ROCK MASS GROUTING

NORWEGIAN TUNNELLING SOCIETY

PUBLICATION NO. 20

ROCK MASS GROUTING IN NORWEGIAN TUNNELLING

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The front page image underlines modern tunnelling as a combination of human resources and sophisticated hardware.

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FOREWORD

The publication "Rock Mass Grouting" is part of the English language series published by the Norwegian Tunnelling Society NFF.

The aim is to share with colleagues internationally information on rock technology, this time with focus on rock mass grouting in tunnelling.

The publication is prepared as guidelines for the planning and the implementation of rock grouting. It provides suggestions for decisions to be taken both during the planning and the construction stage outlining good practise for the execution of rock grouting.

The publication is mainly based on the third edition of NFF "Handbook on Rock Grouting for Underground Projects" prepared in the Norwegian language, dated July 2010.

The target group includes consultants, construction project owners and all grouting practitioners.

The Working Group behind the Norwegian version Handbook of July 2010 was made up of the following members: Hans Olav Hognestad, BASF; John Ivar Fagermo, AF Gruppen; Alf Kveen, Norwegian Public Roads Administration; Lise Backer, Norwegian National Rail Administration; Eivind Grøv, SINTEF/NTNU; Erik Frogner, Norconsult; Arnstein Aarset, NGI.

Behind the preparation of the present publication other members were engaged; foremost Aslak Ravlo, Thor Skjeggedal and Jane Lund-Mathiesen. NFF express sincere thanks to authors, contributors and supporters.

Norwegian Tunnelling Society

Oslo, May 2011

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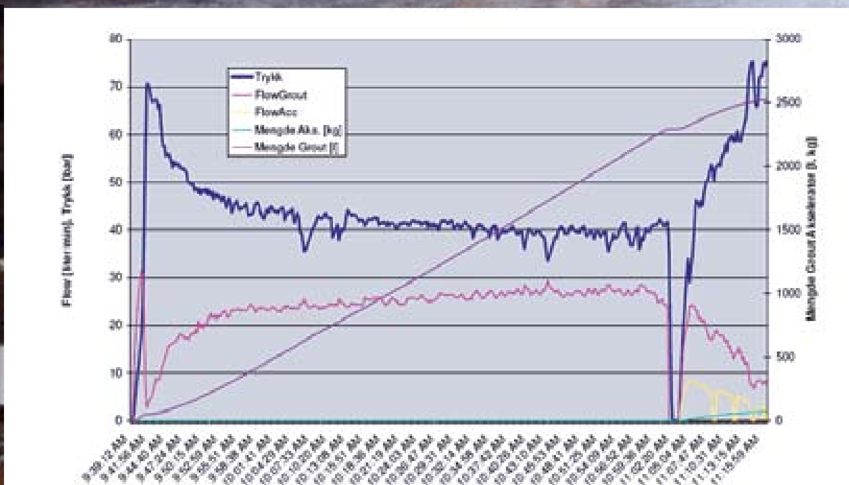

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INTRODUCTION

In the field of rock grouting constant advances are being made in the development of equipment, methods and materials. One major improvement is the implementation of measures relating to health, safety and the environment (HSE) in grouting operations in tunnels and rock caverns.

This publication presents common, standard rock grouting practice in Norway today. The book provides guidelines for the practical execution of grouting in rock caverns, focusing primarily on equipment, materials and operations at the working face. There are brief descriptions of each of the operations involved in rock grouting. Much of the advice in the book is most applicable to projects involving systematic grouting, but may also be helpful in connection with the project design phase or performance of grouting as deemed necessary according to the actual site conditions.

For the most part, this guide focuses on cement-based grout, but alternative grouting products and methods are also discussed. The aim of the book is to show that there are a number of ways of achieving a satisfactory result. Normally, there will be several underlying basic principles, and a technical/financial optimisation tailored to the individual project.

During the previous rock engineering project named "Environment and Community Friendly Tunnels" that took place during the period 2001 to 2004, a great deal of empirical data concerning the relationship between subsidence potential, rock permeability, the vulnerability of the natural surroundings, water inflow criteria and sealing technology was gathered and systematised. The findings have been incorporated in revised editions of domestic Codes, and Standards and and have also been taken into account in contract drafting for new projects involving systematic grouting. Most of the basic material in this publication is also based on the same findings.

Towards the end of the publication some supplementary background material is presented concerning theoretical relations that have an impact on choice of sealing strategy, materials selection and grouting procedures. Guidelines are also provided for determining requirements with respect to the sealing of rock caverns and tunnels, as well as principles for establishing what decision-making criteria will be crucial for when and how rock grouting should be carried out.

During the work it became clear that there is a need to introduce a set of requirements for training and certification for grouting operations, on the same lines as for personnel who are to plan, perform and check sprayed concrete operations.

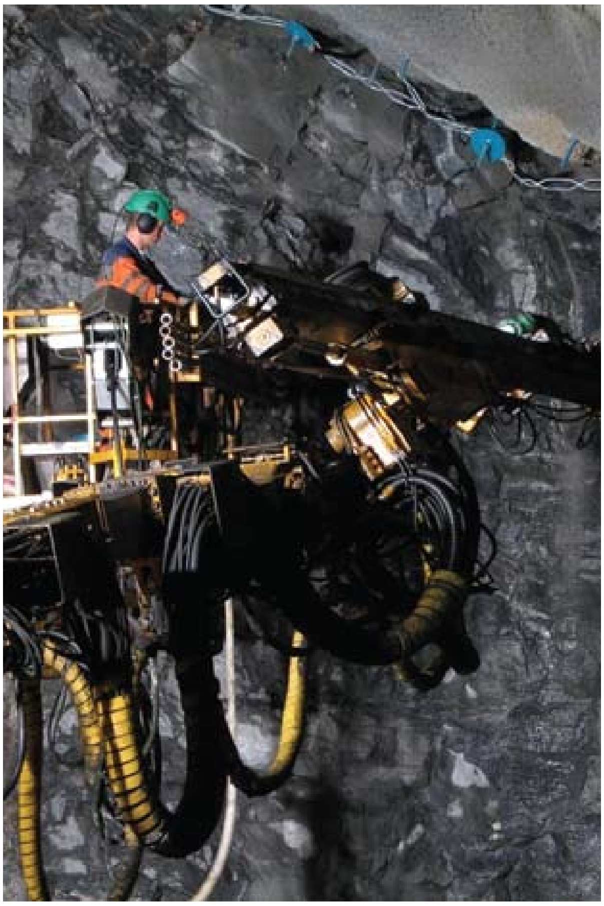


Figure 1. Investigation drilling at profile 1614, change of rod. Photo Svein Skeide, Statens vegvesen.

GROUTING PRACTICE IN NORWEGIAN UNDERGROUND ENGINEERING

It is a indeed a great pleasure to be able to present to the international tunnelling society a publication by the Norwegian Tunnelling Society that is summarizing years of extensive application and development of the rock mass grouting technique. Rock mass grouting has been an important part of tunnelling in Norway, intentionally to assist the contractors in the execution of practical works when hitting water inflow to the tunnelling face, but focus has turned and now it is an important factor in enabling tunnelling to take place without causing adverse impact on the above ground facilities. Times are changing.

GEOLOGICAL AND HYDROGEOLOGICAL SETTING IN NORWAY

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

Throughout Scandinavia, general rock mass conditions are favourable for such utilisation. The geological setting is dominated by igneous rock types such as granite, together with metamorphic rocks of various types and origins like gneiss, shale etc. The host rock is more or less intersected by weak zones, which may have an intense tectonic jointing, hydro-thermal alteration, or be faulted and sheared, constituting significant weaknesses in the rock and making the rock mass far from homogenous. These conditions may require rock strengthening measures.

The host rock in Scandinavia general varies from poor to extremely good rock quality according to the Q-system. The zones of weakness can exhibit great variation in quality, their Q-classification ranging from “extremely poor” rock mass at the lower end of the scale, to “good”, with width extending from only a few centimetres to tens of metres. The stand-up time of many of these zones may be limited to only a few hours.

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 10-7 to 10-8 m/sec. This implies that the most conductive zones in the rock mass must be identified and treated. Further, an appropriate solution must be determined to deal with such zones and to prevent the tunnel imposing an adverse situation in the groundwater regime, in terms of a lowered groundwater. Such an approach may not be restricted to one single measure to be executed, rather as it may consist of a series of various measures and actions to be taken during the tunnelling works.

ROCK MASS GROUTING BY PRE-GROUTING

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 publication will address various reasons for such groundwater control, and provides a description of the modern concept for pre-grouting for underground tunnelling as developed in Norway during the last decades. Further, some project examples will be provided to describe the current practice in Norwegian tunnelling.

Rock mass grouting has become an important aspect in tunnelling and underground excavation, particularly whilst executing such work in urban areas with a highly developed surface infrastructure and also in areas which due to various reasons are sensitive to fluctuations in ground water levels. The public focus on tunnelling work has increased during the last decades, not only project cost and schedules are scrutinised carefully, but also the consequences caused by ground water lowering on the surroundings such as flora and fauna, building settlements etc. This article presents some guidelines on how to approach these situations taking into account rock mass grouting as a main measure to deal with the ground water in rock tunnelling. Use of sensitivity analysis is one tool at hand to identify the vulnerability of the surroundings with respect to ground water draw down caused by tunnelling activities.

Tunnelling in urban areas may be subject to a maximum allowable water inflow level of 2-4 liters per minute per 100 meter tunnel, whilst in other areas such as sub sea tunnels the maximum allowable level can be fixed to 30 litres per minute per 100 meters. Actually, there is a wide range of acceptable inflow levels, still rock mass grouting by use of cement based material is the main material. However, during the last years new material technology has improved the possibility of modifying the cement with e.g. accelerators to control the setting time, plastizisers to improve the pumpability and penetrability, whilst micro silica secures a stable grout until its hardened. Highly efficient and mobile grouting units enable computer aided mixing and pumping, independent injection can take place in several grout holes simultaneously and the applicable pumping pressure may reach 100 bar if required. Water control by rock mass grouting has developed and reached a high-tech level in material technology as well as to the equipment at hand. Following this development it is important that the industry is capable of utilising the concept correctly as it is required, in project specific situations.

Unpredicted water inflow in tunnels, lack of knowledge and insufficient contractual tools to handle such project implications are the most frequent reasons for cost and program overrun in tunneling projects worldwide. By utilizing pre-grouting knowledge and technology we claim that the tunneling industry has a powerful technique to reach predictable costs and construction programs.

Reducing the permeability of the rock mass by pre-excavation grouting has become important in tunnelling and underground excavation, particularly whilst executing such work in urban areas with a highly developed surface infrastructure and in areas which are sensitive to fluctuations in ground water levels. The public focus on tunnelling work has increased and changed recently. Consequences caused by ground water lowering on the surroundings such as flora and fauna, building settlements etc are scrutinised thoroughly. This article presents an update on pre-excavation grouting as a main technique to deal with the ground water in rock tunnelling.

Pre-(excavation) grouting has developed from being the tunnelling activity with low status and limited contractual attention to become the key performance indicator for all parties involved in urban tunnelling projects. Urban tunnelling history and frequent contractual disputes is the best evidence that this technique will benefit future projects. Pre-grouting has reached a high-tech level in material technology as well as to the equipment at hand. It is utmost important that the industry is capable of utilising the technique correctly as it is required, in project specific situations. 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, cut by cracks, joints and discontinuities it is not homogenous and its characteristics can vary greatly within short distances.

Tunnelling may cause a drawdown 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. Requirements to the surrounding environment may be another restriction on 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.
The primary objective is to make the tunnel tight enough for its purpose.

TIGHT ENOUGH FOR ITS PURPOSE!

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

Prevent an adverse internal environment. Tunnels and underground openings are 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 e.g.; lowering the groundwater table causing settlements of buildings and surface structures in urban areas; and disturbing the existing biotypes, natural lakes and ponds in recreational areas.

Maintain hydrodynamic containment. Unlined 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. Water control in this context is to provide a containment to prevent leakage of stored products.

On behalf of the Norwegian Tunnelling Society (NFF) we wish you a pleasant reading and hopefully you will find lots of interesting and useful tips and hints on how to perform your rock mass grouting and where to include these efforts to be best for your project. And for your information the tunnelling society in Norway has earlier provided publications that are available for the greater tunnelling society. A lot of lessons have been learned over the last centuries and these are available through our publications and could constitute an asset for professionals who are seeking experience based information and knowledge on the aspect of rock mass grouting.

Figure 2. Tunnel cut for Eiksund subsea tunnel. Starting a descent of 300 metres. Photo Svein Skeide, Statens vegvesen.

Jurong Caverns, Singapore

GROUTING CHECKLIST

The checklist indicates important aspects to consider during a grout project.
A, B and C are important during the planning stage. C, D and E are more important during project implementation
For the Developer/Owner all are of importance.

A. Surroundings and the rock cavern

- Ground conditions
- Buildings
- Waterways
- Biotopes
- Conductivity of the ground
- Nearby underground installations
- Influence area
- Sensitivity
- Application
- Objectives
- Functional requirements

⇩

B. The bedrock

Geology
- Jointing
- Tensions
- Filling of fissures
- Rock type

Hydrogeology
- Water table
- Water flow in the rock
- Chemical properties

⇩

C. Grouting strategy – systematic vs. grouting as deemed necessary

Geometry of grout cover
- Penetration depth
- Length
- Number of holes
- Overlapping

Grouting agent
- Materials selection
- w/c ratio
- Hardening
- Admixtures

Stop criteria
- Volume
- Pressure
- Migrating leaks

⇩

D. Execution

Equipment
- Mixer and agitator
- Hoses and couplings
- Rods and packers
- Communication

Grouting agent
- Supplier
- Delivery plan

Grouting crew
- Experience
- Organisation
- Contingency plan
- Risk analysis

E.Documentation

Drilling report
- Length
- Angular deviation
- Inflow per hole
- Special conditions

Grouting repor
- Amount per hole
- Standing pressure
- Recipe(s)
- Migrating leaks

Quality
- Stability
- Setting
- w/c ratio
- Storage time
- Cement quality

⇩

F. Results

Geology
Inflow measurements
Groundwater level
Damage to environment
Waterproofing

Documentation of execution
Documentation of quality
Post-excavation grouting

SETTING UP FOR GROUTING

CLEARING

Before starting to set up for grouting, the area at the working face should be cleared of debris and pools of water. There will be a great deal of movement along the tunnel walls during the grouting process and therefore there may be a need to inspect the stability of the wall behind the face. Thorough clearing facilitates this work and helps to reduce the risk of injuries.

ACCESS ROUTES

During a grouting operation there will be much coming and going from the storage area in which the rods and packers are kept and from the grouting rig. This area should be kept clear for safe movement. It is good operating practice to have a well-levelled floor and adequate lighting.

SPACE REQUIREMENTS

The area in which the grouting rig stands should be level and large enough to allow personnel to move around the rig. It must be possible to supply additional grouting material unobstructed. It would be an advantage for maintenance work on grouting rods to be done at the grouting rig.

Figure 3. Preparing for grouting.
Photo Svein Skeide, Statens vegvesen.

LOCATION OF THE WATER PUMP

If it is necessary to remove water from the working face, a clearly defined depression should be established which collects water from the whole face area. In the event of grout leaks, spillage or flushing, cement slurry will end up in the water, and therefore to prevent this from blocking the pumps and the pipelines, the pump should be located slightly above the bottom of the sump. One solution to prevent PP-fibres from blocking the pumps may be to place a filter around the pump which allows water through, but stops the fibres.

REINFORCEMENT OF THE CAVERN PRIOR TO GROUTING

The pressure used in grouting operations is substantially higher than atmospheric pressure. If such pressure is allowed to act on faces parallel to the tunnel surface, cracks may appear and block fallout may be initiated. It is therefore of utmost importance to reinforce the cavern well with bolts and sprayed concrete, and if necessary heavier reinforcement, before grouting is to be performed. When grouting operations are to be carried out in weakness zones, it is often necessary to reinforce the working face with bolts, and here it may be advantageous to use fibreglass bolts

Bolts should be grouted into the rock before the grouting work starts to prevent leaks in boltholes.

DRILLING OF INJECTION HOLES FOR GROUT CURTAINS

DRILLING METHODS

In most tunnel and rock cavern grouting operations, the grout injection holes are drilled using the tunnelling rig (top hammer drilling). The tunnelling rig is well suited to drilling injection holes for normal grout cover lengths (18 – 24 metres). Longer drilling lengths may result in borehole failure. Two and three-boom rigs are used for normal longhole drilling.

The flushing pressure when drilling injection holes with the tunnelling rig should be at least 15 bar. It is important that the flushing pressure and the water volume are large enough to prevent the cuttings from clogging up the cracks in the rock. A flushing pressure that is too low reduces penetration and the holes tend to be oval and result in large deviations.

Once drilled, the holes should be flushed out with a mixture of air and water. Where the rock type is soft, a prolonged flushing of the borehole wall with radial nozzles and high water pressure (150-250 bar) ought to be considered. Tests have shown that insufficient flushing reduces the Lugeon values in the boreholes by 10-50%. In very poor rock masses caution must be exercised when using high pressure so as to ensure that a hole is not lost.

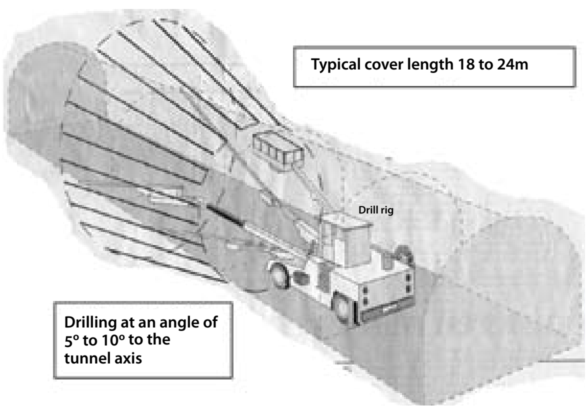


Figure 4. Drilling injection holes for grout cover using a three-boom drill rig.

GROUT CURTAIN DRILLING PLAN

The project owner will normally prepare draft drilling plans for different sealing requirements. The number of holes per grout curtain and the length and direction of the holes must be adapted to the quality of the rock and the sealing requirements that are to be met. The jointing characteristics of the rock, in particular, are of critical importance in the drafting of the drilling plan, and it must be possible to adjust the plan according to the drill string behaviour observed and the results from completed grouting

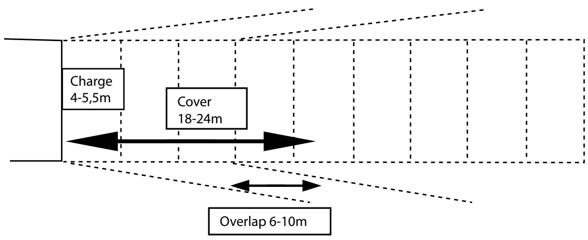


Figure 5. Example of grout curtain and charge length seen in the longitudinal direction

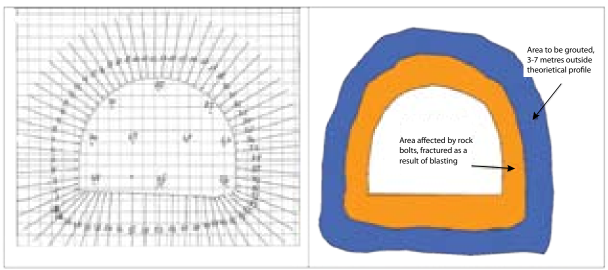


Figure 6. Example of grout cover for a cross-section of 110m2, Lysaker – Sandvika

There are special requirements as regards the design of the grout curtain at points where there are changes in the geometry of the tunnel or rock cavern, such as at intersections for cross-cuts, profile enlargement for recesses and in double-tube tunnels.

Sealing criterion	Stringent requirement	Average requirement	Low requirement
At the start	0,5 – 1,0 m	1,0 – 1,5 m	1,5 – 2,5 m
In critical sections	1,0 – 1,5 m	1,5 – 2,0 m	2,0 – 3,0 m

Table 1. Typical hole spacing for different watertightness requirements

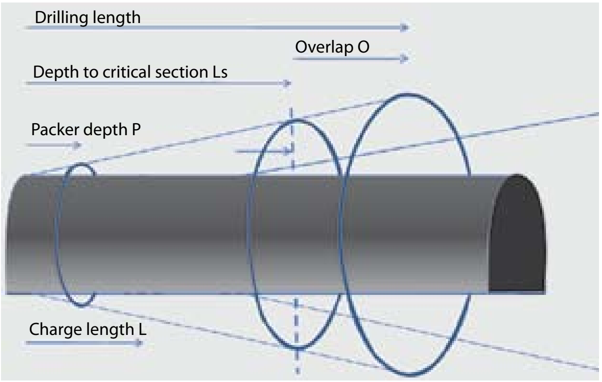


Illustration showing location of critical section, Ls

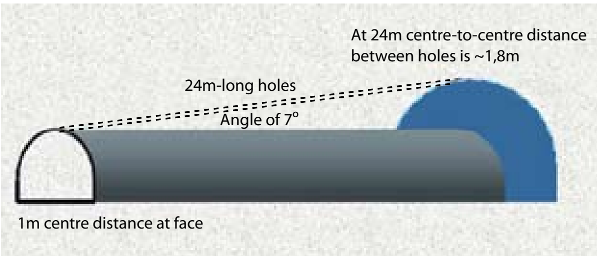


Figure 7. Example of theoretical relation between hole spacing at the start and in the rock mass. Borehole deviation comes in addition

In addition to the hole spacing, the distance between the curtain intervals is also decisive for how watertight the rock mass will be when pre-grouted. It may be valuable to check actual hole spacing at a so-called critical section, that is to say, a cross-section where the packers are placed in the next grout cover. The hole spacing in this section depends on the outward angling of holes, distance between the grout curtains and the spacing of holes at the start.

CUT-OFF GROUT CURTAIN

When there is minimal rock cover it may be appropriate to establish an extra "cut-off cover" that is injected at low pressure to fill voids and wide cracks in a "zone" outside the actual grout cover. The purpose of this is to seal the rock mass sufficiently to be able to grout at higher pressure in the main cover without the grout coming up on the surface or disappearing outside the intended zone.

ROD HANDLING

The drilling of long holes calls for lengthy drill strings with many drill rods. Positioning rods in and removing them from the drill string is heavy work, which can advantageously be carried out mechanically. Today most modern tunnel drill rigs can be delivered with equipment capable of providing mechanical assistance in rod handling. Further developments and improvements of rod handling equipment for longhole drilling is necessary in order to improve daily working life for the tunnel workers.

BOREHOLE DIAMETER

Ideally, the borehole diameter needs not be greater than necessary to obtain a stream of cement that does not set in the hole. The need for long holes with little deviation will often determine the borehole diameter. In general, deviation becomes smaller as borehole diameter increases, due to larger and more rigid drill strings. It is important to be aware that the force on the packer is almost proportional to the square of the borehole diameter. This means that the borehole diameter should not be too large. The Standard Code of Process requires a minimum diameter of 45mm. Today it is usual to use a borehole diameter of between 45 and 64mm.

Drilling method	Hole length	Hole diameter	Typical penetration rate per drill hammer	Average capacity for 3-boom rig	Normal deviation requirement
Top hammer	18-24 m	45-64 m	1,5 – 2,5 m/min	60-90 dm/hour	<5%

Table 2. Typical data for drilling grout injection holes

DRILLING STRAIGHT HOLES

The jointing of the rock and any foliation or stratification will impact strongly on drilling deviation. Deviation can be reduced by using rigid drill rods and/or drill bits with a long skirt for increased control. The use of an extra rigid rod (guide rod) closest to the bit gives smaller deviation, but may result in substantial rotational resistance in long holes and is therefore hardly ever resorted to.

It is crucial that the operator of the drill rig orients the feed beam properly at the start and that excessively high feed pressure is not applied during drilling. Modern drill rigs have substantially larger drilling capacity and better precision than earlier. Normally the full-time equivalent account in tunnel contracts is based on 60 -90 drill metres per hour. It is, however, important to be aware that high penetration rate may adversely affect precision and result in large borehole deviations.

DEVIATION MEASUREMENT

When grout curtains are extensive and requirements as to watertightness are stringent, deviation measurements should be made. Borehole deviation measurements are an important part of directional drilling. Where deviations are large, the scope of deviation measurements should be increased as required. If the deviations are due to geological conditions, a suitable countermeasure may be to adjust the drilling plan. The start of drilling should, if necessary, be adjusted according to the deviations detected, or supplementary holes may be set. A requirement in place for long-hole drilling is that the holes should have less than 5% deviation from the theoretical plan at the end of the borehole.

Where pre-reinforcement bolts are used during tunnel driving, the probe used for measuring drilling deviation may be affected by the steel in the bolts. Similarly, it should be remembered that some rock types may have a magnetic impact on such measurements. A simple method of obtaining an indication of drilling deviation is to mount a light on a rebar and pass it into the borehole. If the light disappears after just a few metres, this indicates excessively large deviations.

POSITIONING AND MARKING HOLES

The boreholes are to be positioned in accordance with the current drilling plan, and holes in the tunnel periphery are normally started outside the contour blast hole periphery.

For the documentation of the grouting work, it is important that the holes should be marked with the same number that is used to indicate them in the drilling plan. The numbers should be spray-painted directly on the working face so that they are clearly visible from the tunnel floor.

BOREHOLE LOGGING USING MWD

When drilling grout injection holes, it is common practice to keep a record of:

- Feed pressure
- Rotational pressure
- Rate of penetration
- Drilling problems
- Loss of flushing water
- Water inrush

This information can be logged automatically on the drill rig. Interpretation of the drilling parameters or "Measuring While Drilling" (MWD) is the designation for the collection and interpretation of drilling data from the drill rig. This data is calibrated and interpreted by software such as Rockma. The results from this interpretation may provide valuable information about variations in hardness, rock type variations, degree of jointing and whether the drilling encounters water. This allows for adjustment of the grout curtain, adaptation of the rod lengths and determination of limits for grouting pressure and choice of a suitable grout.

