as will be explained below.

4 CONSTRUCTION METHODS

4.1 Excavation methods

All the Norwegian subsea tunnels have been excavated by drilling and blasting. In this connection it is worth mentioning that this is now always done with modern hydraulic drilling jumbos with high capacity drilling machines for drilling of blast holes. This equipment is also used for the efficient drilling of probe holes and holes for pre-grouting and control, most often with diameter 45-50mm.

4.2 Probe drilling and pre-grouting

The standard pattern for probe drilling developed more than 15 years ago, and is now used with small variations only. Fig. 4 shows a typical example. The length of the probe holes is typically varied between 24 and 33m, to best suit the rhythm of blasting rounds. Depending on the length of the holes, probe drilling takes place each 3rd, 4th or 5th round, always with a minimum overlap of 8m (two blasting rounds). Longer percussion holes are seldom used, also because deviation increases with the length of the hole.

The number of probe holes varies from a minimum of 2 holes, increasing to 4-6 or more when jointed zones or low cover is expected.

The extent of probe drilling varies between projects; typical total average is 2-7 metres of percussion holes per metre tunnel. In addition, core drilling is done from the face (usually in weekends) when weakness zones are expected from the refraction seismic profiles. As an example of extensive use of percussion probe drilling, in the 7.2km Oslofjord tunnel it was probe drilled 6.9m/m tunnel in total, and core drilling ahead of the face constituted 0.7m/m tunnel on the 2.1km subsea section, Aagaard & Blindheim (1999).

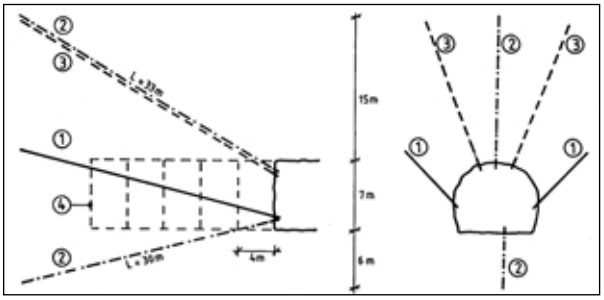

Fig. 4. Typical percussion probe drilling pattern for two-lane road tunnel
1. Probe holes at all sections
2. Additional holes in expected weakness zones
3. Alternative upper holes in sections with low rock cover
4. Overlapping of holes (min 8m) for each 5th blasting round

The water inflow into the probe holes is measured, and the criteria to trigger pre-grouting is typically 5-6 litres/min. Water inflow in blast holes will also trigger grouting. Control holes are drilled after each grouting round to check the result.

The grouting is invariably done as pre-grouting. This is the only way to ensure full control and to meet the tightness requirement.

The grouting is predominantly performed with cementitious material, and includes standard industrial cement, rapid setting cement and micro-cements of different fineness grade. When open joints with significant seepage is detected, the grouting is performed directly with high pressure (e.g. with cut-off pressure up to 60 bar) and relatively thick mix, without the traditional time consuming trying out of gradually thicker mixes starting from w/c = 2. A typical drill pattern is shown in Fig. 5. If necessary to achieve sufficient tightness, overlapping rounds are performed.

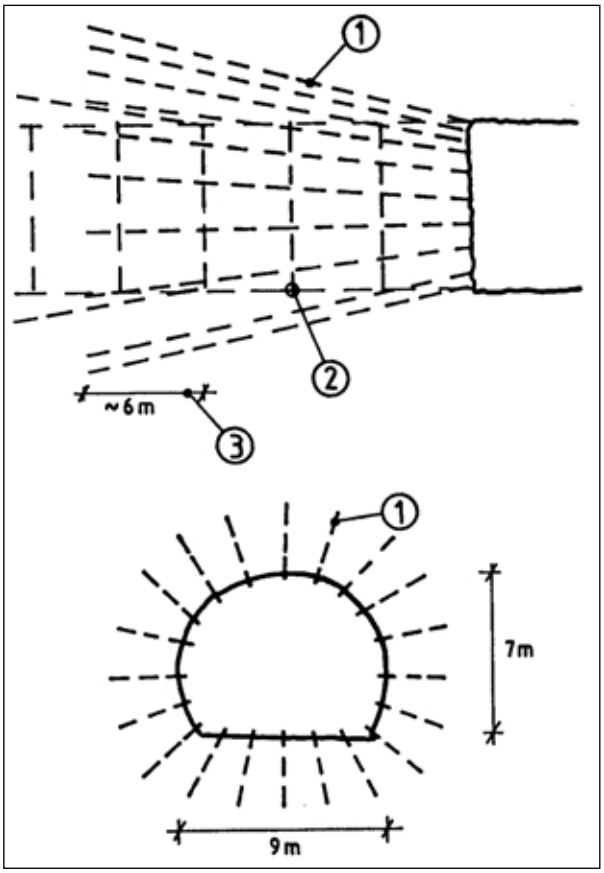

Fig. 5. Typical drill pattern for pre-grouting for two-lane road tunnel
1. Full cone of 45mmm holes. 18m length
2. Blasting rounds 4m
3. Typical overlap between cones

According to the standard contract practise, the decisions regarding grouting are taken by the owner, in consultation with the contractor about practical performance. The contractor gets paid according to the actual performed work (metres drilled and applied grouting material) and for standstill of tunnelling equipment according to tendered prices. This applies also for the probe and control holes.

The consumption of grouting materials has varied from project to project. For most tunnels, it has been below 50kg/m tunnel in total, Melby & Øvstedal (2001). For the Godøy and Frøya tunnels, both tunnels leading to islands close to the continental shelf, the consumption was 430 and 200kg/m respectively. For the Bjorøy and Oslofjord tunnels the consumption was 665 and 360kg/m respectively, but this was due to exceptional conditions, as will be explained below

Typical consumption per grouting round may vary from 1 to 50 tons and the total time per round from 2.5 to 48 hours. Procedures are adapted to reduce the need for fully overlapping rounds.

4.3 Contractual aspects

As mentioned above, the whole process of probing and grouting for the purpose of water control is basically owner controlled in the normal unit price contract. In some cases the owner's representatives will decide all details for all grouting rounds. However, the standard contract does not prevent the owner from utilising the practical experience of the contractor by allowing him to adapt to the conditions. As long as an agreed framework procedure is followed, and the overall tightness requirement is met, the contractor may select the suitable type of cement, length and number of grout holes etc, Blindheim & Skeide (2001).

As the whole process of grouting is rather complex with many variables, and the natural variation in the rock mass cannot be 'controlled', it is usually beneficial for the outcome if a constructive co-operation is established between the parties.

4.4 Special situations

Contingencies for emergencies are included in the contracts. For water problems alone, this relates to normal and extra pumping capacity. For example, for the 7.2km Oslofjord tunnel, the contractor was required to pump up to 350 litres/min/km and to mobilise pumping capacity for unforeseen inflows of 1000 litres per minute for each tunnel face.

For combined water and stability problems, as can be found in fault zones and weakness zones, it is required that movable concrete casting lining shields, with the possibility of face closure, shall be mobilised before the tunnel face moves out below the sea.

Extreme conditions may sometimes be found. For example, a down-faulted zone of loose or poorly consolidated sandstone created an extreme situation in the Bjorøy tunnel. It took very extensive grouting before it could be passed after several months of delay, Holter et al. (1996).

Another extreme situation occurred at the recently completed Oslofjord tunnel, where a glacially eroded channel along a known fault zone proved to be much deeper than expected, resulting in no rock cover. The channel, which had been in-filled with loose sediments, was detected at safe distance with probe drilling from the tunnel face. A by-pass transport tunnel was driven through the fault zone at a deeper level and allowed continued tunnelling under the fjord, while the loose deposits under 120m water pressure were frozen and tunnelled through, Blindheim & Backer (1999).

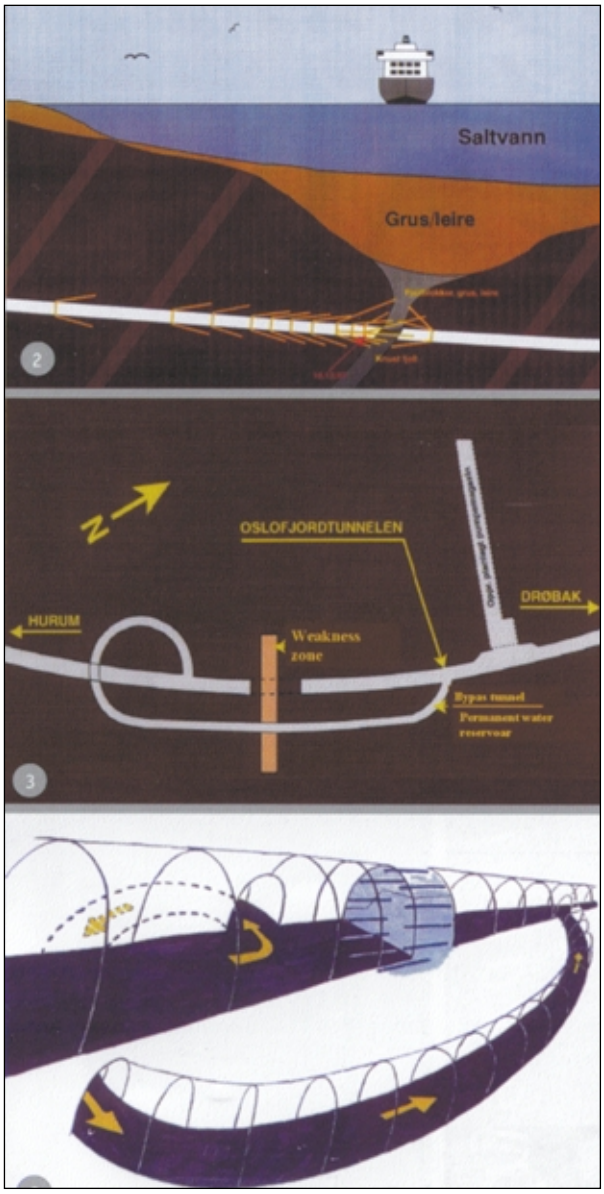

Fig. 6. Freezing of in-filled erosion channel along a fault zone under 120m water pressure.
1. Longitudinal section of tunnel with indicated probe drilling towards the zone (Saltvann = salt water, Grus/leire = gravel/clay)
2. Plan view of by-pass tunnel through the fault zone at a lower elevation
3. Perspective sketch of by-pass and indicated zone for freezing

5 OPERATION

5.1 Seepage development with time

Although one may expect that water seepage may increase with time due to dissolution of minerals etc, experience shows that in almost all cases, the seepage is constant or reduces with time, Melby & Øvstedal (2001). This is likely due to that particles in the joints move and create natural filters and that some minerals swell and close the openings.

5.2 Durability of materials and structures

In order to ensure durability of rock support measures, strict requirements are implemented. Rock bolts have double corrosion protection, with hot-dip galvanising and epoxy coating. I addition, full cement grouting is normally employed. These measures appear so far to give sufficient protection, as no serious corrosion damage has been observed.

Sprayed concrete is specified to a minimum 45 MPa uniaxial compressive strength to provide low permeability. Experience shows that poorly performed shotcrete, applied in thin layers (less than 3cm) or of less than 30-35 MPa strength, will leach in wet areas and loose strength due to salt water seepage. Black steel fibre reinforcement does not corrode inside sprayed concrete of low permeability and a minimum thickness of 7cm, Davik (1996)

For the pumps and pumping pipes strict requirements are implemented, as the combination of salt water and pollution from the traffic provides a very aggressive environment. Polyethylene is used for the piping, if pressure allows, otherwise corrosion resistant steel qualities are used.

5.3 Operation and maintenance costs

The dominant operation and maintenance costs in the road tunnels are for lighting and ventilation. Pumping costs are relatively low. If the buffer reservoir has ample capacity, the pumping time is low and pumps can run during night when electricity may be cheaper. In some tunnels, pumping has been hampered by algae growth, which tend to vary cyclically with time. Chemicals have been tried to limit the algae growth, but this has proven not to be necessary. As a precaution, the drainage systems are now over-dimensioned.

Electrical equipment, pumps and piping have had to be replaced in some tunnels because of corrosion. Corrosion has also damaged the inner shielding or lining for drip protection made of aluminium. This has resulted in the need for replacement work to start in two tunnels so far. Thus re-investment costs can be significant. See Melby & Øvstedal (2001) for details.

6 CONCLUDING REMARKS

6.1 Ongoing developments

The ongoing development of site investigation methods will improve the possibility of predicting water problems ahead of construction, not at least connected to the use of efficient refraction seismics and new interpretation methods (Lecomte et al. 2000) and directional core drilling, Dowdell et al. (1999)

A newly developed drill hole deviation survey tool may now be used as part of the work rhythm for drilling of probe and grout holes, Tokle (2001). This will improve the reliability of the information gained from the holes and may reduce the number of overlapping grouting rounds needed to reach the tightness requirement, as supplementary holes can be placed as needed at once.

Still, construction methods and contracts will always have to count on large variations, and also in these fields it is possible to develop more effective systems and contracts. A co-operative approach, to utilise the experience of both the owner and the contractor, is favourable

Especially for the installations, it is necessary to develop further the requirements and selection criteria for more corrosion resistant materials and solutions in order to reduce maintenance and re-investment costs.

6.2 Applicability of principles

It is believed that the principles described can find a much wider use world wide than today, also in tunnels with significant traffic load and in other geological formations. Some adaptation will of course be necessary, but the potential reward is reliable and safe tunnels at reasonable costs.

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7 LESSONS TO BE LEARNED FROM LONG RAILWAY TUNNELS

Anders Beitnes
SINTEF

ABSTRACT Today's tunnelling, often taking place in urban or valuable recreational areas have to face an increased focus on protection of the groundwater. The paper describes the challenges in Romeriksporten, a 13,8 km long railway tunnel through gneissic rocks completed in 1999. Despite a major pre-grouting effort, the requirements were far from being met on a certain section, much due to a difficult geological situation and high water pressure. The consequence was leakage that led to subsidence in clay in an urban area and damage on forest ground. A huge post excavation tightening task took place within a 2,2 km long section of the tunnel, mainly by grouting the rock mass 4 – 10 m outside the periphery. This effort alone lasted for more than one year, imposing a major delay on the opening of the railway. The problems caught great attention in media, among politicians and also among the public. The experiences have had substantial impact on new projects, boosting both improved planning and new research for more efficient pre-grouting methods.

1 INTRODUCTION

Groundwater control is a primary environmental concern while tunnelling. This has become one of the real challenges to the civil engineering community during the latest years. A number of coincidental factors have contributed to this fact:

- Earlier negligence have led to damage and law-suits, as well as a bad reputation of the tunnelling industry,
- Growing awareness of the groundwater's importance and its vulnerability,
- Longer and deeper tunnels with increasing hydraulic pressure, larger potential inflow and wider influence zone,
- Tunnelling through areas with increased public awareness and attention,
- Gradually less tolerance towards negative effects of even important infrastructure development,
- Ban on effective, but potentially toxic resin grouts, and
- High demands on construction time schedules and budget control.

A Norwegian railway tunnel named "Romeriksporten", completed in 1999, may be an illustrative example of many of these factors. The lessons learned from it shall be briefly discussed in this paper. Another railway tunnel, the Lieråsen tunnel, is referred to for comparison to the situation 30 years earlier.

2 SHORT PROJECTS DESCRIPTIONS

Romeriksporten is a part of the 42 km long high-speed link from the City of Oslo to its new airport, running parallel to the old railway towards the north. Lieråsen tunnel was built to shorten and to strengthen the important line between Oslo and the south-east part of Norway.

	Romeriksporten	**Lieråsen**
Length (in bedrock)	13,8 km	10,7 km
Function	Twin line, single tube	Twin line, single tube
Typical net section	102 m2	68 m2
Excavation method	Drill and blast, drained rock periphery	Drill and blast, drained rock periphery
Support methods	Extensive use of wet shotcrete + rockbolts, 5% cast in place concrete lining	Some dry shotcrete + bolts, 30% full lining, both precast and cast in place
Construction period	1995 – 1999	1965 - 1973
Geology	Precambrian gneiss, some limestone and shale; influence from nearby large fault has induced wide zones of jointed rock with some clay fill	Permian granite partly weathered in wide zones, some sedimentary rocks and hornfels,
Stress situation	100 – 250m overburden. Large sections with small minor (horizontal) stress	100 – 200m overburden. Remanent stresses, complex pattern with partly high major (horizontal) stress
Terrain above	Partly suburb with houses founded on soft clay, partly forest and little soil. High natural groundwater level	Partly suburb, some peat, mostly forest and little soil. High natural groundwater level
No. of drives (faces)	5	2
Grout types used during excavation	Cement, micro-cement, acrylic resin	None
Inner lining	2,5 km precast concrete vault, rest: PE-foam with 6 cm shotcrete in wet sections	Precast concrete in roof (15%) locally: corrugated aluminum cladding
Post excavation grouting	Enormous effort during 1 year on a 2,2 km section	None

Table 1. Some characteristic data

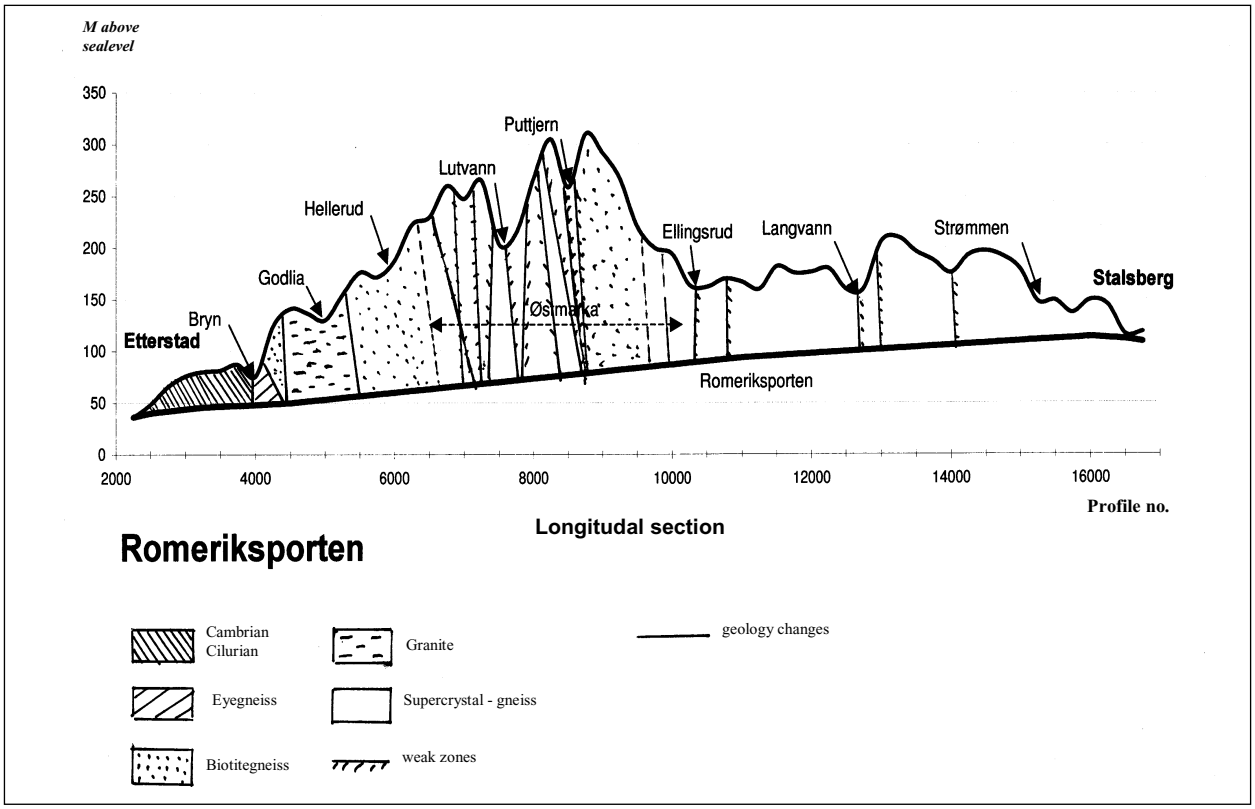

Figure 1. Longitudinal section of Romeriksporten

3 PRE-INVESTIGATIONS AND FUNCTIONAL REQUIREMENTS AT ROMERIKSPORTEN

3.1 Investigations

The investigations in the planning process focused on geology and faults, weak zones, jointing patterns and rock quality. Tunnel routing was more or less given based on fixed points at the ends and limitations in sections with minimum rock cover. No core drilling or pumping tests was performed to map the geo-hydraulic systems. Intensive pre-grouting was foreseen however, mainly connected to faults.

