

# Water control in Norwegian tunnelling

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**ABSTRACT:** The rock mass is a significant barrier in itself. However, as it is a discontinuous material, its hydraulic characteristics may vary widely, from an impervious medium to a highly conductive zone. As a consequence, for groundwater control, it is normally standard procedure in Norwegian tunnelling to include pre-grouting for the purpose of reducing the permeability as well as consolidating the rock mass. This procedure has developed from the early tunnelling projects in the city of Oslo, through unlined, high pressure water tunnels for hydro power projects, oil and gas storage and sub-sea rock tunnels to the current generation of urban tunnelling. This article lists briefly the various reasons for such groundwater control, and provides an overview and an introduction to a cost effective tunnelling concept developed in Norway during the last decades. Further, some project examples will be provided to describe the current practice in Norwegian tunnelling.

## 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. On the other hand, it allows a great potential for hydropower development. It was important in Norway to establish a tunnelling philosophy which enabled the development of an appropriate infrastructure with such extremities to be overcome. Also, the hydropower constructions required an extensive use of tunnels and underground caverns and contributed to the development of this tunnelling concept. In the 1970's the oil and gas era in Norway began and underground facilities were used for transport and storage of hydrocarbon products. Norwegian tunnelling concept 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. Water control is an important aspect of this concept.

This article focuses on providing an outline of water control in Norwegian tunnelling. The Norwegian Tunnelling Association (NFF), associated with the ITA, has published a pamphlet recently presenting a number of various papers on the issue of water control in Norwegian tunnelling. The publication, no. 12 in a series presenting Norwegian tunnelling, includes a detailed description of the various elements in this concept and provides a number of project descriptions to exemplify the concept. This

article summarises the aforementioned publication by the NFF.

The rock mass itself is often an excellent barrier, having a significant capacity with regards to its impermeable and tightness characteristics, but owing to its nature, it is not homogenous and its characteristics can vary greatly. In Norway, shotcrete based tunnelling is the dominating tunnelling support, versus, a waterproof concept with cast-in-place concrete lining. Important elements for the development of the fundamental philosophy of our tunnelling concept are as follows:

- \* groundwater control by rock mass impermeabilisation using pre-grouting,
- \* enhancing the self supporting capacity of the rock mass, and
- \* constructing a drained structure of the rock mass in combination with rock support.

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

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 is required. Projects have been realised where the allowable inflow was in the range of 2-10 l/min/100m. The primary objective is to employ methods that aim at making the tunnel tight enough for its purpose.

This concept compares favourably with a waterproof membrane and secondary cast-in-place concrete lining in typical hard rock conditions. Such conditions may, however, change rapidly, from competent, crystalline rock types such as granites and gneiss, to poor weak zones, often highly permeable. The tunnelling concept has been applied in metamorphic rock types too, appearing fairly impervious.

Another aspect characterising this tunnelling concept is that of decentralised authority for decision making. Typically in Norwegian tunnelling the decision making is as close to the tunnelling activity as possible, to include the competence of the tunnelling crew and the tunnel supervisors. Based on predefined systems for rock support and rock mass grouting, the tunnelling crew is authorised to implement the design according to rock mass conditions encountered, i.e. the active design principle. This is applicable as a result of a contract practice which in Norway mainly has mainly been based on risk sharing through an extensive use of unit rates for different materials and activities.

The benefits associated with the Norwegian tunnelling compared to other tunnelling concepts, such as those including water-proof lining are: significant savings by means of shorter construction time; reduced construction costs; and greater flexibility in use. The application of the concept may, however, be restricted to areas in which the rock mass conditions are favourable with respect to the tunnel stability, i.e. the stand-up time of the rock mass is such that soft ground conditions do not exist. Further, the concept is limited to rock mass permeability consistent with a jointed aquifer with permeable zones in a less permeable rock mass.

## 2 PURPOSE OF WATER CONTROL IN UNDERGROUND TUNNELLING

Why make the tunnel or the underground opening a dry one? The answer seems, as far as can be understood by the author, to be threefold.

\* *Prevent an adverse internal environment.* For various reasons tunnels and underground

openings are associated with strict requirements to obtain a safe and dry internal environment. In many cases such requirements do not allow water appearing on internal walls or roof in the tunnel.