MEASURING WATER INFLOW FROM BOREHOLES

The most efficient method of measuring water inflows from the boreholes is to use a "measuring rod". This is a short injection rod with an open packer that can be unscrewed from the hole. Packers are tightened slightly, so that the water is passed out of the rod. Using a stop watch and a bucket, the amount of water is measured over a given time interval.

It may be wise to indicate measured inflow directly on the working face as work progresses (preferably in another colour than the hole number). This will ensure that measured inflow will be reported for the right hole.

Water inflow can also be collected and measured using simpler equipment, such as a rubber hose that is passed into the hole, but then there is a danger that not all the water will be detected and measured.

Where stringent requirements are set as regards maximum inflow, it is important that the inflow is measured once the water flow has become stable after drilling, so that the packer can be set in the holes as soon as possible. All holes with leakage are measured even if there is a through opening into the neighbouring holes and the "same" water is measured several times. If there is an opening between holes, it should be noted which holes are involved in order to better the interpretation of total water inflow through the grouting holes.

Figure 8. Example of a severe water inrush in a tunnel on Iceland (Photo: Bjørn Hardarson)

Figure 9. High pressure water inflow (Photo: AF Group/John I. Fagermo)

THE GROUTING RIG

GROUTING RIG REQUIREMENTS

Grouting rigs are becoming increasingly subject to strict requirement specifications relating to grouting capacity and documentation of work performed. When simultaneous injection of several cement types is required, the grouting rigs should be equipped with multiple hoppers, several mixers, tanks for intermediate storage of ready mixed grout and multiple grouting pumps.

Modern grouting rigs are designed to be a safe and comfortable workplace with shielding from sources of annoyance such as dripping water, noise and dust. The rigs should be equipped with good lighting, both in the direction of the face and in the vicinity of the mixers. Any lifting of crew members should be carried out using certified and approved personnel hoists. There should also be requirements regarding the certification or approval of other sub-components, couplings etc.

In projects with lower capacity requirements, and where grouting is not a central part of production, typically in smaller grouting operations and in connection with sporadic grouting, smaller grouting units can be used. Such units may be equipped with one to two pumps and have a manual system for feeding cement into the mixers.

Figure 10. Example of equipment found on a modern grouting rig

HOPPERS

In projects where there is a need for grouting using both industrial cement and micro-cement, the grouting rig should be equipped with two hoppers. It will then be possible to alternate between these two cement types without a long break in the grouting work. The hoppers should be equipped with tight lids, so as to prevent water from dripping into the cement. As a rule, cement is supplied in large bags, and therefore a bag-breaker should be mounted above the hopper intake, so that the hoppers can be filled without crew members having to be close by.

MIXERS

Mixers for cement suspensions should be high-speed mixers (> 1500 rpm) which are capable of separating individual particles from each other. This is particularly important when mixing micro-cement. Normal mixing time is about 2 minutes.

The equipment required for high-grade grouting should include a stirrer or agitator which keeps the grouting mixture in suspension and from where grouting mortar is pumped. This will ensure that the grout pumping can take place non-stop without any break between mixtures.

CAPACITY OF GROUTING PUMPS

The grouting pumps should have ample capacity for the maximum pressure that is to be used. A capacity of 100 litres per minute at about 80% of maximum applied pressure is desirable.

AUTOMATIC LOG SYSTEM

The grouting rig should have equipment for automatic logging of all parameters that are included in the injection reporting. These should include:

- Volume of different recipes injected in each hole
- Pressure
- Start and stop time for different mixtures

OVERVIEW OF THE JOBSITE

The operator of the grouting pumps should have a good overview of the jobsite from the operator control panel. When hoses are to be changed frequently, it is extremely important to have good communication between the operator and the person connecting the hoses to and disconnecting them from the rods. It may be helpful to use an intercom system.

Figure 11. Example of a control panel on a modern grouting rig

WORK PLATFORM

Work at heights requires the use of approved personnel hoists or work platform. The reach of the personnel hoist should be such that all the rods can be reached from one position. The hoist must be operable from floor level in case an emergency situation or power cut or the like arises.

Figure 12. AMV Grout plant (Andersen Mekaniske verksted AS) (Photo: AMV Group)

Figure 13. AMV automatic 3 line grout plant (Photo: AMV Group)

Figure 14. Modern grout rig in action (Photo Svein Skeide, Statens vegvesen.)

PACKERS, RODS, HOSES AND COUPLINGS

All photos on this page by courtesy of Ivar F. Andersen, CODAN AS

Figure 15. Packers with a seat for up to 60 bar on the left and up to 100 bar on the right.

Figure 16. Complete rod with accelerator nozzle (demo version with shortened rod)

AREAS OF USE FOR DIFFERENT TYPES OF PACKERS.

At high pressures, that is to say, at pressures of more than 60 bar, the packers used should be adapted to such elevated pressures. At the same time, it must be ensured that the rod is chained to the working face. Packers that are designed for high grouting pressure (60-100 bar) must not be mistaken for low-pressure packers (< 60 bar). The rods have a different design at the point where they press on the packers, and therefore these two systems are not interchangeable.

Single-use packers are usually available with a length of about 15 cm (rubber). Packers can also be supplied as double packers which are about 30 cm long. These are constructed for better "bite" (friction) in the hole.

SINGLE-USE PACKERS

This is the most commonly used packer type in rock grouting. The packer has a valve at the front and is placed in the borehole with the aid of the grouting rod. There are different variants of locking washers at the back of the packer. The locking washers help secure the packer in the hole, allowing the rod to be unscrewed after completed injection. Packers will then remain in the hole, whilst the rod can be cleaned and made ready to be used again (see Figure 20).

All photos on this page by courtesy of Ivar F. Andersen, CODAN AS

Figure 17. High-pressure packer with hydraulic tap and eye for chain on the left and a safety packer with extra tensioning on the packer body on the right (demo version with shortened rod).

Figure 18. Packers with mechanical expansion sleeve

REUSABLE PACKERS

Reusable packers are normally used for measurement of water loss and special grouting jobs. An example of a single multi-use packer is a standard packer from which the locking washer has been removed, so that it returns to its original diameter after the rod has been unloaded. Hydraulic packers are a type of packer which is set using water pressure. After use, pressure in the packer can be release and the packer can be removed from the hole. Hydraulic packers are usually available in 1.0 metre and 0.5 metre lengths (Figure 19).

Figure 19. Example of hydraulic packers

Examples of areas of use for hydraulic packers:
- Extremely poor quality rock
- Grouting of sheet piling
- Stage grouting – where grouting is started at the bottom of the hole and the packer is pulled up progressively whilst pumping is continued, in stages, until the desired height is reached
- Cases where large holes are to be filled with cement.

POSITIONING OF RODS AND PACKERS

Grout injection rods are found in lengths varying from 1 to 6 metres. The most common rod length is 3 metres. Packers are normally set 1.5 to 2.5 metres inside the borehole, but this distance must be adapted to the quality of the rock at the face. In special cases it may be relevant to place packers 10 to 15 metres inside the hole.

STANDPIPES – A SOLUTION FOR A LOT OF WATER AND HIGH PRESSURE

If water at high pressure is encountered during drilling, it may be extremely difficult to insert a packer. In order to succeed, it may be helpful to cut off the tip of the packer so that some of the water can be passed out through the rod. If this is not sufficient, it will be necessary to drill relief holes, so as to allow some of the water to be passed out of the grouting hole. Several relief holes may be required. All packers that are set in these holes must be without a tip, thereby ensuring that water has a passage out through the rod.

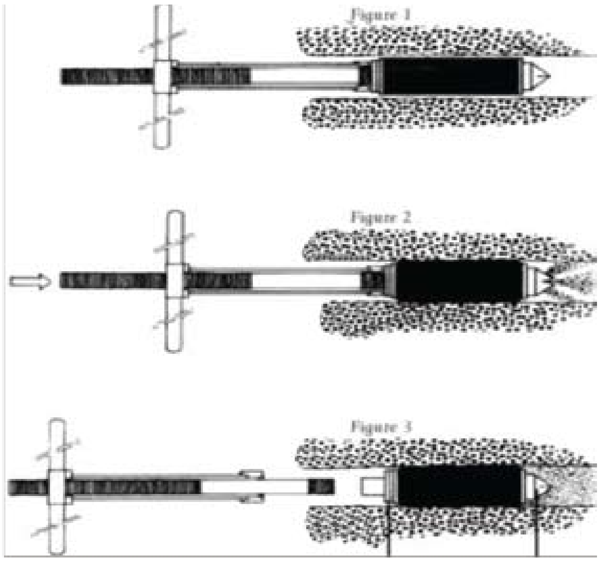


Figure 20. The principle of packer placement.

In extreme conditions conventional packers cannot be used, and so-called standpipes may then be a good alternative. A standpipe consists of a 4" steel pipe which is cemented in place about 2 metres inside a sufficiently wide hole in the face. It will be advantageous to use polyester cartridges to secure the pipes in order to save time. When the pipes have been secured, an adapter with a valve is mounted on the pipe so that the pipe can be closed when a large amount of water is encountered. Drilling then takes place through the standpipe. When the drilling is completed, the drill string is withdrawn

Figure 21. Example of a situation involving large water inflows. (Photo: John Ivar Fagermo, AF Group)

and the standpipe is closed. The grouting hose is then connected directly to the standpipe, the valve is opened and grouting commences.

GROUTING THROUGH SELF-DRILLING INJECTABLE RODS

When it is necessary to grout in very poor quality rock, it may be expedient to use injectable, self-drilling rods. The rods are grouted in place before they can be injected at pressure.

PRESSURE CLASSES FOR GROUTING HOSES

Grouting hoses must be of high quality to be able to withstand grouting pressure of up to 100 bar. A single layer hydraulic hose is the minimum used. Hoses should have an internal diameter of which results in as little pressure loss as possible from the pump to the grout passing the packer. The pressure loss in the system as the grout is run through should be documented. Hoses with an internal diameter of 3/4" are normally used.

COUPLINGS

It is common practice to use quick release couplings (Camlock) and standard ball valves when grouting at a pressure of up to 60 bar.

The quick release coupling is not certified for higher pressure, and so when grouting at a higher pressure it is usual to use hydraulic taps and screw the grouting hose directly onto the rod.

Figure 23. High pressure water inflow from probe holes (photo: CODAN/Ivar F. Andersen)

SERVICE APPARATUS

There should be ready access to spare parts to allow hoses and couplings to be replaced as soon as critical damage or wear is detected. It would be advantageous to have spare hoses on site. Pulsating in the hoses as a result of pressure variations will lead to wear, and the hoses should be laid so as to prevent them from rubbing directly against sharp rocks to any great extent. After each grouting session, hoses and couplings must be checked and replaced before a rupture or failure occurs.

See also the next chapter with regard to maintenance routines.

Figure 22. Constituent components and an assembled standpipe rod.

Figure 24. Injector for accelerator with non return valve (photo: CODAN/Ivar F. Andersen)

Figure 25. Sand pope with blow out preventer (photo: CODAN/Ivar F. Andersen)

Figure 26. Grout packers secured by anchored chain. (Photo: AF Group/John I. Fagermo)

Figure 27. Grout packers secured by anchored chain. (Photo: AF Group/John I. Fagermo)

MAINTENANCE ROUTINES

MAINTENANCE ROUTINE CHECK PLAN

The grouting equipment must be subjected to strict maintenance routines. Cleaning is extremely important. The following cleaning and maintenance plan may serve as guidance:

CLEANUP

Thorough cleanup must be performed to prevent cement from hardening in production apparatus and in the grouting rig. Such maintenance must be facilitated by ensuring that a high-pressure washer and a work bench (see Figure 28) are available in the vicinity of the grouting rig.

Component	Maintenance frequency	Comments
Hoppers	Feed screws should be checked regularly and cleaned when necessary. Hoppers must be emptied in the event a prolonged stoppage.	the screw can be checked when the hopper is run to empty.
High-speed mixer	Cleaning after each round of grouting. Impeller/paddles should be replaced every other year.	Thorough cleaning is vital. Inadequate cleaning may result in set cement falling into the cement being mixed and causing operating problems during grouting. Impeller/paddles/knives will become worn and gradually give poorer mixing results. They should be replaced when they no longer give the same shear force in the mixing process.
Stirrer (agitator)	After each round of grouting.	Thorough cleaning will prevent a build-up of cement, also at the top of the mixer. All surfaces must be checked.
Grouting pump	After each round of grouting.	Must be dismantled, cleaned and lubricaded internally with acid-free vaseline.
	After each round of grouting.	Check of wear on connecting pieces, threads and split pins.
	After each round of grouting.	Check for wear and damage. Damaged hoses must be replaced.
Grouting rods	After each hole.	Thorough cleaning and lubrication of threads.
Packers	Check the packers that are going to be used before each round of grouting.	Check there are no production faults in the locking mechanism.
Taps/Ball valves	After each hole and during grouting. When grout takes are large, functionality should be checked by opening the tap.	Important to check functionality, particularly of pressure relief valves.
Grouting rig	Regular cleaning/lubrication, at a minimum after each round of grouting.	Anti-seize oil should be applied after rig has been washed. It is advantageous if the rig is equipped with a high-pressure washer for continous cleaning during the grouting process.
Scales	Every other month, or after 200 tonnes.	Measuring cells should be checked.

Table 3. Maintenance routines

Figure 28. Example of a work bench on a grouting rig. (Photo: John Ivar Fagermo, AF Group)

As soon as holes are fully grouted, the rod can be unscrewed and removed. It is recommended that the rod be washed externally and internally at once to prevent a build-up of cement on the steel (cement hardens in the grouting rod).

MAINTENANCE

Grouting rods must be maintained. It of utmost importance that the threaded portion be kept clean and in good condition, and it is recommended that the threads be greased at regular intervals. If the threads are not kept in good condition, it will not be possible to tighten the packers properly. Clean threads are therefore vital with a view to safety.

In addition to safety, it makes economic sense to ensure that the grouting rods are well maintained. Taps and rods are expensive, and they can be used many times if they undergo necessary maintenance. A sound piece of advice is to have a work bench equipped with a vice and other tools at the working face. This will make it easy to maintain rods during the grouting operation.

POWER SUPPLY AND DISTRIBUTION

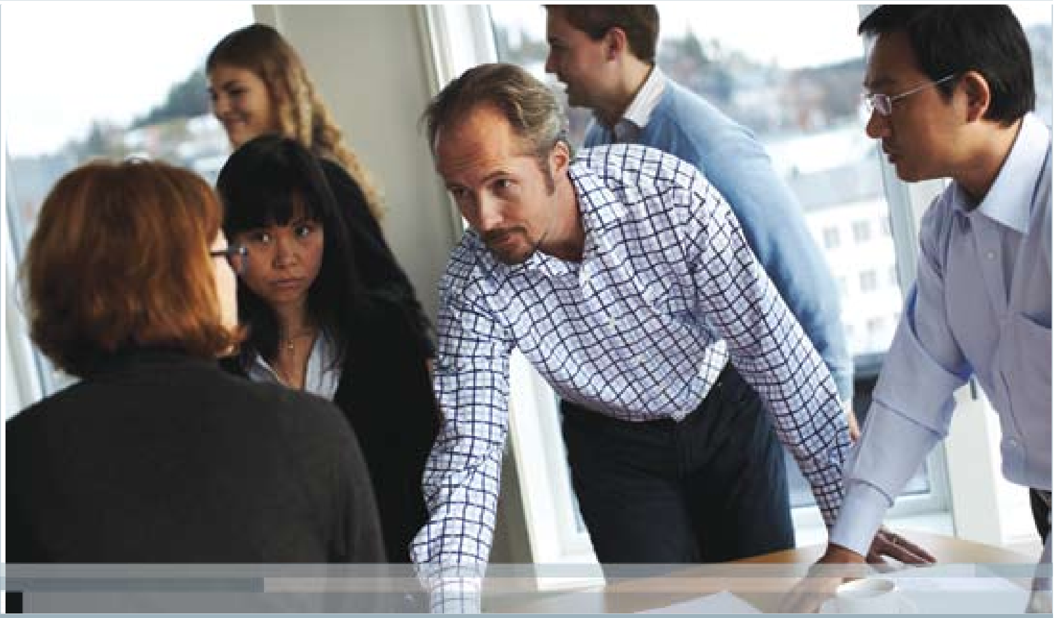
Most grouting rigs are electrically powered and their operation without electricity is limited. The power supply is therefore of critical importance during grouting. A power cut may lead to mixers standing still whilst filled with ready-mixed cement. Hoses may be left under full grouting pressure and access to couplings on rods above ground level may be limited. It is an advantage if the grouting rig is equipped with washing equipment that works at high pressure even if the rig is without power. Similarly, the work platform or basket should be capable of being operated without electricity (by diesel power). In general, it is important that the power supply should be subjected to maintenance and checks as per regulations. Depending on the conditions, the need for a back-up generator should be considered.

FAILURE OF COUPLINGS

Rapid release couplings deteriorate through repeated use and should be replaced when the locking mechanism becomes worn. Taps must be opened/closed at regular intervals to prevent them from becoming stuck.

Figure 29. Safety chain fixing allow rotation of the grout pipe. Make it possible to leave the packer in the hole, and remove the pipe (photo: CODAN/Ivar F. Andersen)

Figure 30. Certified lock bricks for Codan packers, 100 bars (left) and 60 bars (right) (photo: CODAN/Ivar F. Andersen)



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GROUTS

GENERAL INTRODUCTION

It is usual to distinguish between cement-based and non-cement based grouts. Cement-based products are the most common grouting material used for rock grouting in Norway. Non-cement based grouts are most frequently used in post-excavation grouting, when there is a need for an extra round of grouting, or in special cases such as in the event of major water inrushes, and are therefore described in Chapter 13 "Post-excavation grouting".

CEMENTS

In principle, all types of cement can be used for grouting, but the coarsest will have limited penetrability in fine fissures. Coarsely ground cements should therefore only be used to fill large fissures and voids.

If cement is to satisfy the functional requirements that apply to rock grouting, it must be mixed with different admixtures. This relates in particular to the requirement that grouts should be stable, they should flow readily and they should set as quickly as possible when they are placed in the rock.

STANDARD GROUTING CEMENT – INDUSTRIAL CEMENT

In the Norwegian Public Road Administration's Code of Process 025 standard grouting cement is defined as cements with a particle size of D95 > 20 µm. This is the most common grouting material, and the cement quality is known as industrial cement or rapid cement, equivalent to CEM I 42.5 RR. This cement typically has a Blaine of 400 – 450 and D95 < 40 µm. It should be noted that this cement has very many of areas of application, of which rock grouting is just one. Industrial cement is therefore not specially made for grouting.

This cement normally has too much bleeding at w/c ratios above 0.6 -0-7, and it is usual to start grouting with w/c = 0.8 or w/c = 1.0. Silica slurry is added to the grouting mix to prevent bleeding.

MICRO-CEMENT AND ULTRAFINE CEMENT

Micro-cements are ordinary cements that are extra

Type and size or fissure/opening (fissure material)	Typical Lungeon value	Grout mix
Open channels Karst (boulders/gravel)	>50	Standard cement CEM I 42.5 RR with addition of sand/gravel
Large fissures, > 1 cm opening (coarse gravel)	10 - 50	Standard cement CEM I 42.5 RR with addition of silica or P/expanding aggregate
Medium fissures 0.3 - 1cm (gravel)	3 - 15	Standard cement CEM I 42.5 RR with SP admixture and silica
Small fissures 0.01-0.1 cm (coarse to medium sand)	1 - 5	Micro-cements with SP admixture and silica
Minute fissures <0.01 cm (fine to medium sand)	< 1	Finest micro-cement with SP admixture /silica and/or silicates, epoxy, colloidal silica, polyurethane
Special cases	Running water	Consider accelerator/expanding admixtures or polyurethane. See Chapters 8(8) and 13

Table 4. Choice of grouts

finely ground. Micro-cement exists in different degrees of fineness. All types of cement which have D95 < 20 microns are regarded as micro-cement. In Code of Process 025 referred to above, ultrafine cement is defined as a material with D95 < 10 microns.

The most frequently used micro-cements in Norway have a D95 < 16 microns. It is important to be aware that there is a great difference between the properties of the different micro-cements. Some harden very slowly, whilst others harden rapidly. Before grouting is started it is important to know the properties of the chosen cement. When working with quick setting cement, it must be ensured that hardening does not take place in the grouting equipment. On the other hand, if a slow-setting cement is used there will be a longer wait before check drilling can be carried out.

Micro-cements have a large specific surface and hence are more chemically active (partly temperature dependent) than industrial cements, and therefore place greater demands on the mixing process in order to prevent flaking and reduced penetrability.

There is also a large difference in the stability of the different micro-cements. Some have practically no bleeding at all at a w/c = 1.0, whilst others may have more than 10% free water at a w/c = 1.0.

If a stable micro-cement is used, it is possible to grout with a w/c 1.0 without any danger of bleeding channels occurring. However, if the cement is unstable, measures to stabilise the cement should be considered.

STABILISING COMPOUNDS

To obtain a good grouting result, stability of the grout is important. Stability means that the cement mix is able to retain the water so as to prevent bleeding or separation, that is to say, prevent the cement from sinking to the bottom and free water from remaining on the top. If this takes place inside rock fissures, water-bearing channels will arise in the rock which may cause residual leakages. These residual leakages are extremely difficult to seal in a possible extra round of grouting. To reduce this problem, stabilising additives may be used. In addition to being stabilising, it is important that they do not noticeably increase the viscosity. If the viscosity becomes too high, this will have an adverse effect on penetrability in the fine cracks or fissures (high viscosity means a mass more resistant to flow).

In Norway silica slurry is most commonly used for this purpose. Silica slurry is a suspension of amorphous silicon dioxide of minute particle size dissolved in water. The silica slurry acts as a stabilising compound, so that it is possible to pump stable masses with higher w/c

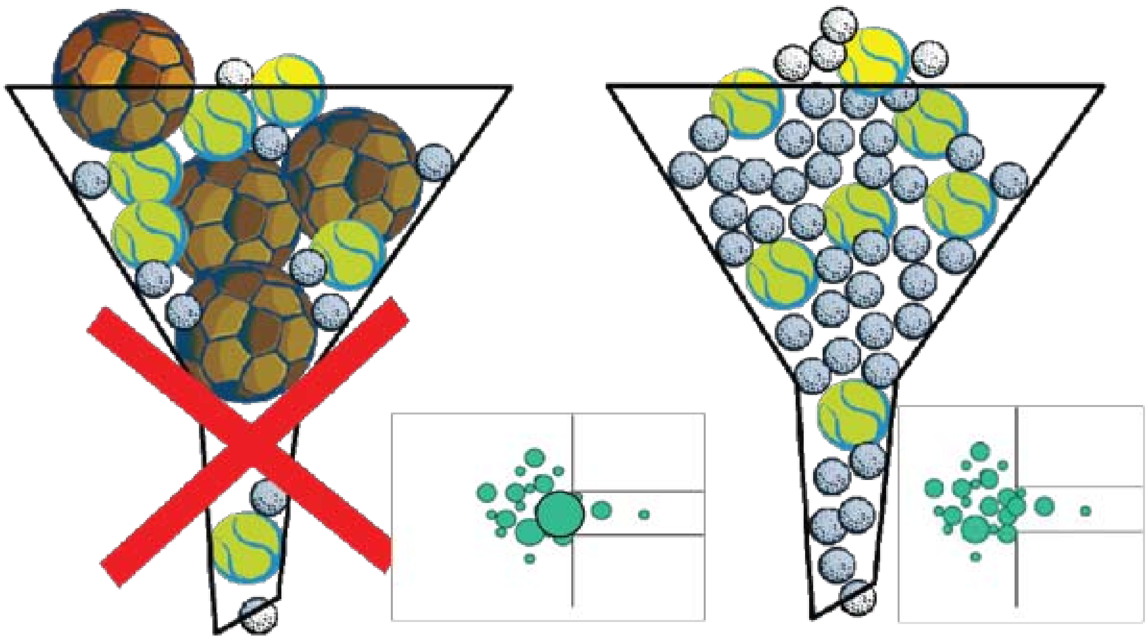


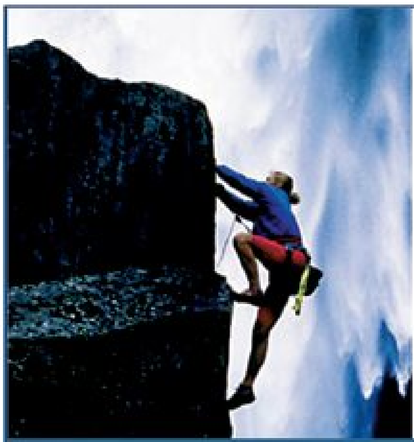
Figure 31. The coarse particles will prevent the small particles from penetrating into fine cracks. In the figure on the right there are no oversize materials and this results in better penetration.

ratios than could otherwise have been used. A normal dosage in most cases is between 5 and 10% of slurry on the basis of the cement weight, provided that a silica slurry of 50% solids is used.

In other countries bentonite is also used as stabilising additive. Bentonite is a clay with special properties which renders the cement mix stable and "smooth". The problem with bentonite is that its particles are in the form of small flakes. When they lie crosswise, these flakes have a larger diameter than the cement and may reduce the penetrability of the mix. Hardened grouting material with added bentonite also has lower shear strength than mixes with no added bentonite.

A common feature of all stabilising agents is that they retard the setting process, with the result that it takes longer for the grouting mix to stiffen. No matter what stabilising agent is used, it is important that it should have smaller particles than the cement. If it does not, it will assume the behaviour of oversize materials and prevent penetration (see Figure 31).