3.2 General requirements

In the planning stage for Romeriksporten, partly without expressive statements, it has been foreseen to keep the inflow below 20 - 30 litres/min/100m in forest areas. That level was based on a record of low-level damage from other tunnelling projects in the Oslo area. For the section passing under the urban areas, the requirement was not to lower the pore pressure in clay more than some 2m, by close follow-up of piezometers. It was expected to fulfil this requirement by keeping the inflow below 10 - 15 litres/min/100m.

3.3 Contract requirements to control water leakage

As customary for Norwegian tunnels, the requirements for in-leakage were not reflected in the contract, as the Client was supposed to have "hands on" and the power to decide the pre-grouting effort and procedures. The contract did, however, have a set of items and requirements to detect and to prevent excess water leakage. As this was a unit prize regulated contract, no major risk rested on the contractor unless he had evaluated his prices falsely.

The tunnelling contract describes investigation holes, or probe holes, to detect possible leakage structures. Pre-grouting was to be decided on basis of recorded in-leakage from those drillholes. Normal probe drilling pattern included 3 – 6 holes, each typically 23m long, all the way through the tunnel. Further, capability should be at hand at all times during excavation for grouting by use of single packers. Grout was to be mixed by standard mixers and pumped by a pressure capacity of minimum 50 bar. Materials specified were standard cement, rapid cement, fine-grained micro-cement (supplier specified by contractor), as well as a resin grout with minimum viscosity (supplier specified by contractor). Also a quick setting grout was specified.

4 TUNNELLING EXPERIENCES ROMERIKSPORTEN

4.1 Conditions encountered during construction

Tunnelling in both the granitic and shistous gneisses south of the Oslo graben, revealed a lot more diffuse and

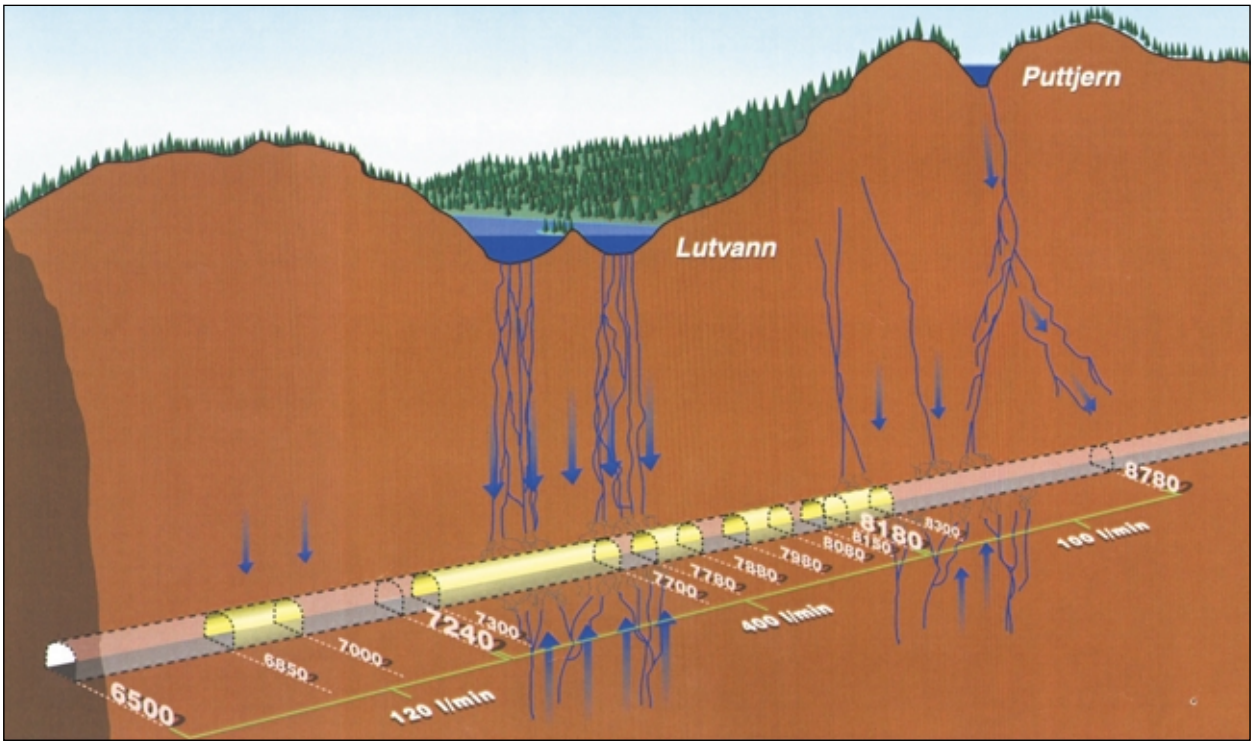

Fig. 2 Artist's impression of tunnel through Lutvann forest. Governmental groundwater inflow limits shown for 3 different sections.

evenly distributed leakage than expected. Systematic pre-grouting had to be used in various sections, like under the populated, urban area of Godlia – Hellerud. Decisions regarding grouting were made on the base of probe drilling, using a criterion of 5 l/min leakage from all investigation drillholes. Lugeon tests (water pressure testing) were not performed. Grouting consisted of one or two rounds with typically 30 holes of 23m and 8m overlap. The advance was slowed down considerably, as each round took some 10 – 18 hours.

Upon entering under the forests of Lutvann, it was expected that less grouting effort was needed due to a higher level of acceptable leakage. It turned out, however, that it became even more difficult and time consuming to comply with the "target" leakage. This was partly due to higher hydraulic head and partly because of more complex and extensive jointing in faults and tension zones. Up to 4 rounds were grouted before control drilling gave acceptable inflow. Finer cement was used frequently, and as poor results came out, even a large amount of acrylic resin (Siprogel) was gradually introduced. At a few occasions, further advance was decided even with excessive inflow in control drillings. Along with the water problems, a contractual dispute arose on the question of time schedule. Slow advance for other reasons on the encountering tunnel face contributed to that.

In all, 37% of the total tunnel length ended up being pre-grouted. The effort included 275 km of drillholes, 5 400 tons of cement, 1 300 tons of micro cement and 340 tons of resin grout.

Despite the large pre-grouting effort, the requirements were far from being met in certain sections. Damages started to develop, and the situation became a big news issue. The headlines were boosted by health and environmental issues connected to the use of acrylamide-containing Siprogel (later: Rocha-Gil) both in Sweden and in this project. In the national perspective, Romeriksporten became a "tunnelling scandal" amongst the public.

4.2 Monitored effects on groundwater

In a particular section at Hellerud, substantial pore pressure lowering developed and houses built on clay gradually began to subside. During excavation, control measurements in drillholes after pre-grouting seemed fine. But flow records in the drainage ditch later showed that some sections of the tunnel had 3 - 4 times the predefined, acceptable inflow of 10 l/min/100 m. Consequent detailed investigations of the overburden revealed a steeply undulating bedrock surface probably connecting with local fissure systems. Therefore, the real tolerance may have been an even smaller inflow rate. As a remedy, 25 infiltration wells have been instal-

led in order to re-establish the pore pressure. All house-owners were promised full compensation for their loss and inconvenience, and a repair campaign was launched.

The phenomenon that attracted most public awareness, however, happened in the forest area used for recreation. A particular pond in a swampy depression in the forest above the tunnel suddenly sank a few meters. Later, the water level in the 2 km2 lake Lutvann began to sink too, a small creek dried out and subsidence developed in narrow valleys with visual effect on large trees. Unfortunately, the groundwater drainage coincided with two consecutive years of low rainfall. A section of 2,2 km of the tunnel became subject to special concern. An inflow of more than 2 500 l/min was recorded after excavation was completed in autumn -97. By then a reduction of approximately 300 l/min already had been obtained by supplementary grouting.

An extensive monitoring program was established with some 30 observation wells and detailed flow measurement in the tunnel. A pump-line was established for refilling/irrigation of the most affected forest area. At the same time, a specialist group was employed in order to develop the best strategy for correction works in the tunnel.

4.3 Post excavation remedies to reduce leakage

At the end of the tunnel excavation, it was recognised that the impact on the groundwater inevitably would become quite substantial, even if measures were taken to reduce the effect considerably. Public scrutiny of the project led to the establishment of a formal provision, which implied that the owner had to apply for governmental approval for groundwater impact. This was a new arrangement in Norway. Under the strain of public interest, a rather long process ended up with detailed inflow limits in certain sections, supported by studies of biotypes and hydrogeology. Accepted inflow equalled some 25 – 30 l/min/100m, i.e. the same level as anticipated as acceptable in the plans.

Advised by the experts, a major effort was initiated in terms of post-excavation grouting. The tunnelling contractor was employed on a cost reimbursement basis in a separate contract to carry out the work. As a principle, extra grouting fans c/c 5 m were introduced to create a new tightened zone 4 to 10m outside the tunnel all around the periphery in all sections with recorded inflow above the criteria.

The work became much more difficult and less rewarding than anticipated. Up to 6 crews for drilling and grouting worked in the tunnel at the same time. The biggest problems arose with the floor section. Drilling downwards required special drill-rigs, a concrete slab had to be poured for logistic reasons, and all sorts of problems were encountered, such as hole stability, packer failure and back-flow of grout materials. The work also revealed that the major part of excessive leakage came through the floor section, much due to a thin or insufficient pre-grouting fan and later blasting for the drain ditch. Apart from that, back-calculation of leakage vs. hydraulic head has shown that the permeability reduction in densely jointed rock has been as good as ever was obtained in earlier tunnels by use of cement grout, i.e. in the size order of k = 10-7 m/sec.

A lot of useful experience was gained throughout the post excavation tightening works. Tests with different micro-cements revealed variations in quality and a substantial difference in the effect of rock temperature.

Between November 1997 and January 1999, a continuos effort on the 2,2 km, with a cost equal to that of excavation of the whole tunnel was performed. Still, the requirements were met only for 2 out of the 3 sections. As a consequence, an installation for re-infiltration by injecting water back from the tunnel was constructed. Experience so far has shown that it is a reliable and non-disturbing remedy, and that it only has to be run in periods with low rainfall, being automatically controlled by the water level in observation wells at the surface.

5 THE LIERÅSEN TUNNEL

When the Lieråsen tunnel was constructed, the major concerns with respect to in-leaking water were not to interrupt the excavation and not to make operational hindrances for the railway. Also here, the planning stage investigations were aimed at rock cover and weakness zones.

Lieråsen tunnel encountered sections with high inflow rates, much the same as in the central part of Romeriksporten, but no leakage control except local drainage in the tunnel was introduced. Immediate groundwater lowering dried out small ponds and led to subsidence in swampy terrain above the tunnel.

There is little record of public concern, though cultivated landscape was affected. It has even been mentioned that somebody found the drainage helpful, as new housing development was established in the area.

6 LESSONS LEARNED

6.1 The inevitable uncertainty in rock tunnelling

In rock tunnelling, no matter the level or amount of pre-investigations, there is inevitably an uncertainty left, or risk, which has to be appropriately dealt with during tunnelling. For Romeriksporten, much criticism has been raised against the planning process, since such unexpected damages have occurred. This is a claim that must be taken seriously by the relevant parties. Pre-investigations for tunnels with respect to potential damage to the groundwater and the vulnerable environment dependant on it, can certainly be improved. It does not, however, mean that just multiplying current methods and routines will get us much further. In this case, an extensive programme of seismic profiles, detailed mapping from core drillings and even permeability tests, would neither have led to substantially change in the tunnel design or in the tunnelling procedures, nor prevented the partly poor sealing result and damage during construction.

Uncritical follow-up of all suggestions of interesting "nice to know" investigations, put forward by experts, parties and bodies who have their own agenda, will in fact act as a threat to a rational use of funds and could lead to a counter-productive planning process.

New aspects have to be included:

- Routine risk analysis, both for guiding targeted investigations and for realistic uncertainty assessment as part of the decision making process,
- Detailed vulnerability studies of potentially influenced environments, and
- Methods to predict the degree of difficulty and effort needed in permeability reduction, leading to improved design of grouting or other means for water sealing. (This will be more complex than mapping in situ permeability!)

Still, considerable uncertainty will probably remain concerning prognosis of leakage treatment in rock tunnels. The uncertainty of this will always be different from that of excavation costs and rock reinforcement. Even pilot tunnels will be of poor value, as they may introduce the damage originally to be prevented, and they will make the pre-grouting more difficult.

On the other hand, the uncertainty concerning damage to the surroundings must be removed. A best possible assessment of the geo-hydraulic model and a thorough mapping of vulnerable biotypes should be standard, consequently leading to well founded and differentiated requirements vs. acceptable inflow in separate tunnel sections.

6.2 Improving public relations

When a tunnel project, such as Romeriksporten, has the potential for becoming "front page news", which may have a variety of different reasons, there will always be a hunt for scapegoats. That is OK as long as it targets in a fair way, but informants with different agendas will pop up and "co-operate" more willingly with journalists than those who know the realities. The times and conditions of Lieråsen tunnel, built 30 years earlier, will never return. A well working democratic society is depending on critical media, and decision makers must be prepared to stand up and face publicity. But, in fear of that, we must not aim at 100% proof solutions; that would cost

too much and even prevent socio-economic favourable developments. Some keys to improved public relation in such cases are:

- Do a proper job in first hand, so that uncertainties are a documented part of the project basis and risks are handled professionally,
- Communicate uncertainties (before) and difficulties (during) the construction, and
- Demonstrate correction capability based on a set of values broadly accepted in the community.

It goes without saying that most civil engineers have a lot to catch up with in this field. Some organisations buy their way out with help of professional public relation employees, but that may not work well unless they are very well informed in the special tunnelling business.

The public interest and awareness of tunnels, boosted by Romeriksporten, have given inspiration for a new seriousness in planning and follow-up of tunnelling works. Promising new co-operation relations have emerged between engineers, hydro-geologists and experts on botany and the environment.

6.3 Know-how and decision making

It is the author's experience and a clear lesson from Romeriksporten that the crucial element in successful pre-grouting is the skill and understanding by those involved at the tunnel front. The more demanding the tasks are becoming and the more sophisticated the tools will be, even more important is this aspect going to be in the future. For jobs above a certain degree of challenge, qualified and experienced engineering geologists should take part in the daily observations and decisions related to pre-grouting. Investigation drillholes ahead of the tunnel may be utilised for much more information and give improved basis for decisisons, compared to customary practice.

In a large scale, it has been shown that pre-grouting, properly performed, is as much as 20 times more cost-effective as post excavation grouting with similar resulting permeability. A certain influence of high groundwater pressure has also been observed: Even if pre-grouting becomes more challenging, the problems with post-excavation grouting may become even bigger. To postpone the "last" effort of grouting until later, should therefor not be an option in serious tunnelling.

A stressed contractual situation will influence negatively on the perfection of methods at the tunnel face, which one need in challenging leakage control. It is an interesting issue to further develop the contractual conditions and the incentives, so that a contractor is inspired to take more interest in the development of efficient methods and to optimise the effort towards the requirements. In the Nordic tunnelling tradition, this will imply a change, and it will only be accepted if the contractor still can be protected from severe financial risks.

6.4 Need for improved methods and materials in difficult conditions

The new focus on the potential danger to the environment from a range of efficient "chemical" sealing agents, have put on the agenda a strong need for improved performance of cement grouting or equal materials. In Romeriksporten, it has been found that by use of stable micro-cement grouts, a maximum effort could bring the average "permeability" (k) in jointed zones down towards some 5x10-8 m/sec, corresponding to a "hydraulic diameter" of the remaining conductive joints about 0,1 mm in moderately jointed rock. That may seem to represent "state of the art year 2000". This is confirmed from the experiences with leakage problems in the challenging railway tunnel Hallandsås in Sweden. For the strongest inflow demands and at high hydraulic heads, it is necessary to obtain permeation of joints half that wide, i.e. 0,05 mm. This might be a realistic goal for further research and development, e.g. by improving the cements and additives, but probably not by just milling ordinary cements even finer. Customary micro-cements seem to have inconsistent or unstable properties, and a quick formation of clusters is probably starting to obstruct efficient permeation only few minutes after leaving the colloidal mixer. Further, the reaction development is very much dependant on temperature and w/c ratio. At 7 – 8°C (as in Nordic bedrock) almost nothing happens with certain cements. Portland clinker types seem to work best at such low temperatures.

The means to distribute injected materials sufficiently complete into a designed collar around the tunnel need to be varied with the local conditions. A much larger and

more sophisticated range of efforts have to be included in the "tool box", including such as:

- High pressure (100 bar or above) pumps in several parallel lines, individually governed to suit any stop criteria,
- Firmly controlled, locally adjusted drillhole patterns. Challenging conditions may require as much as 100 holes per fan, both for efficient access to finer joints and in situations with lack of counter-pressure (effective minor stress),
- Outer, separately grouted collar in a): weak zones to avoid "break-through after excavation, and b): for preliminary sealing of coarser veins and thereby to enable use of high pressure in the main fan,
- Continuos quality control of mixed grout, related to strict requirements, e.g. considering filtration stability and reactions sensitive to pot time and actual rock temperature, and
- Methods for proportionally adding of agents that can accelerate grout stiffening and occasionally plug larger veins.

6.5 Long term effects

The unsustainable effects, both on nature and on properties, have become among the most important aspects in tunnelling. Large efforts must be allowed in any budget if needed to avoid damaging impact. Those effects however, which cannot be avoided, or only at an incomparable cost vs. the impact value, have to be declared through a profound decision and concession process open to the public. Such serious handling of the matter has now become the standard procedure for new tunnelling projects in Norway.

Both nature and mankind seem, however, to have a remarkable tolerance to groundwater lowering. No one asks anymore about the rather dramatic lowering of groundwater and change in the botany system above Lieråsen tunnel. Long term effects of that kind are still to be evaluated at Romeriksporten, but in view of the corrective works, none but minor changes in natural habitat are expected.

If damaging groundwater/pore pressure lowering can not been avoided, it is shown at Romeriksporten that re-infiltration of sweet water can work on a permanent basis, provided it is subject to consistent management and maintenance.

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8 INFLOW CRITERIA FOR A RAILWAY TUNNEL IN THE GREATER OSLO AREA

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ABSTRACT: The paper presents the leakage criteria and tunnel grouting strategy for the projected doubled-tracked rail-tunnel Jong-Asker in the Oslo-area. Investigations have focused on the consequences of groundwater drawdown, overburden settlement and nature vulnerability. Procedures for evaluating leakage rates, groundwater drawdown and subsequent groundwater level depression cones are presented as well as estimates from analytical calculations. Results from a parameter study employing a numerical groundwater model where several tunnel conditions are investigated. This study shows several relationships between the permeability of the grouted zone and drawdown that are presented and compared with empirical data. The analytical calculations show that the groundwater drawdown increases when the difference between the grouted zone permeability and the natural bedrock permeability decreases. It also shows that the groundwater level depression cone is strongly influenced by the geological structures. Water balance estimates are extremely important in defining the maximum allowed tunnel leakage rate.

1 INTRODUCTION

During the construction of transport tunnels in urban areas, there have been some recent incidents where leakage has caused the drawdown of the phreatic surface and, consequently, caused damage to structures due to settlements and negative effects for the environment. As a consequence, the design of future urban tunnels with respect to minimizing the water inflow is of uttermost importance. The overall aim of the design will be to choose a strategy for inflow control that will minimize the cost of the society to handle disturbances and damages due to an unwanted reduction of the phreatic surface.

During the focusing of what is “acceptable disturbance” of the environment, and, hence, an acceptable inflow of water, the tunnel designers will have to address the relevant parameters that will be important for the inflow control of urban tunneling works. By investigating the ground conditions, both at the elevation of the tunnel and within the boundaries of the influencing area above the tunnel, certain parameters will be decisive in classifying the acceptable level of leaking water volumes.

By the use of analytical functions that assume homogenous conditions, a rough idea of the drawdown of the phreatic surface can be estimated. But for more complex geological conditions such estimates may be far from the actual results. Norconsult has been using both 2-D and 3-D model simulations (VisualMODFLOW) in this project.

2 Calculations of groundwater drawdown and inflow to A tunnel
The area of influence due to leakages into the tunnel will depend on many factors:
The area of the tunnel
The depth of the tunnel
The original permeability of the rock
The permeability of the grouted zone around the tunnel
The original gradient of flow
The recharge due to precipitation

By using analytical functions one can get an idea of the drawdown around a tunnel. The following functions have been suggested:

(1) $$q = \frac{2\pi K h}{\ln\left[\frac{h}{r} + \sqrt{\left(\frac{h}{r}\right)^2 - 1}\right]}$$ (Lei, 1999)

(2) $$q = \frac{2\pi K h}{2{,}3\log[2h/r]}$$ (Freeze & Cherry, 1979)

(3) $$q = \frac{2\pi K h}{\ln\left[2\frac{h}{r} - 1\right]}$$ (Karlsrud/NFF)

Where:
q = leakage (m3/m/s)
K = hydraulic conductivity or permeability around the tunnel (m/s)
h = distance from tunnel to equipotential (m)
r = tunnel radius (m)

The above functions do not take into account the thickness of the grouted zone around the tunnel. The functions give the inflow depending on an average hydraulic conductivity of the rock. The following function is introducing the thickness of the grouted zone and its corresponding hydraulic conductivity.