\* *Prevent unacceptable impact on the external, surrounding environment.* Tunnelling introduces the risk of imposing adverse impacts to the surrounding environment by means of eg; lowering the groundwater table causing settlements of buildings and other surface structures in urban areas; and disturbing the existing biotypes, natural lakes and ponds in recreational areas.

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

## 3 HYDROGEOLOGICAL CONDITIONS IN NORWAY

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

The rock itself is in practical terms impervious, and the porosity is negligible. This means that the permeability ( $k$ ) of a sound rock specimen is likely 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 very typical jointed aquifer where water occurs along the most permeable discontinuities. The permeability of the rock mass consisting of competent rock and joints may typically be in the range of  $10^{-8}$  m/sec. This implies that the most conductive zones in the rock mass must be identified and treated. Further, an appropriate solution must be determined to deal with such zones and to prevent the tunnel imposing an adverse situation in the groundwater regime, in terms of a lowered groundwater. Such an approach may not be restricted to one single measure to be executed, rather as it may consist of a series of

various measures and actions to be taken during the tunnelling works.

#### 4 TYPICAL SUPPORT APPLICATIONS

Using the term “unlined” tunnelling must not be misunderstood in such respect that the tunnels are constructed without any rock support at all. In general, permanent rock support consists of rock bolts and sprayed concrete, and only extremely good rock mass conditions may constitute an exception with no support measures. Water and frost protection may be taken care of by installing freestanding systems which are not load bearing, except for traffic loads, and do not contribute to the rock support.

The capability of the initial, primary lining structures are utilised to the extent the quality of the rock mass requires. Such an initial primary lining is often sprayed concrete with a thickness ranging from 50 to 200 mm in combination with rock bolts, commonly fully grouted and corrosion protected, some times installed in a systematic pattern. The latest sprayed concrete technology, with alkali-free accelerators, allows greater thickness to be applied. Sprayed concrete based support structures may in many cases adequately replace massive, cast-in-place concrete structures or pre-cast segments, (Grøv 2001). Such applications provide a long term use of sprayed concrete support compared to a temporary use as primary lining before a later, final concrete liner is installed.

The sprayed concrete mix design can be determined based on the actual project specific requirements that exist for the properties of the concrete. Today, the technology associated with sprayed concrete allows the designer to select amongst a number of properties of sprayed concrete: bond strength, strength, stiffness, ductility and permeability. This indicates that the sprayed concrete holds a great variety of properties that can be customised for any specific project. The challenge for the designer is to identify the relevant and applicable quality of the sprayed concrete.

Furthermore, the developments of spraying robots allows efficient application, with accurate dosage of admixtures and additives to cope with situations that require particular demands. At the same time additives have been developed that allow for example; a fast setting concrete with high early strength without influencing its final strength, the possibility of applying up to a 200 mm thick layer in one spraying operation etc.

Rock mass impermeabilisation is taken care of by pre-grouting ahead of the tunnel face. The effect of

improved stability in this respect is also an important momentum, (Roald et al. 2001).

#### 5 BRIEF METHOD STATEMENT

Norwegian tunnelling applies the philosophy based on: pre-grouting for groundwater control; self-standing capacity of the rock mass; and a drained support structure. The design and construction of any tunnel according to this tunnelling concept have to take into account these constituents.

##### 5.1 Groundwater control

Groundwater control is achieved by probe drilling ahead of the face followed by pre-excitation grouting (i.e. pre-grouting) of the rock mass. 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 removed from the tunnel periphery to outside 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 in the grouted zone within the rock mass. The pre-grouting technique has been particularly important for the successful construction of some 25 sub-sea tunnels with strict focus on keeping water inflow control.

Pre-defined grouting criteria will govern the progress of the tunnelling works. The tunnel will not be allowed to advance until these criteria have been met, which includes that more than one grouting round may be needed. In areas highly sensitive to groundwater fluctuations probe-drilling and pre-grouting shall be executed continuously along with the tunnel advance, e.g. such as every 20 to 30 m and with a specified overlap between each round according to project specific requirements. A pre-grouting round typically includes 10 to 30 holes, drilled in a specified pattern to create a trumpet shaped barrier in the rock mass, (Figures 1–2).