Figure 32. Operator moves packer.
Photo Svein Skeide, Statens vegvesen.



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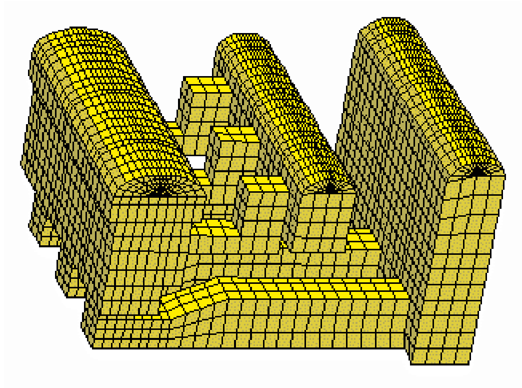

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SUPERPLASTICISERS – MIXING THE GROUT

Superplasticisers (dispersants) are used in most cement-based grouts. These substances are added to cause the cement particles to disperse and not clump together. Superplasticisers break the electrical bonds causing the clumps break down so that the end result is a finer mix with better penetrability.

- Normal dosage of superplasticiser is 1.5 – 2% of the cement weight.

To mix a grout, water and dispersant are first introduced into a mixer. Once this has been done, the cement is added and, if applicable, silica slurry. The ingredients must be mixed well, but at the same time it should not be mixed too long. If it is, the friction heat that is generated in the high-speed mixer might cause the cement to harden in the mixer. The mixer should therefore never be used as a storage tank.

- Recommended mixing time is normally 2 minutes.

On hot summer days with warmed grouting cement and hot water, the cement will react much faster than normal. The usual rule of thumb applied therefore is that that for each 10oC the temperature rises, the open time (set time) is halved.

As soon as the mix is ready it is passed into the final mixer (agitator). The agitator is simply a storage tank with a stirrer. When grout takes are small, a new mix should not be made before the agitator is almost empty, thereby ensuring the mixed grout is always as fresh as possible.

Water

For the grout to behave as anticipated, it is important that the water quality is good. Today it is usual to use recycled water (cleaned operating water) for many processes in underground works. Recycled water is also often used for grouting. However, the use of recycled water may cause a number of problems such as very fast setting and hardening of the grout. The recycled water often has a high pH value, a higher temperature and contains salts from explosives. It is the salts in particular that cause very fast hardening, with a danger of clogging the equipment and preventing penetration into drilled holes. If problems of excessively rapid hardening arise, the water quality should be checked. It may be necessary to adjust the pH value of the water, or in the worst case replace the recycled water with mains water.

ACCELERATOR FOR CONTROLLED CURING

Controlled curing is a fairly new resource in rock grouting. In cases where back pressure is not obtained after a given amount of grout has been pumped in, the addition of an accelerator is a solution that may provide pressure build-up. If this system is used properly, it can save large amounts of grout and pumping time. When a round of grouting is terminated with controlled cure time, the hole is "plugged". In most cases, termination at a desired pressure allows the drilling of control holes or new charge holes to be started more quickly. Controlled curing is a useful expedient when there are migrating leaks in and behind the working face, and with large grout takes.

An alkaline-free accelerator of a similar type as used in shotcreting is normally used for rock reinforcement.

The accelerator is added via a nozzle at the front of the rod (see Figure 16). This means that there is no accelerator inside the equipment and therefore no problems with hardening cement in hoses and equipment. To utilise controlled curing fully, it is advisable to use an accelerator pump that functions as the grouting pump's "slave". This will ensure that the dosage of accelerator is correct regardless of the flow rate of the injection pump.

It is important to be aware that controlled curing is not as effective with all cements. The best effect is obtained with some micro-cements. The combination of an efficient dosage system and micro-cement will make it possible to control the open time from 20 minutes down to about 3 minutes.

When using industrial cement with a low w/c ratio, accelerator may be used to expedite the setting process, but experience has shown that it is difficult to control the open time to the same extent as when using micro-cement.

EXECUTION OF INJECTION GROUTING

REQUIRED SKILLS AND COMPETENCE

Modern injection grouting using complex grouting rigs has become an advanced and specialised technique which calls for skills and an understanding of the work. It is therefore essential that key personnel who are involved in the injection work, whether in the employ of the construction project owner, the contractor or the consultant, have an in-depth knowledge of grouting procedures. All key personnel should be familiar with the measures and adjustments of the grouting scheme that will have to be implemented to obtain the goals of the grouting operation.

To ensure a common understanding of the purpose of the grouting and sealing strategy, it is a good idea to have a common grouting seminar for all personnel who are to participate in the grouting operation on behalf the owner and the contractor. In addition, regular technical meetings should be held between the contractor and the owner to summarise and follow up on findings. Possible necessary changes and improvements in grouting procedures are examples of what might be discussed at such meetings.

The contractor's supervision team must be skilled and experienced in grouting. The foreman in charge of the practical grouting work must either have up-to-date experience or be trained for the job. When rock conditions are difficult and when sealing off running water and performing post-excavation grouting, experience and basic knowledge of the behaviour of water in rock masses is required for the choice of drilling plan, recipes and stop criteria. When watertightness requirements are high, it is important that everyone involved is informed of these requirements and the consequences of departing from them.

When using a modern grouting rig, there must be one person to control mixers and pumps and keep a log of the

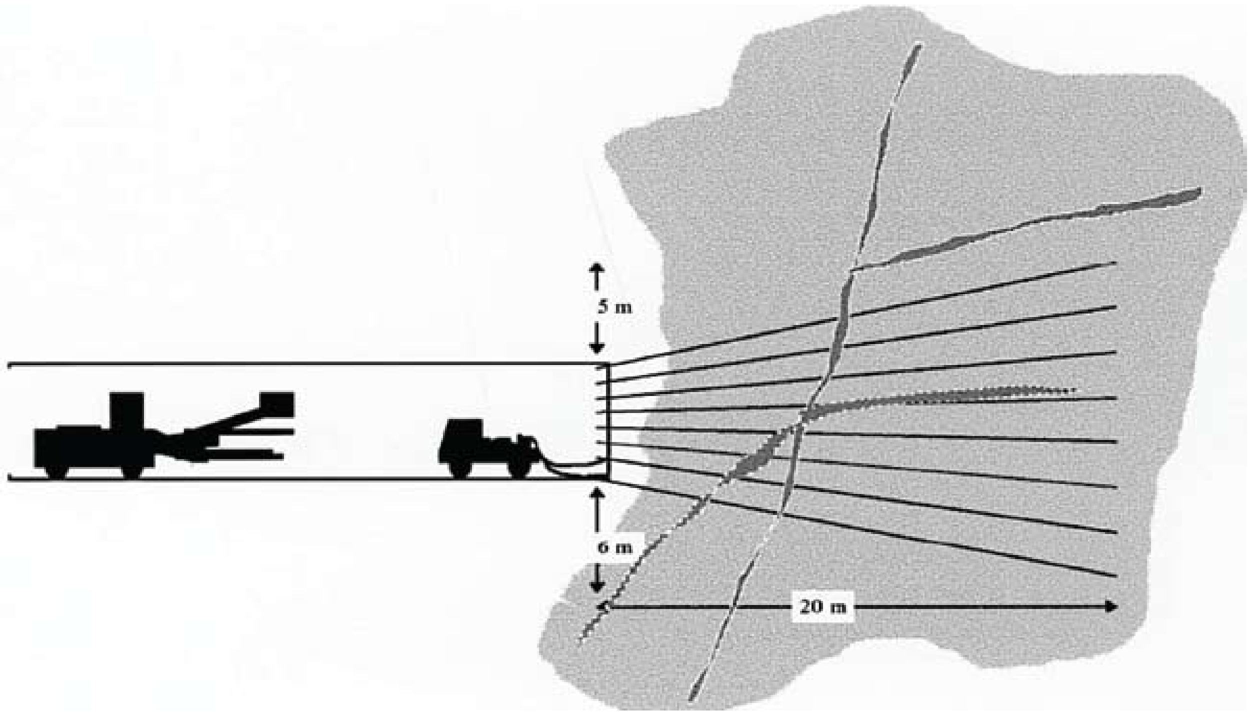

Figure 33. Illustration of the principle of pre-excavation grouting.

Grouting procedure

Jernbaneverket

Grouting work to be carried out in accordance with the following scheme:

1.) Recipe w/c = 0.8 Industrial cement Amount: approx. **300 litres**
2.) Recipe w/c = 0.6 Industrial cement Amount: approx. **600 litres**
3.) Recipe w/c = 0.5 Industrial cement Amount: approx. **600 litres**
4.) Recipe w/c = 0.8 as under point 1, with about 7% micro silica added.

If prescribed pressure is not reached after point 1, pumping to be carried out according to point 2.

If described pressure is not reached, injection into this hole should be terminated with a mixture of industrial cement and accelerator.

In the upper half of the cover it may be an advantage to try to terminate all holes wiht an addition of accelerator.

The injection rate should be limited to 40 litres/minute

If migrating leaks occur in the face, cement should be used that is mixex with accelerator or caulk.

The following pressures should be used:

Rock overburden	Boreholes in ceiling and walls	Boreholes in floor and in face
0-5 metres	20 bar	30 bar
5-15 metres	40 bar	60 bar
>15 metres	80 bar	80 bar

Figure 34. Example of grouting procedure from the Bærum Tunnel

production. In most cases this is the grouting foreman. One to two persons move hoses between the rods and make ready equipment during the grouting operation, whilst another one to two persons must take responsibility for the delivery of the material to the grouting rig. Proper routines and good communication are essential in grouting work. It is therefore an advantage that the work team is well established or has clear procedures for the job. It is important that the supervisor or grouting foreman and the owner's control engineer have discussed details of the grouting scheme such as adjustments of stop criteria and the sequences of holes.

In underground excavations where extensive grouting work is expected, the contractor's crew must have solid grouting skills. They must be able to refer to projects and grouting work experience of a similar type. Similar requirements must be made of personnel who are to plan, carry out and check shotcreting work.

GROUTING PROCEDURES

When drawing up grouting procedures there are a number of factors to be taken into account. First of all there are the stipulated inflow criteria, the size and location of the underground works, and the type of rock mass in which the excavation work is being done. The check list in Chapter 2 gives an overview over all elements involved in the grouting process.

In general, it can be said that there are two different grouting methods: systematic grouting and grouting as deemed necessary according to the actual site conditions.

In the following sections, chapters 9.3 through 9.7, short extracts from grouting procedures from some selected tunnel projects are given. Most are examples of grouting strategy involving systematic pre-grouting, but the example from the T-connection involves grouting as deemed necessary.

GROUTING PROCEDURE FROM THE BÆRUM TUNNEL, A DOUBLE-TRACK RAILWAY TUNNEL

The 6,5 km Bærum Tunnel runs between Lysaker and Sandvika on the west side of Oslo. Developer is Jernbaneverket (the National Rail Administration)

Data from the Bærum Tunnel:

- Rock type: cambrosilurian sedimentary slate, lime and sandstone rock with intrusive dikes
- Tunnel cross-section: approx. 100sqm
- Circumference: approx. 42m
- Number of holes: 63 holes in the crown + 10 holes in the face, giving a total of 73 holes
- Hole spacing at start about 0.67 metres
- Cover length: 23.5 metres
- Cover every third charge/approx. 15 metres and overlap approx. 8 metres
- Stipulated inflow criterion: 4 litre/minute and 100 metres
- Results measured once tunnel blasting completed: approx. 2.0 litre/minute/100 metres

Figure 35. Support in progress. Photo Svein Skeide, Statens vegvesen.

PROCEDURE FOLLOWED IN THE EXCAVATION AND GROUTING OF A ROCK CAVERN AT MONGSTAD

Mongstad is situated on the Norwegian west coast, some 50 km north of Bergen. Mongstad is a hub in the Norwegian oil and gas processing industry, covering landfalls, refinery, storage facilities and export facilities. The actual cavern is constructed for
storage of liquid propane at atmospheric pressure in an unlined rock cavern. A design criterion requires the inflow into the whole cavern to be less than 15 litres.

The rock cavern has the following dimensions:
Height: 33 metres, Width: 21 metres, Length: 134 metres
The floor lies at 83 metres below sea level
(The North Sea is immediately outside the cavern portal.)

The bedrock consists of light and dark anorthositic gneiss, amphibolite and gabbro.

- Systematic pre-grouting with a cover length normally of 24 metres
- Maximum distance between boreholes in the grout cover should not exceed 2.2 metres at the end.
- Control hole round after every grout cover
- Cover geometry was designed so that the rock mass should be as watertight as possible 8 metres outside the theoretical blasting profile.
- End pressure for grouting was set at 80 bar.
- 30 000 metres of grouting holes were drilled
- About 410 tonnes of micro-cement was used.
- After the cavern was finished, a total inflow of 2 litres per minute was measured.
- 4000 metres of holes were also drilled for water infiltration above the hall.

1. Excavation of ceiling section – pre-grouting around the ceiling and wall down to about 2 metres above the first bench
2. Excavation of first bench, pre-grouting of walls.
3. Excavation of lowermost bottom bench, pre-grouting of lower part of the walls and the floor.

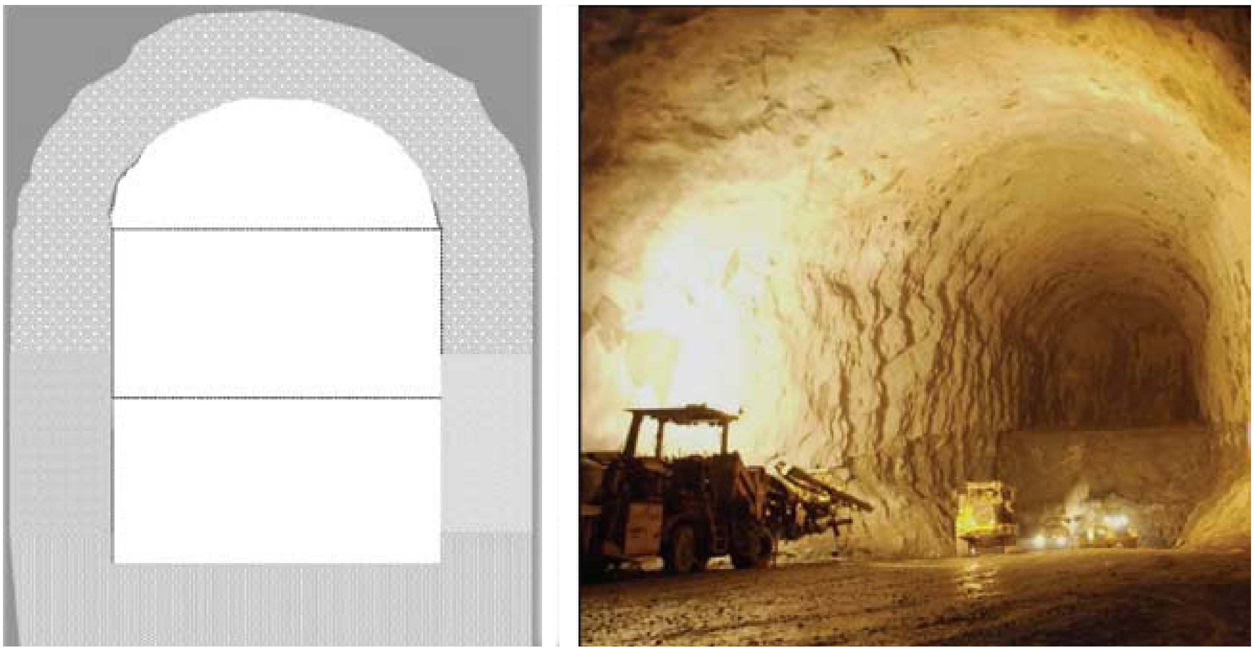

Figure 36. Excavation and grouting of the propane storage cavern at Mongstad.

EXAMPLE OF GROUTING PROCEDURE FROM THE T-CONNECTION (SUBSEA TUNNEL)

The T-connection is a new east-west connection between European Highway 39 in Tysvær and main Rv. 47 road on Karmøy, and a new north-south connection between Fosen and European Highway 134 in Haugesund.

The project includes two subsea tunnels with a T 11.5 profile. One of them is 3977 metres long and runs under Karmsundet, with a maximum depth of 139 metres b.s.l., whilst the other is 3764 metres long and runs under Førresfjorden, with a maximum depth of 136 metres.

Developer is National Roads Authority, Region West.

The functional requirements of the tunnel set the criterion for the acceptable amount of water inflow. The procedure employed therefore is to drill four 28-metre-long probe holes, two in the shoulder and two in the floor. Grouting is carried out when water inflow in excess of two litres per minute from individual holes or more than 5 litres per minute in total is measured.

Note to contractor: Holes for grout cover should be drilled as a standard cover of 36 holes. Length 24 metres. All holes should be grouted at a pressure of up to 60 bar. If amounts of more than 2000 litres per hole are used, the owner should be consulted. A Logac* report should be presented when grouting has been completed. Industrial cement of a normal recipe should be used, starting with w/c 0.8. After 500 litres, change to w/c 0.6. When grout feed holes are large, caulking should be considered. At regular intervals

Figure 37. Injection in progress. Photo Svein Skeide, Statens vegvesen.

NRPA Region West	Project 300141	Profile No.	No.

CONTROL REPORT

PROJECT: T-connection

Doc. Date:	Rev. date:	Responsible for doc.

Contract:	K1 Håvik – Mjåsund – Hellevik **Mjåsund**
Contractor:	AF Gruppen
Owner:	NPRA Region West
Subject:	X Technical quality ☐ Contract basis ☐ HSE ☐ Orientation ☐ External environment ☐ Other ....
Title:	**Grouting Mjåsund**

Note to contractor: Holes for grout cover should be drilled as a standard cover of 36 holes. Length 24 metres. All holes should be grouted at a pressure of up to 60 bar. If amounts of more than 2000 litres per hole are used, the owner should be consulted. A Logac* report should be presented when grouting has been completed. Industrial cement of a normal recipe should be used, starting with w/c 0.8. After 500 litres, change to w/c 0.6. When grout feed holes are large, caulking should be considered. At regular intervals

Figure 38. Example of grouting procedure from T-connection.

**) LOGAC*
The LOGAC system is a computer based recording system for sampling and storing of data during grouting operations.
The logged parameters are
- *flow*
- *pressure*
- *volume*
- *time and real time.*

The system is designed for field operation and the logged data are stored on a PC card.

Drilling Plan established by the contractor

Standard charge report

Project

Firm	AF Anlegg
Site	T-connection
Tunnel	Hellevik
Face	Hellevik
Note	Grout 35 holes T 11.5

Charge data

Drilling metre (m)	980,001
Area contour/at start (m^2)	92,266
Area contour/bottom (m^2)	92,266
Area borehole at start (m^2)	87,760
Area borehole bottom (m^2)	336,688

Number of holes

Grout	35
= Total	35

Graphic presentation

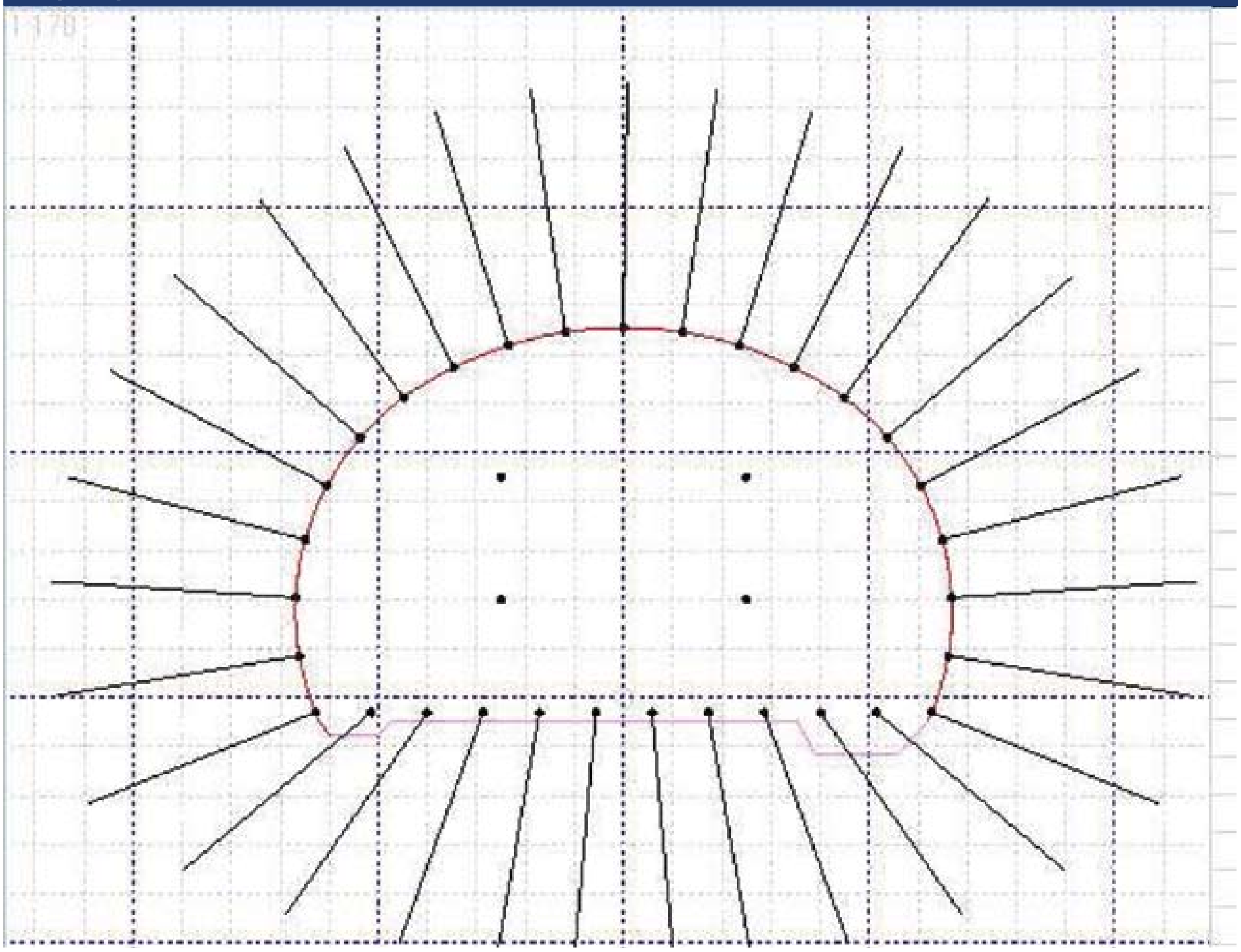

Figure 39. Example of a drilling plan from the T-connection

GROUTING PROCEDURE FOR THE LØREN TUNNEL (HEAVILY TRAFFICKED MOTORWAY)

The Løren Tunnel runs between Økern and Sinsen on Ring Road 3 in Oslo. The tunnel is 1200 metres in length, with 915 metres running through rock and 300 metres running through concrete culverts at the entrances. The Løren Tunnel has two tubes, each with three lanes and access ramps, giving a theoretical blasting profile with a span of up to 15 metres. A closely built-up area lies above the tunnel and inflow criteria are stringent, which means that there is systematic pre-grouting along the whole extent of the tunnel.

Hole length 23 metres
Penetration depth 6 metres outside the theoretical profile in the ceiling and floor and 5 metres outside the theoretical profile in walls
2 rounds between each curtain
Charge length 5.5 metres (full charge length)
Number of holes = 44
Number of holes in profile = 37
Number of holes in face = 7

Statens vegvesen

Procedure using standard grouting cement
for the Løren Tunnel (industrial cement)

- The end pressure for injection grouting should be no more than 80 bar.
- Where grout takes are small (< 100 litres), the grouting pressure should be increased to about 80 bar and maintained for five minutes before any grouting is completed in that hole.
- Where grout takes are large (> 1500 litres), grouting can be terminated at an end pressure of 30 bar.
- The following recipes are used primarily for grouting:

Recipe	Water/cement ratio	Dispersed microsilica in relation to dry weight
3	1.0	10%
5	0.8	10%
6	0.7	5%
8	0.5	No silica slurry

- Always start grouting with recipe 3.
- A change of recipe is made according to the following principle.
  - if there is a grout take in one hole of 680 l of recipe 3 without pressure build-up, change to recipe 5.
  - if there is a take in the same hole of 345 l of recipe 5 without pressure build-up, change to recipe 6
  - if there is a take in the same hole of 330 l of recipe 6 without a pressure build-up, change to recipe 8.
  - if end pressure is not reached after 600l of recipe 8, terminate grouting.
- If there is a gradual build-up of pressure without end pressure being reached, consider using the same recipe beyond the given amount. If migrating leaks occur, try changing to a thicker recipe before the given amount is reached. An evaluation of whether it is necessary to change recipe must be made in each individual case.