(4) $$q = \pi K h \frac{2}{\ln\left[\frac{(r+t)}{r}\right]}$$ (NFF)

Where
t = thickness of grouted zone (m) (otherwise as above)

These functions do not take into account any stratification of the geology or any cross-flow gradient. However, these analytical functions will work well for conditions that can be regarded as homogenous and where the cross-flow gradient is small. For some of the chosen cross-sections of the Jong - Asker project these functions gave roughly the same results as the 2-D models.

Lindblom (1999) has pointed out that the important factor or parameter is not the leakage rate, but it is the consequences of the leakage that should be the main parameter for the grouting strategy.

3 TUNNEL SECTIONING OF THE PROJECT

The Jong - Asker project is located just south-east of the city of Oslo - in a densely populated area. The use of tunnels will shorten the railway alignment, minimize the requisition of land and reduce the nuisance during construction and operation of the railway.

The geology of the area is mainly cambrian-silurian sediments; shales, schists, siltstones with calcerous elements and sandstones. These rocks are overlain by permian basalts and porphyry and have intrusions of dolerites (diabase).

The support of the tunnel will be mainly bolts and shotcrete. It is estimated that on average 8 bolts and 4,5 m3 of shotcrete will be used per m tunnel. However, the tunnel is sectioned into 5 categories of support measures. The poorest rock quality along the line will be a major weakness zone with Q = 0.03.

The tunnel will encounter a number of different conditions, and thus it is important to section the tunnel and classify these sections according to the anticipated problems. These are:
1. The potential of settlements of structures founded on soft materials
2. The use of groundwater and springs along the future line
3. The use of the ground for recreational purposes
4. The presence of areas with special biotopes

The classification of the potential areas that could experience settlements was based on the ground investigations carried out, supplemented with soundings down to bedrock. The areas were classified as follows:
1A. Areas that could experience settlement of more than 80 mm
1B. Areas with a potential settlement of 40 - 80 mm
1C. Areas with settlement potential of less than 40 mm

More than 100 households are currently using groundwater from local wells as their primary or secondary source of fresh water supply. The households using such groundwater as their primary source will all be supplied with municipal water.

In cooperation with local associations dealing with the preservation of recreational areas, a list of the areas along the future railway line were established. These areas were classified based on their sensitivity to a potential lowering of the ground water level and their popularity as recreational areas. By combining the sensitivity for potential damages to the environment and due to settlements, three classes of allowable leakage rates were established.
Class 1. Moderately tunnel leakages, 8 - 16 l/min/100 m tunnel
Class 2. Low leakage rate, 4 - 8 l/min/100 m tunnel
Class 3. Extremely low leakage rate; < 4 l/min/100 m

The above numbers were determined using VisualMODFLOW, and will be referred to in the following.

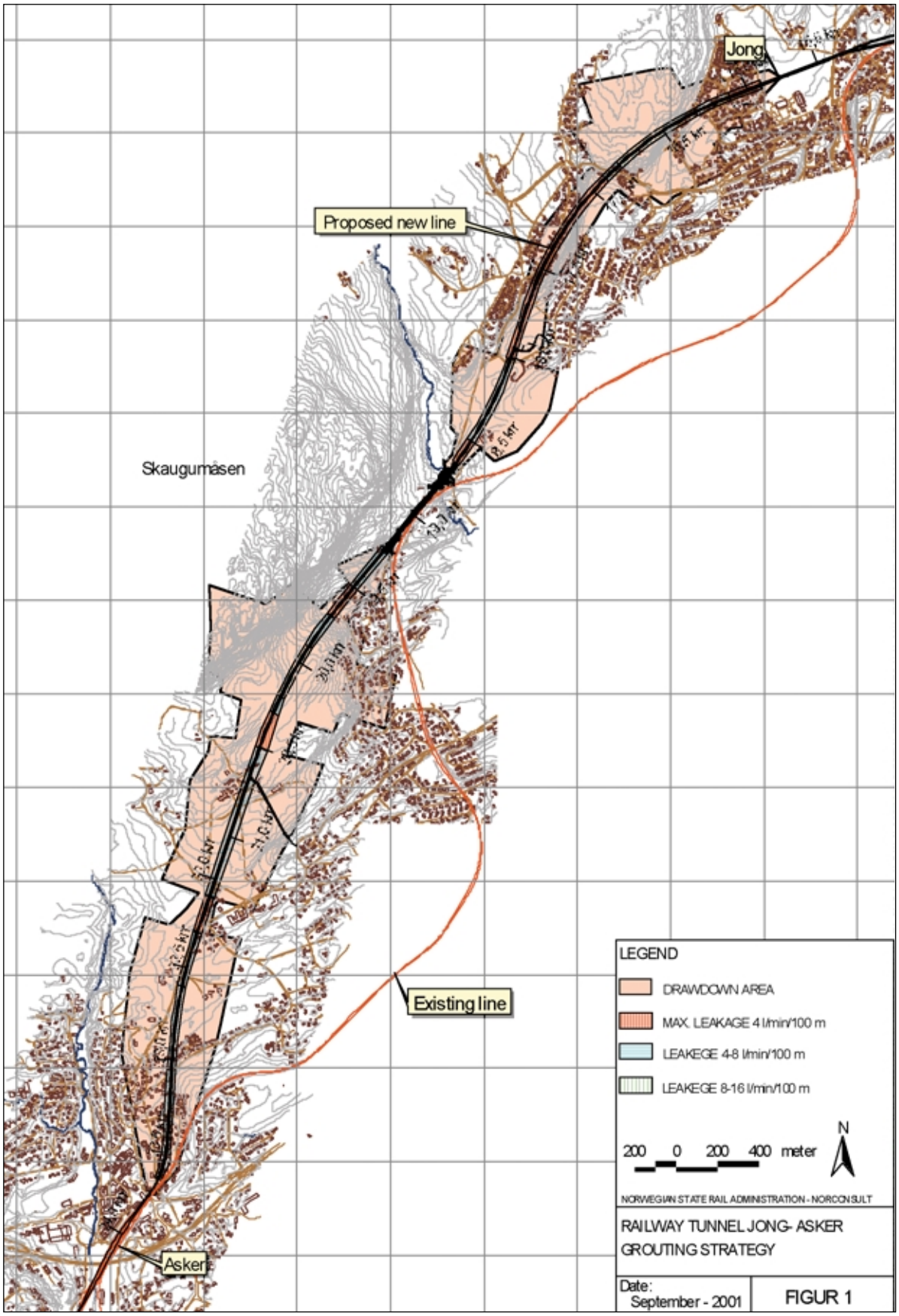

Figure 1. Classification of leakages and potential drawdown areas for the Asker-Jong project

4 NUMERICAL MODELLING

If the ground conditions are well known, the geological conditions are complex or has a distinct anisotrophy and the topography will result in a distinct cross-flow, a numerical model should be applied. Numerical modeling have been performed on some projects. The railway tunnel to Oslo International airport, Romeriksporten (Kitterød et.al., 1998) and the railway tunnels, Frodeåsen, Tønsberg and Jong-Asker.

The simplification of the models will have to be done. Zones in the rock having a high permeability will have to be transformed into porous zones or layers ("Equivalent porous medium" model, Anderson & Woessner (1992), Committee on Fracture Characterization and Fluid Flow (1996). The method of simulating permeable joints/zones using porous medium models have also been published in international magazines, Gburek et al., (1999), Allen & Michel, (1999) and Pohll et al., (1999).

For the Jong - Asker project the geology is quite complex with volcanic rocks from the Permian age overlaying sedimentary rocks (shale, phyllite and calcareous shales). The area has intrusions of near vertical dykes (dolerite), which may in some cases act as a barrier for ground water flow and in other cases acts as water bearing structures with a higher permeability than that of the surrounding rocks.

Initially, the VisualMODFLOW were run using an isotropic, homogenous model with three sets of permeability coefficients of the rock and the grouted zone and three sets of leakage rates into the tunnel (4, 10 and 24 l/min/100 m tunnel).

The model is run using transient flow, which is balancing out to a steady state after approximately 3 years. From Table 1 it is seen that using the leakage rate as the only parameter may give rather large influence radius and may lower the water table by more than 15 m.

The results of the parameter study show that the drawdown area will increase with the depth of the tunnel, and if the ratio between the permeability of the original rock compared to the grouted zone is decreased. In other words having a "groutable", rather permeable rock, will increase the possibility to minimize the drawdown of the water table.

5 water balance

To be able to model the water balance information about evotranspiration, run-off and infiltration into the ground must be included in the input parameters. The permeability of the overburden material will of course have some importance of the infiltration volumes. At the Jong - Asker project some of the overburden materials are marine clays giving very little infiltration to the ground. On the other hand some sands and gravels are overlaying the rock surface and will be acting as an infiltration layer were the more permeable soils are infiltrated from the higher ground levels. At these conditions the precipitation will greatly influence the water balance.

The water balance model can also be used to predict the ground water level at observation wells or springs. This has been done at another project and where the simulations over time could be compared to the observations of the well.

Table 1. Results of the run with medium permeable rock

Permeability of adjacent rock: 5 LUGEON (= 5 10^{-7} m/s)			
Depth ≅ 17 m	Sim. leak.. = 4,2 U.I. = 3 m D.I. = 2 m WTL = 1,3 m RGP = 0,14 L	Sim. leak. = 10,2 U.I. = 652 m D.I. = 336 m WTL = 7,6 m RGP. = 0,45 L	Sim. leak. = 24,2 U.I. = 778 m D.I. = 540 m WTL = 17,2 m RGP. = 2,2 L
Depth ≅ 27 m	Sim. leak. = 4,2 U.I. = 0 m D.I. = 0 m WTL = 1 m RGP = 0,1 L	Sim. leak. = 9,9 U.I. = 642 m D.I. = 330 m WTL = 7,1 m RGP = 0,28 L	Sim. leak. = 24,0 O.I. = 767 m N.I. = 530 m WTL= 15,7 m RGP. = 0,97 L
Depth ≅ 57 m	Sim. leak. = 4,3 U.I. = 0 m D.I. = 0 m WTL = 1 m RGP = 0,055 L	Sim. leak. = 10,0 O.I. = 641 m D.I. = 334 m WTL = 6,9 m RGP = 0,14 L	Sim. leak. = 24,4 O.I. = 775 m D.I. = 539 m WTL = 15,5 m RGP. = 0,4 L

Legend: Sim.leak. (simulated leakage rate, l/min/100 m),
U.I. (upstream influence radius), D.I (downstream influence radius), WTL (water table lowering above tunnel),
RGP (required permeability of grouted zone, 1 Lugeon = 1 10^{-7} m/s)

6 GROUTING STRATEGY AND GROUNDWATER MONITORING

For the railway tunnel Jong - Asker the grouting strategy will consist of pregrouting the tunnel in sequences of 3 or 4 blasting rounds, depending on the classification. The length of each grouting sequence will be 21 - 27 m, and supplementary holes will be drilled depending on the leakage water into the drillholes. However, the primary parameter for changing the grouting effort will be the response of the ground water level. To be able to respond quickly to any change in of the ground water level about 60 wells recording the piezometric levels in the rock and in the overburden are established. The contract is emphazising the need for change of the grouting effort if it is found necessary. The ground water levels are continuously recorded and will be displayed on the internet, using a telephone modem, round the clock for the following up of the grouting effort. Still, water leakage rates down to 4 l/min/100 m is a very low figure indeed, and requires grouting efforts using ultrafine microcements and high grouting pressures.

Grouting strategies for tunnels in suburban areas should be able to cope with alternating ground conditions and the aim should be to minimize the environmental effect. The planning of the grouting effort should be done in such a way that the continuous recording of the ground water levels and the piezometric readings would be the most important parameter for intensifying the pregrouting effort. Measurements of the leakage water in the tunnel are a secondary parameter.

When the overall goal is to minimize the effect of any lowering of the groundwater a pregouting scheme has to be adopted. The grouting program has to include the use of very fine microcements, cement additives and modern equipment and using high grouting pressures.

It is recommended that modeling groundwater flow and water balance calculations are included in the investigation phases. In this way a better understanding of the different parameters importance for the groundwater control that have to be included in the project is achieved.

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Grouting Metro Ring Oslo.

9 PLANNING OF A 25 KM LONG WATER SUPPLY TUNNEL IN AN ENVIRONMENTALLY SENSITIVE AREA

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ABSTRACT: For a major project to provide future water supply for the city of Oslo, a new tunnel is under planning from the Holsfjorden to the water treatment plant at Oset, Oslo. The tunnel will be 25 km long and it will pass a number of environmentally sensitive areas along its route. To improve the predictability and make better prognosis for the consequences of the tunnel on the external environment, a thorough planning process has been performed. This included geological pre-investigations and also a comprehensive study of the hydrogeological conditions. Based on the geological data and hydrogeological study detailed criteria for the rock mass grouting and tunnel lining could be established.

1 INTRODUCTION

During long periods of dry climatic conditions today's water supply capacity in Oslo may be scarce. Therefore, feasibility studies for supplementary water supply to Oslo have been undertaken. These studies conclude that supply of water from Holsfjorden is the best solution. Holsfjorden is located about 25 km west of Oslo, see Figure 1 (Holsfjorden to the west, Oslo to the east). There are lots of lakes around Oslo, however, due to their importance for recreational purpose, their use are restricted. In Holsfjorden, a branch of Tyrifjorden the fourth largest lake in Norway, there will hardly be any effect of the water outtake in question, 4.5 m3/sec at maximum. The water quality in Holsfjorden is also of premium quality, especially 50 - 100m below the water surface.

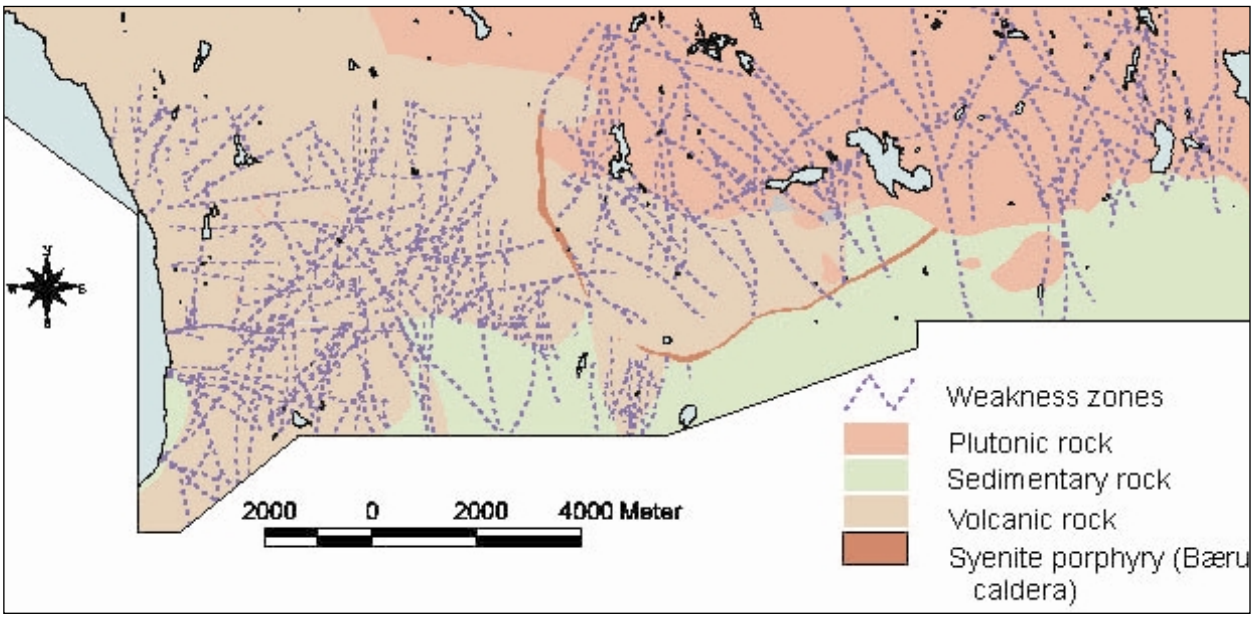

Figure 1. Overview map - geology

Due to hilly landscape between Oslo and Holsfjorden, it is clear that the only alternative for transportation of water from Holsfjorden to Oslo is by use of a tunnel. The tunnel will be designed with a cross-section of about 22 m2 if excavated by drill and blast, about 13 m2 if excavated by tunnel boring machines. A major concern about this water tunnel, which could be routed in various ways mainly dependent on where to start in Holsfjorden and where to end in Oslo, was that it had to go through areas where the surroundings were presumed sensitive to groundwater leakage to the tunnel. Two major groundwater concerns were obvious due to the extensive experience from tunnelling in the Oslo region:

- Pore pressure reduction at the interface rock-soil causing consolidation settlements in marine clays, again causing damage to buildings founded on soil, see Karlsrud (2001).
- Damage to the natural areas in terms of reduced water level/flow rates in lakes/rivers, drainage of bogs, drainage of springs and damage on vegetation.

This article aims at describing the various steps in the process of planning the tunnel, focusing on the environmental aspects.

2 OVERVIEW OF PLANNING PROCESS

The planning of the water supply tunnel described in this article took about one year and included three main phases:

- System studies, including concept for water intake, how to get the water through the tunnel, water treatment concept, etc.
- Location studies; including where to place the water intake, location of the water tunnel and the access tunnels, location of the water treatment facilities and how to get the "ready-for-use" water to the customers taking advantage of the existing infrastructure supplemented with possible new routes for transportation of the water.
- Preliminary design for the chosen alternative.

One main topic during the system studies was if the water was to be pumped from Holsfjorden at 63m above sea level to the water treatment facility in Oslo at either about 80m or 150m above sea level. The conclusion was to transport the water by means of water pressure difference and that all pumping shall take place in Oslo from about 55m above sea level to the water treatment facility.

The location studies included four possible intake alternatives at Holsfjorden, three possible locations of the water treatment facilities, and three main routes for the tunnel between Holsfjorden and Oslo; a southern route, a northern route and a middle route, and various tunnel routes for transportation of the water to the customer. The most important aspects for the tunnel route between Holsfjorden and Oslo were geological and hydro geological conditions and the environmental concerns. Through the study the choice of alternative became pretty obvious since most subjects analysed pointed in favour of a almost straight west-east connection between Holsfjorden and Oslo.

2 GEOLOGICAL PRE-INVESTIGATIONS

The basis for the engineering geological pre-investigations has been existing studies of the Permian geology in the Oslo field. These studies include detailed mapping of rock types and structures such as faults, fracture zones and igneous intrusions. As a part of planning of a new railway in partly the same area as the Holsfjorden-Oslo water tunnel, NGI carried out rock mass classification of numerous locations . During the project period for the water tunnel some additional field mapping was carried out. More extensive, though, were the site investigations in form of drillings and geophysical surveys. These were for the water tunnel:

- Refraction seismic survey: 7.0 km
- Inclined core drillings with water pressure testing: 9 bore holes of altogether 1040m
- Very low frequency (VLF) survey: 6km
- Inclined percussion drillings with water inflow tests: 11 bore holes of altogether 2200m
- Rotary soundings: 25 bore holes
- Shafts and drillings for polluted soil samples

3 GEOLOGY

4.1 Rock types and weakness zones

The bedrock consists mainly of volcanic rock types from early Permian, about 13.2 km, and plutonic rock types from late Permian, about 10.6 km, see Figure 2. About 1.5 km of older sedimentary rock is also found in form of hornfels due to its vicinity to the younger plutonic rock. The distribution of rock types along the 25,4 km long tunnel is shown in Table 1. Table 1 also gives the number of presumed weakness zones in the various tunnel sections, that is faults and fracture zones, perhaps accompanied by igneous rock intrusions.