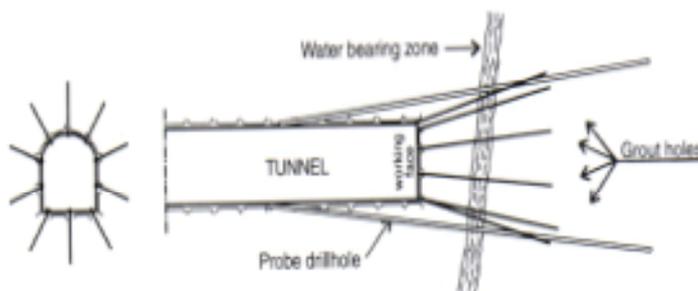


Figure 1. Typical probing and pre-grouting setting

The length of grout holes may vary from 15 to 35 m, and with an overlap of 6 to 10 m between each grout round, if continuous grouting is required. The pre-grouting scheme must cover the complete 360 degrees of a tunnel and include specifications for control holes and success criteria for the grouting work. 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.

Another typical way of assuring groundwater control is by artificial, pressurised water injection, or infiltration in the ground adjacent to the underground openings, often denoted “water curtains”. This method is commonly applied for hydrocarbon storage and also for unlined air cushion chambers in hydropower schemes. Through such arrangements a groundwater gradient acting towards the opening is maintained, as the internal pressure in the caverns (caused by the stored product) is smaller than the groundwater pressure. The stored product is thus contained hydrostatically in an unlined storage facility. Water injection (recharge) may also be applied to restore a groundwater level affected by construction activities.

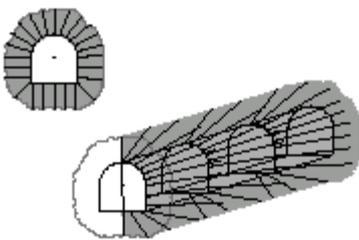


Figure 2. View of pre-grouted zone

### 5.2 Self-supporting capacity

Most rock mass has a certain self-supporting capacity, although this capacity may vary within a wide range (Bienawski 1984). The fact that there is some “stand-up” time implies that the rock mass for a certain time period is not a dead load. 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. The fact that, the rock mass is not a homogenous material should not disqualify the utilization of it’s self-standing and load bearing capacity. Modern computer software codes are available for appropriate modeling of the self-standing capacity of the rock and its interaction with rock support.

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

### 5.3 Drained structure

Another important aspect of the tunnelling concept described herein is that of a drained structure. The rock mass in combination with the rock support should constitute a drained structure. This means that the support measure installed is not 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 which has been subject to an extensive, customized pre-grouting schedule, some seepage may occur. This tunnelling concept includes a controlled handling of excess water at the tunnel periphery and behind the sprayed concrete lining. Excess water must either be piped to the water collection system in the tunnel or taken care of by a water protection system. Drainage can be achieved by means of installing, for example, local collection devices to confine the water and transfer the water via pipes to the drainage system in the tunnel.

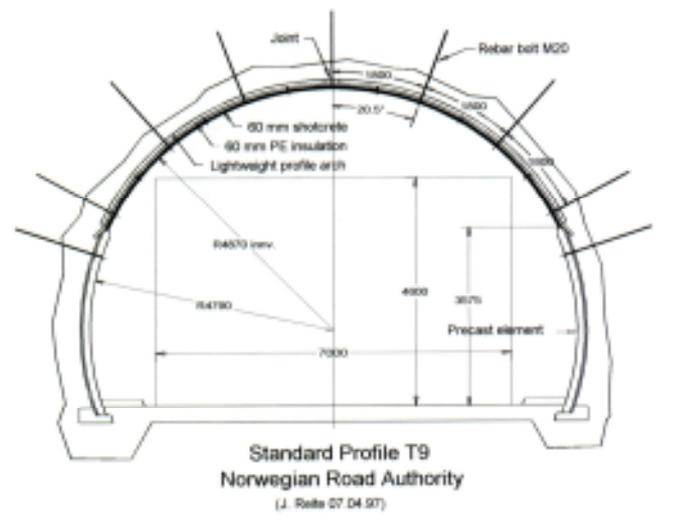


Figure 3. Free-standing water protection system

Recently, sprayable membranes have been launched to enable building a water drainage structure at the rock surface, or as an interlayer between two subsequent layers of sprayed concrete. A water and frost protection system including sprayed concrete is also an alternative. A number of different solutions have been tested in Norway, (Broch 2001a) some are related to low traffic volumes, whilst others are applicable for tunnels with high traffic volumes, (Figure 3). Common for these methods is that they do not interact with the rock mass support measures.