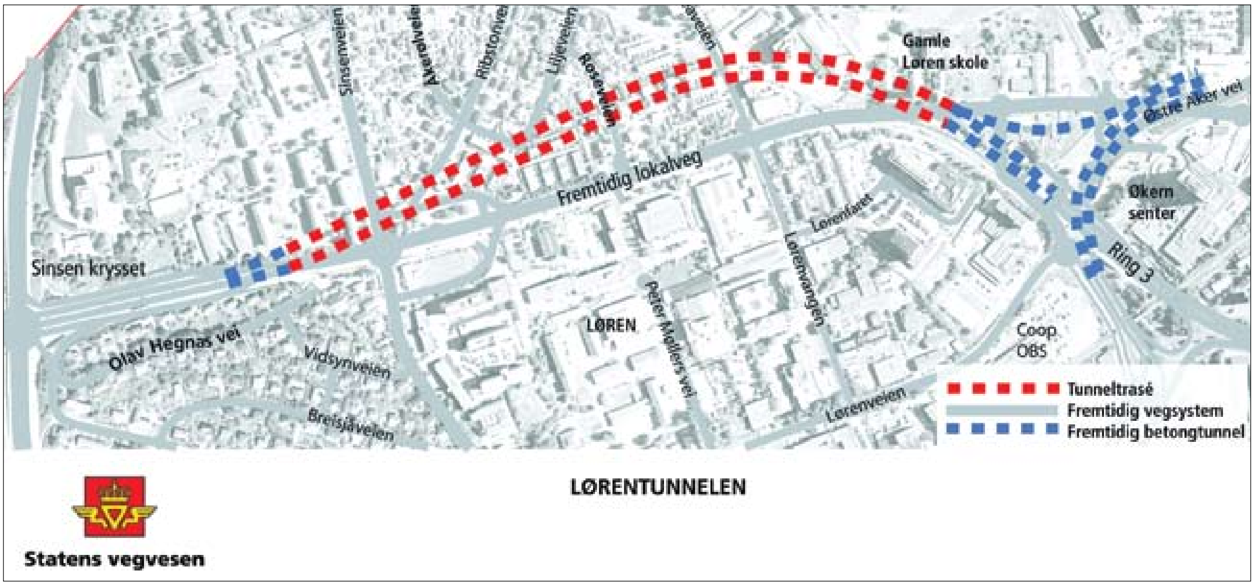

Figure 40. Ringroad 3 has been under reconstruction during decades. Now the challenging Løren section is on including tunnelling.

Statens vegvesen

Procedure using micro-cement for passing deep trench in the Løren Tunnel

Start procedure

- Number of holes for the cover: 89 (55 in the profile, 19 in the face + optional cut-off curtain 15)
- 15 19-metre long holes with a penetration depth of 5.5 metres, located in centre ceiling, are drilled first. MWD data from these holes are transmitted to the owner who assesses whether the holes are to be grouted immediately as a cut-off curtain, or whether they should be included in a full cover.
- If there is no need for a cut-off curtain: Holes for the rest of the grout cover are drilled and grouted. In this case no intermediate holes are drilled in the ceiling.
- If there is a need for a cut-off curtain: 15 holes in the ceiling are grouted in a separate round. Once the cut-off curtain has been established and has hardened, the remaining holes in the cover are drilled, including 16 intermediate holes (inner cover) with a length of 15 metres and penetration depth if 3.5 metres.
- Drilling of grouting holes is allowed in the floor before the need for separate grouting of a cut-off curtain been clarified. Other grouting drilling is not allowed before grouting agent in cut-off curtain has sufficiently hardened.
- Number of face holes in standard cover is initially set at 20 holes distributed over the cross-section with diminishing penetration depth towards the centre of the face. If poor quality rock is detected during drilling of the cover, it may be appropriate to increase the number of face holes.
- If loose masses are encountered during drilling for grouting, the owner must be contacted immediately and drilling in the ceiling stopped.
- Before grouting rods are positioned, packers are prepared in such a way that by allowing the tap to remain open it is possible to identify holes which have internal communication.
- All holes are injected with micro-cement.

Grout pumping of micro-cement should be carried out according to the following procedure:

1. Micro-recipe no. 9 – w/c = 1.0 Amount: up to 340 litres
2. Micro-recipe no. 11 – w/c = 0.8 Amount: up to 340 litres
3. Micro-recipe no. 14 – w/c = 0.5 Amount: up to 800 litres (340 litres in cut-off curtain)

- Max pressure in the ceiling should be 60 bar. Max pressure in the floor 80 bar.
- Controlled curing should be implemented if pressure build-up of more than 30 bar is not obtained at max grout take of 1500 litres.
- Controlled curing should be implemented in the cut-off curtain if pressure build-up of over 30 bar is not obtained at max grout take of 1000 litres.

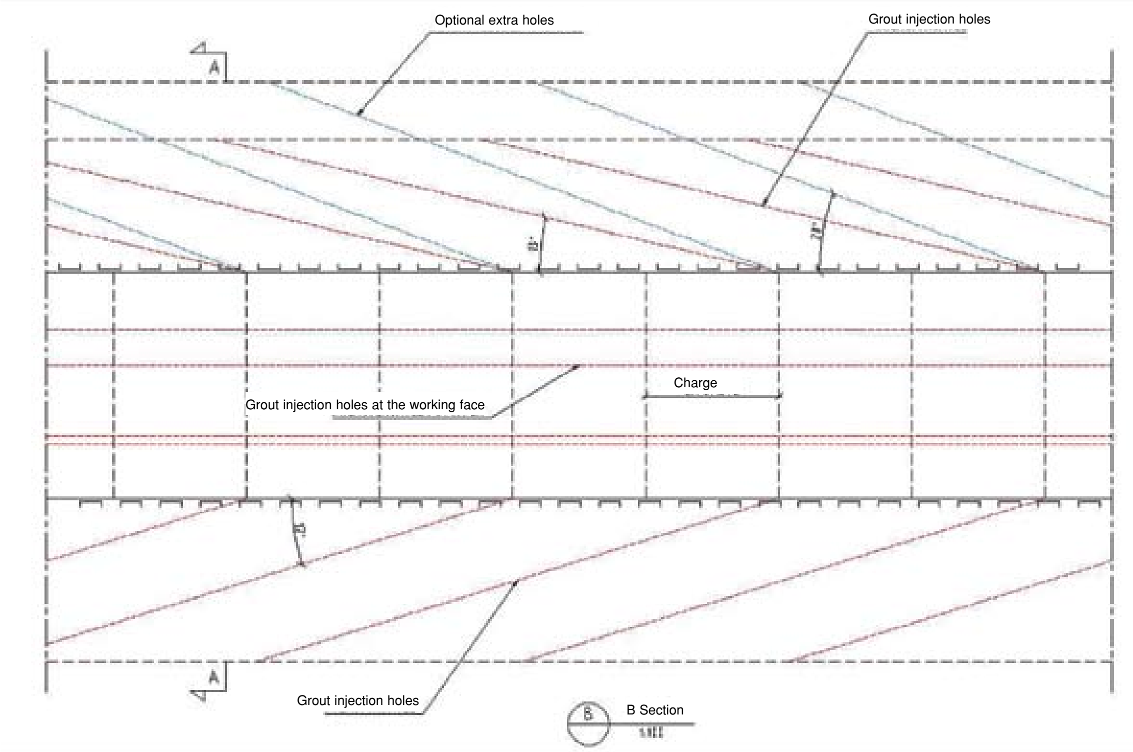

Figure 41. Longitudinal section of grout covers for every other charge in the Løren Tunnel.

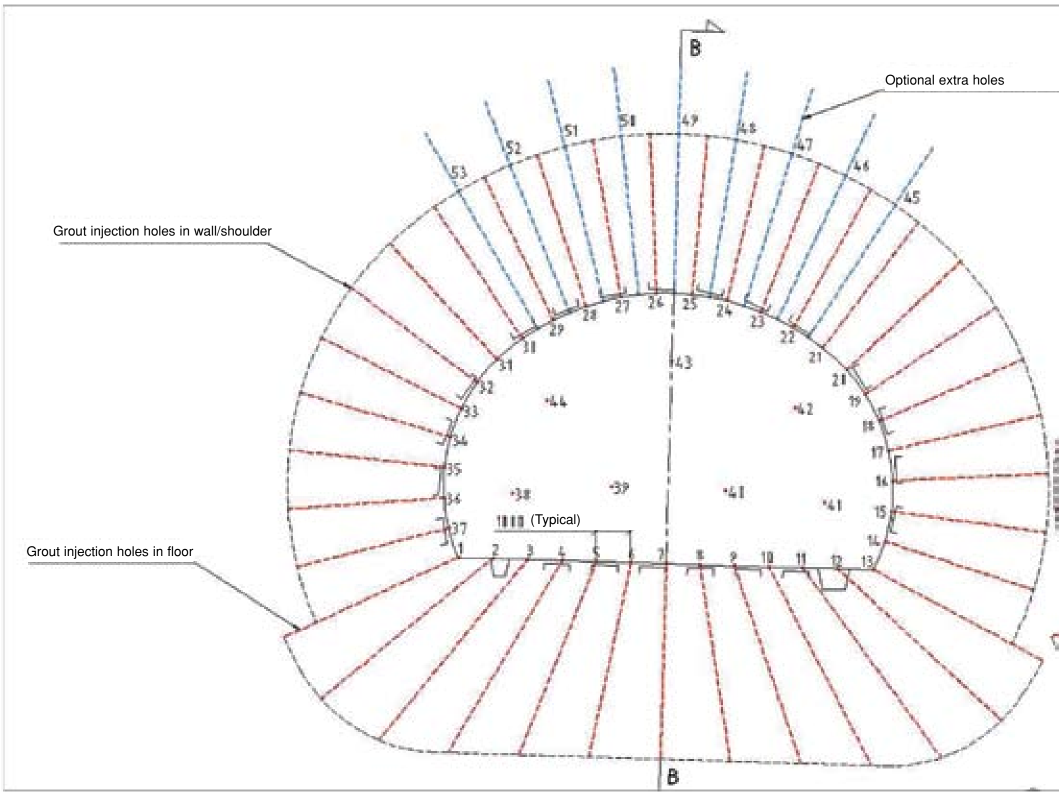

Figure 42. Example of a drilling plan for T9.5 profile of the Løren Tunnel

GROUTING PROCEDURE FOR THE GEVINGÅSEN TUNNEL (SINGLE-TRACK RAILWAY TUNNEL)

Jernbaneverket

The Gevingåsen Tunnel runs between Hommelvik and Værnes (some 30 km north of Trondheim in the middle part of the country). The project is part of the modernising the northbound railway line. The tunnel is 4400 metres long and has a theoretical cross-section of about 70m².

1. Criterion for starting grouting

Grouting is normally started when water inflow for one or more probe or charge holes exceeds the criterion given in the contract. In most parts the criterion is 15 litres/minute with 5 litres/minute in some exposed areas. Total inflow and assessment of the leakage situation in the area will also be of importance in deciding when grouting should be started.

2. Drilling holes for the grout courtain

Standard: Hole length 24 mc/c 1.2 m
Penetration depth 4-5 metres out in the ceiling and walls, 5-6 metres in the floor
Optionally an additional 3-9 holes are drilled in the face as indicated by the controller
Holes from probe drilling should also be grouted.

Normally, holes for a full cover should be drilled, but in the case of concentrated leakage on one side, the owner may decide that only holes for parts of the cover should be drilled.

If there are problems in reaching a hole length of 18 metres, the length can, on agreement with the control engineers, be reduced slightly.

If there is a great deal of leakage it may be necessary to drill some extra holes. This must be assessed during drilling.

3. Injection

Injection should be started at the bottom of the profile. The grouting work should be organised so that a grouting round, once started, will not be stopped until it has been completed, unless otherwise agreed with the owner.

As agreed, micro-fine cement is used (contract's process 81501.31-632). The type of cement can be changed based on observations made during the grouting operation.

Grouting should be carried out according to the following scheme:

1. Recipe No. 9, ie, 200 kg micro-cement and a w/c of 1.0
2. Recipe No. 10, ie, 220 kg micro-cement and a w/c of 0.92
3. Recipe No. 11, ie, 250 kg micro-cement and a w/c of 0.80.

Grouting should be carried out at a pressure of 70 bar.
In view of the prevailing rock conditions at the location and observations made during the grouting, a reduction in pressure may be considered.
The pumping rate should be limited to 40 l/min.

Grouting is commenced using recipe no. 9 (200 kg and a w/c of 1.0). After a grout take of 500 litres in each hole without a rise in pressure to 70 bar, grouting continues with recipe no. 10 (220 kg and a w/c of 0.92). After a take of 500 litres in each hole without a pressure rise to 70 bar, the grouting should also continue with recipe no. 11 (250 kg and a w/c of 0.80) until either a pressure of 70 bar or total grout take of 2000 litres is reached.
If the described pressure is still not reached using recipe no. 11, controlled curing should be implemented.

In the case of migrating leaks in the face, an attempt to stop the leak should primarily be made by using a low w/c and by allowing the hole to "rest" whilst grouting of other holes continues before returning to the hole that did not have full pressure build-up. If this is not sufficient to stop the leak, grout with controlled curing should be used.
When terminating because of pressure build-up, the remaining mix can be diluted for use in the next hole.

The volume of each mix pumped in must be assessed by the person or persons carrying out the grouting together with a control engineer during grouting, and adapted to the conditions prevailing at any given time (rock type, fracturing etc.)

Controlled curing
Controlled curing is used if there are migrating leaks at the face or behind the face, or if it is suspected that the grout is running out on the surface.

Controlled curing agent is otherwise never added before grouting has been carried out as described in points 1, 2 and 3 above (ie, recipes 9, 10 and 11).

4. Extra waiting time
Extra waiting time in addition to one hour's downrigging will be ordered specifically by the owner in each individual case.

5. Control holes
Possible control holes may be ordered by the owner in each individual case. The owner may also order water loss measurement in connection with the grouting operation. Control holes and holes for water loss measurement should not be grouted unless this has been agreed with the owner in each individual case.

6. In the event of a puncture of the curtain
If the curtain is punctured during drilling of control or charge holes, the hole should be plugged with packers (the necessity of this can be assessed depending on how much material is leaking) and a waiting period of another hour should be implemented before drilling continues. Modifications of the given procedure may be implemented by the contractor in cooperation with the control officer.

START-UP HOLES AND INJECTION SEQUENCE

It is common practise to start by grouting the floor holes and then continue progressively upwards towards the ceiling. Even in a situation where, for example, there is a lot of leakage in the shoulder, it is recommended that grouting should be initiated at floor level. One reason for this is that the floor holes are particularly difficult to reach later, and so it is wise to deal with them before they are affected from other holes. It has been found that floor holes with residual leakage are very awkward to grout post-excavation.

Another reason for starting with the floor is that the cement mix is heavier than water, and so by starting at the bottom, gravity is utilised. Water is "squeezed" forward and upward, and cement is injected in, which is often evident by the use of less cement in the uppermost holes.

Earlier it was not usual to set packers in the holes before they were to be grouted and so it was easy to see if there was a connection between some holes. If some holes are found to be connected, these holes should be grouted first before reverting to the original procedure. The reason for this is that there are two or more inlets to the same leakage system. By dealing with these holes one after the other, it can be ensured that the cement does not set and prevent the complete filling of the water-bearing fissure system.

With today's high-pressure grouting, packers should normally be set in all holes that are drilled before pumping of the grout is commenced. The reason for this is the consideration of safety and HSE requirements. When there is nothing but cement is in a hole, it functions as a lubricant and there will be poorer friction between packer and rock. This may lead to the packer sliding out when pressure is applied, thus creating a dangerous situation.

This can be avoided in two ways. One is to flush out the cement in the hole before the packer is placed. This is done most easily with a rigid PVC hose that is passed in and out of the hole.

Another way is to insert a small pin into the valve on the packer or use packers constructed to allow water to run through them before they are grouted (see Figure 43). In this way, the packer remains open, making it possible to see whether there is a connection. When the packer is to be grouted later, the pin will be pumped into the hole, and the packer will work as normal.

REQUIRED CURE TIME

In systematic pre-grouting in underground works with moderate water pressure there is normally no need for extra cure time. In structures with large cross-sections, the time it takes from the start of grouting in the floor holes until charge drilling can start in the same part of the face will be sufficient time for setting/hardening. In some cases, for example, at low temperatures, a possible need for extra cure time can be met by using faster setting cements.

Figure 43. Packers that are open and allow water to pass through before they are grouted.

Figure 44. A modern grouting rig shall be complete, self-contained and prepared for easy transport to the next place of operation. Photo AMV.

Under normal circumstances standard grouting cement has a set time of 8 to 10 hours, whilst a fast setting micro-cement under the same conditions sets in about 2 hours.

In underground installations where the sealing strategy is based on grouting as deemed necessary, situations may arise involving inrushes under high pressure. In such cases there will be a need for extra cure time after grouting has been carried out before excavation can continue.

CONTINUOUS INJECTION

Normally a round of grouting that has been started should be completed as a continuous process. It is recommended that the grouting work should be planned so that the whole grouting procedure can be completed in one operation.

Working time regulations governing noisy operations may pose a problem which restricts the flexibility of efficient production in underground works in residential areas. In works with large cross-sections and extensive grout covers (> 70 holes), in order to make best use of a 24-hour period for non-noisy activity, the cover has been split up. Satisfactory results have been obtained by drilling the lower part of the cover to completion first and grouting from the bottom up, and then starting the drilling at the top. It is always important that all drilled holes are grouted before interrupting the grouting work to drill the remaining holes. If this procedure is not followed, there is a risk that holes will be partly filled with grout because of a through opening between them, and that these holes will be lost.

GROUT VOLUME AND INJECTION PRESSURE AS STOP CRITERIA

Volumes and pressures are normally given in the specification that is drawn up by the construction project owner. There are normally two stop criteria:

- Given pressure
- Given volume

In all rock grouting the aim as a rule should be to reach described back pressure in all holes. This means that w/c ratio, the flow rate and the injection pressure must be adjusted during the grouting operation. Sometimes it is extremely difficult to reach back pressure and the grouting must be stopped after a maximum volume has been injected. For a 21-metre long grout curtain of 30 to 40 holes, a typical maximum volume is 1000-1500 kg cement per hole. It is recommended that the curtain should be seen as a single unit, so that more than the given stop criterion in some holes can be allowed if grout does not run into other holes. In certain "problem holes" it may be a solution to allow the hole to rest and then carefully resume pumping.

In some cases it has been found that grouting at high pressure from the start gives better penetration in finer fissures.

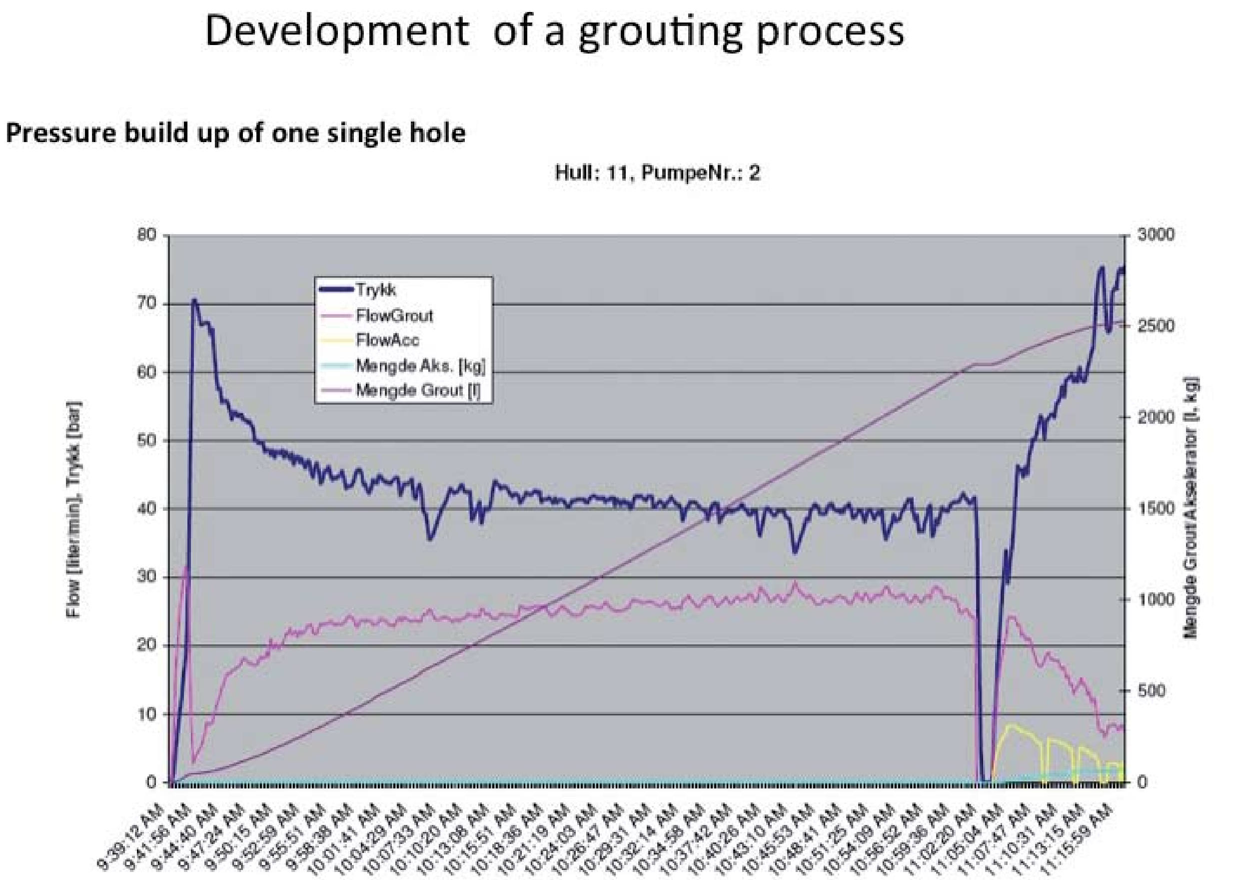

Figure 45. Observations during the grouting of one hole. Total grout take 2500 kg, typical grout flow around 25 kg/min and max grout pressure of 80 bar. The observation period was 1 hr:36 min: 47 sec.

In any case, it is especially important to observe what happens in a start phase of the grouting work so that execution and the stop criteria can be adapted to the conditions.

Another alternative or supplement to stop criteria based on pressure reached or maximum volume may be to use controlled cure (see Chapter 8.8).

Permissible grouting pressure will vary from a few bar up to 100 bar, depending on the overburden, in-situ rock tensions and rock quality. This means that the grouting pressure may exceed the rock tension with a danger of subsequent splitting or jacking of the rock mass. Grout penetrates on a fissure plane and may exert substantial forces. It is important to be aware that grouting in some cases can also lead to a lifting of the bedrock. Surface-parallel fissures can be filled with grouting material and result in a shifting of the rock surface.

THE GIN METHOD

When specifying maximum terminating pressure and maximum cement take per packer placement, it may be sensible in some conditions (highly porous conditions) to combine grout take per drill metre with pressure, a combination that is given as a "grouting intensity". This concept was first introduced in international grouting work by Lombardi (1993) and is called the GIN method. GIN stands for Grout Intensity Number and is the product of the pressure (in bar) and volume of grout injected (in litres per drill metre). See Figure 46.

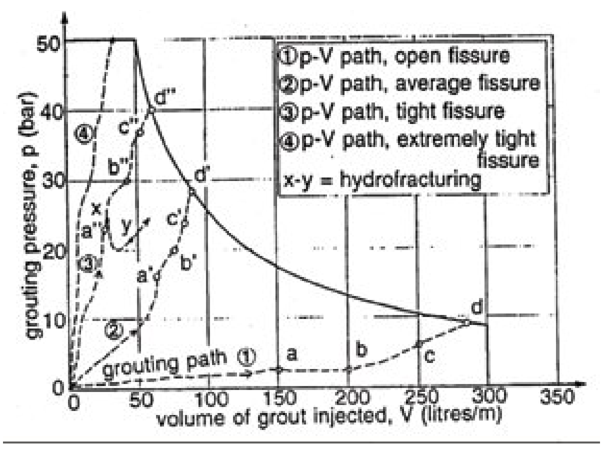

Figure 46. Graph showing the principle of the use of the GIN method

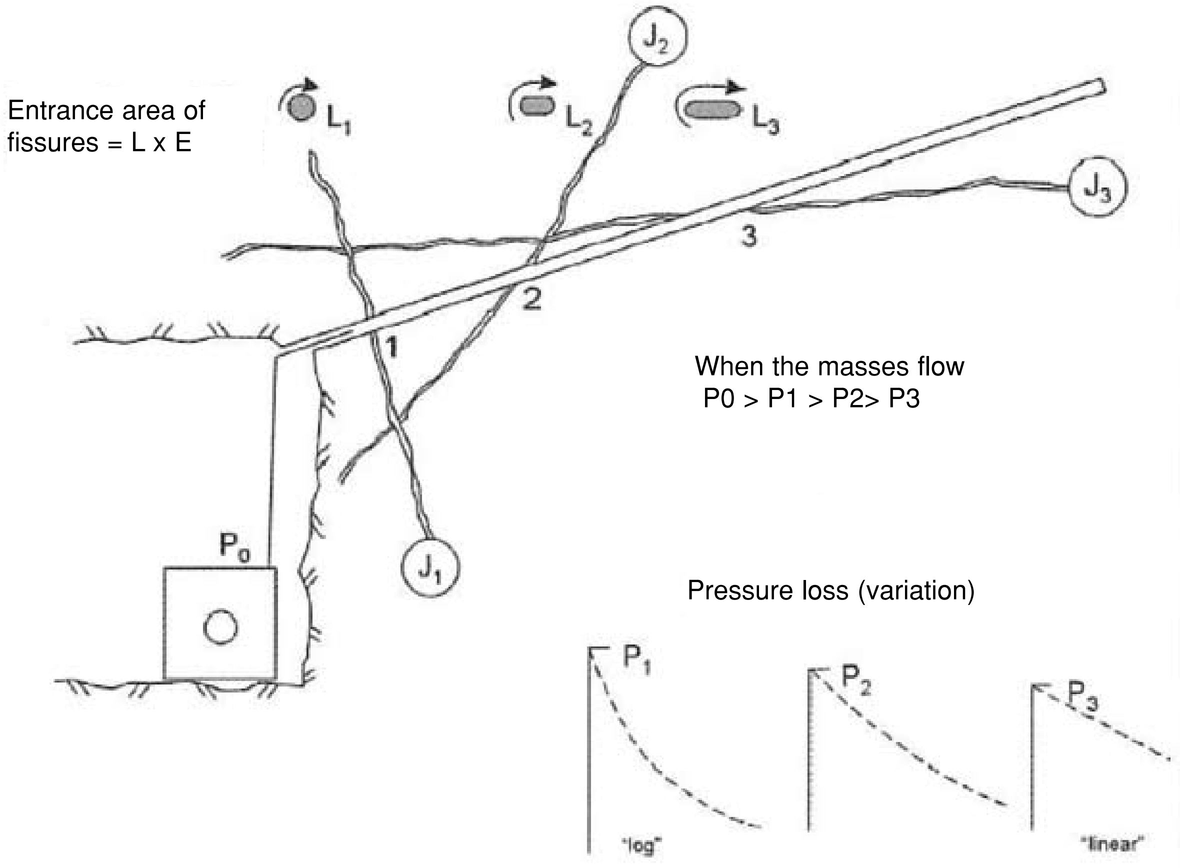

Figure 47. Principle showing pressure loss in relation to distance from the pump.

