ROCK SUPPORT IN NORWEGIAN TUNNELLING

NORWEGIAN TUNNELLING SOCIETY

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1. ROCK SUPPORT IN NORWEGIAN TUNNELLING

The publication “Rock Support in Norwegian Tunnelling” 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 support in tunnelling.

The publication is based on research and experience including achievements during the recent years. It provides suggestions on rock support for decisions to be taken both during the planning and the construction stage. Mapping and exploration methods to determine needs for support are parts of the content. The publication contains project examples demonstrating different situations that require rock support. Preconditions to consider when drafting a contract are also included. The publication does not cover special rock support solutions for TBM-tunnels.

Decisions on rock support are based on competence, available site investigations, common practise and most important, actual observations of the rock mass during excavation.

The basic approach to rock support in Norwegian tunnelling is to utilise the inherent qualities of the rock mass while considering the safety during implementation and operation. Economy during implementation used to be the dominating aspect to consider. Methods like the use of concrete lining as a means of rock support was limited to a last recourse for exceptional situations. Today, however, lifetime costs, “open-time” and environmental aspects increasingly influence the development of tunnelling and hence the selection of support methods.

Our thanks to authors and contributors to the publication, further thanks to the editorial committee of the NFF Handbook no 5 and to the work group that is working on guidelines on sprayed concrete ribs in road tunnelling.

Oslo, May 2010

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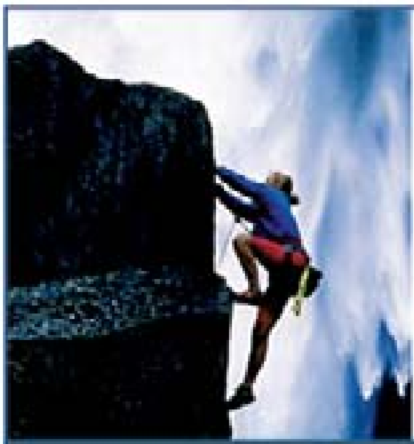

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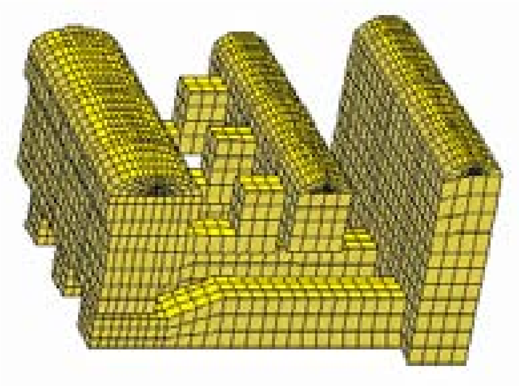

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2. PREINVESTIGATIONS FOR SELECTING ROCK SUPPORT

The purpose of geological pre-investigations related to underground construction projects primarily aims at establishing the geological and rock engineering conditions to be expected.
The investigations assist in selection of tunnel alignment/cavern location and give input to cost estimates for excavation and rock support.

2.1 INVESTIGATIONS DURING THE DESIGN PHASE

The first step includes studies of existing basic material such as literature, maps and air photos. Based on this material one will normally plan further investigations such as surface geological mapping and preliminary sub-surface surveys by means of site visits/mapping, geophysical methods, drill holes and test sampling. Recently some alternative investigation methods like electrical methods (resistivity) and magnetic measurements have been used. These methods have shown a potensial to detect zones with stability problems. NGU (Geological Survey of Norway) has developed a method to detect zones characterized by deep weathering. This deep tropical-type weathering occurred during the Triassic to Jurassic Period (more than 150 million years ago), when Norway was located further south under sub-tropical conditions (about 30ºN). As part of the study so-called awareness maps were developed showing potential "problem zones" containing deep weathering in the Oslo region. NGU has also started to take in use new developments within the use of resistivity measurements. Resistivity measurements can detect thickness, depth and dip of the weakness zones.

Based on a preparatory study it may be possible to identify areas that should be further explored.