## 6 EXAMPLES FROM NORWEGIAN TUNNEL PROJECTS

In the following, water control in Norwegian tunnelling will be exemplified by project references, focusing particularly on the aspect of minimising the impact on the surroundings. Further, some examples will be provided with description of the interface between the tunnel and the surrounding environment.

### 6.1 *Pre-excavation analysis*

Within the city of Oslo construction of rock tunnels have taken place over several decades representing a special challenge due to the frequent presence of soft marine clay deposits above the bedrock. A number of tunnels have been excavated in this environment and significant experience has been gained. This experience shows that even a relatively small groundwater drainage to a rock tunnel will rapidly reduce the pore pressure at the clay/rock interface initiating a consolidation process in the clay. According to Karlsrud (2001), such a consolidation process can lead to settlement of the ground leading to severe damage to buildings, structures etc.

To be able to predict the maximum allowable inflow to a tunnel, Karlsrud reports that a formula has been developed based on the recording and experience from several tunnelling projects in the Oslo area. This formula has been tested by back-calculating a large number of cases.

The formula for calculation of inflow of groundwater to a rock tunnel assumes that:

- \* The tunnel lies in a homogenous media with a constant permeability in all directions.
- \* The tunnel is deep seated, the depth is 3-4 x the tunnelradius.
- \* The groundwater table is not influenced by the leakage.

$$Q = \pi k h \frac{2}{\ln\left(\frac{2h}{r}\right)}$$

where;  $k$  = permeability of rock,  $h$  = depth below water table and  $r$  = equivalent radius of tunnel. Assuming the permeability of the rock closest to the tunnel has been reduced with a factor of 10 by the grouting procedure, the leakage can be determined;

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

where;  $k_i$  = permeability of the grouted rock zone,  $h$  = depth below water table,  $r_e$  = equivalent outer radius of the grouted zone and  $t$  = thickness of grouted zone =  $r_i - r$ .

Further, the experience from the tunnels in Oslo shows that the acceptable inflow may range from 2 l/min/100m to 10 l/min/100m. This implies that inflow magnitudes as described will not create any harm to the surroundings in terms of settlement. Karlsrud suggests that the cost of systematic grouting typically adds some 50 to 70 % of the excavation costs, which is less than the costs of a waterproof concrete inner lining.

A 25 km long tunnel for transport of potable water to the city of Oslo is currently in the planning stage. The planning process includes a sensitivity analysis, as the tunnel will pass an area presumed sensitive to groundwater fluctuations. Kveldsvik et al. (2001), describes a procedure to identify areas sensitive to groundwater drainage. They report that the vulnerable sites are the following: lakes and ponds, bogs of all types, swamps and influent springs and streams.

The basis for the analysis was digital maps of land cover by which groundwater dependent areas could be identified and isolated. The vulnerability of the different areas was classified by the size of the actual catchment area. The most vulnerable areas were those with the smallest catchment area.

Water balance is a term introduced for the purpose of monitoring and following-up the effect of a tunnel construction on the groundwater situation. The importance of restored groundwater balance is discussed by Grepstad (2001). However, as Grepstad (2001) concludes, the groundwater balance is just a conception that implies that the groundwater level is within the range of natural ranges. The groundwater has seasonal changes and also cyclic changes over several years. Consequently, it is necessary to define the level of acceptable inflow to the tunnel, the level at which the water balance is 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.