The purpose of the GIN system is to adjust the radius of influence of the grouting to the borehole pattern used. For Norwegian works, the selected GIN number will vary depending on geological and hydrogeological conditions, watertightness requirements and grouting method.

JACKING

The grout will flow into fissures and channels which are in communication with the grouting hole. As pressure increases, the grout might exceed the tensions in the rock and cause jacking of the rock, thereby opening new channels. This can be observed by following the pressure build-up during the grouting process. The pressure builds up gradually and then remains stable at one level. An abrupt fall in pressure or a sudden increase in flow rate will indicate that the grout has found new channels and voids. This may be a desired effect where there is a lot of clay in the fissures and/or there are problems in introducing grout into the water-bearing channels. In other cases, jacking may lead to undesirably large grout take. The right grout pressure must therefore be assessed on site on the basis of the findings made.

GROUT PENETRABILITY

The penetrability of the grout is related to its viscosity, particle size and pressure. In the case of large fissure volumes, high viscosity (low w/c ratio) can be used in combination with high pressure. As a consequence of the high viscosity, the grout will have substantial resistance to penetration, and since the pressure decreases with the distance from the pump, the grout will barely penetrate into the rock.

COMMUNICATION AND CHANGE OF SHIFT

Proper routines for ensuring good communication are important in grouting work. A successful round of grouting depends on the operator being able to control and maintain an overview over how much of each grout is used in each hole at any given time. This depends in part on how well designed and efficient the grouting rig is. Notes in a suitable form are crucial for maintaining an overview and providing a change of shift status report.

Figure 48. An example of migrating leaks at the surface

GROUT LEAKS

The use of high pressure during grouting may result in the grout being transported far from the borehole. It is therefore necessary to conduct inspections in areas where leaks into the ground may occur. In caverns very close to other caverns, pipelines, structures, or to the surface, it is necessary to establish inspection routines in order to detect possible grout leaks. Such areas should be mapped before the grouting starts. During the grouting procedure it is important that there is communication between the controller at the surface and the crew at the face so that pumping can be stopped immediately when leaks are observed. It is essential to implement clean-up measures before the grout sets

MEASURES TO PREVENT GROUT LEAKS

In the event of fissure patterns with an unfavourable orientation, insufficient overlap between grout covers or in structures where grouting is sporadic, grout leaks may enter the rock cavern.

To reduce problems associated with such migrating leaks at the face the following measures are appropriate:

- Use of longer rods /place packers further in
- Stop leaks with wooden wedges
- Use caulking cord or black cement

For underground works with anticipated grouting needs, and where two parallel tunnel tubes or rock caverns are to be driven in close proximity to each other, the grouting work should be planned so that the work in one passage does not disturb the excavation of the other. In the case of parallel tunnels, one of the faces should be about 50 metres ahead of the other.

The problem of grout leaks behind the face will be less prevalent in systematic grouting than in sporadic grouting. It is in any case important to carry out inspection rounds back along the tunnel in order to check for signs of outward pressure/destabilisation.

In portions where pre-bolting (spiling) is used, the rock mass should normally be grouted before drilling for the pre-bolts is carried out. If an exception must be made, preliminary bolts must be grouted prior to injection grouting in order to prevent grout leaks.

Leaks may occur along radial bolts even though the bolts are grouted. If this becomes a recurring problem during systematic grouting, the design of the grout cover should be altered so that puncturing with bolts is avoided. To stop leaks in bolt holes, safety bolts with a packer of the "Thorbolten"* type may be used.

*) This is a packer connected to the rock bolt that is designed for balanced grouting pressure exceeding the existing water pressure.

CHECKING AND DOCUMENTATION OF GROUTING OPERATIONS

DELIVERY AND CHECKING OF GROUTING MATERIALS

It is important that cement used for injection grouting is fresh. If the cement is old, it will often give a poor result. If there is any doubt about the quality of a cement, it may be wise to compare its set time and bleeding with that of fresh cement. Old cement is usually slower to set and has more bleeding than fresh cement.

When it is expected that grouting injection will have to be carried out in underground works, agreements should be entered with suppliers capable of delivering fresh cement at short notice. It is also important to have a supplier who is able to deliver the right quantity at the right time. A good supplier will, in addition, be capable of performing quality checks of the products.

Bags of cement that are delivered for injection grouting should have the production date stamped on the bag. Furthermore, it should also be possible to trace the cement to its production in order to see the results of routine check tests. The supplier should be able to provide a production certificate for each delivery. This certificate ought to contain data such as set time, bleeding, Blaine and sieve curve.

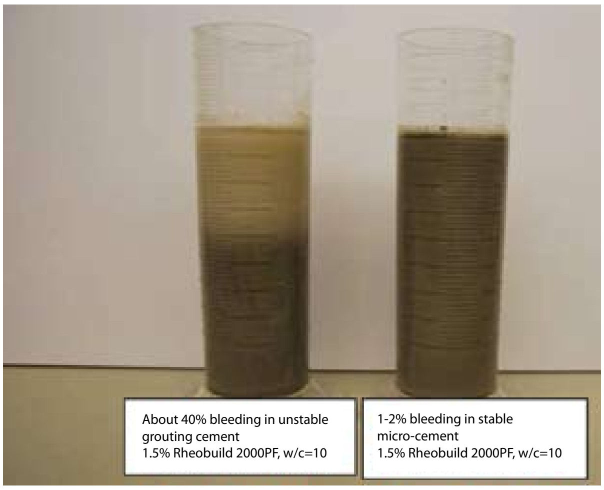

Figure 49. Photograph illustrating the testing of cement bleeding

TESTING OF CEMENT BLEEDING

When cement is tested, it is very important to mix it well. An example of suitable equipment is a common household hand blender. Use a w/c of 1.0. When the grout has been mixed thoroughly, it should be emptied into a cylinder. After 2 hours it can be seen how much free water there is at the top of the cylinder. This water is bleeding water. If much more bleeding is observed in the older cement than in the fresh cement, this is an indication that the cement is too old.

Unusually excessive bleeding indicates that the cement is too old. Do a comparative test with fresh cement, The API Filter Press Test has been developed for drilling fluids in oil wells, but can be used as a test method for grouts.

STORAGE OF MATERIALS

Cements for injection grouting are generally delivered in 1000 kg “giant bags” or in bulk. In Norway, small bags are only delivered exceptionally.

Industrial cement is delivered in woven bags with plastic protection on the top only. This cement is therefore easily damaged as a result of incorrect storage. Micro-cements are basically delivered in bags of the same type, but with an inner sack entirely of plastic. This makes this cement better suited to storage. All cement should be stored in a dry and airy environment. The best solution is to store the cement is a plastic hall with good ventilation.

The worst form of storage is to store cement under a tarpaulin outdoors. Although the tarpaulin protects against direct rain, water will run in under it and a greenhouse-like environment will be produced when later the sun shines on the tarpaulin. The air humidity will permeate into the bags and prehydrate the cement.

Cement should not be stored in the tunnel for any period of time either, as air humidity is high in the tunnel and will impair the quality of the cement. In periods when there is little grouting, it is important to rotate the store so that the oldest cement always comes first, thereby

Figure 50. Specific gravity testing by mud balance scale (Photo: BASF)

ensuring that cement of high quality is always to hand.

It cannot be said categorically that a cement is too old simply by looking at the production date. Storage, air humidity etc. also have a major impact on the quality. The requirement of maximum storage time will therefore be dependent upon the storage conditions. The products that are to be used for injection grouting should in any case be marked with the production week. It will then be possible to adjust the extent to which materials are checked depending on age and storage conditions so as to avoid poor cement in the grouting operations.

CHECKING THE W/C RATIO

It is very important to check the w/c ratio, that is to say, the ratio of water to cement. Water is a wholly necessary part of the grouting material, and a very good “means of transport”. Nevertheless, too much or too little water in the mix can do much to spoil the end result. Too much water will result in a great deal of separation and a spoiled cement paste and difficulty in binding the paste Too little water will result in a paste that is of such high viscosity that it will not penetrate into the fissures in the rock.

Development of the strength of grout is also largely affected by the w/c ratio. Final strength of cement suspensions is almost inversely proportional to the w/c ratio, that is to say a grout with a w/c of 1.0 will have a strength that is 25-30% of the strength at a w/c of 0.4.
The easiest way to check the w/c ratio is to use standard weighing scales (the photograph Figure 49). In the field the w/c ratio can be checked using a so called “mud balance”. See Figure 50.

CHECKING VISCOSITY

A Marsh cone may be used to check the viscosity of the grout. This piece of equipment, whose standard designation is API Marsh Funnel No. 110-10, is a simple funnel with a 4.3 mm opening which is used to measure how long it takes for 1 litre of material to run out.

There are two important factors besides the w/c ratio which affect the Marsh cone time: the addition of a flow agent and initial setting. It is recommended that the grout (sample taken from the agitator) which has, for example, an increase in Marsh cone time of 10 compared with fresh material should be discarded.

Table 5 shows results from tests conducted in a laboratory with a recipe consisting of standard grouting cement + 1.5% of superplasticiser based on the cement weight. For the tests with added silica slurry, 10% silica slurry containing 50% solids is used.

If the time deviates from the values given in Table 5, the scales and other weighing systems on the grouting rig must be checked and adjusted.

Example: w/c ratio = 1.0 with silica means that a w/c ratio of 1.0 is used together with 10% slurry on the basis of the cement weight. The silica is thus not included in the powder quantity, only added in the supplement,

Figure 51. Checking w/c ratio using a Marsh cone

Equipment
- Marsh cone
- 1000 ml measuring cylinder (1 litre)
- Stop watch
- Digital scales

Procedure
- The Marsh cone is filled to just below the screen whilst the cone spout is blocked with a fingertip.
- The fingertip is removed and the stop watch started.
- When the measuring cylinder reaches 1000 ml, the watch is stopped and the Marsh cone time is noted.
- Weight: Set the scales to zero with an empty measuring cylinder, fill the measuring cylinder with 1000 ml and read off on the display.

TESTING OF SETTING

Setting is tested in a laboratory using a Vicat needle (see Figure 52). When the grout is so firm that the needle stops inside the sample body, this indicates initial setting, and when the needle only penetrates 1 mm the material is considered to be set. The test method is described in more detail in NS-EN 480 2. When the grout has set it is sufficiently strong to resist water pressure in the rock mass, and excavation of the rock cavern can continue. To test setting at the face, a type of handheld Vicat needle can be used.

It is also possible to use a plastic cup that is filled with grout, marked and put to one side. The cup must not be allowed to stand on the grouting rig itself as vibrations

Water/cement ratio	Specific gravity	Marsh-cone time	Specific gravity with 10% silica 50/50 slurry	Marsh-cone time with 10% silica 50/50 slurry
v/c 0,5	1,83 kg/l	Not measurable	1,80 kg/l	Not measurable
v/c 0,6	1,73kg/l	50 seconds	1,71kg/l	42 seconds
v/c 0,7	1,66kg/l	43 seconds	1,65 kg/l	37,5 seconds
v/c 0,8	1,60kg/l	38 seconds	1,57 kg/l	35 seconds
v/c 0,9	1,55 kg/l	35 seconds	1,52 kg/l	34 seconds
v/c 1.0	1,52 kg/l	33 seconds	1,46 kg/l	32 seconds
Water	1,00 kg/l	29 seconds		

Table 5 Relation between specific gravity and Marsh cone time with and without silica

from the rig will disturb the setting. The cup should be placed on the floor, so that the grout it contains is exposed to conditions as similar as possible to those to which the grout pumped into the rock mass is exposed. When the grout is so firm that the cup can be turned upside down without it running out, there is sufficient setting to continue operations (drilling) in the tunnel.

CHECKING WATERTIGHTNESS USING MEASURING SILLS

To check how much water runs into the tunnel or the rock cavern, so-called measuring sills can be established. In tunnels the sills are established transversely at suitable points, and the distance between them is adapted to the inflow criteria. For tunnels with strict inflow criteria a typical distance is about 250 metres. It must be ensured that the sill sits snugly in the transition between rock and cement, and therefore it must reinforced, anchored and contact grouted. The height of the sill is dependent upon the tunnel gradient and water volume. A graduated tube must be provided which carries the water such that it is easy to obtain reliable measurements using a bucket and a stop watch. Alternatively, a standard V overflow can be embedded such that the water volume that flows over the overflow can be measured using a metre rule.

When taking measurements it is important to have a stable water table. The measurements must be made in excavation-free periods, preferably in connection with a shut-down lasting several days in order to measure true ingress without any addition of water from operations. It is recommended that measurements be made several times over a period of time in order to ensure that the inflow of water is stable. This is of particular importance in sites with the most stringent requirements as regards ingress of water.

In rock caverns or when driving tunnels on a descending gradient, pump-out water that is collected at the face can also be used to measure inflow of water. In this case, water is allowed to run in over a given period and then the amount pumped out is measured in order to arrive at the same level as prior to the start of pumping.

DOCUMENTATION OF EXECUTED GROUTING

Most contracts require a grouting report to be written. A separate record or grouting report should be written each grouting round. This report should be written up on a separate form and should contain the following information:

Figure 52. Vicat needle for testing setting.

- Part of tunnel or other description of area
- Grouting face (profile number) and grouting round (primary cover, secondary cover, control cover)
- Each individual borehole location in the tunnel cross-section, preferably with a figure showing the grout cover with the hole numbers indicated.
- The length and direction of the borehole
- The depth of packer placement (depends on rod length)
- Measured water inflows, preferably with a figure showing the cover and hole number
- Grout used (type of cement, w/c ratio and admixtures)
- Notes recording grout leaks on and behind the face

  Amount of grout used (volume and/or kg) for each type of grout in each individual hole
- Grouting pressure against time in each individual hole, with an indication of end pressure and back pressure (dwell time) that are reached.
- Grouting time (start/end of grouting and any start/ end of drilling)
- Date, time and signature

LOGAC | WORKSITE: 1 | SECTION: 1 | PRINTOUT DATE: 17.10.2008

Grouting Report

Atlas Copco

	HOLE NO.	HOLE LENGTH [m]	STAGE NO.	RECIPE NO.	INJECTION NO.	W/C	CODE	DATE [yyyy:mm:dd]	START TIME [hh:mm:ss]	STOP TIME [hh:mm:ss]	LOGGING TIME [hh:mm:ss]	VOLUME [Litre]	PRESSURE [Bar]	CHECKS
1	4	23	1	23	1	0,8	None	2008:10:15	18:08:13	18:45:49	00:37:36	174.9	79.2	OK
2	5	23	1	23	1	0,8	None	2008:10:15	18:46:29	18:53:30	00:07:01	38.1	76.7	OK
3	6	23	1	23	1	0,8	None	2008:10:15	18:54:20	19:03:30	00:09:10	18.3	75.8	OK
4	7	23	1	23	1	0,8	None	2008:10:15	19:04:20	19:14:41	00:10:21	53.8	84.2	OK
5	8	23	1	23	1	0,8	None	2008:10:15	19:15:41	19:23:52	00:08:11	15.2	78.2	OK
6	9	23	1	23	1	0,8	None	2008:10:15	19:24:32	19:44:33	00:20:01	308.3	58.5	OK
7	10	23	1	23	1	0,8	None	2008:10:15	19:45:13	20:01:55	00:16:42	28.2	70.1	OK
8	11	23	1	23	1	0,8	None	2008:10:15	20:02:35	20:06:26	00:03:51	47.8	78.9	OK
9	12	23	1	23	1	0,8	None	2008:10:15	20:07:16	20:15:58	00:08:42	49.2	79.0	OK
10	13	23	1	23	1	0,8	None	2008:10:15	20:17:08	20:25:20	00:08:12	48.5	79.8	OK
11	14	23	1	23	1	0,8	None	2008:10:15	20:48:14	21:12:07	00:23:53	324.0	51.2	OK
12	15	23	1	23	1	0,8	None	2008:10:15	21:12:37	21:49:20	00:36:43	258.5	53.6	OK
13	16	23	1	23	1	0,8	None	2008:10:15	21:50:00	21:56:51	00:06:51	52.5	81.1	OK
14	17	23	1	23	1	0,8	None	2008:10:15	21:57:51	22:10:41	00:12:50	49.7	85.8	OK
15	18	23	1	23	1	0,8	None	2008:10:15	22:12:11	22:43:53	00:31:42	136.1	71.4	OK
16	19	23	1	23	1	0,8	None	2008:10:15	22:45:03	22:56:44	00:11:41	2.6	77.9	OK
17	20	23	1	23	1	0,8	None	2008:10:15	23:00:14	23:30:25	00:30:11	61.7	80.2	OK
18	21	23	1	23	1	0,8	None	2008:10:15	23:06:34	23:31:15	00:24:41	70.7	78.7	OK
19	22	23	1	23	1	0,8	None	2008:10:15	22:48:23	23:05:24	00:17:01	42.1	73.4	OK
20	23	23	1	23	1	0,8	None	2008:10:15	22:01:21	22:37:12	00:35:51	282.8	47.1	OK
21	24	23	1	23	1	0,8	None	2008:10:15	21:41:09	22:00:51	00:19:42	253.7	49.3	OK
22	25	23	1	23	1	0,8	None	2008:10:15	21:32:47	21:40:19	00:07:32	62.7	83.3	OK
23	26	23	1	23	1	0,8	None	2008:10:15	21:24:57	21:32:07	00:07:10	55.2	72.7	OK
24	27	23	1	23	1	0,8	None	2008:10:15	21:11:07	21:23:47	00:12:40	53.7	79.7	OK
25	28	23	1	23	1	0,8	None	2008:10:15	21:03:25	21:10:27	00:07:02	53.9	74.7	OK
26	29	23	1	23	1	0,8	None	2008:10:15	20:55:24	21:02:25	00:07:01	55.1	79.1	OK
27	30	23	1	23	1	0,8	None	2008:10:15	20:43:53	20:54:34	00:10:41	55.2	69.7	OK
28	31	23	1	23	1	0,8	None	2008:10:15	20:33:21	20:42:53	00:09:32	56.0	85.2	OK

Figure 53. Example of an automatic print-out of documentation from the grouting rig.

PRINT-OUTS OF THE LOG

Print-outs of the grouting rig log should be included in the grouting report. The rig log contains detailed information about the execution of the grouting round (Figure 53).

A drilling log (interpretation of drilling parameters) should also be included in the grouting report. In addition, information should be recorded about variations in drill penetration, colour of the drilling water and a mention of drilling problems, clay etc.

CONTRACT: LS04 Sandvika Øst
Revision date: 25-06.07 Rev. No. 0

No. TP-12.02
GROUTING REPORT - TUNNEL

Tunnel	Face		Chainage
Start		End	
Date	Time	Date	Time
Drilling			
Grouting			

Hole No.	Rapid litres	w/c ratio	Stop pressure Bar	Hole No.	Rapid litres	w/c ratio	Stop pressure Bar

Probe drilling 12<24m Probe drilling 24<36m	m m	packers		Industrial cement R build 2000PF Silica slurry Rheocem800	kg kg kg kg
Measured water leakage				Accelerator MEYCO SA162	kg
Waiting time at face	Hours				
Type of admixtures		Percentage of cement weight			
Remarks					
Date/signature SKANSKA					

Figure 54. Example of grouting form used at "New double track tunnel Lysaker – Sandvika"

HEALTH, SAFETY AND THE ENVIRONMENT (HSE)

GENERAL INTRODUCTION

Unfortunately there have been a number of undesirable incidents in the rock grouting industry that have left their mark. Several of these incidents have had a high injury potential, and a fatal accident (in 2005) was caused by failure related to grouting work. The causes of the incidents are complex, but a major factor is inadequate knowledge of the risk factors. NFF's Technical Report No. 8 Safety in Rock Grouting, November 2008 is based on the industry's experience of undesirable incidents and measures taken to deal with them.

Splatter from grouting fluid and the ejection of packers have been responsible for a large number of the undesired incidents. Such incidents have often been associated with grouting work at high pressures. Well-established work routines can help to reduce the danger of splatter significantly. Other major factors which contribute to safety include the training of all grouting personnel and good internal communication between crew members.
The use of an intercom between the members of a grouting crew can be extremely useful. Establishment of a safety zone with restricted access ahead of the face and the physical securement of rods between the holes are sensible and commonly implemented measures.

DANGER OF SPLATTER

Use of the correct protective equipment is essential. Protective goggles and gloves should always be worn. Access to eye wash close to the work site is vital. It is also advisable to hold first aid courses with focus on eye injuries. Experience has shown that it may be more important to give good first aid/eye wash than to get the injured person to a doctor. A dust mask should be used in the event of a danger of cement or silica in the air. Frequent hand washing is necessary and the use of moisturiser prevents skin from drying out. An HSE datasheet should be readily available at the work site.

It is also extremely important that all equipment is certified for the maximum pressure that is to be used.

EJECTION OF PACKERS

Some of the major causes of the ejection of packers have been identified:
- Poor fixing
- Inadequate friction between packers and borehole
- Densely fissured rock where packer is placed
- Wrong type of packer is used
- Failure of packer locking system

Good maintenance of rods, with particular focus on the tightening of packers, is crucial in order to avoid poor fixing. Outer tubes should be replaced if there is any damage to the contact face. Old cement must be removed from the threads and, once clean, the threads should be greased between each grouting round. A good rule may be to retighten the packer before the hose is connected to the rod. Follow the supplier's instructions.

Holes that are "greased" with grouting material will in general present less friction. Packers should, as a rule, be placed in the holes before the start of grouting.

Rods of up to 6 metres in length should be available so that the packer can be set far inside the hole if the rock is densely fissured. In cases where it is extremely difficult for the packers to "bite", the entire rod and packer can be grouted in place inside the hole. However, this will result in the loss of the rod.

The frictional force that is to hold the packer in place in the hole is proportional to the borehole diameter. Axial ejection force on the packer, on the other hand, is proportional to the square of the borehole diameter. This means that the danger of ejection of the packer increases with increasing borehole diameter – a fact it is important to have in mind when choosing borehole size.

Choice of packer also requires this factor to be considered. It is important to be aware that there may be an air pocket under high pressure in closed boreholes (boreholes with no inlet). If the packer becomes loose in such holes, it will shoot out of the hole.
Rods can be anchored to the rock or nearby rods to

Figure 55. Chain holding the rod in place when the packer is ejected.

prevent the packer from sliding out of the hole (see Figure 55). Today, grouting equipment certified for high pressure is commercially available.

OBSERVATION OF ROCK STABILITY DURING GROUTING OPERATIONS

When high pressure grouting is performed there is always a danger of the rock mass at the face and in the ceiling being thrust outwards. When terminating grouting holes where there is a pressure build-up and little grout take because the hole is blocked, there will be a danger of buckling of the face and/or ceiling. Most pressure is found close to the tunnel interior. If it looks as though the shotcrete in the ceiling is cracking, grouting work must cease and the crew must withdraw so that the unstable portion can be stabilised.

ENVIRONMENTAL CONSIDERATIONS

Guiding values for all cements that are used in Norway indicate that the maximum amount of water soluble chromium (Cr^{6+}) should not exceed 2 ppm (parts per million). In other words, only minute quantities of water-soluble chromium are permitted. This is important for the working environment as even small amounts of water-soluble chromium can cause eczema if it repeatedly comes into contact with unprotected skin. Gloves should therefore always be worn when handling grouting materials.

GROUTING IN TBM TUNNELS

GENERAL INTRODUCTION

In TBM-bored tunnels there will often be a need for pre-grouting. The object of this grouting may be

- Sealing of water leakages
- Stabilisation of the ground

In addition, it may often be necessary to drill probe holes in order to investigate the ground conditions ahead of the machine.

It is not desirable to drill probe and injection holes through the drill head and normally therefore the collaring of such holes will be carried out through special cut-outs in the shield behind the drill head.