Later and more detailed studies will often be based on iteration. The investigation methods become more and more extensive if/when uncertainties have been discovered.

For homogeneous ground conditions it will be easy to obtain data immediately, otherwise investigations may need several surveys to obtain acceptable overview of the ground conditions. Adjustments of tunnel alignment or relocation of a rock cavern may be necessary to minimize difficult rock conditions.

The investigations should aim at detecting critical areas. Special attention must be given to fault zones, areas with little rock cover and other anomalous rock sections that may require extraordinary rock support.

The investigations must be adjusted to the following elements:

1. Local ground conditions stated as degree of difficulty (geology, access to the area, topography, rock cover, etc.)
2. Type of project and the requirements that must be met in regards to safety/stability, expected (or planned) lifetime, and surroundings.
3. The phase of planning or the implementation of the actual project.
4. The type of contract that will be used for the construction.

Items 1 and 2 are most relevant when assessing the relevant scope of sub-surface investigations for the project. Increased efforts to establish adequate information are necessary if: unusual complicated geology occurs, e.g. alternating rock types, tectonic structures, weakness zones, high rock stresses, zero rock stress, or if larger parts of the planned structure are covered by soil, weathered rock, vegetation or is a sub-sea project.

The requirements to an underground structure include safety and stability during the construction period and during the lifelong operation period.
Another requirement is to minimize negative impact to the surroundings. Blast vibrations, noise, ground water lowering etc. can cause negative impacts to the surroundings.

Depending on the damage potential /consequence category and the difficulty-class, a subsurface project is defined according to NS3480 (NS = Norwegian Standard) in different geotechnical project classes.

The geotechnical project class will determine:
- The required efforts to establish reliable geological data
- The required efforts during the planning stage

And, further, the scope of:
- The geotechnical controls during the construction phase
- The control of the project planning

The handbook no. 021 issued by the Public Roads Administration includes a chapter on geological preliminary studies, including guidelines to carry out the sub-surface surveys during the different stages of the planning.

2.2 INVESTIGATIONS DURING THE CONSTRUCTION PHASE

This includes the investigations being carried out at or in front of the tunnel face in order to map rock conditions in detail. The observations will form a base for later detailed decisions on technical solutions or final design of the permanent rock support.

Basically there are 3 types of decisions that need to be founded on facts from investigations:
- Which measures are necessary to carry out in front of the tunnel face (spiling bolts at face, grouting, and possibly drainage)
- Which level of temporary rock support needs to be in place right up to the work face before the next blast can take place
- Which level of permanent rock support is sufficient to meeting safety and quality requirements during the lifetime of the underground facility

Regarding these 3 objectives there are in principle 2 categories of investigations:
- Surveys, registrations and measurements in the tunnel profile or at the work face after each blast.
- Surveys ahead of the workface interpreting the geological situation before the next blast.

In demanding cases (difficult or complex geology) several survey methods should be considered to safeguard a sound base for decisions on suitable support applications.

If the situation is particularly demanding, or the operational costs are high, thorough investigations ahead of the tunnel work face could be a sound investment.

2.2.1 TOP HAMMER DRILLING

Drilling using normal drilling equipment will provide useful information regarding the rock conditions ahead of the tunnel face. In principle a drill hole of this type is to be considered a pinprick which main purpose is to determine the distance to a certain geological phenomenon such as water leakage, altered zones or noticeable cracks or joints. Equipment (MWD, measuring while drilling) and software (Rockma or similar) register and compare and present graphically different sets of data from the drilling. A interpretation of cracking, rock hardness and water ingress in front of the tunnel face is thereby possible.