More important though, according to Grepstad (2001) is the identification and mapping of various biotypes present in a tunnelling area. A thorough knowledge of the sensitivity of various biotypes in a tunnelling area is necessary before deciding where to concentrate a grouting effort. The knowledge of such biotypes must include such aspects as: local, regional and national context, how drought resistant

the biotypes are and the consequence of groundwater drainage.

Pre-construction assessments can be made, however, it is only by an appropriate monitoring program that the effect on the groundwater regime can be documented during construction. Therefore, it is important to install monitoring equipment to follow-up pore pressure and water levels at the surface prior to any construction starts, and likewise, continue monitoring at the surface during construction and include in addition monitoring from the tunnel. A great variety of different equipment and monitoring equipment is available.

Kveldsvik et al. (2001), also describe their approach to a maximum allowable inflow to the tunnel expressed by the percentage of the run-off in a catchment area. For their project, they arrived at the following differentiation:

- \* Inflow < 10 % of the run-off yields no, or small consequences.
- \* Inflow in the range of 10-20 % of the run-off yields medium consequences.
- \* Inflow > 20 % of the run-off yields large consequences.

The run-off from a given area (Q) is defined as the precipitation (P) minus evapotranspiration (E).

The project described by Kveldsvik aims at a defined minor negative influence on the surrounding environment. This implies that the strongest class prevails, imposing a maximum, allowable inflow of less than 10 % of the run-off. The tunnel will be designed with a number of various sections to meet the local environmental conditions, thus the maximum, allowable inflow will vary from 5 l/min/100m to 40 l/min/100m, being considered as on the “safe side” as regards to possible damage.

## 6.2 Elements of a grouting strategy

Rock mass grouting has reached a technical level where the equipment has become quite industrialised with highly efficient, multi hole pumps that are capable of providing a great range of grout pressures. A wide range of grout materials are available, and the industry utilises the knowledge from concrete to improve the material technology.

An important aspect of rock mass grouting is to choose the appropriate grout material. Recently, materials have been presented that could improve the effect of rock mass grouting, particularly for such situations that call for strict requirements on water leakage. For “bulk” grouting, and elsewhere without a complicated geological setting and strict

leakage criteria, OPC and other in-expensive grouts may be used.

However, there has been a trend towards greater use of micro-cements. According to Roald et al. (2001) the Norwegian company Elkem developed a new cement based grout material to fulfil the following:

- \* grout penetration to achieve a permeability close to 0.1 Lugeon,
- \* stable grout in liquid phase (< 2 % water loss),
- \* reduced shrinkage during hardening (< 1-2 volume loss),
- \* possibility to “guide” the grout into a specified zone surrounding the tunnel,
- \* environmentally safe and durable.

Further, it is important to choose a material, and a grouting procedure that enables completion of the grouting work in one round. Another key aspect of grouting is to focus the grouting works to a limited area surrounding the tunnel periphery and avoid penetration of grout deep into the rock mass. A thorough knowledge of the hydrological characteristics of the rock mass is required for the planning of a pre-grouting scheme, to choose the appropriate grout materials, grout pressure, number and length of grout holes, and grout strategy. However, an important input to the grout design, will also be the experience obtained for the particular project, and monitoring results of water inflow.

The principle of the grouting trumpet is described above. It is important that the trumpet covers the full circle surrounding the tunnel and includes grout holes in the invert too. For a typical grout sequence the grout holes in the invert would be the initial holes to be grouted, then the holes in walls follow and finally the holes in the roof will be grouted. The use of appropriate grouting pressure is as important.

Roald et al. (2001) describes the establishing of the “blocker” zone, by use of a set-controlled cement with variable viscosity. Such an approach can be particularly useful in poor rock mass conditions where experience show that it is difficult to build up a desired grout pressure. Inside the “blocker” zone the ordinary grouting can take place to consolidate the rock mass and reach desired permeability and ground improvement.

The use of high pressure grouting has shown to be effective in good rock mass conditions and in situations with rather impervious rock. Hydraulic fracturing can even be applied to improve the effectiveness of the grouting. However, in poor rock mass conditions care must be used to avoid a too high grout pressure, which could cause a lengthy and

consuming grout effort, and harm to the tunnel surroundings.