4.2 Tectonics

In early Permian the crust in the Oslo field was exposed to large tensile stresses. Deep cracks and faults developed and opened up for magma from the deep. These lava flows accumulated to a total thickness of 1000-2000m. In late Permian large magmas rose to shallow depths in the crust, and solidified as large plutonic bodies of granites, syenites and monzonites. This latest event defor-

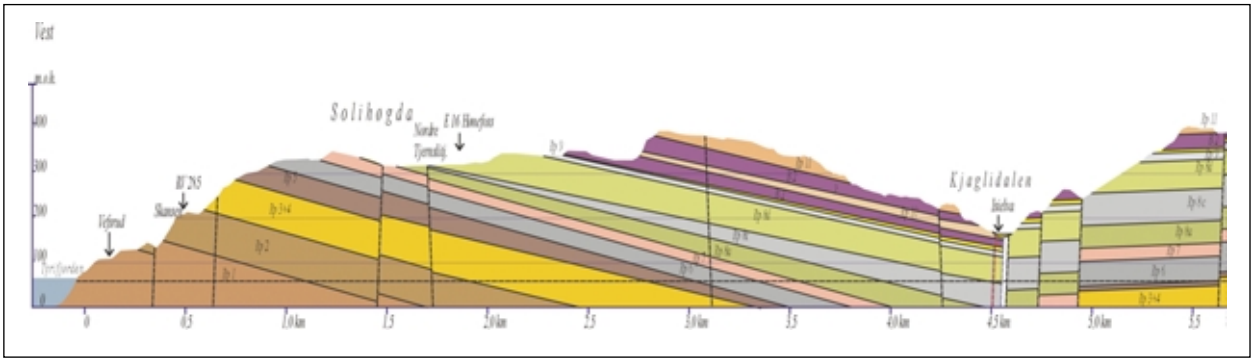

Figure 2. Part of geological section

Chainage	Length	Rock type	Presumed weakness zones (Nos.)
0-8940	8940	Rhombus porphyry	17
8940-9140	200	Syenite porphyry	2
9140-11900	2760	Basalt (B3)	9
11900-14170	2270	Plutonic rock	7
14170-14530	360	Basalt (B3)	2
14530-15640	1110	Plutonic rock	5
15640-16810	1170	Basalt (B3)	2
16810-21680	4870	Plutonic rock	16
21680-23220	1540	Hornfels	10
23220-25400	2180	Plutonic rock	5

Table 1 Distribution of rock types along the tunnel

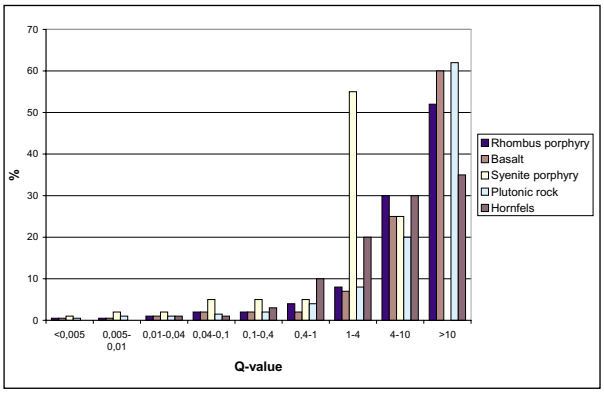

Figure 3. Distribution of Q-values in various rock types

med the sedimentary rocks of Cambro-Silurian age and lifted and tilted the overlying lava flows. Caldera subsidence also took place, with up to 740m subsidence. The structural results of these major geological events are today seen as numerous faults and fracture zones. The most frequent strike direction of the major faults is N-S to NNW-SSE and vertical to sub-vertical dip. Some faults have been cemented by calcite and appear today as calcite breccias. Others are found as up to some tens of metres wide zones with clay, crushed rock and dense jointed rock.

The rock mass, outside of the weakness zones, usually have two joint sets plus sporadic joints to three joint sets, making up about 75% of the observations . Typically, a N-S joint set with vertical to sub-vertical dip, another sub-vertical joint set and perhaps one sub-horizontal joint set, are found. Joint spacing may vary from a few decimetres to two to three metres.

4.3 Rock mass quality

Based on the pre-investigations, the distribution of rock mass qualities has been estimated, see Figure 3. The Q-method has been used for rock mass classification, Grimstad et al. 1993).

Figure 3 shows that the rock mass quality in general is favourable for tunnelling. When the Q-value is larger than ten, which is predominant, rock support are generally considered not necessary. In the poorest rock mass reinforced ribs of shotcrete or cast concrete lining is necessary. A tunnel from Holsfjorden to Oslo will, regardless of route, come across major faults due to their N-S orientation, see Section 4.2. The only way to avoid major faults is to stop the tunnel, which in fact has been done for the most easterly area by the lake Maridalsvann. Refraction seismic lines in the lake and south of the lake, indicated extremely poor rock conditions over long sections, therefore it was decided to use pipelines at the seabed instead of a tunnel.

4.4 Rock cover

The most shallow tunnel depths are around 100m with 70m as minimum. Maximum rock cover is around 350m. About 6500m (26 %) of the tunnel has more than 250m of rock cover.

5 IDENTIFICATION OF NATURAL AREAS WITH varying SENSITIVITY TO GROUND WATER DRAINAGE

Since pore pressure reduction and settlement in marine clays is a rather limited problem for the water tunnel route chosen for preliminary design, the following will focuse on the possible damage to the natural areas in terms of reduced water level/flow rates in lakes/rivers, drainage of bogs, drainage of springs and damage on vegetation.

5.1 Natural areas and principle for affect on groundwater

The possible vulnerability of the following type of natural areas were emphasised:
- Lakes and ponds
- Bogs of all types
- Swamps
- Influent springs and streams

The groundwater level in an area depends on the water balance, which can be described by:
- Inflow
- Outflow
- Storage changes

If a tunnel causes groundwater drainage this will affect the water balance of the area and consequently the groundwater level. This implies that an area with a small inflow will be more sensitive to a given tunnel leakage than an area with a larger inflow. This also implies that ecological consequences of a given tunnel leakage probably will be larger when the inflow is small. The vulnerability of an area is therefore related to the area's inflow and, therefore, the size of the catchment area.

5.4 Methodology

The basis for the analysis was digital maps of land cover by which groundwater dependent type of natural areas could be identified and isolated. The vulnerability of the various types of natural areas was then classified by the size of the actual catchment area.

The size of the catchment area was calculated from a digital elevation model (DEM) with a resolution of 10m ¥ 10m and contour interval of 5m. This was done by calculation of the area that drains to the single points. In the first place the part of the catchment area to the single locations of lakes, ponds, bogs, etc. was calculated. After that the total catchment area to the single locations was calculated by adding all parts of the catchment area upstream of the actual locations. Figure 4 shows the digital elevation model as a so-called "hillshade model". Figure 5 shows the catchment areas and specific runoffs.

The vulnerability based on the size of the catchment areas was classified in the following way, with Class 1 as the most vulnerable:

Class 1: Bogs and lakes/ponds with catchment area less than 0.5km^2

Class 2: Bogs and lakes/ponds with catchment area between 0.5km^2 and 1.0km^2

Class 3: Bogs and lakes/ponds with catchment area between 1.0km^2 and 2.0km^2

Class 4: Bogs and lakes/ponds with catchment area between 2.0km^2 and 5.0km^2

Class 5: Lakes that belong to river systems that run through the model area: large catchment area and little vulnerability.

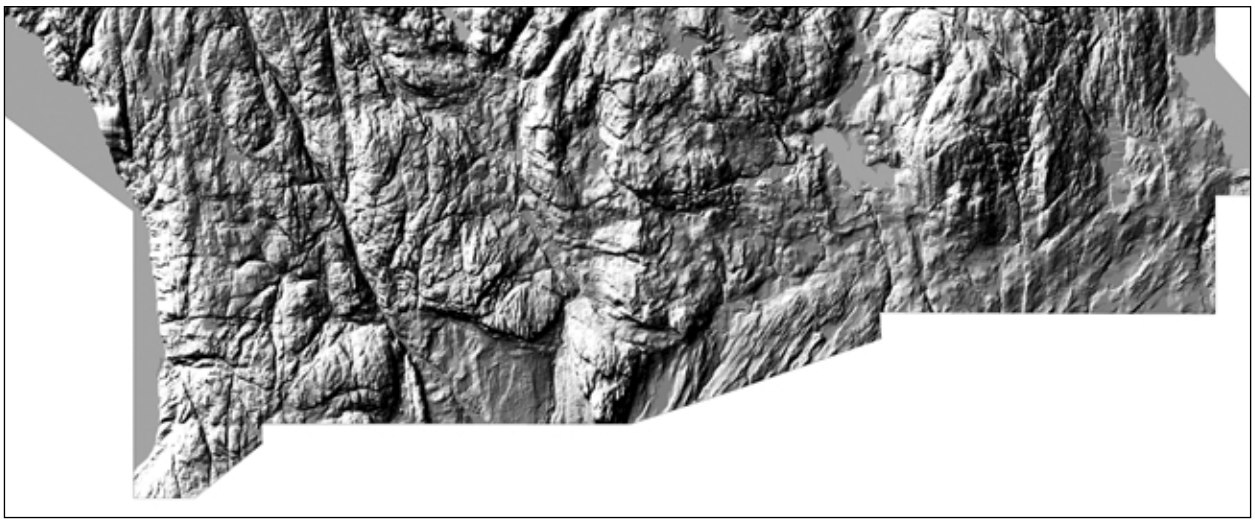

Figure 4. The digital elevation model shown as hillshade

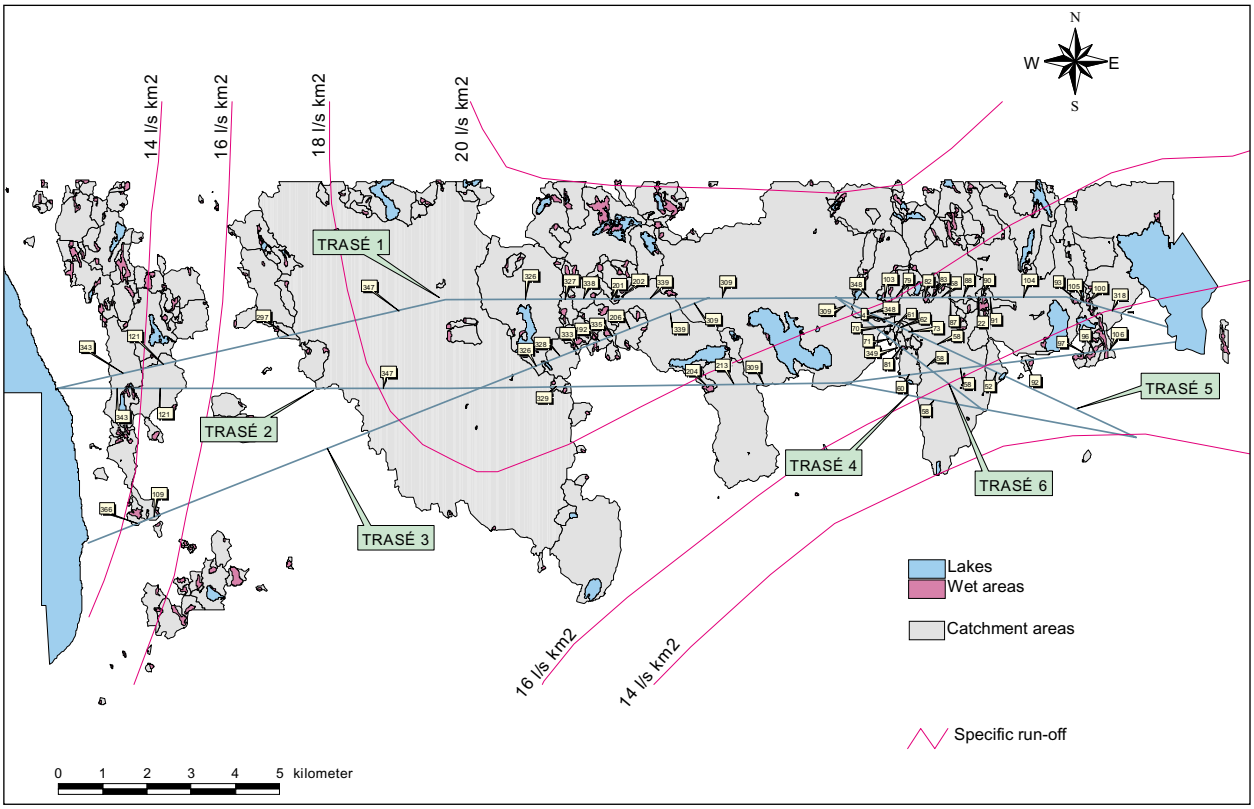

Figure 5. Catchment areas and specific run-offs

Since it is not given that the selected areas from the digital maps of land covers include all the areas with various types of natural areas dependent on groundwater, an additional concavity model was used. The purpose of this model was to identify areas with possible near surface groundwater level, and these areas were described as "potential vulnerable" areas. The various steps in the method are described below:

1. Each point (10m x 10m) in the elevation model has an elevation, z. For each point the mean elevation, z_m, inside a circle with diameter 250m was calculated.
2. The difference between z and z_m, was calculated.
3. Points with $z - z_m < -1$m were recorded in the data set.
4. Points in slopes steeper than 15° were removed from the data set since these areas probably only are dependent on seepage water.
5. The remaining points in the data set were checked to what degree they resulted in continuous areas. Areas that only included one or two points were removed from the data set.

The resulting classification of vulnerability for a part of the total planning area is shown in Figure 6.

The last step of the vulnerability analysis was to check some of the "wet" locations near the tunnel route. By field mapping the (immaterial) value of the nature could be estimated at specific locations, and it was possible to

Figure 6. Classification of vulnerability based on a catchment area model and a concavity model, section of map

classify the locations on a scale from "limited/small" to "high local value". Further, the sensitivity with respect to reduced groundwater table could be classified. The field mapping also revealed that the "potential vulnerable" areas found by the analysis described above, actually include smaller bogs, swamps, etc. However, the actual vulnerable areas tended to be smaller than found by the analysis.

6 RESTRICTIONS ON GROUNDWATER DRAINAGE TO THE TUNNEL

6.1 General

Restrictions on groundwater drainage to the tunnel were evaluated with respect to the following:

1. Vulnerability of natural areas to groundwater leakage
2. Risk for settlements on buildings, see Karlsrud (2001).
3. Total groundwater leakage for the tunnel system (water tunnel and access tunnels) must not exceed 7500 litre per minute, corresponding to 27 litre/minute per 100m tunnel. This amount of water corresponds to 20% groundwater in the water tunnel when the flow is at minimum.
4. Leakage of polluted groundwater is not allowed.
5. The quality of natural groundwater: The blend of Holsfjorden water and groundwater shall not contain concentrations of elements that do not satisfy the requirements to drinking water, and to raw water before processed in a water treatment facility.

6.2 Polluted groundwater

During the design period locations were identified where polluted groundwater was considered being a major concern. For the chosen tunnel route the following main possible sources for pollution were identified):

- CCA (**C**opper, **C**hrome and **A**rsenic) from a closed down impregnation facility
- BTEX (**B**enzene, **T**oluene, **E**thyl benzene and **X**ylene) from leaky buried tanks
- BV (**B**acteria and **V**irus) from sewer pipes

For the possible CCA pollution a rather extensive investigation program was conducted. Due to the moderate concentrations of CCA found in soil and groundwater in combination with the hydro geological conditions at the location, it was concluded that the risk for getting CCA in detectable concentrations in the water tunnel could be neglected. Therefore CCA pollution did not influence on the groundwater leakage restriction in that area.

Possible transportation of BTEX and BV to the tunnel by groundwater was analysed by numerical modelling. "Worst case" scenarios were used in the modelling work, and the conclusion was that either BTEX or BV would cause any problems for the raw water quality. Therefore it did not influence on the groundwater leakage restriction in areas where sewer pipes exist and leaky buried tanks may exist.

6.3 Quality of natural groundwater

Groundwater samples were collected from 19 boreholes along the tunnel route. From eight of the bore holes samples were collected from two different depths. Analysis of the samples showed that the concentration of manganese (Mn) could be a problem. However, the Mn-concentrations varied from very low to very high. In addition, data were available from a water tunnel in rhombus porphyry and basalt south of the western part of the Holsfjord tunnel. Mn-concentrations in samples taken from groundwater actually leaking into this existing tunnel, were generally very low.

An adequate explanation of the very high Mn-concentrations that were detected in some samples was not found. The most likely explanation was that Mn is connected to water bearing faults and dense jointed structures by Mn-coatings on joints that dissolve in the groundwater under low content of oxygen. Leakage from these water bearing structures have to be reduced by grouting and perhaps water tight linings due to environmental reasons. Based on this some scenarios were estimated, and it was found very likely that the Mn-concentration in the raw water would be lower than 50μg/l, which is the maximum allowed concentration in drinking water.

6.4 Vulnerability of nature to groundwater leakage

The effect on the nature based on the rate of groundwater leakage to the tunnel in percentage of the run-off in a catchment area, were classified as follows:

- Leakage < 10% of the run-off: **no** or **small** effect
- Leakage 10 – 20% of the run-off: **medium** effect
- Leakage > 20% of the run-off: **large** effect

The project has a strict requirement that the effect on the natural environment shall be no or small. Thus, the calculations of tolerated groundwater leakage in the various catchment areas were restricted to 10% of the run-off. Where catchment areas had not been calculated because these areas did not include land cover by which groundwater dependent type of natural areas were identified and isolated (see Section 5.2), the leakage rate has been set to around 30l/min per 100m tunnel. Most of these areas contain "potential vulnerable" areas. Due to the overall restriction of a total leakage rate of 7500l/min (27l/min per 100m) for the whole tunnel system, it was necessary to reduce leakage rates to less than 10% of the run-off in some areas. Altogether, the tolerated leakage rates for all the 29 tunnel sections used in the analysis varied from around 5l/min per 100m to around 40l/min per 100m. Overall, the tolerated leakage rates may be considered on "the safe side" with respect to possible damage to the natural areas.

7 HYDROGEOLOGICAL CONDITIONS

7.1 Hydraulic conductivity of the rock mass

Based on experience from usually shallow bore holes for water supply, it is known that the volcanic rocks of the Oslo field, i.e. rhombus porphyry and basalt, yield more water than other rock types in Norway, Morland (1997). The main reason for this is the porous character of the top of the various sub-horizontal lava flows. Core drillings to great depths indicate that large water losses in sub-horizontal layers are found down to about 130m. Deeper down, large water losses are found more sporadically and generally in connection with steep structures such as faults and diabase intrusions along tension joints.

In the Oslo area, the smallest horizontal stress is orientated E-W, and the main structures orientated about N-S tend to have the largest hydraulic conductivity which coincide with major faults and joints.

Several sources for estimation of hydraulic conductivity were available. These included:

- The bedrock well database of the Geological Survey of Norway
- Core drillings with water pressure tests
- Percussion drillings with water inflow tests
- Leakage rates in existing tunnel in volcanic rocks of the Oslo area

Information from more than 30,000 wells are available in the Geological Survey of Norway database. 116 wells from Permian rocks near the tunnel route were included in the analysis of hydraulic conductivity. For each well, information of depth and yield after drilling was provided. The transmissivity, T, for each well was estimated by:

$$\mathrm{Log}_{10}(T) = 1.17 + 1.13 \times \mathrm{Log}_{10}(Q/d), \text{ for } d < 50\text{m} \quad (1)$$

$$\mathrm{Log}_{10}(T) = 0.16 + 0.93 \times \mathrm{Log}_{10}(Q/d), \text{ for } d > 50\text{m} \quad (2)$$

where Q = yield (m^3/s), d = depth of well (m)

Hydraulic conductivity of the bedrock, K (m/s), has been calculated from the transmissivity by T/d.

Figure 7 indicates a good correlation between hydraulic conductivity and depth for wells with depth down to about 90m. For wells with larger depth, hydraulic conductivity seems poorly correlated with depth.

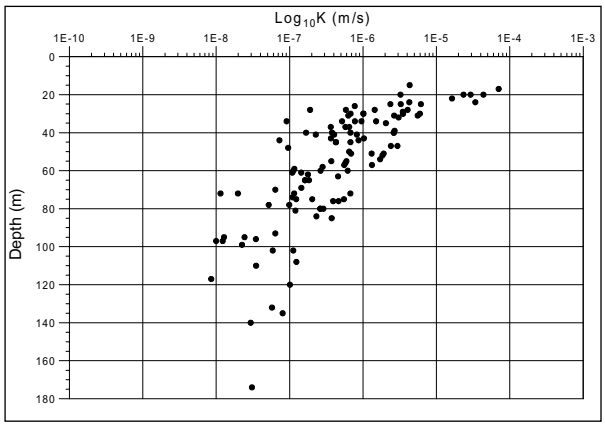

Figure 7. Well data. Correlation between hydraulic conductivity and well depth

Figure 8 shows that the data fit well to a log-normal distribution, and that the most representative value of hydraulic conductivity, the median value, is about 5 x 10^{-7}m/s for wells with depth down to 90m.