During the drilling one will be able to point out observations like:
- Faults with gouge material
- Heavily cracked rock
- Rock with a significant degree of weathering/disintegration
- The distance from the test face to the observed phenomena
- Loose material zones
- Water-bearing zones

Investigations by means of hammer drilling is often a very efficient method to verify the geological interpretation, and subsequently to discuss in detail how far

Type of survey	Costs	What can be established
Ordinary top hammer drilling	Low	-Water leakage -Weakness zones -Improved drillability after the injection
Core drilling	High	Direct sampling of the zone material Cracks. Water leakage
Measurement of water-loss	Low	-The water leading capacity of the rock mass. The ability of the rock mass to lead the water -Any post injection improvements
Seismic tomography	High	Variations in the rock mass, zones, and sections as to seismic velocity

Table 1. Review of relevant survey methods on and in front of the tunnel face during the operational phase

excavation should continue before injection, pre-bolting at face or to conclude that more sophisticated investigation methods must be implemented.

Having carried out the grouting, control drilling will be a swift and efficient method to assess the result of the executed injection work and to decide whether further grouting is needed, or to conclude that the tunnel excavation may continue.

2.2.2 CORE DRILLING

Samples of the rock mass will be gained by means of core drilling method. A diamond type cylindrical core bit fixed at the end of drill rods made as tubes is utilised. The analysis of the rock cores reveals cracks, altered zones etc. This provides efficiently the input for decisions on necessary preventive actions ahead of the next blasts. The testing method can be cost wise reasonable if performed without delaying the tunnel excavation works. If such core drillings, however, cause hindrances or stop the actual ongoing excavation work in the tunnels, the indirect costs could be high. In the circumstances test drilling from a niche may be an option. The core drilling method has its limitations. Loose material or incompetent rock mass may cause core loss.

2.2.3 WATER-LOSS TESTS

Water-loss tests are carried out by pumping water into the rock mass. The test results indicate to what extent water seeps into the excavated rock mass, and indirect to what extent penetration can be expected when selecting grouting agent. The method can also be used after the grouting has taken place to prove the effect of the executed grout work. The method is mainly used for projects with strong requirements to the water control during execution.
The water is pumped through the drill hole, using a certain pressure compared to the static water pressure. When it is possible the water-loss measurements should be carried out in short sections of the drill hole preferably 1-3 metres so that the values for conductivity (stated in Lugeon) become as detailed as possible. Experience shows that it is difficult to predict grout quantities based on the measured Lugeon values. The values, however, can indicate the degree of fracturing and the size of fractures.

2.2.4 GROUNDWATER LEAKAGE AND WATER PRESSURE MEASUREMENTS

A commonly used method for measuring the in-leakage of water in tunnels is the use of buckets in combination with the use of stopwatches. It is a useful method for testing the water flow from long hole exploratory drilling or drilling related to grouting. To ensure that all the leakage water from the hole is being measured it is recommended to install a short piece of a rubber hose into the end of the drilled hole. The diameter should match the diameter of the hole.

For subsea tunnels and tunnels with large overburden high water pressure might occur. Installing a pressure gauge on a packer inserted into the drill hole will ease the measuring
A combination of high water pressure and extensive weakness zones require special precautions during the excavation and the implementation of the rock support means.

2.2.5 GEOPHYSICAL DRILL HOLE TOMOGRAPHY

Drill hole tomography includes seismic, electrical or electromagnetic (geo radar) measuring methods. These methods will give a linear picture of the rock mass between two drill holes, or between a drill hole and a line along the surface. The output of the tests can be interpreted based on a given material parameter in the actual section, a tomogram. The tomogram can reflect the distribution of parameters like seismic velocity, the seismic attenuation, the geo radar velocity and/or attenuation, the electrical conductivity. The observed parameters depend on the selected testing method. The interpretation of the tomogram will contain assumptions on the geology (lithology, "quality of the rock mass", etc). The final results are usually presented in drawings of the tested cross section. To further support the interpretation all other relevant and available geological information must be taken into consideration.

The purpose of the tomographic surveys is frequently a detailed mapping of weak zones previously identified through seismic refraction, geological map analyses or other observations. Tomographic surveys are the recourse when observations by means of other methods are found inadequate.

During the operating phase the tomography can be used to clarify the situation in front of the tunnel face when measuring between the horizontal probing holes, or probing holes and the surface. In this way one will get a more detailed picture of the geological situation prior to decision on whether stabilization measures must be implemented or new blasts/further excavation may take place.
Mapping by means of tomography at the work face will halt excavation, and should therefore be used when there are indications of serious stability problems only.