### 6.3 Organisation of grouting works

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

Blindheim et al. (2001) report the experience gained during the grouting works for the Oslofjordtunnel. Before entering the sub-sea section of the tunnel on the east shore, a comprehensive grouting scheme was enforced to fulfil the environmental requirements. The efforts involved in the grouting scheme included: definition of criteria for allowable inflow, establishing requirements for probing and pre-grouting, follow-up of inflow monitoring. To be able to run this grouting schedule as well as optimising the performance without sacrificing the established requirements, a significant organisational effort was needed, including the co-operation of both the contractor and the owner. During this co-operation, procedures were agreed and authority to adapt to the varying rock conditions was delegated to the tunnelling staff working at the tunnel face. A typical tunnelling contract in Norway would involve the owner as the decision maker, the one who specifies the grouting scheme to be conducted, with all details needed.

The target for the co-operation as described by Blindheim et al. (2001) was to optimise the grouting scheme by particularly focusing on an efficient performance of the works. Guidelines were agreed between the owner and the contractor amongst other things with regards to: change of grout material during the work; use of stop criteria related to grout pressure; need of supplementary grout holes; and probably most important; adaptation of the procedures to fit the actual rock mass conditions as encountered.

The typical Norwegian tunnelling contract is a unit rate type. This type of contract is based on a remuneration to the contractor in terms of quantity used and unit price for all items. This is a very typical risk sharing principle. The owner is the risk taker as far as the geological conditions are concerned. On the other hand, the contractor is the risk taker as far as to the efficiency of the workmen and staff to produce the specified item to the agreed price. In Norway it is typical to allow in the contract that the quantity may change 100 %, without influencing the unit rate agreed. Another particular agreement included in most tunnelling contracts is the "equivalent" time, a time balance sheet where each main tunnel activity has a normative execution

time. This is due to the significant costs associated with the operation of a tunnelling construction site, therefore, if the quantities are changing, the contract value will be adjusted to include a remuneration or deduction for the longer/shorter site mobilisation.

### 6.4 Hydrodynamic containment

More than 80 unlined pressure shafts and tunnels with maximum water heads varying between 150 and 1000 m have been put into operation in Norway to date. Experience from ten unlined air cushion surge chambers have been examined with respect to air leakage. Another large number of unlined hydrocarbon storage facilities have been in operation in Norway for a couple decades.

Common for many of these underground projects is that there is a hydrodynamic containment, in addition to utilising the capability of the rock mass with respect to impermeability and in-situ stresses. The basic principle is that the internal pressure in an underground opening is smaller than the minimum principal stress. The rock mass will, to a certain extent, take care of the containment. At one site the ratio between maximum air cushion pressure (i.e. the internal pressure) and minimum rock cover is as large as 2, but still the minimum stress component exceeded the internal pressure, thus it does not leak.

In addition, for some projects a water curtain may be used to provide an artificial groundwater supply. The experience from air cushion chambers shows that the leakage from the chambers increases significantly when the water curtain is cut off. The hydrocarbon storage facilities are mainly applying a similar concept, where the water injection to the ground may take place ahead of the tunnelling face to prevent groundwater lowering. Thus, there is always a saturated environment during the tunnelling itself and also during operation. The hydrodynamic containment allows the concept of unlined caverns.

During the last 20-30 years several empirical approaches have been developed for the design of unlined, pressurised tunnelling and caverns. Broch, (2001b) describes in details this development and the status of this design concept. The formula developed for the design of unlined, pressurised tunnels:

$$L > (\rho_w \times H) / (\rho_r \times \cos \alpha), \text{ where}$$

$L$  = shortest distance between the point of interest in the underground and the surface,  $\alpha$  = average inclination of the valley side,  $\rho_w$  = density of water and  $\rho_r$  = density of rock.

The experience with the unlined air cushion chambers is also detailed by Broch (2001a).

### 6.5 Tunnelling projects in the greater Oslo area

A recent publication in *Tunnels & Tunnelling*, in July 2001 describes the extension of the T-banering in central Oslo. A 1.7 km long tunnel, with a cross-section of 65 m<sup>2</sup> is currently under construction in a rock mass consisting of slate with limestone concretions, or syenite. The project is subject to strict environmental controls due to its close proximity to dense urbanisation.