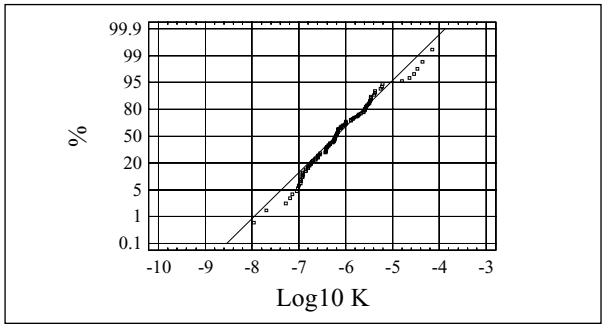

Figure 8. Well data. Log-normal distribution of hydraulic conductivity from wells with depth down to 90m

Figure 9 shows that the data fit fairly well to a log-normal distribution. The median value is about 5 x 10^{-8}m/s for wells with depths between 90m and 180m.
Water pressure testing measuring water losses by an overpressure of 1MPa were conducted systematically in

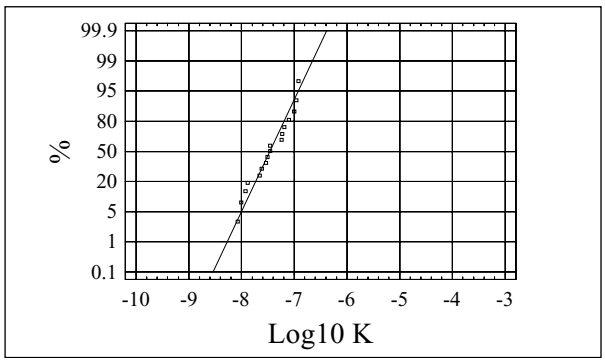

Figure 9. Well data. Log-normal distribution of hydraulic conductivity from wells with depth between 90m and 180m

all nine core drilled holes, from depth 5m to 140m. Figure 10 shows that the data fit well to a log-normal distribution, and that the median value is about 5×10^{-7}m/s. Note that most of the highest K-values are found from depths smaller than 60m.

11 percussion drillings were bored where water-bearing

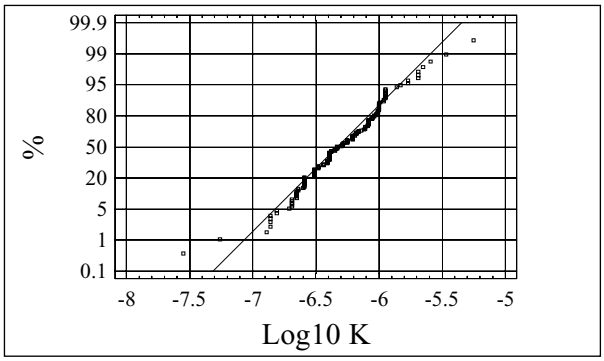

Figure 10. Water injection tests in core drilled bore holes. Log-normal distribution of hydraulic conductivity from depth between 5m and 140m

structures were expected. The largest vertical depth reached by these drillings was 235m. For each 50m interval of drilling the following procedure were followed for water inflow testing:
The hole was air flushed during 30 minutes through the drill string by use of a compressor.
Under the next 30 minutes was the rise of the water level in the hole measured by means of a manometer.

The maximum water inflow during one minute (out of 30 minutes) was taken as the value for estimation of transmissitivity. The transmissitivity was further estimated by Equation 1 and Equation 2.

As the 50m sections of the borehole above the actual test interval were not hydraulically isolated (open bore hole), the net inflow to the actual interval had to be calculated. This was done by subtracting the previous obtained inflow values from the sections above, from the total inflow to the borehole. In Figure 11 – 13, which show log-normal distribution of hydraulic conductivity, these intervals have been named as hypothetical wells.

Figure 11 shows that the data does not completely fit to a log-normal distribution. At least two statistical populations are indicated and K-values higher than about 2×10^{-7}m/s seem to make up a population of its own. These rather high values are presumed caused by structures with medium to large flow of water encountered in the various test sections.

Figure 12 (0 – 100m) and Figure 13 (100 – 250m) show similar results as presented in Figure 11, that is the pre-

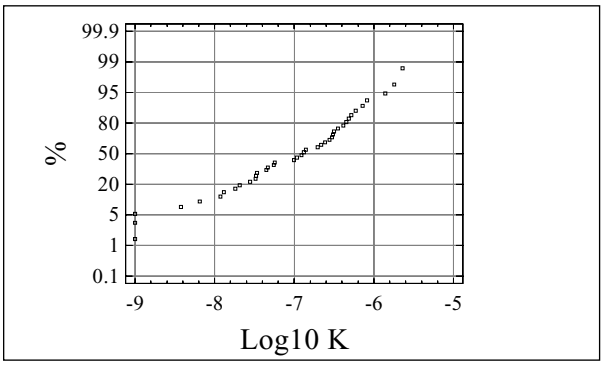

Figure 11. Water inflow tests. Log-normal distribution of hydraulic conductivity from all 40 hypothetical wells, depth 0 – 250m

sence of structures with medium to large flow of water. If these rather high K-values are neglected, the remaining K-values might represent the rock mass between

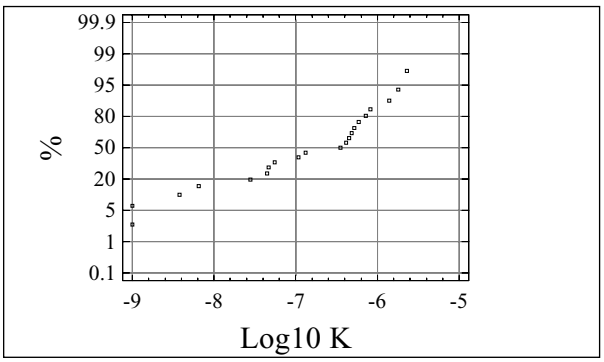

Figure 12. Water inflow tests. Log-normal distribution of hydraulic conductivity from 21 hypothetical wells, depth 0 – 100m

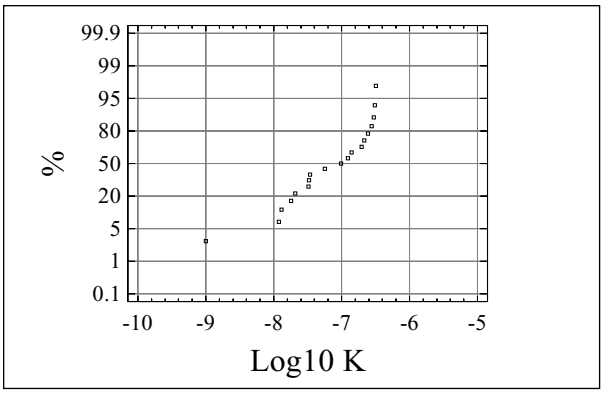

Figure 13. Water inflow tests. Log-normal distribution of hydraulic conductivity from 19 hypothetical wells, depth 100 – 250m

the highly conductive structures. These "rock mass" values are mainly in the range of 5×10^{-9}m/s to 2×10^{-7}m/s for depth 0 – 100m and 1×10^{-8}m/s to 1×10^{-7}m/s for depth 100 – 250m.
Log-normal distributions of transmissitivity, T (m^2/s), for T-values higher than $1 \times 10^{-6} m^2/s$ (corresponding to a

K-value of 2 x 10-8m/s for a 50m long section). i.e. "low-conductivity" sections are left out of the data basis, gave the following result:

- At depth 0 – 100m two ranges of transmissitivity were found: 1 x 10^{-6}m²/s to 3 x 10^{-5}m²/s, and 6 x 10^{-5}m²/s to 1 x 10^{-4}m²/s. The latter interval indicates that structures with large water flow had been encountered.
- At depth 100 – 250m the data were more evenly distributed. The median value is about 1 x 10^{-5}m²/s, and the minimum and maximum values are 2 x 10^{-6}m²/s and 7 x 10^{-5}m²/s respectively.

The existing 5.45km long Kattås tunnel passes through volcanic rocks about 100m below the ground level. Based on back-calculations from measured leakage rates, the conductivity is 7.6 x 10^{-7}m/s for a 250m long section, 2.9 x 10^{-8}m/s for the rest of the tunnel (5200m).

7.2 Conceptual hydrogeological model

Based on the work described in Section 7.1, a conceptual hydrogeological model for the tunnel route was established, see Figure 14. For the rock mass between the "water-bearing" structures the conductivity intervals of 5 x 10^{-9}m/s to 1 x 10^{-7}m/s at depth 0 – 100m and 1 x 10^{-8}m/s to 5 x 10^{-8}m/s at depth 100 – 250m are assumed to represent the intervals of 80% probability. The median K-value of depth 100 – 250m is 3 x 10^{-8}m/s.The medium permeable structure of T = 1 x 10^{-5}m²/s represents the "median water-bearing structure".

As can be seen from Figure 14, the model does not

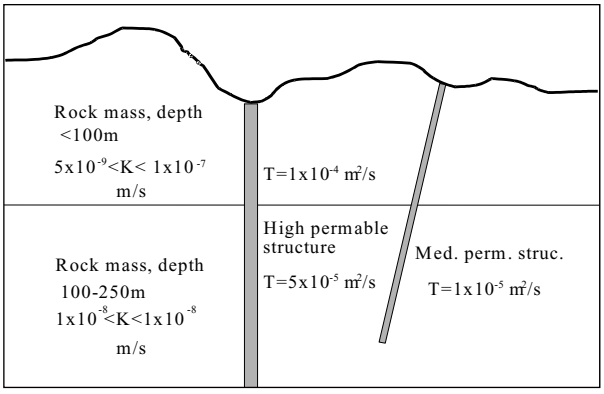

Figure 14. Conceptual hydrogeological model for the rock mass along the tunnel route

include depths larger than 250m. This is due to lack of data from these depths. 26% of the tunnel route has rock cover larger than 250m. However, the conceptual hydrogeological model was used also for depths larger than 250m for estimation of needs for tunnel sealing.

8 ESTIMATION OF EXTENT OF ROCK MASS GROUTING AND WATERTIGHT TUNNEL LINING

Three prognoses for tunnel sealing were carried out based on the conceptual hydrogeological model (see Section 7.2):

- Optimistic prognosis: "little" tunnel sealing. This was based on K20 = 1 x 10^{-8}m/s and T = 1 x 10^{-5}m²/s.
- Most likely prognosis: "most probable" extent of tunnel sealing. This was based on K50 = 3 x 10^{-8}m/s and T = 1 x 10^{-5}m²/s.
- Pessimistic prognosis: "large" extent of tunnel sealing. This was based on K80 = 5 x 10^{-8}m/s and T = 1 x 10^{-5}m²/s.

The extent of tunnel sealing was predicted for various tunnel sections as follows:

1. Calculation of maximum allowed leakage (see Section 6.4).
2. Calculation of leakage based on the conceptual hydrogeological model and K-values as described above. All assumed water bearing structures were given a T-value of 1 x 10^{-5}m²/s.
3. If predicted leakage was larger than allowed, grouting (injection) of the rock mass was "modelled".
4. Firstly, the water bearing structures were grouted. It was assumed that grouting will reduce the leakage to 10% of the original quantity. It was also assumed that each structure in average will cause 30m of grouting along the tunnel.
5. If the leakage was still too large after grouting, the length of the grouted zone between the structures was predicted. It was assumed that the K-value of the grouted zone around the tunnel periphery was reduced to 3 x 10^{-9}m/s, and the thickness of the grouted zone is 7m. A K-value of 3 x 10^{-9}m/s is based on back-calculations from a large number of tunnels in the Oslo region were grouting has been conducted, and leakage rates have been measured. The chosen K-value represents the best results achieved by grouting in these tunnels.
6. If the whole tunnel section length has been grouted and the maximum allowed leakage still is exceeded, the length of a watertight lining was predicted. Firstly, the structures were lined, and leakage eliminated from these. If this was not sufficient, the tunnel between the structures was lined until the leakage was reduced to the allowed rate.

For calculation of leakage rates the following equations were used:

$$Q = 2 \times \pi \times K \times d \times l / \ln(2 \times d / r_e) \quad (3)$$

$$Q = 2 \times \pi \times T \times d / \ln(2 \times d / r_e) \quad (4)$$

$$Q = 2 \times \pi \times K_i \times d \times l / \ln((r_e + t) / r_e) \quad (3)$$

where,
K = hydraulic conductivity of the rock mass
K_i = hydraulic conductivity of the injected zone around the tunnel periphery
T = hydraulic transmissitivity of the water bearing structures
d = depth below the groundwater level
l = tunnel length
r_e = the equivalent tunnel radius (= 2.7m)
t = thickness of the injected zone (= 7m)

The predictions described above resulted in the prognosis given in Table 2.

Category	Injection (m)	Injection (%)	Watertight lining (m)	Watertight lining (%)
Optimistic prognosis	5141	18	825	3
Most probable prognosis	17123	61	2246	8
Pessimistic prognosis	22810	82	2246	8

Table 2 Prognoses for tunnel sealing

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10 GROUTING – THE THIRD LEG OF UNDERGROUND CONSTRUCTION

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Dr S.Roald
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Dr Nick Barton and Assosiates
Tarald Nomeland
Elkem Materials

ABSTRACT: During recent years, some projects in Norway have experienced problems with ground water drainage caused by tunnelling in sensitive areas, and have also suffered hazardous water inflows during construction. These events have accelerated the development of grouting technology, which now offers improved possibilities for environmentally safe and economic viable underground construction. The aim of this article is to focus on the importance of rock mass grouting as an integral part of the tunnelling process, where the dual grouting strategies are leakage control and rock improvement.

1 INTRODUCTION AND AN IMAGE

The technology involved in underground construction has developed considerably during the last decades. Drilling equipment, tunnel boring machines, shotcrete equipment and materials and scaffolding systems have all been improved. The grouting technology has, however, not developed at the same rate or to the same level.

Environmental problems due to lowering of the ground water table, inflows of water stopping or slowing down the tunnel advance rates or problems with durability of the lining systems and installations due to wet conditions in tunnels have been common reading in news papers and magazines almost all over the world.

Lack of free space in the urban areas and major infrastructure needs in developing countries both require more tunnel projects. It is important to have as an overall philosophy that tunnels should be constructed without adversely influencing their surroundings. This philosophy should also be applied outside urban areas because environmental issues are important in areas of recreation and rural industry.

The future challenge is to construct environmentally friendly tunnels in a cost effective manner yet maintaining key safety aspects for the worker and the users. This implies the development of improved construction methods and techniques to reduce the permeability of the rock and provide necessary leakage control.

Images are commonly used to improve both communication and understanding. A three legged chair may be used to illustrate a stable situation. Remove one leg, and the chair becomes unstable. Add one leg, and it does not make any difference to the stability. An image closer to the civil engineering profession is the surveyors tripod. Imagine the surveyors tripod with more or fewer than the three legs. It would not work! The surveyors tripod is like a three legged chair made for rough terrain. For that reason each leg is made adjustable so it can fit almost anywhere.

Which leg is predominant is dictated by the actual terrain. Sometimes a stable position is achieved with one long leg and two short legs, other times the surveyor need three long legs or even three short legs. A minor deficiency in one leg my be corrected by adjusting the

two other legs. A major failure in one or more legs will make it impossible to do the surveying. Finally there is the fine tuning of the surveying table which only "the expert" will achieve.

Figure 1. Special need accommodated by the anisotropic approach.

2 THE OBJECTIVES OF ROCK MASS GROUTING

For an underground project a crucial question can be raised: Why perform rock mass grouting? The answer to this simple but important question also rests on three legs. The main purposes of grouting are:

To establish the tunnel in an economic and safe way.
To prevent environmental impact caused by the tunnel.
To improve the internal environment in the tunnel during operation.

Figure 2. A balanced approach may also be good.

Which one of these "legs" are most important may vary from one project to another. In other words, there are three different grouting criteria that should be evaluated during the planning and design phase of an underground construction project. Thus the governing criteria may vary according to the circumstances, and hence, the grouting strategy needed to conduct an appropriate leakage control may also vary. Recent developments in grouting technology may influence all of them in one way or another.

Prior to establishing a grouting strategy, the above questions must be answered and the objectives of the rock mass grouting must be clarified.

3 CONSTRUCTING THE UNDERGROUND OPENING

Constructing a tunnel normally involves three main activities. In terms of the image of the tripod, the three legs are:
• Excavation,
• Rock support
• Grouting.

Figure 3. The tripod, also an image of tunnelling.

This analogy equally applies to mechanical excavation techniques such as TBM or roadheader excavation of hard to soft rock tunnels as well as to the conventional drill and blast technique.

Excavation and rock support have both reached a more developed technological level than grouting. The interconnection between excavation and rock support have been studied and developed over many years and is very well documented, Barton and Grimstad, (1994). These two activities are therefore recognized as the strong legs

of underground construction whilst the grouting must be considered as the weak leg.

The traditional purpose of grouting in connection with tunnel excavation has been to avoid major water inflows and to reduce the amount of water that has to be pumped out of the tunnel system. In many cases, grouting has only been considered as a contingency measure, used to solve a problem only after a problem of water inflow has occurred. Some people have also been talking about the stabilizing and strengthening of the rock as a result of grouting. These effects are not well described in a scientific manner.

So far, there has been limitations in the material properties and the grouting technology that have constrained the predictability of the rock mass improvement effect from grouting. The recent development in grouting materials and grouting technology in Norway are increasing the possibility to achieve tangible rock improvements (impermeability and strengthening) by grouting. The key performances offered by the recent development in grout materials are:

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

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

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

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

4 HOW GROUTING IS INFLUENCING THE ROCK QUALITY

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

New grouting products and techniques allow penetration in the rock masses with permeabilities as low as 0.1 Lugeon. An interpretation of the physical process involved, has been developed using the Q-system parameters, and using important evidence from 3D permeability testing before and after grouting.

The evidence points to potential increases in the Q-system parameters RQD, J_r and J_w, and potential reduction in J_n, J_a and SRF. Permeability tensors may swing away from the permeable and least stressed joint set following grouting. This is consistent with small individual increases in RQD/ J_n, J_r/J_a and J_w/SRF. Each component may be small, but the combined result is potentially remarkable.

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

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

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

5 COST AND SCHEDULING IMPACT OF PRE-GROUTING

The cost of excavation is highly dependent on rock quality. The rock quality affects time consumption for drilling and charging. In addition the rock support work in poor rock conditions is time consuming. Poor rock conditions may not only increase the cycle time, they may also reduce the length of each round. The time consumption per meter driven is therefore increased disproportionately.

A calculation of the excavation cost as a function of rock quality was made by a contractor at a 100 m2 transportation tunnel in crystalline rock near Oslo. The relative cost is shown in Figure 4.

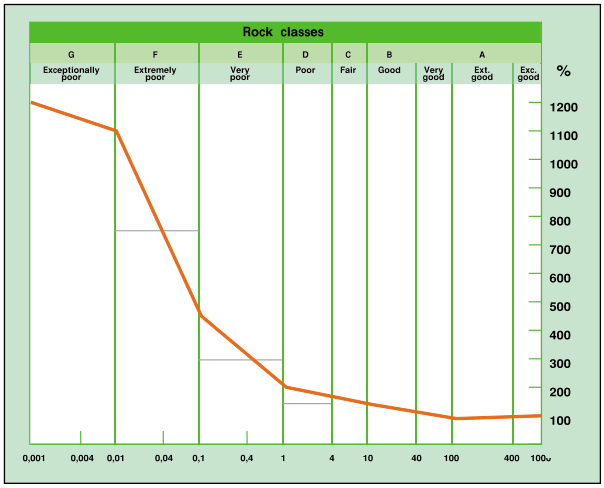

Figure 4. Relative excavation cost in different quality classes. Potential, quality dependent increases in rock mass quality and cost reduction, as a result of efficient pre-grouting.

Cost analysis indicates that improving the rock quality is both cost and time saving. As discussed above, grouting may improve the rock quality by one to two rock classes. Expressed in a simplified manner, in the poor rock categories; money spent on grouting is paid back by cheaper excavation and rock support. A prescribed inflow criterion may also be achieved at the same time.

The Public Road Administration in Norway have the same experience. At the tunnel project Baneheia in Kristiansand a customised grouting program was carried out to maintain the ground water table in an environmentally sensitive area. The cost saving in rock support, water shields and frost protection compensated for the cost spent on the grouting work and the overall saved time. At the project Storhaug Road Tunnel in Stavanger, the Public Road Administration experienced similar effects. A projected, full concrete lining was replaced by shotcrete and rock bolts after pre-grouting the poor quality rock zone, Davik and Andersson (2001)

6 GROUTING STRATEGIES

Ideally, the grout should be placed to form a low permeability zone circumferencing the tunnel. Grout that is transported far away from the tunnel is both a waste of time and a waste of money. Different geological situations require different grouting strategies. Basically, two main strategies can be outlined:

- Pregrouting in a normal jointed rock mass.
- Pregrouting in a heavily jointed rock mass or in weathered/ crushed zones.