In particular in the case TBMs with a diameter of up to 4-5 metres, there will be limited space in the machine area and it will be necessary to have specially adapted drilling equipment. In the case of larger machines, the space in the machine area is better and adaptation of the

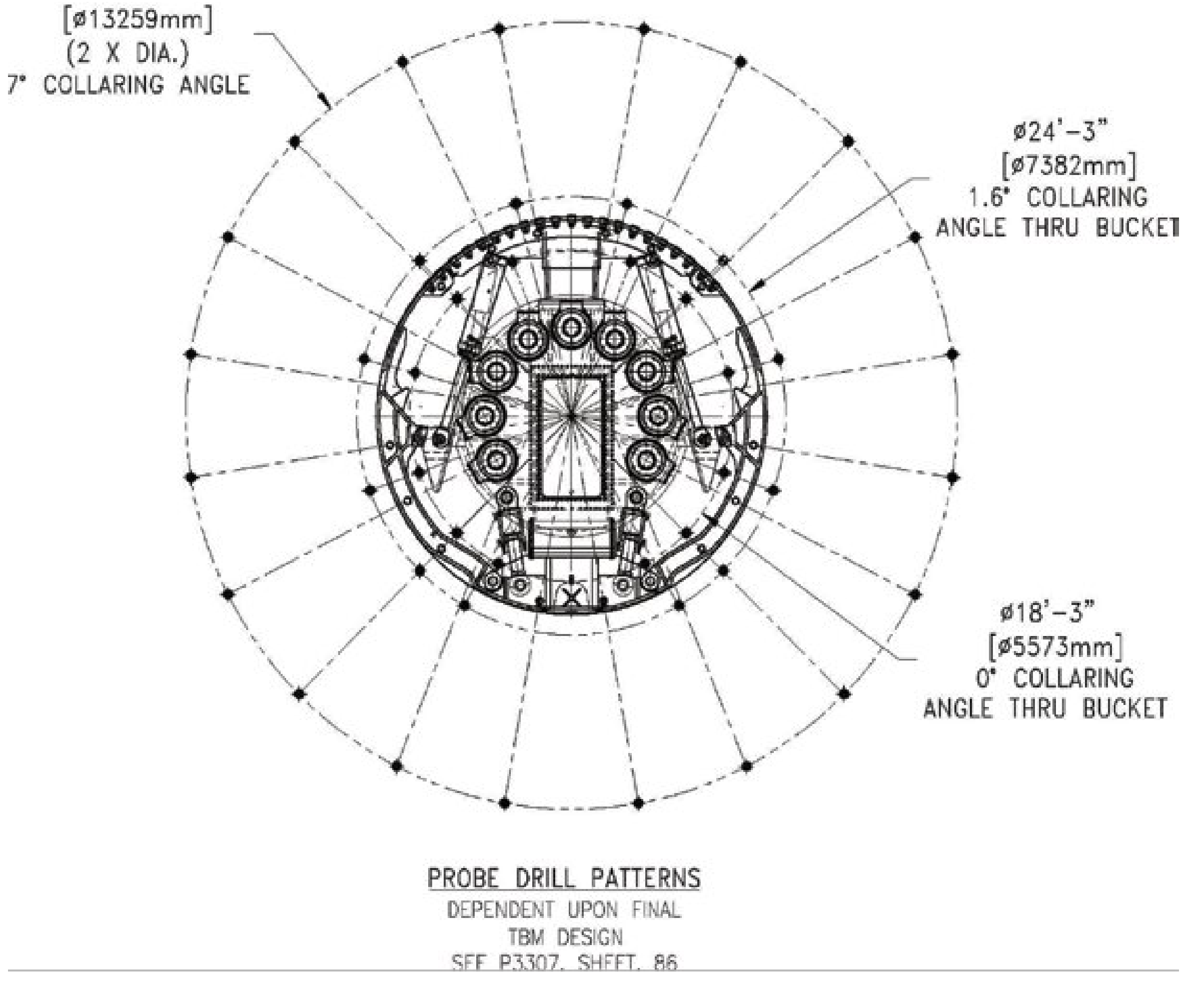

Figure 56. Example of possible location of grouting holes

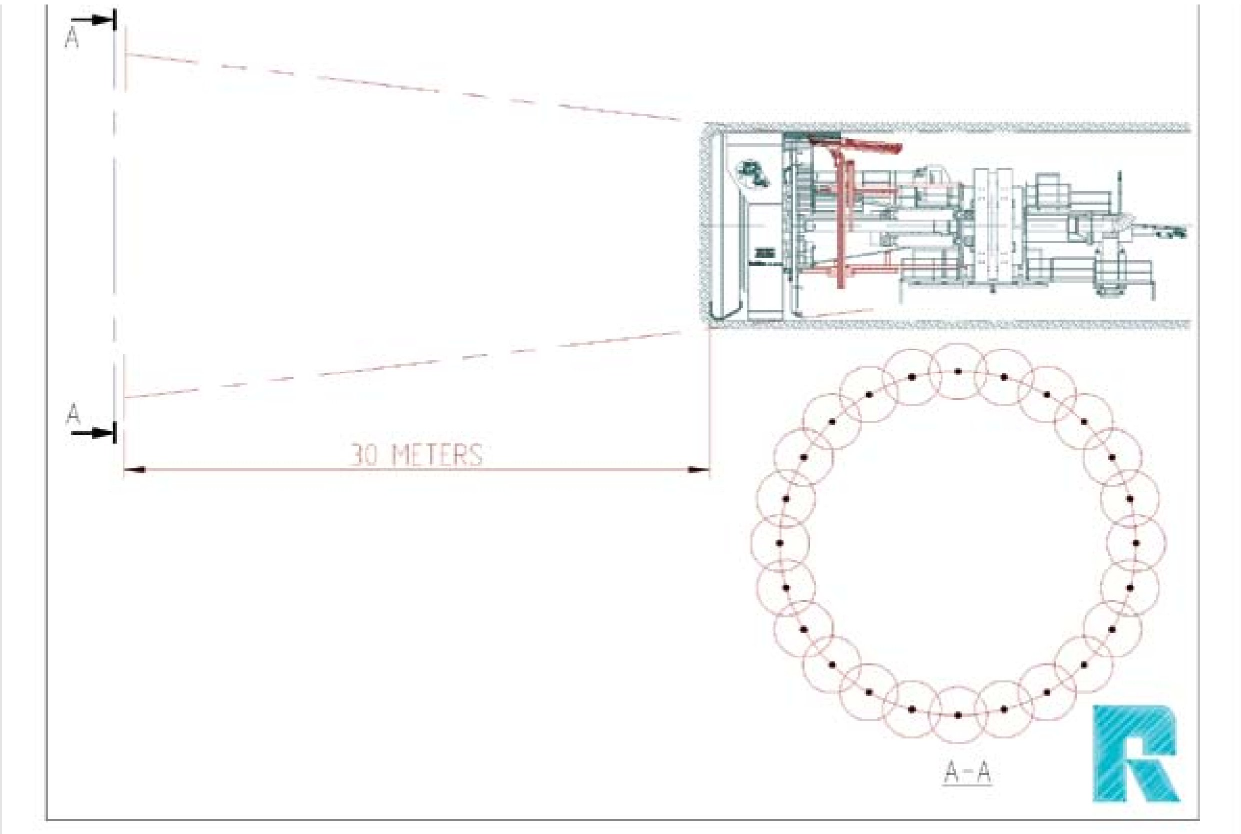

Figure 57. Principle of grouting drilling during TBM driving.

equipment simpler. The possibility of varying the hole locations will vary depending on the type of machine and diameter used. Normally, it will be easiest to adapt the drilling equipment in an open machine. In shielded machines, the space is smaller and it may be necessary to draw the collaring of the grouting holes slightly further back in the machine area. The retrofitting of equipment results in complex and inexpedient solutions.

PRACTICAL CONSIDERATIONS WHEN GROUTING FROM TBMS

Usually it will be the need for watertightness which necessitates the start of grouting operations. Large water leakages into the tunnel may prevent progress being made and/ or it may be necessary to seal so as to prevent the lowering of groundwater with associated settling damage.

In shield driving tunnel excavation using cement rings as a lining, the gap between the concrete segments and the rock must be post-grouted to secure durable watertightness without the use of pre-grouting. But when there is a great deal of water under pressure (more than 70 – 100 m water head), problems could arise with the shield packers, seals and post-grouting. In such cases, pre-grouting will nonetheless be necessary.

The grouting technique will be like that employed in a blasted tunnel. The tunnel boring is stopped and holes are drilled 25-30 metres ahead of the drill head in a cover as shown in Figure 57. These holes are grouted in the usual way before further tunnel boring can be carried out until normally 6-8 metres remain to provide an overlap to the next round of grouting. The packers must be placed at a sufficient distance (normally 2 to 3 metres) ahead of the drill head so that the grout does not escape into the tunnel. This means that the grouting rods used will have to be longer than those employed in a blasted tunnel.

Mixers and pumps are positioned in suitable locations on the rear rig to facilitate the supply of cement and other grouting agents.

STABILISATION OF THE GROUND AHEAD OF THE FACE DURING TBM DRIVING

In some cases large fault zones may be difficult to drive through, and here one solution is to pre-grout these zones so as to increase stability. The drilling procedure for the grouting holes will normally be the same as for water sealing, but it may be necessary to use different grouts, depending on the local conditions.

Figure 58. Probe drills on an open TBM

POST-EXCAVATION GROUTING

GENERAL INTRODUCTION

Experience of post-excavation grouting (or post-grouting) has shown that it is difficult and time-consuming to seal water inflows after the tunnel or cavern has been excavated. It is therefore important to take the time necessary to obtain best possible watertightness during the pre-grouting operation. In those cases where the result of the pre-grouting has not been satisfactory, it has been found that even small leakages may be very difficult to seal in the tunnel after excavation.

When watertightness requirements are stringent, there will sometimes be a need for some post-sealing of the pre-grouted rock chamber. For successful post-excavation grouting, it is essential to have a well drawn-up plan for the operation.

Detailed information about the properties of various grouting products must be obtained from the supplier.

PRACTICAL ADVICE FOR POST-EXCAVATION GROUTING

In general it is recommended that the rock mass should be sealed outside the blast-affected zone. Previous experience has shown that this zone is from 0.5 to 1.5 metres outside the contour. Beyond this zone there is normally a stress transfer (increased normal stress) which may be from half to the whole of the tunnel diameter outside the tunnel profile.

Prior to the start of post-grouting the orientation of the water-bearing fissures should be mapped. In the case of underground structures whose whole contour is covered with shotcrete, engineering-geological mapping of the excavation is extremely useful. The boreholes should be oriented so as to intersect water-bearing fissures approximately 5 to 10 metres outside the profile. Boreholes that encounter the largest leakages (channels) should initially be used as relief holes. The grouting work should start after all the holes have been drilled. The holes that are located farthest from the drainage holes should be grouted first. During this operation, grout leaks should as far as possible be sealed with oakum or the like. Before the grouting is terminated in the hole or holes that produce most leakage water, the rest of the grouting should be hardened for at least 24 hours so that this grout is not washed out as the last hole is "plugged".

Experience from this method has shown that the leakages move when a portion is post-sealed. It is therefore important to drill post-grouting holes at a good distance from both sides of the observed leakages. How great this distance should be depends on the rock overburden of the structure and the degree of fissuring of the rock mass. If nevertheless new leakage points develop, drilling must be recommenced at the fissures believed to be water-bearing and the same procedure carried out again.

It is easier to perform a post-grouting operation where there is concreting or shotcrete that withstands some grouting pressure (1-2 bar). However, the principle of grouting should follow the example given above in that the innermost part of the hole is grouted first (packing placement at 5 metres or deeper), which will allow a higher pressure to be used whilst the main leakage path is grouted first.

There are many types of post-excavation grouting, ranging from grouting for leaking bolt holes and more or less concentrated point leakages from the rock to long stretches where more water than acceptable seeps in.

SUITABLE PRODUCTS FOR POST-EXCAVATION GROUTING OF BOLT HOLES

Sealing of leakages from bolts is perhaps the simplest form of post-grouting. First, the bolt hole must be drilled open about 0.5 metres within the contour and a small packer with a reflux nipple (same type as a grease nipple) inserted. A small pump should then be filled with the grouting product, the hose should be connected to the nipple and pumping started. A suitable product for use in this case is one-component polyurethane (PU) foam. This is a liquid product to which accelerator is

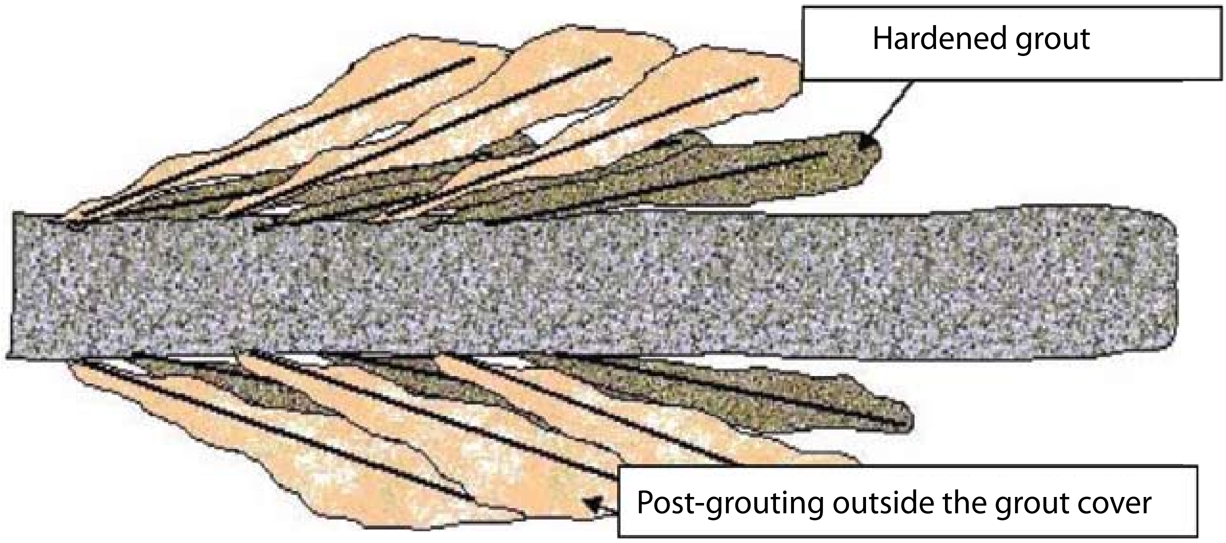

Figure 59. Principle for the execution of post-excavation grouting. Holes for post-excavation grouting should be drilled in the opposite direction of the pre-grouting holes.

added as required. As long as no water gets into this mixture, no reaction takes place. When the material is then pumped into the leakage, it will foam as soon as it encounters water. During the foaming process, the material will continue to move towards the water. A compact foam which seals the water leakage will thus be obtained.

This type of PU foam is a freely expanding foam capable of about 20- to 25-fold expansion. That is to say that 1 litre of foam will expand to about 20-25 litres. The PU foam is freely expanding along the bolt and must therefore work its way towards the water within the rock. Maximum expansion pressure is about 8 bar for one-component PU foam. The foam takes about 2 minutes to react fully.

SUITABLE PRODUCTS FOR POST-EXCAVATION GROUTING OF POINT LEAKAGES

Another type of post-grouting is that required to deal with point leakages from the rock. These leakages are often difficult to seal with cement-based products. An alternative is two-component PU foam. The advantage of using a 2-component foam instead of a one-component product is that 2-component foam also reacts without encountering water.

A two-component PU foam requires a two-component pump capable of pumping equal ? portions of component A and component B. The components come together in the mixer nozzle, where they are mixed before being pumped into the rock through the packer.

There are many variants of two-component PU foam, ranging from those that are slow foaming and have a low foam factor to those that are fast foaming and have a high foam factor. For this type of injection, fast-foaming variants with a high foam factor are usually preferred.

It is recommended that at least two holes be drilled at an angle so that they intersect water-bearing channels about 4 to 8 metres inside the rock. If the leakage is large, several drainage holes must be drilled in which open packers are placed so that the water from the leakage is drained whilst the other holes are grouted.

Grouting of the hole that intersects the leakage closest to the tunnel is then commenced. When the material comes out of the leakage hole, or the leakage stops, pumping is terminated and attention is turned to the next hole. This means that the last part of the leakage channel is sealed first and then PU is pumped backwards into the leakage.

SUITABLE PRODUCTS FOR POST -EXCAVATION GROUTING OF LARGE SURFACES

The principle adopted for post-grouting of large surfaces or stretches in a tunnel, should be more or less the same as that followed in pre-grouting. In this case, the intention is not to block a single leakage but to penetrate fissures so that the water seepage stops.

For this type of grouting, micro-cement or colloidal silica is generally used. It is important to use products with optimal penetrability. Where the rock mass has already been grouted, good penetrability is necessary to improve watertightness. Colloidal silica is a product that provides good penetration and has given good results in work of this type. The equipment for this type of grouting is normally the same grouting rig as that used for pre-grouting.

The post-grouting holes should be drilled in the opposite direction to the pre-grouting holes, and they can expediently be drilled in a herringbone pattern. The holes drilled run deeper than those used in the pre-grouting operation. When the herringbone pattern has been grouted, the next step is to withdraw, say, 5 metres back in order to establish a new cover. This means that the grout of the second cover will "butt" against the first cover and seal the rock towards the tunnel.

Before work commences on a "backward herringbone advance" of this kind, a cut-off grout curtain is generally established. This is a cover that is set at 90 degrees to the tunnel direction. The idea is that this cut-off curtain should block the material in the first herringbone cover.

COLLOIDAL SILICA

Colloidal silica is a special type of silica in liquid form. It is a mineral grouting product which must not be confused with silica slurry. Colloidal silica is usually used where there has been no success in meeting the watertightness requirement with cement-based grouting agents, but it may also be used as pre-grouting material in demanding conditions.

Colloidal silica has a viscosity of 5 mPa·s (milliPascal second) when ready for use. This is very close to the viscosity of water, which is 1 mPa·s at 20oC. Colloidal silica has minute particles with an average particle size of 0.016 microns. These particles are dissolved in water. Initially the particles in colloidal silica are negatively charged. When a salt water solution is added as catalyst, the positively charged sodium particles will cause the silica particles to "collide" with each other, thereby forming a gel. This gel develops strength over time.

Normally ordinary cooking salt, NaCl is used in the salt water solution, but other types of salt, as for instance, road salt may also be used. The more salt that is added the faster the gelling time becomes. Usually a 10% salt solution is used (1 kg of salt to 9 litres of water, which gives a specific gravity of 1.07 kg/l). The open time may be freely adjusted from about 10 minutes to 21/2 hours with full control. Standard open time for rock grouting is 20-40 minutes.

The temperature will have an impact on how quickly the gel is formed, and it is therefore recommended that some

Figure 60. Test showing the spreading of polyurethane.

test mixtures be made at the face to adjust the open time before starting on the actual grouting.
The salt solution is only a catalyst and is not a part of the actual gelling process. Experience has shown that salt will seep out from the process in the first two weeks after grouting, but that this will gradually come to a complete stop.

Standard grouting equipment is suitable for grouting with colloidal silica. Silica and salt water are mixed in the mixer and injected in the normal way. To avoid clumping it is important that the mixer containing colloidal silica is running when the salt water solution is added.

Colloidal silica does not have to be marked as the product is not hazardous to humans and the environment.

COMBINATION GROUTING

In some cases it may be appropriate to mix different grouts in order to solve difficult problems. An example of combination grouting may be to use polyurethane in combination with conventional cement grouting (about 5-10% of the cement volume) to reduce the consumption of cement. The amount of cement can be reduced when the grouting is controlled using a non-water soluble material with controlled reaction process. A one-component polyurethane may, for example, be used as additive to cement when:

- The cement is pumped into the rock without back pressure being reached and the cement consumption is abnormally large.
- The cement does not manage to harden before it runs out onto the face (stopping agent).

To obtain the desired effect, it is recommended that experience be acquired with the mixture from the supplier in question before a solution of this type is chosen.

Figure 61. Insert and expand grout packer in grout hole (Photo: AF Group/John I. Fagermo)

FUNCTIONAL REQUIREMENTS

DETERMINATION OF FUNCTIONAL REQUIREMENTS

When functional requirements are to be set with respect to inflow and outflow of water, there are a number of different factors to be taken into account. These can be split roughly into three groups:

- Consequences of leakages for the area
- Consequences for the actual construction work
- Consequences for the permanent structure

A crucial question is how functional requirements should be drafted and what methods there are for follow-up to ensure that they met. Today follow-up and checking of functional requirements involve the following methods:

- Water loss measurements in the rock before and after grouting
- Inflow measurements (outflow) in the structure
- Pore pressure and groundwater level measurements in rock and loose masses around the structure and measurements of settling in surrounding built-up area.

None of these methods can be used alone. The methods that should be chosen depend upon the requirements made.

ESTABLISHING WATER INFLOW CRITERIA

Considerations associated with water inflow criteria are based on the following factors:

- Purpose of the underground structure
- Location of the structure
- Overburden and size of structure
- Consequences of leakages – economic and environmental (including reputation)
- Safety issues
- Permanent and temporary functional requirements
- Financial aspects

Maximum inflow allowed is usually given in litres per minute per 100 metres of tunnel or rock cavern.

Water ice in the frost zone in a tunnel will occur when there is dripping and minor water seepage.

Leakage from a gas or oil store must be assessed in the light of safety and environmental factors. The Norwegian Climate and Pollution Agency and the Directorate for Protection against Fire and Explosions must normally approve/set the functional requirements.

INFLOW CRITERIA BASED ON ENVIRONMENTAL CONSIDERATIONS

The following consequences must be considered when establishing the tolerance requirements set by the surroundings with respect to the underground structure:

- Possibility of drainage and settling of the surrounding loose masses
- Swelling of alum shale
- Damage to forestry and agriculture as a result of a change in groundwater level.
- Environmental impact on surface water and flora and fauna
- Reduction of groundwater reservoirs (wells)
- Possibility of pollution from leaking gasses and liquids
- Sulphate-containing rock types (for example, PPP road construction project E18 Grimstad – Kristiansand)

Because of the consequences of a lowering of the groundwater level, the watertightness requirement in underground structures in urban areas with building foundations on loose masses is very stringent. It is highly desirable and/or necessary to have pore pressure

	Stringent requirement	Intermediate requirement	Moderate requirement
Allowable inflow	5 litre/min/100metres	10 litre/min/100metres	20 litres/min/100metres
Functional requirements	Sensitive surroundings	Moderately sensitive	Site-dependent

Table 6. Examples of typical inflow criteria

measurements in good time (at least one year) before the start of construction work. In practice, visible leaks at the face in the most sensitive area are not tolerable. This makes tough demands on both the planning and execution of the grouting work. In these cases, there should also be a contingency plan including water infiltration and/or watertight grouting.

Detrimental consequences are not always measurable in economic terms, and damage and drawbacks may arise as a result of drainage over which there was no full overview beforehand. In towns and densely built-up areas, a single tunnel construction cannot be viewed in isolation, but should be considered in conjunction with the impact from existing underground structures and perhaps also future tunnel excavations in the same area. In the Oslo area, this has led to a constant tightening up of the requirements regarding water inflow in new tunnels.

INFLOW CRITERIA FOR THE ACTUAL CONSTRUCTION OPERATION

The following considerations must be taken into account when establishing what inflow requirements should be set with regard to the actual construction operation

- Prevent large water inrushes
- Problems and costs associated with pumping out water
- Specification of maximum pumping capacity for sub merged structures
- Stability of the rock cavern
- Working environment
- Reduced quality of the grouting work
- Determination of maximum inflow at the face with a view to drilling charge holes
- Avoid charge problems
- Extra costs in connection with use of cartridged explosives because of large water leakages
- Determination of maximum allowed inflow of water with a view to a washing out of the driving path

For underground structures where the surroundings make no special demands as regards maximum inflow of water, the scope of the grouting work is determined on the basis of the problems the inflow of water will cause operations. Pump capacity requirements are different depending upon whether operations are carried out at a descending or ascending gradient. Normal pump capacity for a construction pump is about 500 l/min at 50 metres back pressure and 2000 l/min at 15 metres back pressure. The limit value for the size of leakage that can be permitted with regard to operations may therefore be around 1000-2000 l/min. This will vary from structure to structure and will depend on both geological and construction conditions.

When a tunnel is driven on a flat floor, the capacity of the drainage trenches may be too small, making it necessary to have a pipeline and pumps to keep the roadway in order.

INFLOW CRITERIA FOR THE FUNCTIONING OF THE UNDERGROUND STRUCTURE

The following factors should be considered when inflow requirements are to be established on the basis of the function of the underground structure:

- Water ice formation
- Service life of technical installations (corrosion)
- Problems and costs associated with pumping out water (especially in the case of lowest points/subsea tunnels)
- Current leakage in electrical installations
- Significance of a damp environment for quality, life time and operating costs
- Leakage from underground structures
- Traffic and safety aspects.

Different areas of use for the underground structure make different demands on the extent of sealing. Below are some examples of the considerations that should be made for different types of structure:

- **Public transport tunnels**

If the surroundings do not call for a maximum allowed inflow, the extent of the grouting work is determined on the basis of an assessment of safety and life cycle costs. A cost-benefit analysis of the grouting work should then be made in relation to shielding/drainage. In most cases, it is inappropriately costly to grout the rock mass around an underground structure so as to achieve such watertightness that water shielding can be omitted. Where the surroundings call for extensive grouting, the required scope of water and frost protection can in some cases be reduced. The Norwegian Public Roads Administration has a project (Modern Road Tunnels) which is looking at a lifetime perspective of one hundred years for rock engineering projects. An increase in the scope of grouting is one of several measures that are being assessed in order to increase quality and service life.

- **Subsea tunnels**

Subsea tunnels have an endless reservoir of water above them. Service life costs and operating costs in connection with pumping out leakage water must be seen in relation to investments in pre-grouting.

- **Water tunnels**

Lost production because of leakages from the tunnel system at the appropriate operating water pressure must be assessed against grouting costs. Tunnels that normally should be filled with water might stand empty at times. The environmental consequences of inflows of water must therefore be considered. Acceptable inflow must be assessed in relation to downtime associated with maintenance of the tunnel or installations.

- **Gas stores and storage of LNG**

When assessing leakage from gas storage caverns where gas is to be held under pressure, it must be taken into account that gas more easily escapes through fracture zones and fissures than water. As a rule of thumb, the ratio of the penetrability of water to gas (air) can be assumed to be 1 to 100. Required tightness is so great that the use of advanced grouting technology must be expected. In practice, this will mean that the sealing work may be a substantial economic factor. In the case of gas stores, the problem of pollution is also crucial in determining the extent of grouting. One problem that must be taken into consideration is jointing as a result of thermal tensions caused by cooled gas penetrating into existing fissures.

- **Pressure tunnels**

Leakage from pressure tunnels may arise where smallest main tension is less than the water head. There are examples of leakage from such structures with some major damage and huge repair costs as a result. Even though grouting will help to reduce the leakage in the fissure system, there will still be a danger of a wash-out of the fissure material which will result in increased leakage.