The tunnel passes under deep glacial deposits which hold groundwater and surface structures. The rock cover varies down to as little as 15 m, except at the portals. Above, the surface structure must be preserved through strict limitations on water leakage to the tunnel. Permitted inflow to the tunnel has been calculated to 7 l/min/100m as a minimum in the most critical areas to 14 l/min/100m as a maximum. The tunnel has been sectioned according to the inflow criteria, and systematic pre-grouting will take place in sections with a maximum inflow < 10 l/min/100m. The permanent support will be handled with the use of sprayed concrete and rock bolts, except for a 200 m long section with low rock cover, that will be supported with a concrete lining.

Another infrastructure project in the close vicinity of Oslo is the Asker – Skøyen railway extension. The project, a twin-track railway includes the Asker – Jong tunnel, is due to commence excavation in 2002 and is described by Johansen (2001). The tunnel alignment passes several populated areas and also areas for recreation.

The influence area, i.e. the area affected in terms of groundwater drainage resulted from tunnelling activities is important for the determination of the inflow requirement to the tunnel. The tunnelling activity may influence in such ways as; reducing the pore pressure in the area, dewatering of swamps and bogs, and reducing the capacity of adjacent wells. Using various techniques, including analytical and numerical modelling, 2D and 3D simulations, Johansen (2001) reports that the project arrived at various classes of allowable inflow to the tunnel. The following classes were established:

- Class 1, moderate leakage, 8 – 16 l/min/100m.
- Class 2, low leakage, 4 – 8 l/min/100m.
- Class 3, extremely low leakage, < 4 l/min/100m.

The tunnel has been divided into sections with uniform characteristics, and each section is given designated classes. One factor being decisive to the

sectioning has been the potential of causing ground settlements, thus the following applied:

- \* Areas with potentially > 80 mm settlement.
- \* Areas with potentially 40– 80 mm settlement.
- \* Areas with potentially < 40 mm settlement.

Another factor considered for the determination of the leakage rates was the preservation of recreational areas. A list of such areas was established and the areas were classified according to their sensitiveness to a potential lowering of the groundwater level.

### 6.6 Experience from recent road tunnels

Davik et al. (2001), report the experience from 5 road tunnels recently constructed in Norway. The article, prepared as a part of a major nationwide research project on Norwegian tunnelling focuses strategies for pre-grouting of tunnels. A brief update on these projects, which were all constructed during the years 1997 to 2001, is described below.

The Tåsen tunnel, in Oslo, is a twin tube tunnel, each tube with a length of appr. 950 m. The rock mass was mainly sedimentary rock such as clay shale and limestone, interfaced with igneous dykes, occurring occasionally as quite permeable. The total measured inflow after excavation was in average 13 l/min/100m in each tube, which is slightly above a strict pre-defined criteria of 10 l/min/100m. Groundwater control was achieved partly by systematic pre-grouting and partly by dedicated pre-grouting of highly fractured dykes.

The Svartdal tunnel, located within the city of Oslo, is also a twin-tube tunnel. The tubes are 1450 and 1700 m long respectively, with a minimum rock cover of only 2.5 m. The rock mass consisted of highly fractured clay shale of poor quality, very poor quality alun shale and good quality gneiss. A maximum inflow rate of 5 l/min/100m was required, with particular focus on the concern of settlement of buildings above the tunnel. However, only a total of 260 m of the two tunnels was subject to systematic pre-grouting. The pre-grouting was based on measuring the leakage from probe holes, continuously drilled ahead of the tunnel face.

The Storhaug tunnel, is located outside Stavanger in the south-western part of Norway. The tunnel is 1260 m long and is excavated in different types of phyllites with very low permeability. The tunnel passes under an urban area with houses built between 1900 and 1950. Several of these houses are founded in peat moor by means of traditional wooden piles while some are “floating”. The maximum leakage was after due consideration set at 3-10 l/min/100m, the lower value for the section of

the tunnels closest to the peat moor area. To reach such a low value, systematic pre-grouting was required. Leakage measurements conducted after the completion of the tunnel yielded a total of 1.6 l/min/100m.