2.2.6 MAPPING, REGISTRATION AND OBSERVATIONS

Several incidents with rock fall in relatively new tunnels underlines the importance of thorough mapping of the rock mass before applying sprayed concrete.

The rock engineering mapping of a tunnel must include:

- All conditions that are important to stability
- Especial critical conditions that require rock support
- Information regarding the geology in front of the work face

This will mainly include registration of information on strike, dip, fractures and fracture magnitudes, fracture material with emphasis on swelling clay and indications of rock stress. It is also important to register changes in the rock mass that might then also change the need for rock support. The results of such mapping should be presented in drawings with vertical cross sections to give an accurate picture of any weak rock zones that may affect the tunnel excavation.

The mapping should also include a simple, visual registration of the conditions of the already carried out support in case there are any visible deformations such as cracks in the sprayed concrete or squeezing in parts of the tunnel profile.

It is important that the rock engineering mapping takes place systematically in agreement with given guidelines. Regardless of whether the Q-method will be used or not, the Q-method parameters will be a useful checklist when mapping. The use of predefined rock support classes will contribute to a predictable running of the project. For stability situations that require heavy rock support particular assessments must be made.

2.2.7 MEASUREMENTS OF DEFORMATIONS

Measurements of deformations can be executed by using tape-extensometers from installed measuring bolts or by using theodolite towards installed measuring points (measuring bolts with a reflective platter). In this case the tape extensometer will provide you with the most accurate result (approximately 1/10 mm accuracy on the measurements). The measuring of the deformations can in cases with minor rock cover be carried out from the surface. Such approach was adopted for the Gjøvik Olympic Rock Hall. (Span 60 m+)

The results are presented graphically, showing deformation and time. In this way one can see if the deformations are declining towards a stable situation, or if the deformations continue unfavourably.

When excavating tunnels in rocks that are exposed to large scale deformations the measuring of deformations are necessary to document that sufficient stability support has been installed.

2.2.8 INSPECTIONS OF CLAY ZONES AND ALTERED MATERIAL

During the excavation of tunnels and caverns the occurrence of clay zones and weak materials will have a great impact on the stability support one must install. It is therefore important that the extent and the propagation of these zones are looked into carefully. If unfavourable clay zones or indications on the presence of swelling clay are observed, four test indicators may be useful:

- Colour test
- X-ray diffraction test to determine minerals
- Measuring of swelling pressure
- Free swelling test