- **Warehouse caverns and public halls**

In the case of rock caverns for use by the public and for goods storage, the scope of grouting will normally be determined by construction-technical conditions and life cycle costs. If moisture-sensitive products are to be stored, shielding/drainage will almost always be necessary. Sealing can be weighed up against required temperature/fresh air system.

SPECIAL GROUTING IN THE TRANSITION BETWEEN LOOSE MASSES AND ROCK

For breakthrough in rock tunnels that are in areas prone to settling, or for shafts that run through both soil and rock, in an especially sensitive area or for special use, sealing measures are required that are tailored to the situation in question. The use of old mines as stores for dangerous waste is one example of projects where special requirements must be set for the grouting scheme.

Grouting is carried out in several phases with, for instance, three packer placements. A combination of polyurethane and cementitious grouting is used, and the first grouting phase establishes a tight curtain or mantle. Subsequent grouting phases can thus be carried out with better back pressure.

Knowledge and experience gleaned from the grouting of dam foundation beds in many dam projects provide a good basis of sealing strategy. However, there is not the same access to a cleared rock surface. A great deal of thought must therefore be given to the transition zone and the most shallow rock mass. A sealed transition should first be established between sealed support walls (slotted walls, sheet piles, tubular piles, secant piles) and rock. Several methods may be applicable; geometry and geotechnical conditions will determine which is the most appropriate for the situation in question. In sand and gravel, jet grouting may be a good solution, whilst in clay it will probably be necessary to resort to a special jet grouting. This involves the grouting of flushed-out voids (EC-1 Triplex jet grout). In solid, permeable masses, soil grouting with “tube-a-manchette” may work. However, it should be borne in mind that these methods have been developed primarily for obtaining rigidity and strength, not for sealing. A one-component PU will contribute to increased watertightness. For shafts and tunnel openings, the sealed support wall should be moved some way into the rock. This is possible by using, for example, secant piles and tubular piles.

Sealing must often be done via long casings in soil/support wall and is carried out using a dense hole pattern in several intervals with a gradual increase in pressure and repeated drilling through the “surface rock zone”. When the inflow criterion is stringent, the result must be assured before internal excavation in the soil section can be completed. One example of complex sealing can be taken from Møllenberg in Trondheim. This tunnel system, under construction from 2010-2013, comprises, among other things, a transition from loose mass tunnel to rock tunnel. There are strict requirements as to water inflow. A typical situation is 15 metres quick clay over rock where the cutting and tunnel opening are established. The pile wall has been made using tubular piles that are drilled 1 – 2 metres into the rock and cast in place using cement mortar. Holes for the grout cover have been drilled in guide pipes with a centre distance of about 70 cm to a depth of about 10 metres beneath the bottom of the cutting. Grouting will be carried out in several phases with repeated drilling. The material used in the grouting work is a combination of PU and cement, the PU chiefly being used to obtain a mantle in the shallowest zone. Pile walls with grout covers constitute a closed frame that is tested for watertightness by test pumping before internal excavation beneath groundwater level is started.

Type of structure	Required watertightness	Surroundings	Scope of grouting	Grouting agent	Alternative/ supplement
Power station tunnels	No loss of water which results in financial operating consequences	Change in groundwater	Sporadic (as deemed necessary)	Cements	Lining (steel, concrete)
Public transport tunnels					
/in built-up areas	Avoid damage to the environment	Lowering of groundwater level and settling	Systematic grounding	Cements	Shielding, drainage. Water filtration
	Water/ice problems in tunnel			Chemical agents	Watertight grounding
/subsea	Pump capacity/ finances	Independent	Based on systematic probe drilling	Cements	
	Water/ice problems in tunnel			Chemial agents	Shielding, drainage, watertight grounding
/others	Water/ice pro-blems in tunnel. Requirements in driving phase	Lowering of grondwater	After probe drilling in premapped zones	Cements	
Oil/gas stores	Gas-tight, prevent pollution	Pollution	After probe drilling	Cements	
	Operating expences			Chemical agents	Water filtration
Goods stores	Dependent on type of store	Lowering of groundwater, settling	Dependent on type of store and goods	Cements	Shielding, drainage
				Chemical agents	

Table 7. Conditions of significance in deciding a grouting strategy

CRITERIA FOR THE EXECUTION OF GROUTING

In underground works with strict requirements as regards inflow or leakage and demanding ground conditions, it is necessary to have comprehensive groundwater monitoring in order to determine the grouting scheme. In addition to the conditions dealt with in the table, it may also be necessary to take into account service life and operating costs.

TECHNICAL AND FINANCIAL CONSIDERATIONS IN CHOOSING GROUTING STRATEGY

For a number of the public transport tunnels that have been excavated in the Oslo area in the last 10 years, the costs of rock grouting have been in the same range as the costs of excavation and reinforcement. It is therefore very important to make a thorough assessment of both the necessary extent and choice of grouting scheme when planning projects where there is a need for extensive rock grouting. In such assessments, the following must be taken into account:

- Necessary inflow criteria
- Grouting method
- Suitable equipment
- Suitable materials

Maximum allowed inflow of water determines how extensive the grouting must be, but as a rule there will be a number of solutions that can be applied. The number of holes necessary in the cover must be assessed against how high pressures may be and the groutability of the rock mass.

The costs of pre-grouting will be considerably higher if more than one round of grouting must be carried out before tunnel driving can continue. For some definable situations it will be necessary to grout in two rounds. These may be situations where some wide channels must be "closed", reflux must be "closed off" or fissures with little closing pressure must be pretensioned or an outer cover must be laid before the main grout cover can be put in place (Chapter 4.2). When or if it is considered probable that one grouting round will suffice, the hole distances and grouting materials should be chosen so that the watertightness requirement is actually met.

Penetrability of industrial cement is lower than that of micro-cement, which means that in heavily groutable rock masses this must be compensated for by a denser hole pattern. A relevant consideration will therefore be to compare costs for extra drill metres with the difference in price between, for example, micro-cement and industrial cement. This is a simplified presentation and will to a great extent be dependent on the fissure pattern of the rock mass.

Another aspect of the total picture of the costs is associated with value increase and costs connected to waiting and stoppage in the excavation operation. This aspect must be considered when selecting method, equipment and materials. It must be borne in mind that that even the finest cements may not reach leakage channels with which there is no direct communication, even at high pressure and with materials of high penetrability.

When inflow criteria are strict and the rock mass is difficult to inject, holes must normally be drilled in a dense pattern in order to increase the probability of encountering the leakage channels, so that they can be grouted.

CONTRACTS

DEVELOPMENTS IN FORM OF PAYMENT FOR GROUTING WORK

Historically, good tunnel production has often been associated with high advance rate, that is to say a maximum number of tunnel metres per week. In a situation of this kind, the need for grouting was often seen as “a nuisance and a delay” compared with the “real” production. In contract descriptions in recent years, terms such as “stop time” and “downtime” have usually been replaced by “grouting work” and “grouting time”, and all involved parties have a greater understanding of the fact that grouting work is an essential part of tunnel production. This is a positive development.

INCENTIVES, RISK AND DISTRIBUTION OF RESPONSIBILITY

It is a good main rule that form of payment and incentives in the contract endeavour to place risk and responsibility with the party who is best positioned to influence the risk. This means to say, for example, that the productivity or capacity risk that depends on geology and ground conditions should not be borne by the contractor, but by the owner.

There may also be conditions that the contractor is best positioned to influence, whether productivity as a result of quality of equipment or personnel, which it would be unreasonable for the owner to bear the risk of.

An important principle for distribution of responsibility is that it should be balanced optimally between the parties, and that the parties manage to deal with any disagreements underway in a professional and flexible manner.

AGREED FORM OF PAYMENT

The previous form of payment, whereby the contractor’s work associated with grouting operations was basically only to be paid according to a fixed unit price for the grouting material independent of expenditure of time, is not advised. The later models where payment for grouting operations is made primarily on the basis of time spent are regarded as a balanced form of payment which to a far greater extent puts the risk where it belongs and ensures a better result as regards both quality and HSE.

It is worth considering whether a distinction should be made in the tender documentation between grouting time within and outside normal working hours in the tunnel.

An invoicing principle of this kind, which is something close to working on a cost-plus basis, requires close dialogue and follow-up between the parties, and mutual trust. Capacity of equipment and staffing requirements and skills etc. should be described in as much detail as possible in advance, but at the same time, the parties must manage to adapt/change the scheme required. Time spent on grouting work should give corresponding construction time through an equivalent time account.

As the Code of Process is formulated, drilling of grouting holes is paid according to price per metre. Since drilling capacity in fact depends on the nature of the rock, it may be sensible to consider to time-dependent costs for drilling being billed according to a separate rate for drill rig and crew. A principle of this kind was used by the Norwegian National Rail Administration on the Jong-Asker project (Oslo area.). A drawback of the drilling being paid according to hours worked is that discussions may arise as to capacity achieved in relation to the requirements set in the contract (often 80/90 drill metres per hour).

For all costs and work associated with the actual grouting, payment is made according to agreed time rate and real time expenditure, grouting time. Here, three separate grouting lines are often required, and discussions may arise if not all lines are in use. In this case, the owner and the contractor together must find the right balance between capacity and adequate control of pumping/grouting progress.

Settlement for materials is made according to materials costs and amount of grouting agent supplied.

Whatever the form of payment, an optimally correct estimate for the different amounts (drill metres, kilograms, hours, equivalent time etc.) in the competitive tender documents will be a vital success criterion.

HYDROGEOLOGY AND ROCK GROUTING

WATER FLOW IN THE ROCK MASS

Most rock types in Norway have little capacity for transporting water (low hydraulic conductivity). The groundwater flow takes place along secondary structures or fissures and channels that are formed when the rock is subjected to tension and temperature changes. The rock mass on mainland Norway is thus a fissured aquifer and the significance of pores is negligible. Different rock types have different fracturing intensity and pattern. Fracturing is determined by the mechanical properties of the rock type and the geological history of the area.

Water conductivity in the rock mass is dependent on the hydraulic conductivity of the individual fissure, and on connections within the fissure network for the area. Structures in the rock mass such as fault, folds and rock type boundaries have a much higher fissure intensity than the rest of the rock type. These zones can, as a result of the high fissure intensity, have much higher hydraulic conductivity than the adjacent rock type. Fault zones may be both hydraulically conductive and sealing. In zones where there has been a great deal of movement in the fault zones, crushing may have resulted a lot a fine material which may render the zone sealing in relation to the areas around it, thereby creating a groundwater barrier.

The most common approach to estimating flow in a rock mass is to reckon on laminar flow between two parallel plates. In reality, however, much of the flow may take place through “channels” that occur in the intersection between two fissures. “Channels” may also be formed as a result of shear deformations along a rough (uneven) fissure surface or as a consequence of local wash-out or dissolution of filler material in fissures.

MODELLING OF GROUNDWATER FLOW IN ROCK

Inflow of groundwater into a tunnel installation in rock and the effect on pore pressure and groundwater level in the surroundings is a very complex modelling problem. The major reasons for this are:

- A rock mass is a highly complex flow medium, where hydraulic conductivity of fissures and channels varies considerably within short distances, often in a very complex 3-dimensional pattern.
- The rock masses are often overburdened with soil masses which also may have a very varying hydraulic conductivity, for example, from gravely sand with typically $k = 10^{-4}$ m/s to clay with typically $k = 10^{-9}$ m/s.
- Complex topography in the area that entails substantial differences in running off and infiltration. Efficient infiltration or feed of groundwater is of course also highly dependent on the hydraulic conductivity of the loose masses and the rock, and whether the tunnel is located in urban areas where precipitation is collected in surface water facilities or in undisturbed open land.
- Because of variations in effective infiltration throughout the year, there will always be variations in groundwater level and pore pressure even though the leakage conditions or hydraulic conductivity in the ground are otherwise constant around the tunnel.
- If unsaturated zones and the water-storing capacity of the soil or rock are included, the modelling of water flow and its effects becomes even more complicated. These aspects may be important for modelling transient states in areas of loose masses.
  In addition to the modelling of flow problems per se being very complex, preliminary surveys to determine the true variations in the loose masses and the hydraulic conductivity rock present a major challenge.

WATER BALANCE

The natural water balance/groundwater flow is determined by a number of conditions as shown below:

- Precipitation
- Size of area of precipitation
- The amount of the precipitation that evaporates
- The amount of the precipitation that runs off on the surface

These parameters determine how much water actually infiltrates into the ground. In addition, the following parameters will determine the behaviour of the water in the ground.

- Permeability of soil and rock
- The storing capacity of the soil and rock masses
- The presence of and infiltration from open groundwater reservoirs (lakes, rivers, streams)

The impact of a tunnel leakage on the groundwater balance will depend upon the same factors. The effect of the tunnel on pore pressure and groundwater level will also depend upon the depth of the tunnel below groundwater level. A key concept in this connection is the influence zone, that is to say, the area around the tunnel where the pore pressure and groundwater level are affected.

CALCULATION OF INFLOW INTO A ROCK CAVERN

For a tunnel located in an ideal isotropic homogeneous rock massif with constant permeability at a certain depth below groundwater level, the inflow into the tunnel can be calculated from

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

wherein
h = depth below groundwater table
r = radius of the tunnel
k = permeability of the rock.

FLOW INTENSITY

If the groundwater level is to remain unaffected, the supply of surface water locally must correspond to the calculated flow intensity through the rock surface.

The natural effective supply of surface water in the form of precipitation is considerably smaller than the mean precipitation seen on an annual basis. The effective infiltration coefficient depends on a great number of factors such as topography, the nature of the surface, soil and evaporation. Little information regarding these factors is available for urban areas, but it is assumed that only 15% of the precipitation in the centre of Oslo is introduced into the groundwater.

CONSEQUENCES OF INFLOW OF WATER INTO UNDERGROUND STRUCTURES

RELATION BETWEEN INFLOW OF WATER AND INFLUENCE DISTANCE

Findings from the project “Environment and Community Friendly Tunnels” have shown how the influence distance varies with the distance from the tunnel.

RELATION BETWEEN WATER INFLOWS AND PORE PRESSURE REDUCTION

Problems of pore pressure reduction are often related to deep trenches. When a clay-filled deep trench has reduced pore pressure at the rock surface, a consolidation process will be initiated, and the pore pressure towards the top of the clay will gradually drop. If the clay layer is of a certain thickness, and there is some access to surface water, the upper groundwater level will barely be influenced. The equilibrium pore pressure on stationary flow is then determined by the permeability variations throughout the clay layer, and is linear if the permeability is constant with the depth.

Conditions affecting the natural water supply to deep trenches are the depth, width and length of the trench and the topography of the area. In addition, the following factors should be considered:

- To what extent are there more permeable water-bearing deposits (gravel, moraine) in the transition between clay and rock.
- The extent and orientation of highly water-bearing fissure zones in the rock (eg, diabase passages, crushed zones or faults/tension fissures) and the extent to which they are directly traversed by the tunnel.
- The success of the sealing achieved and how the leakages in the tunnel are distributed locally.

With the geological conditions that exist in the Oslo area, experience has shown that leakage in a road traffic tunnel at about 30 metres depth, without any form of sealing, is on average from 10-40 l/min per 100 metres of tunnel. The potential impact of the pore pressure in the bottom of deep trenches is therefore very great. For comparison, the part of the Fjelllinjen Tunnel that was

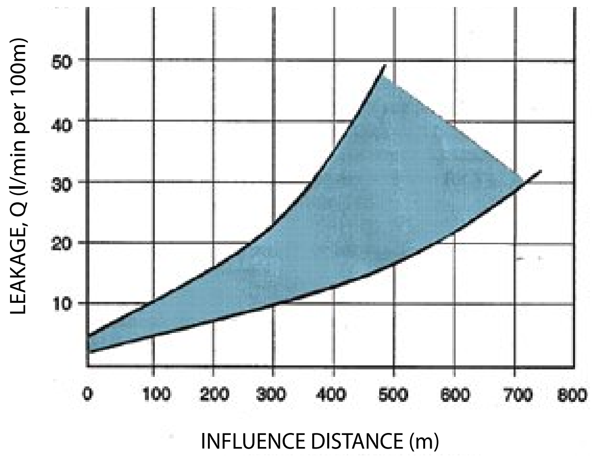

Figure 62. Influence distance in relation to inflow level

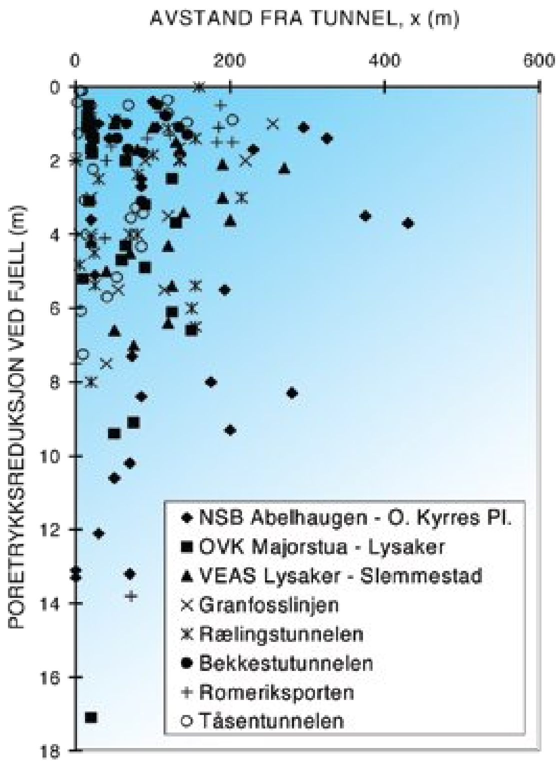

Figure 63. Measured pore pressure reduction at the rock surface in relation to distance from the tunnel

Leakage, q (l/min/100m)	Pore pressure reduction in rock Δuf (m)	Maximum influence distance (m)
< 1	0	0
1 - 3	0 - 2	0 - 200
3 - 5	2 - 4	100 - 400
5 - 10	4 – 6	200 – 600
10 - 20	6 – 8	300 – 800
20 - 40	8 - 10	400 - 1000
> 40	10	>500 - 1000

Table 8. Relation between inflow in the tunnel, q, and impact of pore pressure in deep trench immediately above the tunnel, Δ uf, based on findings from tunnels in the Oslo area.

sealed with permanent pre-grouting has an inflow of about 2 l/min/100 metres.

Experience from tunnel constructions in cambrosilurian slate rock types in the Oslo area has been that the greatest challenges with water penetration have appeared on contact with different hypabyssal rocks.

SUBSIDENCE AND SUBSIDENCE BEHAVIOUR

If the potential for subsidence as a consequence of a tunnel installation is to be evaluated, a detailed mapping of the loose masses along the route must be made first. The surveys should basically cover the entire potential influence area. See Figure 64.

It may be wise to use seismic technology in order to obtain a picture of the depth and extent of clay-filled deep trenches. Ground drilling will be a wholly necessary supplement, also in order to calibrate the seismic gear.

Reliable data as regards the properties of the clay deposits must also be obtained for each individual deep trench. Of particular importance is in-situ pore pressure, whether and to what extent the clay is preloaded (overconsolidated), and modulus number (stiffness).

Anticipated subsidence and subsidence behaviour as a result of a given pore pressure reduction at the rock surface can be calculated using classical consolidation theory. Consolidation parameters may be determined by oedometer tests on representative samples. It is of particular importance to determine the preconsolidation pressure of the clay with high accuracy.

Pressure probing (CPTU probing) may be useful in determining how the preconsolidation pressure varies in the depth direction and across an area. It will normally call for some sound test results which can provide the basis for local correlations between result of pressure probing and preconsolidation pressure determined through reliable tests.

It is absolutely essential to chart the in situ pore pressure state. Without this information it is not possible to determine whether or to what extent the clay has a preconsolidation pressure larger than the prevailing in situ effective stress. It is also important to chart whether or to what extent the area has previously been subjected to temporarily or permanently lower pore pressure than the prevailing state.

The stationary pore pressure that will be reached after a long time can usually be assumed to vary linearly from a depth of 2-4 metres below natural groundwater level or 2-4 metres below the bottom of the uppermost solid dry crust and down to rock. See Figure 66. The reason for this assumption is that the uppermost dry crust / the weathering zone has substantially higher permeability than the underlying non-weathered clay. If there is clay of varying permeability in the deeper horizons, it must be remembered that the stationary distribution will differ from linear distribution.

Both theoretical considerations and measurements in connection with earlier tunnelling works show that the groundwater level in a clay area is not affected to any substantial degree even in the case of the largest tunnel leakages and pore pressure reductions. This is attributable to the fact that the water volumes which flow through the clay, even under large gradients, are very small compared with the natural supply of groundwater through natural infiltration from the surface.

ALUM SHALE AND CHANGES IN GROUNDWATER LEVEL

A permanent lowering of groundwater in an area with alum shale will cause the shale to disintegrate, which

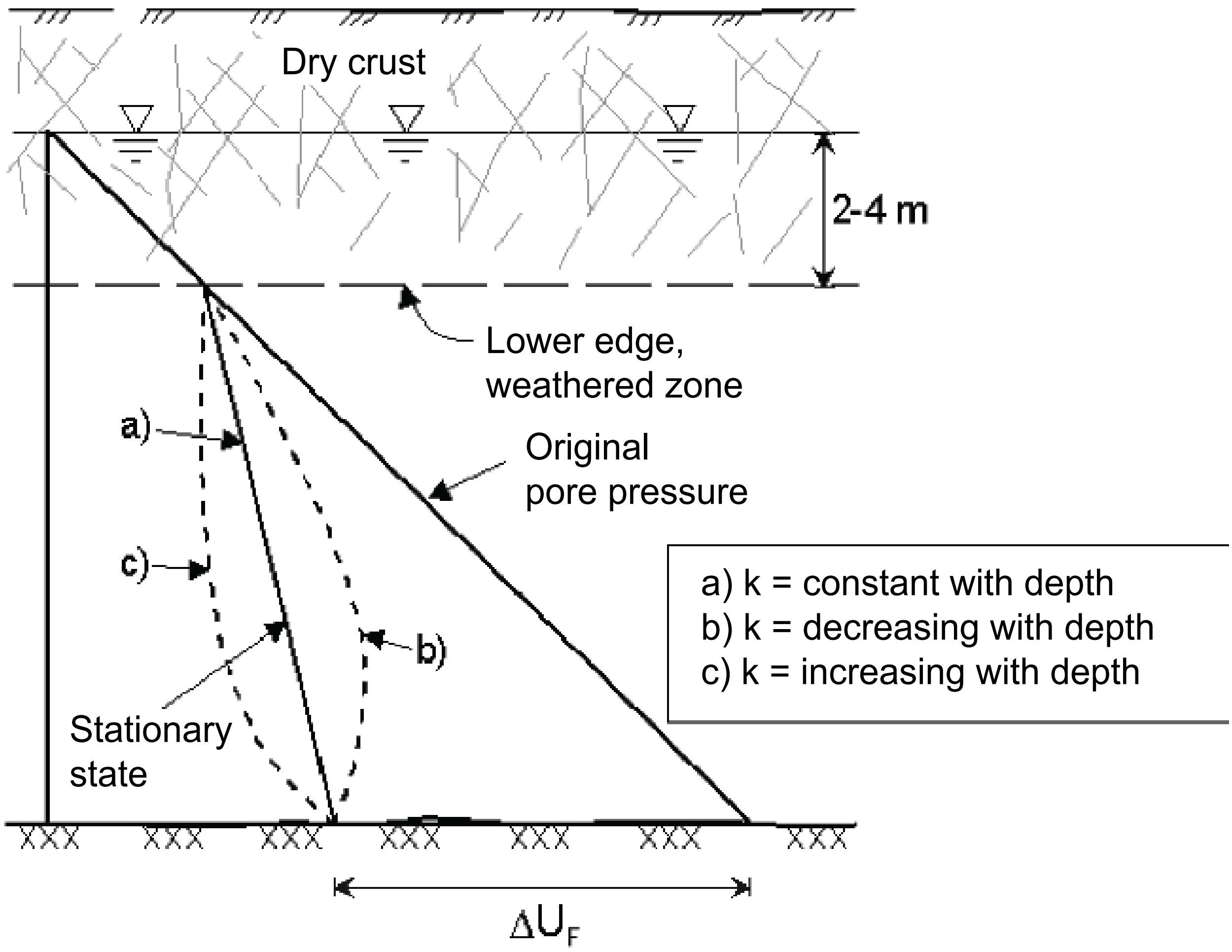

Figure 64. Example of a typical stationary pore pressure state

may cause shrink-swell damage to buildings and be the cause of acidic and sulphate-rich water.

Non-weathered shale may also swell due to a more chemically dependent mechanism. On the supply of oxygen-rich water, the sulphide minerals of the shale may oxidate slowly and lead to a steady swelling which may continue for decades.

Tunnel installations may cause both forms of swelling, due in part to temporary or long-term lowering of groundwater with subsequent supply of water again. Underground works can alter flow gradients and thus water flow rates and volumes which, in turn, may result in a larger oxygen supply than before.

For both mechanisms, the swelling direction is normally on the schistous plane. The swelling pressures rarely rise to more than 1.5-2-0 Mpa and loads from structures of the same size as the swelling pressure will counteract swelling.

Non-weathered alum shale contains varying amounts of comminuted sulphide in the form of pyrites and a special form of magnetic iron-pyrites. When weathering occurs, the sulphides become sulphates. If the sulphate concentration is the groundwater becomes high enough, the typical sulphate attacks on concrete made using Standard Portland Cement will occur. Moreover, the acidic water is highly aggressive towards reinforcement and other metal structures.

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TUNNEL LEAKAGE AND ENVIRONMENTAL ASPECTS

Figure 65. The tarn Puttjern which is situated above the Romeriksporten tunnel. It was nearly drained due to tunnel leakages, and later, due to response from neighbours, restored with grouting and permanent water infiltration (Photo: L. Erikstad).