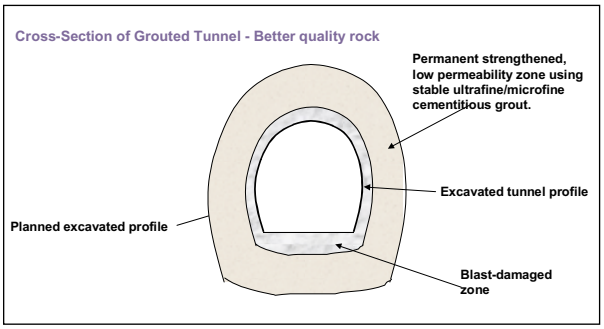

Figure 5. Pregrouting – good rock

For both situations, it is important that the grout-fan has a sufficient no of holes, so that the grouting scheme can be completed in one round. That also means that the grouting operation should start from the bottom and move upwards, see Figure 5.

It is important to complete the grouting in one round both from a technical and from an economical point of view. If grouting fails, the next round must be located outside the grouted area because many of the water bearing joints may be inaccessible due of the first grouting round. A marginal increase of costs to complete the grouting in one round will save the cost for the second round.

Grouting in good rock conditions: In good rock conditions it may be fairly easy to predict how the grout will spread out from each grout hole. Sometimes, it is also possible to watch the communication from one hole to another. The normal sequences of grouting in good rock masses are shown in Figure 5 If some open structures are hit in part of the area, these areas can be treated as poor rock conditions.

Grouting in poor rock conditions: Poor rock conditions require special measures. Before establishing the low permeability zone by stabilized grout, the spreading of the grout has to be controlled. This can be done by using a set controlled cement with variable viscosity to establish an outer "blocker" zone. After establishing the "blocker" zone, further grouting of the low permeability zone inside the blocker zone can take place as indicated in Figure 6.

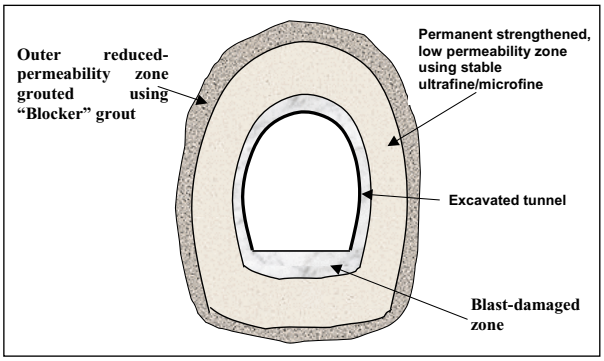

Figure 6. Grouting in poor rock conditions using Elkem Multigrout.

Grouting in good rock conditions can be done with very high grouting pressure. Small deformation is likely to occur, and is desired to help consolidate the rock mass. This means that a grouting pressure well above the minimum principle stress in the rock mass is required. The very high grouting pressure is required in order to penetrate the finest joints. This can be achieved with the fine particle sizes now available.

The poor rock condition generally requires significantly lower injection pressures. Normally, the grout should penetrate the weathered rock masses and consolidate the mass. Some times hydraulic fracturing is desirable. However the circumstances should be very carefully evaluated, especially in cases with low rock cover.

7 MATERIAL REQUIREMENTS

From a grouting technology viewpoint, the grout material should flow like water and harden like ice, - and remain like ice in the long term!! Durability and environmentally safe mean "What you put into the rock, should remain in the rock and be rock like".

Multigrout is a high performance, high durability grouting system, developed by Elkem ASA, a Norwegian company operating on a global basis. It is the spin off of 25 years of research work in the field of high performance, high durability concrete.

The Multigrout system rest on three legs:

- *High performance materials:* Elkem provides the materials. All materials have long term durability.
- *Grouting technology:* The Multigrout team will adopt best pratice grouting and transfer the technology to the users of the Multigrout system.
- *High performance equipment:* The development of the grouting equipment, including the provision of improved drilling and grouting equipment on TBM´s, will accelerate to reach a high performance level.

Figure 7. The Multigrout- system has three strong components.

The Multigrout constituents form a tripod too:

- *Pure Portland Cements:* Dispersed OPC, Microcement or Ultrafine Cement, dependent on the requirements.
- *GroutAid®:* High technology microsilica slurry. Improves penetration (see fig..) stabilizes the grout (see fig..) and improves the durability (see fig….).
- *Thermax™:* A mineral based, controlled setting cement, the best tool known to control the flow of the cementitious grout within the rock- a requirement for an engineering cost-/time- and quality controlled grouting process.

Figure 8. "Multigrout" materials are three-fold too.

The length of the "material legs" is dependent on several factors. The materials can be used together in different portions to suit the specific conditions. Sealing fine joints requires fine cements, open joints requires less fine cement and special attention to shrinkage problems, backflow into the tunnel (from small leakages or high volume water-fighting) or large consumption of grout without any pressure build up requires more use of the universal, material based grouting tool "Thermax".

8 ACHIEVEMENTS MADE BY ULTIGROUT

The leakage into a tunnel is mainly determined by the hydraulic conductivity of the rock mass, the head of water and to some extent disturbances caused by excavation and by the tunnel cross section. In practice, it is normal to measure the amount of water that leaks into a given section, e.g 100 meter in a fixed time (i.e l/min/100m). This figure is more understandable for the people involved in the project, and it can be verified more directly by measurement. The leakage into the tunnel can also be stipulated as a contract requirement.

As a practical rule of thumb, given ground water pressures of about 10-20 bar and medium rock conditions, the leakage into a tunnel is expected to be within the following ranges when grouting with:

- Rapid cement (100-140 m): 20-50 litres pr 100 meter and minute.
- Micro cement: (<30μ): 10-20 litres pr 100 meter and minute.
- Ultrafine cement: (<15μ): 2-5 litres pr 100 meter and minute.

(refer the Romeriksporten, Storhaug, Baneheia tunnels):

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11 THE MERÅKER PROJECT - 10 KM OF TUNNEL IN 12 MONTHS

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ABSTRACT: New powerplants, tunnels and dams have been built at Meråker in Central Norway. A total of 44 km of tunnels with cross sections varying from 7 m^2 to 32 m^2 have been excavated in hard rock formations. Ten kilometres were excavated by a High Performance Tunnel Boring Machine (HP TBM). This paper gives special attention to the TBM drive and equipment selection, including planning, site organisation and performance.

1 INTRODUCTION

Meråker is located 80 km east of Trondheim. Five small hydropower stations were built in the years 1890 to 1915 to provide electric power to the Meraaker Smelter. In 1987 it was decided to upgrade and replace four of the generating facilities, increasing the annual output from 200 GWh to 590 GWh.

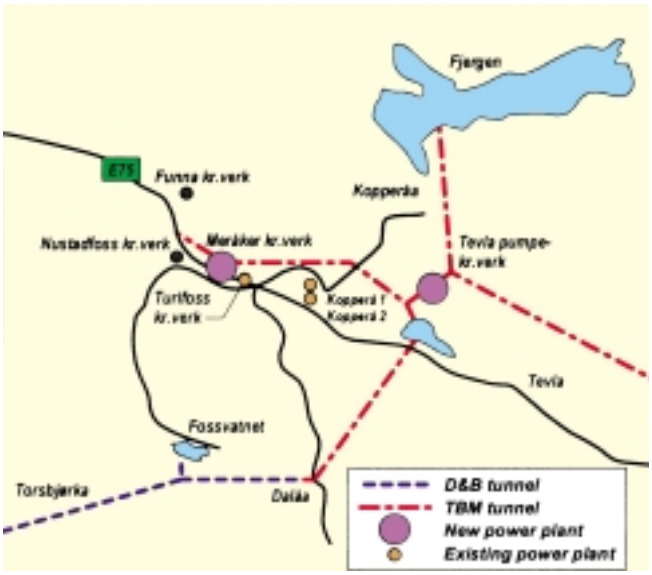

Fig. 1 Overview of the Meråker Project

2 TIME SCHEDULE

The construction started in September 1990, and the power plants, tunnels and dams were commissioned 1994. By the end of 1992, all 44 km of tunnels had been completed, approximately nine months ahead of schedule.

3 CONTRACTS

The client, Nord-Trøndelag Energy, invited bidding, and the civil works were divided into two contracts: The Tevla Power Station contract including 17 km of tunnels and two rock fill dams, was signed with the VSF-Group, a joint venture consisting of A/S Veidekke and Selmer A/S.

The Meråker Power Station contract with 27 km of tunnels, was awarded to Merkraft, a joint venture consisting of Eeg-Henriksen Anlegg A/S and A/S Veidekke.

4 GEOLOGY OF THE TBM DRIVE

The rock types expected in the project area consisted mainly of Cambrian and Ordovician metamorphic sediments with metagabbro intrusions. The metagabbro was considered extremely hard and massive with unconfined compressive strength up to 300 MPa.

Six different rock types were anticipated along the tunnel.

Fig. 2 Geological Profile.

To obtain the best possible basis for the tender, Merkraft supplemented available geological information with its own comprehensive survey before and during the tender period. The survey was carried out in cooperation with the Norwegian University of Science and Technology (NTNU) and the Norwegian State Power Board (Statkraft).

The tests from the 10 km Torsbjørka - Dalåa indicated that the rock parameters varied from a relatively soft phyllite with a Drilling Rate Index (DRI) of 60 and Fissure Class II+ or more, to a very hard metagabbro with DRI=32 and Fissure Class 0+; according to the NTNU Classification System of Report 1-88.

The formations of graywacke and sandstone appeared as mixed face conditions. Hence, the high rock strength and the wide variation in boreability characteristics became important issues in the selection criteria for a TBM.

5 SELECTING THE TBM

During the 1980's several major projects in Norway were completed in hard massive gneiss and granite by the TBM method. These were all excavated using state of the art machines, designed with capacities up to 222 kN per cutter. Though most projects were successful, technically and financially, actual field tests and studies showed that the projects could have been done even faster.

Experience showed that the machines including hydraulics, electrics, main bearing, cutters and cutter housings could not sustain the high thrust levels needed in the massive, abrasive and hard rock types. It became obvious that a substantial ROP gain would be realised if machines and cutters were designed to take higher thrust.

In the softer rock formations several machines also showed torque limitations at a rate of penetration of about 5 m/h.

In 1989, Statkraft decided to put into operation a new generation machines for the Svartisen hydropower project. During the period 1989-1992, three Robbins HP machines were purchased for boring more than 30 km of tunnels, with diameters of 4.3, 5.0 and 3.5 m respectively.

With the above mentioned experience in mind, the contractor specified that the TBM should:

- Be able to efficiently bore through hard massive metagabbro, greenstone, graywacke, mixed face conditions as well as the softer phyllite.
- Be able to increase the diameter to 4.2 m without changing the basic HP concept.

These requirements lead to a Robbins HP TBM, designed to bore with an average cutter load of up to 312 kN per cutter using 483 mm cutters.
Close co-operation between Statkraft and The Robbins Company lead to further development of the HP concept and ideas, eventually to the final design and manufacture of Model 1215-265.
Brief Machine specifications were as follows:

- Maximum recommended cutterhead load: 7,900 kN
- Cutterhead power: 1,340 kW (335 kW x 4)
- Cutterhead RPM: 13.4
- Number of cutters: 25 single discs
- Machine weight: approximately 200 tons

6 MACHINE DEVELOPMENT CHANGES

The experience at Svartisen with the 3.5 m diameter HP TBM 1215-257, indicated unusually high cutter wear and cutter bearing seizures in the transition area of the cutterhead when the average cutter load exceeded 265 kN per cutter. This clearly limited the practical and effective use of the machine at full thrust, and meant extra costs and downtime for cutter changing. The cutter profile and cutter spacing for the 1215-265 were therefore revised and changed according to Fig. 3, to provide a better load distribution and a closer spacing in the critical zone.

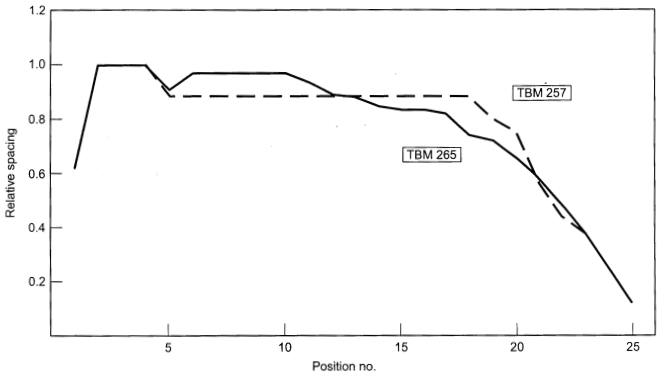

Fig. 3 Spacing 1215-257 and 1215-265

The modification turned out very satisfactory, and made it possible for the first time to bore with a sustained load of 312 kN per cutter.

The Svartisen Project and the Hong Kong Cable Tunnel Project experiences were transferred, and lead to additional modifications.

7 BORING PERFORMANCE

Fig. 4 shows that the make 111-265 bored 10 meters per hour and more during certain periods. The torque did not cause any problems and the muck handling system performed well.

ROP turned out to exceed the contractors scheduling prognoses. From the figure 5 one will also see that the ROP out-performed the NTNU 1-88 prognoses.

During the first four-week period of boring more than 1,000 meters tunnel had been achieved. The entire tunnel was finished six months ahead of schedule.

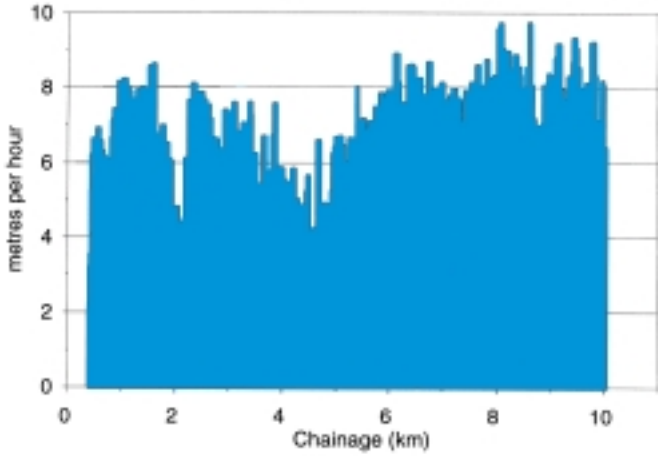

Fig. 4 Actual ROP over the Tunnel Length

7.1 Advance Rates and Utilisation

Norwegian regulations allowed Merkraft to operate 100 shift hours per week only. In spite of this limitation, the actual advance rates at Meråker averaged 253 meters per week, or almost 100 meters more per work week than ever before achieved in Norway. During the first four-week period of boring more than 1,000 meters tunnel had been achieved. The entire tunnel was finished six months ahead of schedule.

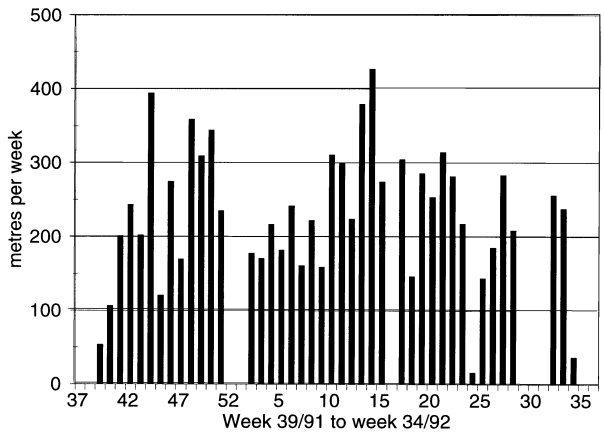

Fig. 5 Weekly Advance Rate

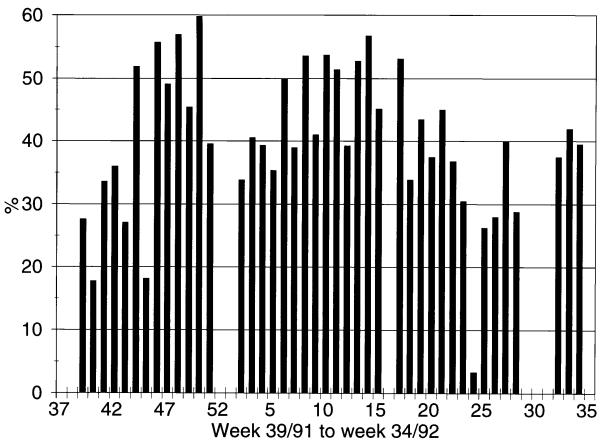

Fig. 6 Machine Utilisation

The TBM utilisation rate, defined herein as actual boring hours in percent of available shift hours, are shown in Fig. 6. The reduced utilisation towards the end was due to in-sufficient muck transport capacity over the last four km (two muck trains and one locomotive).

Increased average ROP and thus reduced construction time had been well possible if one additional locomotive and one California switch had been made available for the phyllite section towards the end of the tunnel.

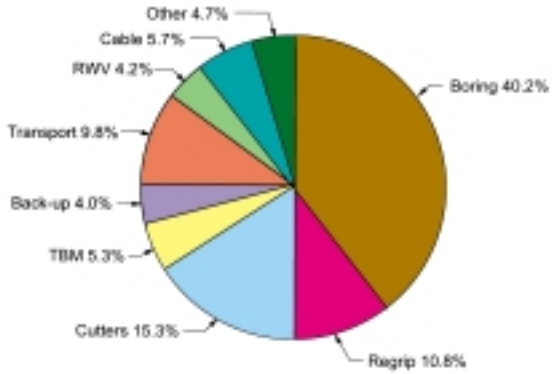

Fig. 7 Average Machine Utilisation

The results from Meråker can be summarised as follows:

- Best ROP over one single shift: 9.54 m/h
- Best shift (10 hours): 69.1 m
- Best day (2 x 10 hours shift): 100.3 m
- Best week (100 shift hours): 426.8 m
- Best month (430 shift hours): 1,358.0 m
- Average ROP: 6.4 m/h
- Average weekly advance rate: 253.0 m

8 CUTTER WEAR

The greenstone turned out to be the most difficult to bore. It caused the greatest amount of cutter wear and downtime for changing cutters. The variations of the mixed face conditions were also major factors for the high cutter consumption.

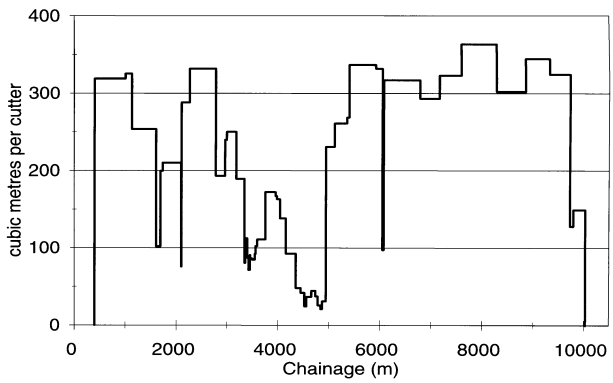

Fig. 8 Cutter Life

The great variation in cutter life was surprising. The TBM never had a ROP lower than 4 meters per hour. The cutter life varied from about 300 m^3 to 30 m^3 per cutter ring.

The overall downtime due to cutter changes was kept on a reasonably low level. This result was partly due to the improved load distribution made possible by the modified cutter profile and spacing. The modifications also meant changing more cutters in series, hence saving time.

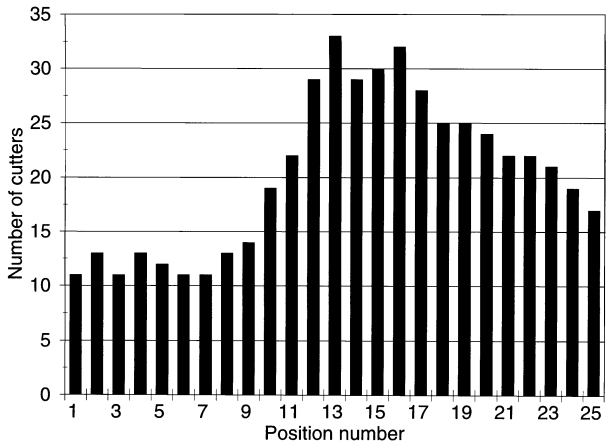

Fig. 9 Cutter Changes

9 MUCK TRANSPORT

The transport arrangement from the heading to the silo unloading station consisted of two muck trains, each with nine 10 m^3 bottom dump mine cars. The two trains were moved in shuttle service, maintaining an average speed of 25 km/h turn-return to the heading sloping 0.2-0.8% uphill. Each train load had a capacity of three boring strokes, equal to 4.5 m tunnel production. A rail bound back-up with towed California switch ensured a high production system.