The colour test is a qualitative method based on colour

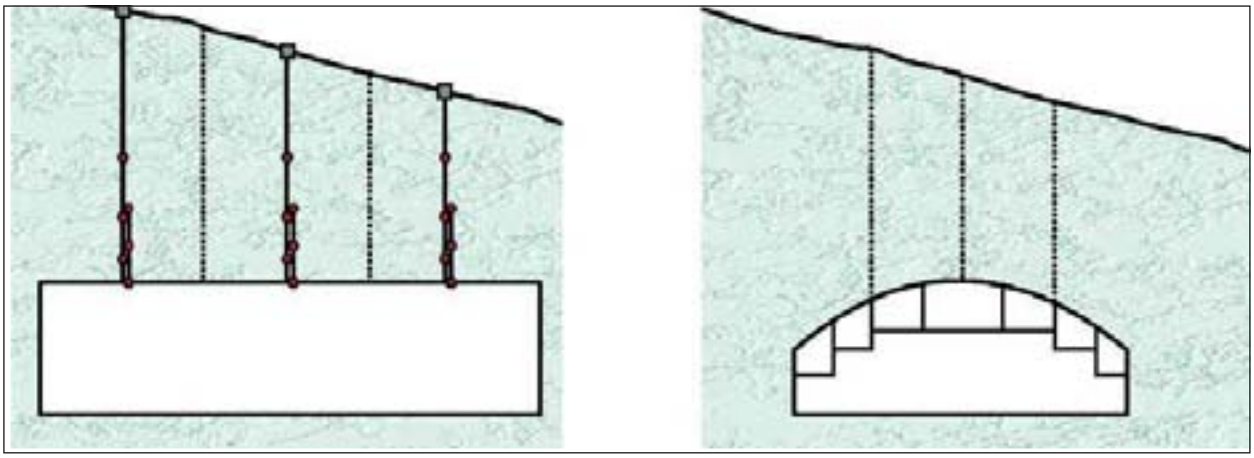

Figure 1. Longitudinal section and cross section that show examples of the placing of the instrumentation during the blasting of a large cavern; span=60m, length=90m

reactions to distinguish between different types of clay. It is a quick and simple method to determine whether there are any swelling minerals or not. It is mainly the minerals of the smectite group that have swelling qualities, for instance montmorillonite, which will become red or red violet when adding a bit of malachite green.

The x-ray diffraction of the rock material can determine whether there are any minerals that can have an impact on excavation and rock support methods; and if available weak zone materials also may give indications on the formation of the zones, their character and size. The method is time consuming.

Examples of rock type- and altered zone material that can be of special interest in this connection:

- Swelling clay material where the water absorption leads to reduced shear strength
- Other swelling minerals (for instance anhydrite)
- Swelling rock types (alum slates)
- Clay minerals and rock types that will provide very bad adhesive qualities for sprayed concrete
- Minerals and rock types that are especially smooth or weak and therefore will influence the stability

Depending on the geological history of the zones, the size, shape, and consistency can vary greatly. Central parts may contain different contents of minerals as well as different quantities and sizes of the secondary rock fragments. The fine-fragmented zones with crushed rock particles may contain clay in thin layers on innumerable sliding planes. Simple or complex clay zones have respectively one or several divided, central clay- containing sections and scattered clay content in the crushed rock. Foliation joints or tectonic are usually clearly defined cracks with crushed material, possibly scattered flags from the secondary rock. In the alteration zones circulating water can have transformed the secondary rock. Hydro thermally transformed rock has been exposed to high temperature or very ion-rich solutions (fluids). Before selecting a support method for clay containing faults or clay containing altered rock, it is of paramount importance to analyse the clay content and the swelling potential.

When swelling clay (montmorillonite) is observed, the normal procedure is to undertake a free swelling test. In order to measure the swelling pressure of the material directly, one must use an odometer with a stepless load variation. The pressure will be measured on a specified area, of a sample with constant sample volume over a specified period of time (24 hours). It should be underlined that the measured swelling pressure is not equal to the expected pressure towards the support structure about to be installed.

The free swelling volume (FSV) is the volume of the water that the test sample takes up during the sedimentation process (V_1), expressed in percentage of the volume of the dry material submerged into water, V_2:

$$\frac{V_1 - V_2}{V_2} \times 100 = FSV\%$$

Project	Material < 20 µm %	Free swelling %	Swelling pressure MPa
Rana power station	-	200	1,04
Rafsnes water tunnel	23	232	1,05
Sira-Kvina Power station	2	170	1,76
New Osa power station	12	140	0,30
Åbjøra power station	13	210	0,89
Øvre Otra power station	5	195	0,95
Hjartøy subsea tunnel	10	450	0,95
Ormsetfoss power station	10	167	0,62
Ormsetfoss headrace tunnel	46	125	0,34
Baneheia tunnel, E-18, Kristiansand	-	133	0,18
Stallogargo tunnel, Rv94, Finmark	-	135	0,20
Hanekleiva	-	140-150	0,16-0,19

Source: Public Roads Authority/NTNU

Table 2. Examples of measured values of clay zones from selected constructions

Several experiments have shown a connection between free swelling (FS) and the obtained swelling pressure. Roughly, the connection can be expressed like this:

FS> 150%	Very active
FS 120-150	Medium active
FS 80-120	Hardly active
FS< 80	Not active

The Norwegian Group for Rock Mechanics (NBG) has suggested the following classification based on the swelling pressure of the clay at a constant volume:

Hardly active	< 0.1 MPa
Medium active	0.1 - 0.3 MPa
Active	0.3 - 0.75 MPa
Very active	> 0.75 MPa

2.3 USE OF SPECIAL REFERENCE GROUPS

For projects where geology is of utmost importance it may be useful to establish a standing special reference group for technical support when /if matters of extraordinary geological complexity should arise. It may be regarded as a supplementing tool for unbiased competent advice. Such reference group should consist of 3 individuals with wide experience and technical competence within the topics where technical problems may be expected.
The technical group shall provide technical assessments and professional advice, especially when uncertainties regarding the technical conditions arise and difficult judgements and decisions of technical character must be made. The duties will normally not include the handling or participation in disputes or matters concerning economy.

It is important to clarify mandate, composition of the group and its organisation. One approach to the selection of group members is for each of the two parties to select one member, and then for these two members to jointly agree on the third member that will also be head of the group. The situations when the reference group should be activated depend on the contract format. For an Owner managed unit price contract the reference group will function as an independent support for the Owner's organisation and his advisors. The Reference Group may also be invited to verify technical solutions proposed by the Owner correspondingly, in a turnkey type contract managed by the contractor, the reference group may serve as decision making support for the contractor and/or as a verifier of the proposed technical solutions.

Main type of situation (range of use of a reference group)	Role/mandate	
	Fixed price/ Procurement and construction	Unit price contract Building owner-run contract
Follow-up on the planned operation when faced with difficult conditions	Control of the project engineering	Control of the project engineering.
Surprising, extraordinary conditions	Decision making support for the contractor, verifying body for the owner	Participating in the planning of the technical solutions
Quality audit	Routine use of the reference group in order to verify the competence of the developer	Only used when needed, decision making support

Table 3. Relevant situations where a reference group may be useful

3. DETERMINING NECESSARY STABILITY SUPPORT

The necessary scope of stability support is determined through structural analysis, numerical models or experience based methods. The assessment must be based on the actual geometry and its influence on the stability, the geological mapping and the observations at the work face. Finally, the purpose of the underground structure establishes a decisive factor when deciding the support method, quality and support quantities. While excavating very difficult geological sections ongoing update, modification or verification is necessary to provide correct support in the decision making process.

3.1 THE ROCK MASS COMPARED TO THE FUNCTION OF THE SUPPORT

Correct dimensioning of heavy support must be based on an accurate understanding of the rock conditions and how the rock support is going to work. In principle the situation where weak rock material may occur, is divided into two main categories:

a. Rock conditions where local reinforcement of the rock mass along the contour of the tunnel will produce stability
b. Rock conditions where reinforcement of the rock in combination with the bearing capacity of the rock mass itself, do not establish a satisfying permanent stability of the structure

These two main categories refer in practice to the mechanical stiffness of the rock mass compared to the mechanical stiffness of the support construction. A useful practical “border line” between these two main cases would be to compare the E modulus of the rock mass to the E modulus of the support construction (for instance a concrete support structure). Furthermore, the rock stress situation in the actual area has an important impact.
If the dominating parts of the rock mass have a low stiffness (low E modulus) the rock stress will over time lead to deformations. Thereby the rock support globally will be exposed to stress from the rock mass, thus becoming a load bearing structure.

If, however, the dominating parts of the rock mass have a high stiffness (high E-modulus), the rock mass will absorb the stress without significant deformations taking place. Loads will in such cases affect the rock support locally only.