Severe leakages in the 14.5 km long railway tunnel Romeriksporten between Oslo and the new airport some 50 km to the north initiated a research project on tunnel leakage and environmental aspects.

The construction work took place in the late nineties. The leakages caused damage on the surface, both to vegetation and to buildings. The aim of the research project was to study the effects of groundwater leakage and to develop procedures to quantify maximum allowable water inflow to a tunnel based on the possible or acceptable impact on the surface environment.

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

NUMERICAL MODELLING

Modelling may be used to simulate the hydro-geological conditions before and after tunnel excavation, and to evaluate the relative importance of the different parameters used in the models. In this way, important information about the groundwater conditions may be provided in an early stage in the planning process. The hydraulic conductivity of the bedrock in Norway is generally low, with groundwater flowing along joints and weakness zones. The usefulness of numerical flow models will depend on realistic geological and hydro-geological input data and the boundary conditions established for the model.

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

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

EXPERIENCES FROM 2D MODELLING

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

A second approach involves analysis of a local area with hydro-geological parameters and the interaction with overlying sediments included in the model. In a preliminary study the model was built to illustrate a typical landscape situation with, in a vertical section, bedrock in a local depression or valley bottom, overlying sediment layers and a water saturated area on top representing vulnerable nature elements. Simulations were performed to study how water inflow to a rock tunnel will affect the vulnerable surface area. The simulations were performed with varying hydraulic conductivities of each of the layers in the model. The results indicate that relatively small changes in the groundwater table may have an impact on the water saturated zone on the surface. The thickness and type and hydraulic conductivities of the individual sediment layers are the most important parameters. For example, a clay layer will seal off the groundwater and cause less water inflow. The relative position of the groundwater table adja-cent to the local depression will also affect the amount of water inflow into the tunnel.

EXPERIENCES FROM 3D MODELLING

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

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

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

The model provides a three-dimensional image of the water flow in the rock mass, and more accurate results compared with results obtained from 2D models.The limitation of the 3D model is the need for computer power, which in this case have put limits to the size of the area of investigation. Saturated transient and steady state calculations were performed to predict the amount of water leaking into the tunnel. The results from the simulation show that water inflow is high, with a large drawdown of groundwater. This is mainly caused by the fault zone which contribute significantly to the fracture network in the model.

The effects of cement injection of the tunnel were modelled by reducing the transmissivity of joints cross cutting the tunnel. The results show that a reduction in the leakage rate is observed only after a significant reduction in transmissivity. An extensive injection of the fault zone was shown by comparison to be more effective than a moderate injection along the whole tunnel, although the leakages tend to increase on both sides of the injected section of the tunnel. Before cement injection a leakage rate of 900 l/min./100 m was predicted in the fault zone, with a signi¬ficant drawdown of the groundwater table, which would in effect drain the model. Reducing the transmissivity in the fault zone by a factor of 200 will result in a leakage rate of 50 l/min./100 m, and a lowering of the groundwater table of 5 m.

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

TUNNELLING EFFECTS ON THE GROUNDWATER TABLE

The effects of tunnel excavations on the groundwater table were shown by collecting data from several wells in the close vicinity of recently built tunnels. As would be expected, groundwater drawdown becomes less significant away from the tunnels. Changes which are caused by the tunnels are not observed beyond 200 to 300 metres from the tunnel axis. The available data shows, however, no clear correlation between leakage into the tunnels and the measured groundwater drawdown.

In general, leakages of more than 25 litres/minute/100 m tunnel causes significant drawdown of the groundwater table (more than 5 to 10 m), and a leakage rate of 10 l/min./100 m or less causes a groundwater drawdown of 0 - 5 metres.

ACCEPTED LEAKAGE IN NATURAL LANDSCAPE

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

A mapping programme with systematic registration of the vegetation above several tunnels with documented leakages was carried out by the Institute of Nature Research. The aim was mainly to acquire new information about the relation between damage to vegetation and tunnel leakages. Systematic field mapping were not performed previous to the excavation of these tunnels, and the mapping programme thus focused on the registration of visible damage. Surprisingly little damage to the vegetation was recorded in this mapping. Some effects of drainage are easily identified such as ground settlements and dried-out ponds, but for the most part damage to the vegetation is not evident. It was concluded from the study that this may be due to the fact that the actual damage is unsignificant or is healed, that damage to certain species is undetectable due to lack of pre-tunnel investigation or that the scale of detail in the registration is not the appropriate for this type of investigation.

The areas most sensitive to groundwater draw down as a result of tunnel excavation are gene-rally quite small compared with the area above the total length of the tunnel. A method is pro-posed to locate potentially vulnerable areas and to classify the vulnerability of nature elements at the early stages of tunnel planning. The method involves an initial identification of the potentially sensitive areas from regional mapping. Features such as local depressions in the terrain are isolated, as these often contains water-dependent vegetation. The vulnerability of each area identified is then classified by the size of the local catchment area, for example calculated from digital hillshade models, and field mapping of the local geology and hydrogeology.

The method is well adapted to the most common geological situation in Norway, with a relatively thin layer of soil lying on top of crystalline magmatic or metamorphic rocks. Local depressions in the terrain usually coincides with lineaments such as weakness zones in the bedrock, and this combined information is of importance both in finally establishing the tunnel route, decision of the excavation procedures and in the planning of sealing measures to protect the most sensitive areas.

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

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

The level of acceptance for each area may be converted into a maximum allowable leakage rate along the respective sections of the tunnel.

PROCEDURE TO DETERMINE ACCEPTED LEAKAGE RATE IN SENSITIVE LANDSCAPES

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

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

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

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

The pore pressure reduction decreases with distance from the tunnel, with an average of 2 metres per 100 m horizontal distance from the tunnel. The measurements are locally affected by the thickness of the sediment deposit, sediment types, joints and weakness zones

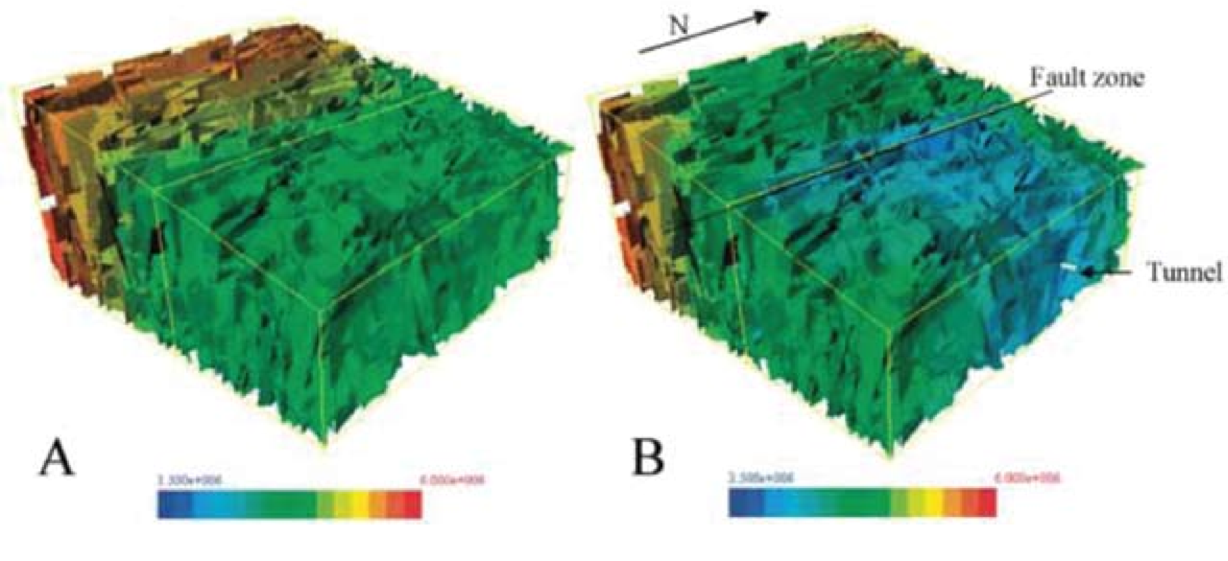

Figure 66. Example of presentation of the steady state pore pressure distribution in a section of the Lunner tunnel.
A: The pre-tunnel situation, with geological data and the model fracture network.
B: Modelled effects of pore pressure change after tunnel excavation. The water will tend to flow towards lowpressure areas (blue)

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

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

An accepted maximum settlement is related to the value and the type of type structures on the surface. For example, two major construction projects in Oslo have requirements to maximum settlements in the sediments above the tunnel excavation sites of 10 mm and 20 mm respectively, in order to keep the possible influence on surface structures to a minimum.

PROCEDURE TO DETERMINE ACCEPTED LEAKAGE RATE IN URBAN AREAS

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

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

WATER LOSS MEASUREMENT AND PERMEABILITY

ROCK JOINTING AND PERMEABILITY

Permeability designates the ability of a material to transmit water or gas. If laminar flow is assumed and it is assumed that the liquid used is incompressible, permeability will be a material constant for the given liquid (viscosity) that is defined in Darcy's law:

$v = k \cdot i$

wherein
v = flow rate
k = permeability coefficient
i = hydraulic gradient

Darcy's law assumes a homogeneous material. However, the permeability of a rock mass will largely be determined by the permeability of fissures and scars. For this reason a distinction should be made between the terms mass permeability and fissure permeability (- conductivity).

Mass permeability designates permeability in an assumed homogeneous material. It may also be referred to equivalent permeability or average permeability.

Fissure permeability provides the conductivity for a physically limited water route. The practical execution of, eg, a water loss measurement (Lugeon test) will be carried out in rock on a scale which means that what is measured is a form of fissure permeability whilst the result, with few exceptions, is passed off as a mass permeability.

The permeability coefficient, k, is measured in a laboratory using Darcy's formula:

$k = q/A \cdot i$

wherein
$q = m^3/sec$
k = permeability coefficient (m/sec)
A = area of sample (m^2)
i = hydraulic gradient

DEFINITION OF THE LUGEON VALUE

Lugeon (L) is defined as the amount of water in litres that is injected into a borehole per minute and per metre borehole at an overpressure of 10 bar.

Lugeon: Q(litre) 10(bar) / time(minutes) measuring length (m) overpressure (bar).

A Lugeon unit that is calculated by measurements of the water loss in a borehole will, with assumed homogeneous conditions, correspond to an average permeability, k, at ~ 1 · 10-7 m/s.

THE LUGEON TEST

This is the most frequently used test in Norway and to some extent in other countries today. The test was originally carried out according to the following criteria:

1) Hole diameter: 46-76 mm
2) Max. pressure: 10 bar
3) Measuring length: 1-2 m

The Lugeon value is thus calculated on the basis of measuring the volume of water that is pumped into the borehole in a 5-minute period at constant overpressure. The measurements should be taken until two subsequent measurement periods give the same injected volume. Measurements in rock of low permeability will require a longer time before the "steady state" is reached.

Today 50 mm boreholes are usually used, but experience has shown that it has no significant effect on the water loss result if the diameter is, for example, 40 mm or 70 mm.

Measurement of injected water amount, Q, is normally made using a mechanical water volume gauge. When water loss is small (< 1 Lugeon), the accuracy of the counter in a water volume gauge may be too low to record the exact water volume. If greater accuracy is required, measuring vessels from water can be pumped should be used. Water gauges must be checked at regular intervals.

Per definition the Lugeon value must be calculated at an overpressure of 10 bar. This means that the fissure water pressure (back pressure) around the borehole must be known. In permeable rock this can be measured in the borehole before or after the test. Where the inflow is small, it is proposed that the hole be "pumped up" and that drop in pressure against time be recorded.

The test can be carried out with both single packers and double packers. In Norway, single packers are most common. Usually the water loss measurements are carried out at intervals of 5-10 metres and this interval length is used in the calculation of the Lugeon value. However, in rock conditions with few leakages, the water will be distributed over such a great length that the value will be lower than, for example, the required value, even though some fissures in the hole are groutable. In many projects, therefore, a standard calculation length of, for example, 3 to 5 metres is used. It is thus assumed that the leakage is spread over this distance instead of the whole test length. What Lugeon value is the most correct is an academic question, but for moderate to slightly jointed bedrock gneiss it must be a recommendation that a calculation length of 3-5 metres be used.

GLOSSARY

The glossary below contains definitions of terminology used in rock grouting. The glossary is an extract of that given in European Standard NS-EN12715 "Execution of special geotechnical work – Grouting", supplemented with common terms in rock grouting.

Agitator	A low-speed mixer which keeps the grout in suspension after it has been mixed in a high-speed mixer.
High shear mixer	A high-speed mixer used to ensure that the mixture is as homogeneous as possible so that individual particles in the cement are in suspension. The term colloidal mixer is also used to designate a high shear mixer.
Set	The condition reached by a grout when it is no longer plastic, usually measured by penetration or deformation. Initial set refers to first stiffening and final set refers to the attainment of adequate rigidity.
Set time	The period of time from when the material is mixed until a significant change in rheological properties occurs. Set time is dependent on volume and temperature and can be measured in several ways.
Durability	Durability against mechanical and chemical degradation.
Bingham fluid	A substance that has both viscosity and cohesion.
Blaine	The fineness of cement is usually expressed as Blaine. Blaine is the specific surface of the cement, and is expressed in m2 /kg. For example, if the surface area of all cement particles in 1 kg of cement is 600 m2, this will give a Blaine value of 600. NB. Blaine is sometimes expressed as cm2/gram. The number obtained then is 10 times greater. A Blaine of 6000 cm2/gram means the same as Blaine 600.
D95	The particle size of cement is usually expressed as D95, which means that 95% of the cement particles are smaller than this size. A typical D95 may be, for example, 50 microns for industrial cement.
Dispersing agent	A substance that alters the surface tension of colloidal drops such that they do not merge but are kept suspended.
Double packer	A device consisting of a pair of seals that is assembled on a grout tube at a certain predetermined distance and which is used to limit the grouting to the ground between the two seals.
Effective pressure	The actual pressure in the grouting material in the ground.
Epoxy mortar	A resin grouting material containing several components which has extremely high tensile strength, compressive strength and bond strength.
Filter press	An instrument used to measure the filtration properties of a grout.
Flow	Term used to refer to the flow rate of the grouting material from the pump into the rock massinn i bergmassen
Yield stress	The lowest shear stress at which extension of a material increases without increase in load.

Flow cone	A device for measuring the consistency of grout, where a pre-determined volume of grout is permitted to escape through a precisely sized orifice. The time of efflux is used to indicate consistency.
Fluidifer	An additive which enhances the flow properties of the grouting material by reducing its viscosity.
Gel strength	The shear strength of a gel. This can be measured at a fixed time after a gel has been mixed or dissolved or when the gel is fully developed.
Gel time	The time it takes for a grout to gel after being mixed.
GIN method GIN value	GIN stands for "Grouting Intensity Number". The method utilises this value as a parameter to delimit the area of grouting to a largest GIN value. This value is arrived at by multiplying the volume of the injected material (in litres) by the grouting pressure (in bar) per metre borehole.
Groundwater level	At groundwater level the pore water pressure is equal to atmospheric pressure, ie, the water level that becomes established in a hole dug in the ground.
Resin	A material that constitutes the base of an organic grout system, such as acrylic, epoxy, polyester and urethane.
Cure time	The time it takes for the grouts to reach design hardness.
Hardening	Increase in strength of a grout after setting.
Hardener	In a two-component grouting liquid, the chemical component that causes the base component to cure.
Fast setting	That a freshly mixed grout stiffens quickly, usually whilst substantial heat is generated. This stiffness cannot be removed and nor is it possible to make the material plastic again by continuing to blend without adding water.
Hydraulic fracturing	Fracturing of the ground caused by the injection of water or grout under pressure that is higher than the local tensile strength and local ambient pressure.
Industrial cement	A cement quality with a particle size of D95 > 20μm. The same cement quality is also known as standard grouting cement or rapid cement.
Grouting material	A pumpable material (suspension, solution, emulsion or mortar) which stiffens and sets over time.
Grouting cement	Standard grouting cement is defined as a cement quality with a particle size of D95 > 20 μm. The same cement quality is also known as industrial cement or rapid cement.
Grouting pressure	The pressure applied during the grouting process and which is measured at specified points (pumping pressure).
Chemical grouting fluid	Any grouting material characterised by being a solution, ie, that it has no particles.
Colloid	A substance composed of fine particles that are dispersed in a continuous medium. A colloidal particle has a size of between 5 and 5,000 Ångström.
Colloidal mill	The same as a high shear mixer, that is to say a high-speed mixer used to ensure that the mixture is as homogeneous as possible such that all individual particles in the cement are in suspension.
Consistency	The relative mobility of freshly mixed grouting material and its ability to flow. The usual measurements are slump for stiff mixtures and flow for more fluid grouts.

Contact grouting	The grout is injected into the boundary between concrete structures and the ground or rock mass.
Particle distribution	The particle distribution or sieve curve of the cement is also important for its ability to penetrate. Just as it is important to have a good sieve curve for shotcrete aggregate, it is important to have a fine curve for the cement. A well-graded sieve curve will give a more stable mixture than cement with a singe-graded sieve curve
Lugeon value	A relative unit for transmissivity measured in litres per minute per metre borehole with a diameter of 75 mm at an overpressure of 1 MPa in rock.
Solution	A liquid formed by dissolving chemicals completely in water to give a homogeneous liquid with no solid particles.
Marsh viscosity	Viscosity tests carried out using a Marsh cone. The time it takes a given volume to flow through the cone, expressed in seconds, is called the Marsh viscosity. See also flow cone.
Micro-cement	A very fine cement with an even particle distribution curve where D95 < 29 µm.
Mud balance	Method of measuring the viscosity of grout. The instrument checks volume weight and hence the water to cement ratio.
Newtonian fluid	A true fluid that exhibits constant viscosity at all rates of shear. A Newtonian fluid has no yield stress.
Overburden	The thickness of the rock and soil material lying over the hole to be grout injected.
Oversize	Oversize is a designation used if a fine cement has an excessive proportion of large particles. The large particles will block small fissures in the rock, thereby preventing the smaller particles from penetrating into the rock. Blaine, D95 and oversize all say something about the ability of the cement to penetrate into fissures and cracks.
Packer	A device which is inserted into a borehole to isolate one part of the hole from another. A packer is usually an expandable device that is actuated mechanically, hydraulically or pneumatically.
Permeability	A measure of how easy it is for a fluid to pass through a porous medium.
Polyurethanes	Chemical resins that react by forming foam.
Pozzolan	A siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value, but will, in finely divided form and if moist, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.
Rapid sement	A cement quality with a particle size of d95 > 20 µm. The same cement quality is also known as standard grouting cement or industrial cement.
Rheological properties	The properties that control the flow of a fluid or plastic solid substance.
Batch	Quantity of grout mixed at one time.
Sedimentation	The particles in a grout which, because of gravity collect at the bottom of a container when the material is not stirred.
Cement mortar	A grouting material where the main binder is cement.

Silica slurry	Silica slurry is a suspension of fine silicon dioxide dissolved in water. It is used as a stabilising agent to allow grouting to be carried out using grouting materials with higher w/c without separation occurring.
Foam	Foams that are used in grouting consist of solids surrounding air, usually closed pores. They are formed either by injecting gas into a grouting material or by a reaction between grouting material and groundwater which releases gas.
Specific grout take	The measured amount of grout injected into a volume unit of the ground, or a length unit of the grouting hole.
Radius of diffusion	The theoretical distance the grouting material migrates from the point of injection.
Stable suspension	In stable suspensions, less than 5% clear water should be separated out in 2 hours uppermost in a 1000 ml cylinder with an inner diameter of 60 mm at a temperature of 20oC.
Superplasticiser	Admixtures that increase the workability of the mortar and reduce the viscosity of suspensions.
Suspension	A mixture of liquid and solid materials. It behaves like a Bingham fluid when flowing and possesses both viscosity and cohesion. Particle-containing suspensions contain particles that are larger that the clay fraction, whilst colloidal suspensions contains particles of clay size.
Shrinkage	A reduction in the volume of the grouting material.
Syneresis	The exudation of liquid from a set gel that is not subjected to stress, accompanied by contraction of the gel. Syneresis takes place over a period of months.
Thixotropy	The property of a material that enables it to stiffen in a relatively short time when not stirred, but which when stirred or manipulated is given a very soft consistency or becomes liquid with high viscosity, and that the process is completely and wholly reversible, ie, the viscosity of thixotropic liquids decreases with increasing shear rate and returns to its original value after it has been regenerated for some time.
Admixture	Any ingredient of a grouting material (eg, fluidifier, stabilisers) other than the basic components that are used to alter the properties the grouting material has as solid or liquid.
w/c ratio	The water: cement ratio is the ratio of the weight of the water to the content of dry cement in a grouting material. A mixture of 80 litres of water and 100 kg of cement will mean a w/c ratio of 0.8.
Water content	The ratio, expressed as a percentage between the weight of water in a given grouting material and the weight of dry, solid particles. sjonsmateriale og vekten av tørre, faste partikler.
Water separation	The autogenous flow of mixed water into, or water that flows out of, recently placed grouting material.
Vicat needle	A Vicat needle is used to determine whether the cement has solidified. Roughly, it can be said whether the cement is hard enough for further operations when a Vicat needle does not penetrate further than 10 mm into the cement.
Viscosity	The internal fluid resistance of a substance which makes it resist a tendency to flow.

REFERENCES

http://www.tunnel is the English language home page of NFF, focusing on general society activities and information on rock engineering technology.

Under section Publications one will find information on Norwegian tunnelling technology in general. NFF has prepared a number of English language publications, each focusing on selected rock engineering topics. These are available for free downloading.

For rock grouting the industry has made available a wide range of machinery, equipment and materials.

Typical suppliers for grouting operations commonly used in Norway are
BASF, CODAN, DENEEF, RESCON-MAPEI, TESS. These may be reached on the following web addresses:

http://www.basf-cc.no /
http://www.codan-gummi.no/portal/
http://www.mapei.no /
www.deneef.no /
www.tess.no

Above internet pages are prepared in the Norwegian language. Most pages, however, will guide you to partners, subsidiaries or mother companies globally.

ACKNOWLEDGEMENTS

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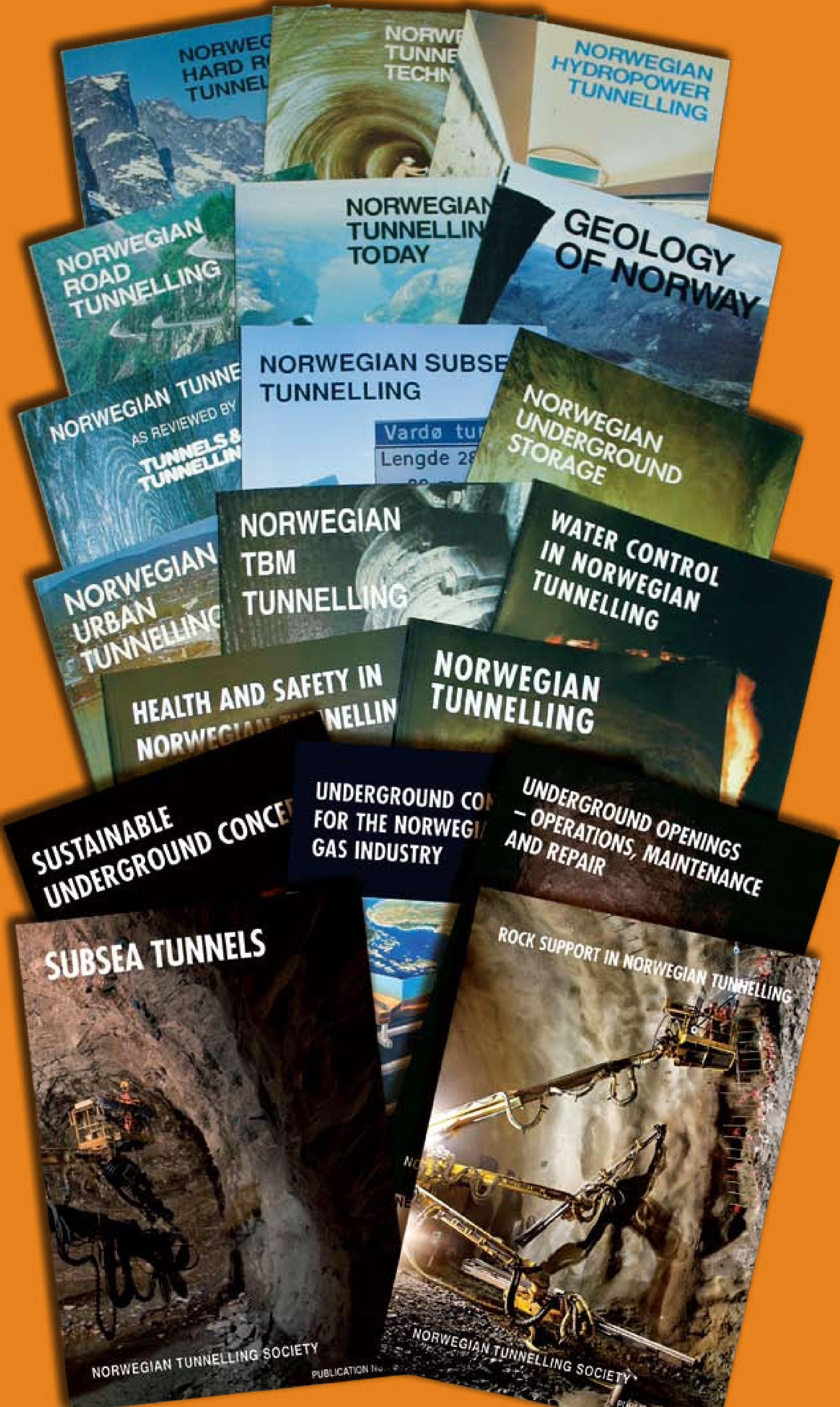
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