The mine cars dumped the muck into a 300 m^3 silo, placed within the mountain. A subcontractor with commercial type trucks transported the muck up the 1:8 decline access tunnel and to the disposal area approximately 500 m outside the portal.

Due to high penetration rates of more than 10 m/h, the tunnel muck haulage itself lacked adequate transport capacity on the last 4 km of the 10 km long drive. However, the cost to install a bypass switch, an extra locomotive and extra personnel was estimated to be higher than the actual cost of waiting. The weekly advance rates were anyway far ahead of schedule, and no special bonus was offered by the client for early completion.

10 ASSEMBLY/DISASSEMBLY

The 265 TBM was shipped from Seattle to Norway in components not exceeding 87 tons. The shipment arrived at the site in August 1991 and the TBM was assembled underground. Three and a half weeks later the TBM started boring. Detail planning, experience and staff training, including extensive training for key personnel at the manufacturer's plant, made such short assembly period possible.

Once the tunnel was completed, the cutterhead was disassembled and removed through an existing 100 m long intake tunnel. The rest of the machine and back-up travelled out on the tunnel rails.

11 SITE ORGANISATION AND STAFF

Norway has long been recognised for its cost efficient tunnelling. Some of the main reasons may be the low number of staff, crew flexibility and capability, and the use of modern and well maintained equipment. At Meråker, 16 men covering three shifts were employed, each working the regular 33.6 hours per work week.

This crew covered all operations including boring, rock support installation, mucking, work shop and cutter repairs. Small crews should only be considered when experienced, flexible, dedicated hands are available. Fair pay and bonuses do the rest.

The crew at the face worked on a rotation system work face to improve teamwork. One operator controlled the TBM and the filling of trains from the cabin mounted on the back-up. One mechanic, one electrician and one locomotive driver handled all the other duties.

The crew was paid based on actual production. This meant that the machine had to be properly maintained and repaired to prevent downtime.

The TBM site management included five persons. These also supervised the 5 km long 20 m^2 Drill & Blast tunnel and the tunnel intake construction.

12 ROCK SUPPORT

The contractor's site investigation during the tendering period indicated only a minor need for rock support when using the TBM method. The amount actually needed ended up even less. Over the 10 km length, only 140 bolts and 44 m^3 of shotcrete were used for support. The client's original estimate included 900 bolts, 300 m^3 of shotcrete and 200 m^2 of liner plates for the TBM tunnel. On the Drill & Blast tunnels, the bid documents turned out to be nearly accurate with an average

of 250 bolts and 33 m^3 shotcrete per km.

The average production rates for the Drill & Blast tunnels were approximately 80 meters per week per tunnel heading.

Tunnelling in Norway by tradition utilises the bearing capacity of the rock itself, using support only as and when needed. The smooth excavation provided by the TBM, the small cross section and the rock quality were major reasons for the small amount of rock support required at Meråker.

13 SUMMARY

The Meråker Project has demonstrated a new standard in tunnel production. The high advance rates and reduced need for ventilation give possibilities for longer tunnel drives, reduction of overall construction time and cost savings.

The production planning, the choice of machine and back-up, and the experienced and dedicated staff were the main ingredients to successful hard rock tunnel boring performance.

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12 DETERMINATION AND CO-OPERATION IS CRUCIAL FOR ROCK MASS GROUTING IN ORDER TO SATISFY STRICT ENVIRONMENTAL REQUIREMENTS

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O. T. Blindheim AS
Svein Skeide
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ABSTRACT: This paper describes the successful efforts to make the eastern land section of the Oslofjord subsea tunnel sufficiently watertight. Strict requirements had to be satisfied to protect the lakes and connected wetlands in the recreational forest area close to the community of Drøbak, and to prevent settlement damage to houses in the township itself. The efforts included definition of allowable seepage and requirements for probing and pre-grouting, and follow-up measures such as sectional measurements of seepage. For this process, which is basically managed by the tunnel owner, it was found necessary to enter into a constructive discussion with the contractor on the detailed methods to optimise performance without sacrificing quality. During this co-operation, framework procedures were agreed and authority to adapt to the varying rock conditions was delegated to the face staff from both parties. The resulting tightness met the requirements, no damage to the environment occurred, and cost and time impacts were minimal.

1 INTRODUCTION

1.1 Project description

The Oslofjord tunnel is a subsea road tunnel with 7.2km total length, which constitutes a key element in the Oslofjord Connection, a project which by 26.5km of roads, bridges and tunnels connects the eastern and western side of the Oslo fjord about 50km south of Oslo. Different aspects of the project have been described in several papers, see e.g. Haug & Øvstedal (1999) and Øvstedal (2001) for general description and Blindheim & Backer (1999) about the freezing of a section in soil under 120m water pressure. The tunnel was completed on schedule in 2000 by the contractor Scandinavian Rock Group for the owner Norwegian Public Roads Administration.

The present paper addresses primarily the land section of 2.8km length on the eastern side of the fjord. This section starts at a portal in the farming and forest area surrounding the small township of Drøbak. The tunnel alignment passes below a recreational forest area close to idyllic small lakes (old water reservoirs), and further under the centre of the township. Here, some of the old wooden houses were founded partly on timber fleets in a poorly refilled ice-cutting dam above a fractured zone in the bedrock.

Fig. 1. Cut-out map of the eastern land section of the Oslofjord tunnel (yellow line), passing under a forest area with small lakes and the township of Drøbak.

The land section of the tunnel is an environmental measure in itself, as it allows the through-traffic to bypass the township. It would become an unacceptable paradox if the tunnel should be allowed to drain the ground water and thereby cause damage to the environment. Other aspects of the environmental protection and landscaping, which had a high priority on the project, are beyond the scope of this paper.

1.2 Site investigations

During the period of planning and design, the site investigations concentrated on the subsea section, where the most serious risks are found. This is covered elsewhere (Aagaard & Blindheim, 1999). However, engineering geological pre-investigations based on field inspections were performed for the land sections and aimed at identifying the more important problems for the tunnelling. This included stability problems and need for rock support, but addressed also the risk of water inflow to the tunnel and need for ground treatment under residential areas. In the traditional manner, these aspects were planned to be dealt with during tunnelling. No detailed risk or special environmental assessment for water seepage was done ahead of construction, as has later become practise.

It was known from experience with water supply wells in the area that the geological formation could show a large variation in permeability. The main rock type is Precambrian gneiss with bands of darker and lighter minerals, and frequent pegmatite intrusives. It was expected that both small and spread out seepage could occur, and that some of the joint sets, opened by tensional tectonic movements in the vicinity of the Oslo graben, could cause large localised inflows. Naturally, for a subsea tunnel the general preparedness for large water inflow was high.

2 TIGHTNESS CRITERIA

2.1 Requirements in the contract

The tunnel contract was a normal unit price contract according to the standard format of the Norwegian Public Roads Administration. In these contracts the selection of criteria for water tightness and when to perform grouting is decided by the owner, to some extent in consultation with the contractor, to adapt to the situation and experience gained during tunnelling. The contractor gets paid for the actually performed works of drilling probe and grout holes by the metres of holes, by consumed amount of the different type of grouting material, and for standstill of tunnelling equipment during grouting, according to the respective tendered prices. The contract included the normal patterns for probe drilling and grout hole patterns, which have been used successfully for more than 20 years, as reported in several Norwegian papers.

The only requirement expressed by figures in the contract was the target of allowed final seepage, as normal set to 300 litres/min/km in total (corresponding to 30 litres/min/100m). This relates mostly to the dimensioning of the pumping facilities and has no relevance for the possible environmental demands of e.g. sensitive land sections of the tunnel.

At the beginning of the construction period, the detailed requirements to the pre-grouting criteria were defined by the owner, after advice by his geological engineering consultant O. T. Blindheim AS. The basic criteria was that a seepage out of one probe hole (or other drill holes in the face) of 3 litre/min would trigger pre-grouting. This applied to all land sections, including below the township. This was considered a strict requirement, based on experience from tunnelling through similar geological formations and environments, justified by the need to avoid draining of the small lakes in the area and damage to residential and other properties. It involved an assessment of the geological and hydrogeological conditions, based on normal engineering geological practise and experience.

For the subsea sections 6 litres/min was chosen as a trigger level. The criteria also included directions about the number of control holes to be applied after grouting etc.

2.2 Environmental assessment

At the time, it was not yet a requirement for special environmental assessment for risk of water seepage in the quality assurance manuals of the Public Roads Administration. However, the owner's project management realised the need for a closer assessment of the sensitivity to effects of tunnelling below the attractive recreational forest area. This hydrogeological assessment was performed by Jordforsk (a specialist hydrological and hydrogeological advisory company) during the early stage of the tunnelling.

The assessment confirmed that the small lakes (or rather old water reservoirs left unregulated), and the adjacent peat areas, were likely underlain by clay-rich marine deposits on top of the bedrock, thus reducing the infiltration to the bedrock. The risk of rapid drainage by tunnelling below could still be significant, as the catchment areas were small. A water seepage into the tunnel of 20 litres/min/100m was evaluated as acceptable for the natural environment, provided there were no large concentrated seepage. It was recommended to reduce the seepage to 10 litres/min/100m in view of the interest in the area for recreational use.

The effect of 'sealing' by the marine deposits on top of the bedrock could not be assured without extensive additional site investigations, which would partly defeat the purpose of leaving the forest area in peace. It was decided not to thrust this effect, and to act as if close connections could exist between the joint water in the bedrock and the water regime above.

Based on these assessments, it was believed that the already chosen grouting criteria would suffice. It would be necessary to ensure a consistent performance of the pre-grouting, i.e. avoid remaining concentrated seepage, and not only to follow up, but also to document the results.

3 FIRST RESULTS

3.1 Increased probing needed

On the first section from the portal only minor seepage was encountered. On a few locations, small seepage identified by probe holes or blast holes triggered pre-grouting. After the tunnel had reached 1 km in, the measured seepage was about 20 litres/min/100m on the average, however, most of this came from one short section. After one concentrated seepage had been passed without being detected by the probe drilling, it became clear that the result of the grouting needed to be improved before the more sensitive areas were approached. It was decided to step up the probe drilling, which had been performed according to 'Level 1' consisting of 2 holes of 30m length. The number of holes were increased to 'Level 2' with 4 holes of 30m and the direction of the holes was adapted to better intersect the joint sets that appeared to carry most of the seepage. Four control holes were drilled after each grouting round to check the result.

3.2 The initial grouting

The grouting was initially performed with standard industrial cement ('Rapid RP-38'), and in some cases with fast setting cement (Rescon's 'Cemsil') at locations with larger seepage. In some locations, where both open and narrow joints occurred, micro-cement (Rescon's 'Microcem 650' from Blue Circle) was also taken into use.

The pattern of grout holes was initially 23 holes of 20m length, which were drilled with the blast-hole drilling jumbo, as the probe holes. This pattern was increased to 28 grout holes to reduce the spacing, and a few holes were added in the face. The length of the holes were increased to 23m to get minimum 8m overlap for 3 blasting rounds of 5m between grouting rounds.

In some sections 3-4 overlapping grouting rounds were necessary, mostly 2 had been sufficient. The pumping pressure was set at 45 bar initially, and increased to 60 bar after 1km. Occasionally pressure up to 70-80 bar was applied. The stop pressure was held at 30-45 bar, where achievable.

The rock mass conditions varied from one round to the next. The different joint sets were more or less present with a varying persistence. The pegmatitic intrusions were more jointed, but with irregular discontinuities. Some joints were rather tight or clay filled, and difficult to grout. Thus, the contractor wanted to try a finer micro-cement. As such cement was not priced in the contract, this was accepted by the owner on a trial basis at first. The results were promising, and Microcem 900 or A12 Spinor were taken into use according to availability.

After the first kilometre the grouted length of the tunnel was 198m, i.e. 19% of the section length of 1058m. The total length of grouting rounds was 467m, giving a degree of overlap of 467/198 = 2.4. The total material consumption was 222 ton, which gives an average of 1120kg/m on the grouted length and 210kg/m on the total section length. The latter figure was slightly less than the overall average in the Bill of Quantities, however, most if this ad been expected to be used on the subsea section.

3.3 Need for improvement

As planned, a complete re-evaluation of the situation was performed by the geological engineering consultant after the first kilometre of tunnelling. This included an assessment of the assumptions, the experienced rock mass conditions, the characteristics of the jointing, the results from the grouting, remaining seepage, potential impact on the surroundings etc.

It was clear that in spite of the improvements achieved so far, it was necessary to improve further both the resulting tightness and the efficiency of the process. A strategy to reduce the remaining water seepage was developed. The technical measures included improved probing (6-8 holes for each 3rd blasting round), increased number of grout holes to reduce the distance between grout holes down to 1m at the inner end, reduced length of grout holes to reduce deviation, use of high pumping pressure (60 bar with minimum 30 bar stop pressure), types of cement, selection criteria, etc. The need for a flexible adaptation to the varying conditions was emphasised. This required further efforts as will be described below.

4 NEED FOR CO-OPERATION

4.1 Decisions on principles

After these initial experiences, the owner realised that it was not going to be easy to satisfy the tightness requirement. Due to the sensitivity of the subject, a high degree

of assurance to avoid damage to the environment was wanted.

On most of the first kilometre, the grouting was ordered by the owner without detailed discussion with the contractor. As the effort was stepped up, both on probe drilling and on the grouting, several work meetings took place between the representatives of the owner and the contractor, including also the geological engineering consultant. This helped the systematic trying out of different types of cement, the criteria for their selection, and details of the stop criteria.

It was however obvious that the total effort of grouting could potentially impact the time schedule and cost significantly. Based on the re-evaluation mentioned above, the owner's project management made the following crucial decisions:

The criteria for triggering of grouting should not be compromised in any case;
- The use of micro-cements outside the contract would be allowed, after selection criteria and a negotiated price had been agreed;
- The performance should be optimised by co-operation with the contractor, drawing on his experience on practical performance.

The determination to achieve the target tightness whatever it took, but to minimise the time and cost impact by constructive co-operation, was conveyed in both organisations.

4.2 Co-operation on performance

This higher level of co-operation than demanded in the contract resulted in the agreement of a set of 'framework' grouting procedures. These aimed at adaptation to the varying conditions to achieve satisfaction of the requirements and at the same time allowing efficient production.

In detail, the target was to minimise the number of overlapping grouting rounds, i.e. to reach the satisfactory result faster. It was allowed to use cement outside the types priced in the contract, according to specified selection criteria (e.g. for holes with more seepage than 25 litre/min grouting would start with industrial cement, for less than 25 litre/min with micro-cement grade 650, and for less seepage than 10 litre/min, micro-cement grade 900 could be used directly). Guidelines were laid out for change of material during grouting, stop criteria with respect to pressure, supplementary holes etc.

Most important, it was agreed that the procedures could be adapted according to the conditions, within the agreed framework, after the judgement of the owners shift controllers (most of them had geological engineering training) and by the contractor's face crew during execution. It must be realised that the typical Scandinavian face crew consists of experienced and multi-skilled tunnel workers. This decision delegated authority to adapt the procedures to the staff from both parties that were closest to the performance of the works.

5 FINAL RESULTS

5.1 Adaptation to varying conditions

The detailed performance of the subsequent work provides a wealth of detailed experience, which is worth a detailed analysis, but which is beyond the scope of this paper. However, it can be mentioned that the target of satisfying the tightness requirement with fewer overlapping grouting rounds was achieved, although the total consumption per grouted metre tunnel remained about the same, as will be shown below.

5.2 Tunnelling under Drøbak township

Most houses in Drøbak are founded on rock, moraine or not so sensitive marine deposits. However, in the centre of the township, an old ice-cutting dam had been poorly filled in, and houses in this area are founded wholly or partly on timber fleets, and would be extremely sensitive to any lowering of the water pressure or the ground water level. Four piezometres were installed around the area, and preparations made for 1-2 water infiltration wells. The tunnelling passed just below this area, at the same time crossing a fault zone. The pre-grouting was done according to the developed detailed procedures, and no effect of the tunnelling was detected, neither on the pore pressure, nor on the elevation of the houses.

5.3 Probing methods and results

The systematic use of 6-8 probe holes ahead of the face proved to be sufficient to identify the potential seepage joints. The length of 23 or 30m was chosen to best fit the sequence of blast rounds, ensuring a minimum overlap of 8m. Half of the probe holes were typically set in the middle of the face, aiming at an angle of 30-45° to the tunnel axis in order to cover the complete front of the grout hole 'cone'. The others were collared in a normal manner pointing 15° outwards from the tunnel contour.

Although this level of probe drilling gives 2-3 times more probe holes than normal, it does not represent a significant cost or delay to the tunnelling when it becomes a routine part of the work cycle. The cost of the probe drilling on the east land section came to approx. 1 million NOK (9 NOK ~ 1 USD) or about 0.25% of the total cost of the complete tunnel, or less than 1% for the land section only. This can be seen as an inexpensive insurance against not finding and passing through too large seepage, which would result in time-consuming and very expensive post-grouting, and delays to other works.

The trigger level of 3 litre/min was kept throughout and proved sufficient. The same applied to the selection criteria for the grouting materials, depending on the level of water seepage out of the probe holes.

5.4 Pre-grouting methods and results

The total consumption of cementitious grouting materials on the complete land section on the east side of the fjord of 2817m was 667 tons. This constitutes an average of 237kg/m tunnel, which is 10% more than the total average for the whole tunnel in the Bill of Quantities of 215kg/m. Distributed on the grouted section of 687m only, which constitutes 25% of the total land section, the consumption is 970kg/m grouted tunnel. Accordingly, the improved procedures had reduced the consumption per meter grouted tunnel compared to the first kilometre. The need for repeated grouting rounds also decreased, reducing the time impact significantly.

See Table 1, which gives an overview of the different grouting materials used on the complete eastern land section. Note that the ratio between normal and coarse cements to the micro-cements is about 60/40.

5.5 Post-grouting methods and results

On the section where a few joints with concentrated seepage had been passed without being detected by the probe drilling, and for 15-20 rock bolts that had penetrated the grouted zone around the tunnel contour, a limited post-grouting program was performed. Although the overall seepage on the section with localised seepage satisfied the average requirement of 20 litre/min

Grouting material	**Consumption, kg**
Industrial cement (Rapid)	385 710
Fast setting cement (Cemsil)	9 925
Coarse filter cement (Mauring)	100
Micro-cement, grade 650	124 670
Micro-cement, grade 900	126 775
Micro-cement, A12	12,180
Micro-silica (GroutAid)	7,143
Total cement/silica materials	**666 503**
Polyurethane (Taccs)	1390 litres

Table 1 Consumption of grouting materials for the eastern land section of the Oslofjord tunnel.

100m, it was considered worthwhile to try to treat each of the seepage points, especially since most of the local seepage was coming from either one joint or a bolt hole.

The post-grouting was done with a separate small crew without disturbance to the tunnelling using light drilling equipment and polyurethane, which was available in the contract. The result was satisfactory as many of the larger concentrated seepage points were stopped or reduced significantly. Some were only partly reduced, or more often, spread out or just 'chased around'. As the use of polyurethane material became questioned for health reasons, this small post-grouting scheme was discontinued. However, at this point, the environmental criteria were satisfied, and no further post-grouting was required.

It is worth mentioning that not at any stage was post-grouting considered as an equal alternative to pre-grouting. The use of systematic post-grouting drill patterns was never considered necessary, due to the determined emphasis on pre-grouting. The minor post-grouting program that was performed must be considered as a 'fine-tuning' with a minor economic impact, decided by the owner's project management as a measure to be on the safe side.

5.6 Resulting tightness

A satisfactory water tightness was achieved on the whole land section. Most sub-sections had no or spread out minor dripping only. At a few locations water were running rather than dripping, but in so small amounts that it did not constitute a threat to the environmental criteria.

The total average remaining seepage was estimated, based on the measurements for the different subsections, to 9 litres/min/100m, which reduced to 8.4 litres/min/100m at the end of the construction period. This was not only below Jordforsk's recommended requirement to avoid damage to the environment of 20 litres/min/100m, but also below their 'idealistic target' of 10 litres/min/100m.

In detail, on one section of 200m only the remaining seepage was 35 litres/min/100m. However, pore pressure measurements confirmed that this did not influence the ground water level. Measurements of pore pressure performed from holes drilled upwards from the tunnel in the area adjacent to the lakes confirmed that the ground water level below rocky outcrops remained less than approx. 5 m below ground level, unchanged or increased since the first measurements when the tunnelling reached the area.