The Bragernes tunnel, in Drammen is located some 40 km north-west of Oslo and is a ring road leading the traffic around the city centre. The tunnel is 2300 m long, and was mainly excavated in volcanic rock mass, regarded as highly permeable, with a hydrostatic water pressure on average of 100 m. The maximum allowable inflow was 10 l/min/100m. Systematic pre-grouting was conducted merely throughout the entire tunnel length. The pre-grouting scheme was designed to adapt to the local conditions allowing modifications to take place based on geological conditions aiming at meeting the leakage criteria with only one grouting round. The total inflow was measured at an average of 10 l/min/100m, with a minimum in the most critical section of 8 l/min/100m.

The Baneheia tunnel, in Kristiansand, in the southern part of Norway consists of two parallel tubes, each appr. 750 m long. The rock mass consisted mostly of low permeable gneiss and dykes of pegmatite. The rock cover is 10 to 40 m, and the tunnel passes under a popular recreational area with 19 m rock cover. Due to the sensitivity of this recreational area the maximum allowable inflow was set totally at 60 l/min, or equally to 2.1 l/min/100m. This required a systematic pre-grouting for 95 % of the tunnel length, as it was found that sporadic grouting was insufficient. Implementing a standard grouting scheme with high capacity incorporated in the excavation cycle enabled an efficient tunnelling advance. The total inflow of 1.7 l/min/100m, measured after completion of the tunnelling works reflects a successful grouting process.

A conclusion from these projects is that the use of a standardised, systematic grouting scheme through the whole tunnel is most advantageous for groundwater control and surprisingly also for the excavation cycle. Further, Davik et al. (2001) concluded;

- \* that superplasticizers and silica additives increased the penetrability and pumpability for grouting and micro-cements;
- \* that increased grouting pressure (up to 90 bar) yielded better penetrability and grouting capacity;
- \* that reduced w/c ratios improved the quality of the grout; and
- \* that the pre-grouting efforts improved the rock mass stability.

## 7 COST ASPECTS

Taking standard cost into account, cost comparison figures are available in published articles. According to Garshol (1997) the relative comparison 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.

Based on Aagaard et al. (1997) Figure 4 shows the cost comparison for unlined versus cast-in-place concrete lining. 1 USD is equal to appr. 9 NOK.

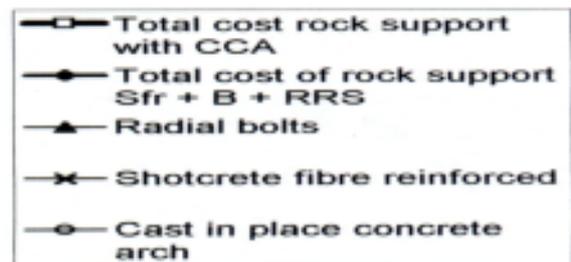


Figure 4. Cost comparison of applicable methods

That is, with increased quality of the rock mass the unlined alternative is more favourable. In adverse rock mass conditions it seems as if the two alternatives are more or less equal from a cost comparison point of view.

For an example situation the cost aspect can be estimated. For a tunnel size of 60 m<sup>2</sup> and in normal rock mass conditions the following cost figures can be derived. The excavation costs are set to 100.

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

Table 1. Cost comparison

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

Applying the above cost approximations an 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.

For a 2-shift arrangement, working 10 hours per shift and 5.5 days per week a typical tunnelling progress for example a 60 m<sup>2</sup> cross section would be in the range of 50 to 60 m, 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.

## 8 CONCLUSIONS

This article has presented an outline of water control in Norwegian tunnelling, a concept developed in Norway during the last decades. The concept is based on utilising the self-supporting capacity of the rock mass, supplemented or strengthened by use of rock bolts and sprayed concrete to establish the permanent rock support, whilst groundwater control is achieved by use of pre-grouting the rock mass.

The main advantages of this tunnelling concept are as follows:

- It provides a flexible approach.
- It utilises the capability of the rock mass, with respect to both its self standing capacity and impervious characteristics.
- It is cost effective.
- It reduces the construction time.
- It fulfils the requirement for strict water control in urban tunnelling, and also for tunnelling projects focusing on the practicality of water pumping during excavation.
- It provides full documentation of all activities.

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