The seepage on the section below the township itself, with vulnerable houses with poor foundations in the infilled ice-cutting dam, was well below the criteria, down to approx. 2-3 litres/min/100m. However, such low levels are needed to avoid settlement damage. In this area, piezometres installed from the surface detected a normal variation in ground water level of within 0.5m, and no variation related to the tunnelling passing below. Settlement checking of the house did also not detect any effect of the tunnelling during the period of construction until 2 years after the passing.

During the whole construction period, water level measurements confirmed that the tunnelling did not affect the small lakes in the area. These measurements actually became one of the rather pleasant tasks of the site staff, as the walk through the forest to the undisturbed lakes gave a reward in itself, as Figure 2 illustrates.

Fig. 2 Idyllic Øvredam loved by everybody living in the area, and saved by the successful water control scheme during tunnelling below.

To avoid water dripping on the asphalt driveway, normal water shielding was done by polyethylene foam (fire-protected by sprayed concrete), according to normal procedures, see illustration in this publication (Blindheim & Øvstedal, 2001).

5.7 Time and cost impact

Compared to the contractor's initial time schedule, the tunnelling through the 25% part of the land section that needed extensive grouting caused a delay of approx. four weeks. This was later recovered, also because the subsea tunnelling from two faces benefited from the improved procedures. Accordingly, the break-through on the subsea section was on schedule.

The cost of the grouting measures (including probe drilling) on the east land section came close to 6 million NOK (9 NOK ~ 1 USD), which constitutes approx. 1.5% of the cost of the completed 7.2km tunnel. As some of this cost anyway was expected passing the built up areas, the final cost increase became insignificant.

6 THE REST OF THE OSLOFJORD CONNECTION

6.1 Grouting of the other sections

Much less grouting was necessary on the land section on the west side of the fjord, giving an average consumption of only 74kg/m.

The total grouting on the whole tunnel of 7.2km averaged 360kg/m, almost 30% of which was used in a short section in an eroded fault zone with no rock cover, that was extensively grouted as preparation for freezing (Blindheim & Backer, 1999).

For the five shorter tunnels under land on both sides of the Oslo fjord, no grouting was performed.

6.2 Resulting tightness

The total remaining seepage on the complete subsea section was at completion 26 litre/min◊100m.

Including the subsea section and the land sections at both sides, the total seepage for the 7.2km tunnel was 22 litres/min/100m, which is well below the target of 30 litres/min/100m.

For convenience and safety of the drivers, frost insulated drip protection (polyethylene foam with sprayed concrete as fire protection) was installed in all sections with remaining seepage. This amounted to 18 m2/m tunnel in total, corresponding to approx. 80% of the tunnel contour.

7 CONCLUSIONS

From the experience with the water tightness control for the eastern land section of the Oslofjord tunnel it can be concluded:

- It is possible to achieve the degree of tightness as required to protect the environment with respect to drainage of ground water, from the joint water in the bedrock, pore water in soil deposits and surface water in lakes above.
- Although in this case the target was achieved without detailed analysis ahead of construction, the environmental assessment and establishment of specified criteria shall preferably be done before tunnelling starts. A complete system of follow-up, flexible procedures and measurement of resulting tightness must be implemented. The evaluation of all factors and results by experienced specialists in geological engineering and the timely decisions by the project management are crucial for the result.
- It is vital that the owner's project management in a decisive manner secures the constructive co-operation of all involved. Besides the normal support from his specialist consultants and follow-up staff, procedures must be established and implemented that draw on the experience of the contractor, including his foremen, shift bosses and face crew.
- If these basic measures are neglected, even the use of advanced grouting material technology can not secure satisfactory results.
- The emphasis has to be on pre-grouting, if reasonable targets with respect to time and cost shall be achieved. Post-grouting must not be allowed to become a substitute for pre-grouting.

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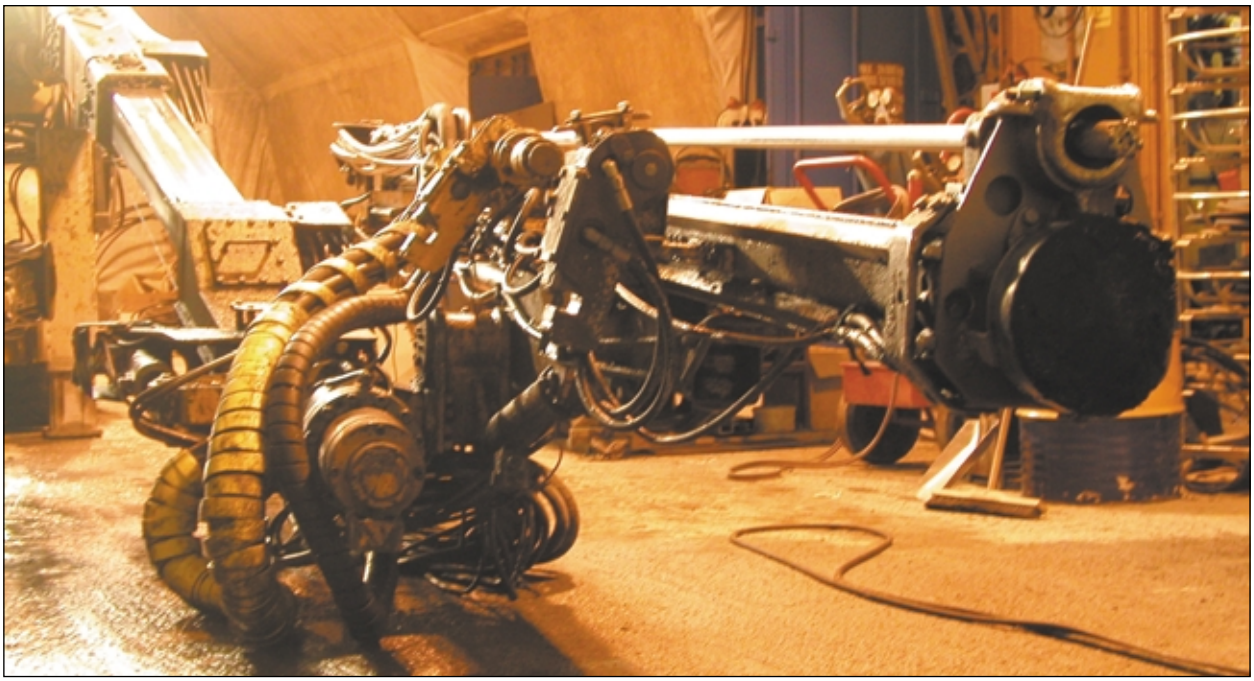

Figure 3. Heavy duty drilling equipment.

13 WATER CONTROL – REASONABLE SHARING OF RISK

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ABSTRACT: A fair and just contract between the Owner and Contractor is essential when dealing with water control in rock tunnelling. The Norwegian tunnelling contract system implies that the owner is the risk-taker for the amount of grouting needed to meet water control requirements. The contractor's risk is limited to the execution of the work in the best and most effective way, hence the unit price contract system is dominating the water control work.

When the quality of the product is of importance, i.e. minimising the impact of tunnelling on the surroundings, the owner should take the risk of time used to have the grouting made to his satisfaction. The finishing date in the contract should be regulated for used grouting hours compared to a suggested number of grouting hours in the tender. This matter is currently subject to discussion whether it may be an even better solution for the owner to pay per hour used for grouting and in addition pay materials per kilo when the surroundings are of great importance.

1. THE NORWEGIAN TUNNELING CONTRACT SYSTEM

Norwegian Tunnelling contract philosophy is based on a simple principle of survival: "expect the unexpected". When forming a contract for a tunnel or cavern project, there must be allowance for eventualities. Typically in Norway specifications and unit prices covering the entire range of possible rock conditions and configurations are included in the contract and flexibility is cultivated. Woe betide the contractor who installs an unsuitable and dangerous supporting structure simply because "that's what the contract specifies".

The Achilles heel of any excavation project is not the drilling and blasting, or boring operation, but the support work needed to stabilise and secure the physical void left in its wake. This is not only crucial for the safety of the entire installation upon completion, but also for the safety of the construction workers during construction. The purpose of all support work is to re-establish the stability of the rock as quickly, effectively and cheaply as possible. This is not something that can be completely planned on paper, years in advance, sitting in a comfortable office suite with a beautiful view. Decisions have to be made on the spot, kilometres into the mountainside or scores of meters below the ocean floor, with crumbling, leaking rock masses overhead. The contract must be flexible enough to apply to almost every eventuality or occurrence of changing geological conditions.

For more than 20 years, the contract system has proven that the use of a flexibility results in lower total costs, greater safety in tunnel construction and reduction of disputes, arbitration or litigation.

The Norwegian Tunnelling Contract System includes the following main elements for a typical tender document:

1. Tender papers include geological and geotechnical reports conducted by experienced geological engineers, showing all investigations and surveys that have been conducted in connection with the project. These give the contractor a general idea of the rock conditions and specify all the facts and assumptions.

2. Based on these reports, the tender papers list the types and amounts of rock support and grouting to be included in the tender sum, the Bill of Quantity representing the expected level of such work. The tender papers also include alternative unit prices for work, such as driving pilot headings, probe-drilling, grouting, etc.

   The various types of support and grouting works and their requirements and conditions are clearly described in the tender documents.

   The support and grouting work is divided into two main categories: The work executed at the tunnel face (which delays the progress of the tunnelling) and behind the tunnel face (which does not delay the progress of the tunnelling). The tender also defines the types and amounts of rigging and equipment required for support work (steel form, shotcrete and injection equipment, special bolts, etc.) that the contractor must have available at the site, on a stand-by basis. Tender prices are specified for all these special items.

3. Most importantly, the tender documents must include a number of different measures that shall be deliverable during construction. This condition allows the most suitable grouting technique to be applicable at any time dealing with different situations at the tunnel face.

4. Delays or savings in total construction time due to increased or decreased amounts of support work is dealt with in the following way:

   In the tender, the contractor must specify "the equivalent time" for each type of support work. These are only used as a key for extending or shortening the construction time. Theses figures make it easy to calculate the consequences of changes in types and amounts of support and grouting work at a later date.

An important requirement of the Norwegian Tunnelling Contract System is that both the client and the contractor must have skilled and experienced representatives present at the site. When problems arise at the tunnel face, the parties must be able to inspect and evaluate the situation together and find the best solution without delay and dispute. The contractor who has correctly priced various support alternatives will be able to advise without being concerned about "ulterior motives". The contract already specifies prices for such work. The representatives at the site or a consultant must have the expertise and the authority to decide on the best alternative, so that the client pays for the solution that is technically the correct one.

The idea behind the Norwegian Tunnelling Contract System is to financially account for all possible eventualities at the tender stage. Thus, all parties can work as one team during the construction phase to effectuate the plans and overcome difficulties quickly and efficiently.

2. THE PRINCIPLES OF RISK SHARING

The principles used for sharing the risk in Norwegian tunnelling contracts were introduced in the 1970's. Earlier contract practices did not provide for enough flexibility with regard to the sharing of risk and responsibility between owner and contractor when ground conditions became adverse and penalty dates where being overrun. Still more time was lost during the discussions that followed, and in some instances the parties had to go to court.

In a paper by Kleivan (1988) the most significant arguments for the introduction of a risk sharing principle in tunnel contracts were presented. According to Kleivan these were:

- The ever present uncertainty of what the actual ground conditions are: Field investigations normally include few, if any, core drillings, and from surface observations alone it is limited how accurately the conditions at tunnel level can be predicted at the time when the contract is signed.
- The cost: If the contractor should assume all risk he would necessarily ask for a high price in order to be safe, a price that in most cases would exceed the actual cost, thus in the end giving the owner less value for his money. Should on the other hand the actual ground conditions be such that the contractor goes broke during the contract period, also this situation may have an undesirable effect on the owner's economy.
- Court proceedings in the wake of a contract are costly and time-consuming, and may do more good to lawyers than to the parties involved.

The main principle of risk-sharing in tunnel contracts while using time-equivalents is to establish tools for converting amounts of various items into construction time needed. These tools can later handle any contingency arising from changed ground conditions. Future discussions on regulation of the construction time where needed, or on related costs incurred will be easily concluded. All such problems are foreseen and the answers are to be found in the contract.

The formula developed to meet this end contains the following ingredients:

- Contract documents where relevant types of rock support and grouting measures that may possible be needed are described. Each one of these activities shall be specified in the Bill of Quantities, as a separate pay-item. The unit price for that item shall maintain its validity even if the actual quantity should deviate widely from the one given in the contract documents.
- Time parameters, quoted in the contract documents, by which any one of those activities expected to influence on progress can be converted into time intervals that reflect the time this activity consumes. Such work will have different impacts on the construction time when it is performed at the tunnel face where it interferes directly with the excavation cycle, and when it can be postponed to be installed behind the face where it interferes only marginally with the excavation cycle. Therefore two sets of time parameters must be quoted to cover both situations.
- Provisions in the contract for adjusting the contractual construction time, calculated by means of the work volume presented in the Bill of Quantities (the reference ground conditions) and the time-equivalent system described above, to the actual ground conditions encountered during construction.

A great advantage of this “time-equivalent system” is that the contractors incentive to meet the penalty deadline will be maintained; changed ground conditions and changed volumes of rock support may shift the completion date one way or the other, but then strictly in accordance with the rules and regulations laid down in the contract.

The reference ground conditions, reflected in the Bill of Quantities, should be based on the best judgment of the owner after an evaluation of all geological and geotechnical documentation from pre-investigation.

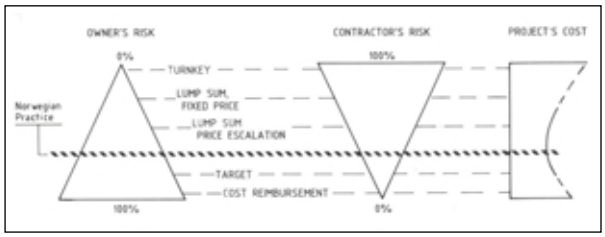

Figure 1. Risk sharing according to type of contract, and assumed influence on the project cost

3. PROVISIONS FOR REGULATION OF CONSTRUCTION TIME

In today’s tunnelling projects, strict environmental requirements prevail, calling for a high quality grouting to be performed. In such cases it is considered to be the best solution to incorporate a regulation of time. It is easy to understand that shifting the full risk on the contractor for time used for grouting do not carry sufficient incentive to use a long time on grouting to obtain a first class result. As the need for grouting in many cases is related to the concern of negative impacts on the neighbourhood, it is of greater interest for the owner to decide the quality he wants to obtain, than it is for the contractor.

Based on these motives, it seems to be the best solution that a regulation of the contract period depends on time used for grouting. This may be regulated in a separate “time-balance” sheet.

Usually the owner put an assumed quantity of hours for grouting in the tender. This amount of hours is regulated for the difference to actual used hours for grouting. The contract period is regulated hour by hour.

4. FUTURE CONTRACTUAL EVELOPMENTS FOR ROCK MASS GROUTING

4.1 Focus on the aspect of “time”

During recent years an increased number of tunnels have been constructed in urban areas. This has moved the focus of the tunnelling industry from the amount of inflow the tunnel, to the impact of draining of the ground water above the tunnel. The quality of the result of the grouting procedure concerning the surroundings is in these cases of highest importance. To obtain a more common goal for the involved parties concerning the final result of the grouting, the Norwegian Standard NS 3420 Specification texts for building and construction, will be revised to allow the possibility to compensate for the actual time used for grouting in addition to the cost of the materials.

By using this contract format the owner will take the risk for the aspect of time and time depending costs for grouting. The contractor takes the risk as far as having relevant equipment at his disposal for the grouting work. This is considered to be a fair share of risk when the demands for the result of the grouting are special high. The cost and the quality will better match each other when the contract format encourages a correct consumption of time and material.

It is also considered as an advantage to keep the risk sharing of today with responsibility for consume of time on the owner, and responsibility for the execution on the contractor, in situations where the grouting is not critical as far as the environment is concerned.

For the contractors, remuneration based on time, rather than quantity, will improve their capability of meeting the leakage criteria within the first grouting round. Benefiting from an efficient grouting procedure will automatically draw the attention of all involved parties towards a development of materials and methods in use for this kind of work.

Not only materials and methods are expected to develop, improved relations between the owner and the contractor are almost as important to obtain a time efficient grouting procedure.

Risk sharing and cooperation.

4.2 Principles of execution

It has been a common procedure to measure leakage of water from boreholes as criteria for decision when to start a grouting procedure. This is still used for grouting of dam foundations.

For tunnels in urban areas, however, the case will usually be to try to obtain a tunnel as water tight as possible, minimizing the drainage effect on the surroundings. The need for grouting is decided by the sensitivity to settlements of the ground above the tunnel or the sensitivity of existing biotypes to groundwater lowering. When a borehole ahead of face has been made, it should preferably be used for pre-grouting. Test has proven small connection between water entry into the bedrock compared to the entry of grouting materials, see figure 2.

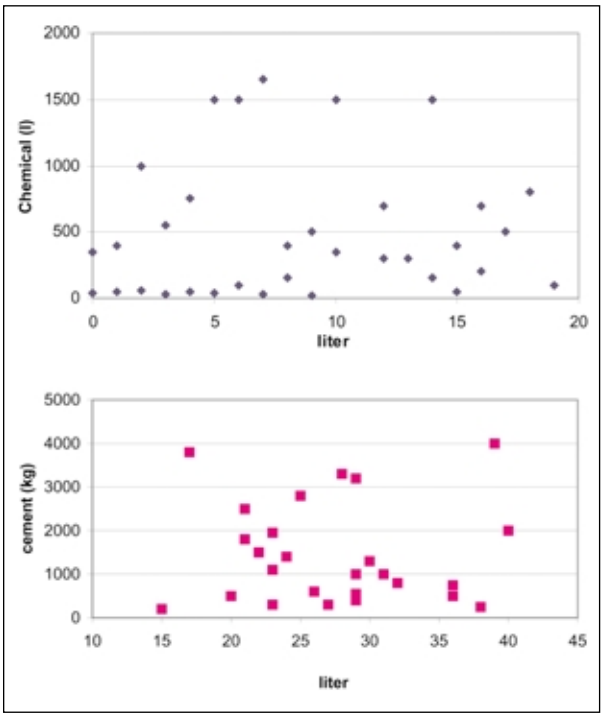

Figure 2. Measured relationship between Lugeon in liters and used grouting material in kg

The common procedure that has developed in areas sensitive to ground settlements, is to perform a systematic grouting in a first round. Further grouting is decided by the entry of grouting materials and after new combined test and grouting holes.

The most used procedure is to establish a grouted trumpet with 20 – 40 meter long boreholes with a spacing of 1 - 2 m between each. The next round is repeated every 12 meter. When the amount of grouting materials is high, the distance between each trumpet can be shorter.

It is important that everyone who is involved in the grouting work at the tunnel face is familiar with the aim of the grouting and the principles used for grouting. They then will be able to take the correct decisions while executing the grouting work. It is of importance to be familiar with the maximum allowed grouting pressure, and the maximum allowed grout material per hole, before the grouting is terminated in any specific borehole, and a new borehole is made for grouting in a later round.

Growing concern related to the owners and contractors organisations of such work has been seen. An efficient procedure is also very much depending on the construction management; decision at the tunnel face is important; a trustful co-operation between the involved parties; defined requirements of success-criteria.

5. EXPERIENCE

In figure 3 the relationship between planned and used amount of grouting for 4 tunnels in urban areas is demonstrated. The need of a minimum drainage effect on the ground water above the tunnel has been the focus of decision making.

Further, in figure 3 the same relationship is tested for 3 subsea tunnels. In these cases the design criteria is the amount of water that needs to be pumped out of the tunnel.

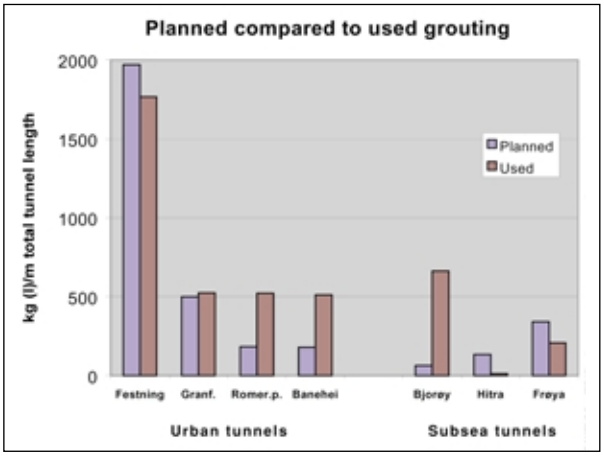

Figure 3. Planned grouting compared to used grouting for 4 urban tunnels and 3 subsea tunnels

The relation between planned and used amount of grouting points to the difficulty in predicting the correct estimate of such work.

The gained experience leads to the conclusion that the contract between the owner and the contractor should be based on a flexible system available to perform the optimal quality compared to cost within the contract.

LITERATURE

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