There are two situations needing special attention additionally:
either
(i) very low rock stress zones (e.g. low rock overburden, or stress-relieved zones), and
(ii) very high rock stress zones (e.g. very big rock overburden or when big tension anisotropy occurs)

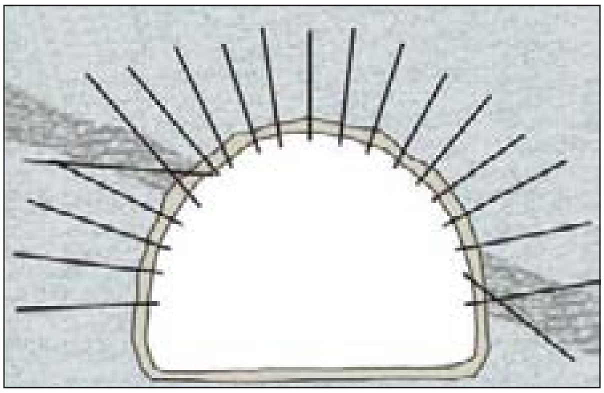

Figure 2. Example a), weakness zone in otherwise competent rock. Local support acceptable as permanent support.

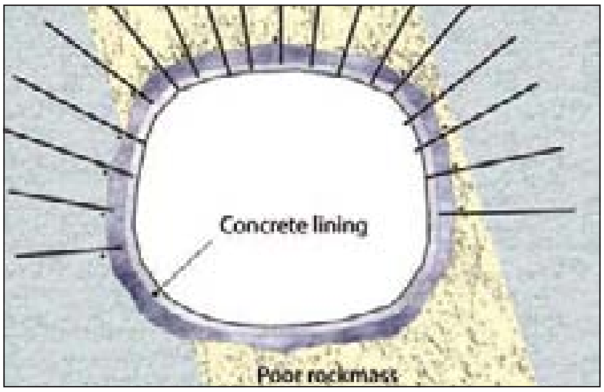

Figure 3. Example b),temporary support by pre-bolting, radial bolts and sprayed concrete. Permanent support by in-situ conrete lining

3.1.1 SITUATIONS WHERE LOCAL ROCK REINFORCEMENT PROVIDES STABILITY

A long-term stability reinforcement of the rock mass presupposes that the rock mass in itself is stiffer than the support construction. Rock support usually consists of lining (sprayed concrete) in combination with steel reinforcement or special support of the local weakness phenomena (bolting). In this way a reinforcement of the rock mass is literally obtained provided the rock mass is sufficiently stiff to withstand the rock stress without deformations taking place.

Rock conditions that belong to this category will in most cases constitute what frequently is described as "altered zones". The practical work for such conditions has been to include the rock mass as integrated part of the rock support and to use other rock support means (bolts, sprayed concrete etc) that have the necessary long-term durability to function as support throughout the projected operating lifetime of the project.

Rock support is limited to walls and roof in the contour of the tunnel. Rock conditions belonging to this category are mainly hard rock with cracks where altered zones with very dense cracking, crushing, and significant weathering/ disintegration and clay material with a relatively small thickness occur. As long as these altered phenomena size wise are smaller than the radius of the underground opening the mechanical qualities of the rock will determine the global stability situation. A local reinforcement (support) of the altered zone is possible when the tangential stress in the contour of the opening is lower than the uniaxial compressive strength of the intact rock material.

3.1.2 CASES WHERE PERMANENT STABILISATION MUST BE ABLE TO TAKE THE FULL LOAD

Long-term stable reinforcement of the rock material (where the rock is the most important load carrying element) is not possible when the dominant rock material has a lower E modulus than that of the installed rock. When the thickness of the weak rock material (rock material with a low E modulus) exceeds approximately 1 diameter of the cavern, deformations that lead to a global load take in the support construction will occur. A long-term durable support must therefore be dimensioned for full load take in case such situation should take place. In practice this means a continuous rock support construction that includes the entire contour including the floor. The support construction must have a geometry that enables it to obtain evenly divided compressive stress. Therefore the design aims at establishing a circular or semi-circular cross section.

Rock support under such conditions must be dimensioned exceptionally based on pure mechanical considerations where geometrical conditions matter. The use of empirical models containing rock mass classifications is not suitable and will often lead to the disregard of critical and decisive conditions that will affect the stability, as these are easily overlooked when using those methods.

Hard (stiff) rock material that is affected by very high rock stress belongs to this category.

3.2 ASSESSMENTS OF LOADS

3.2.1 LOADS RELATED TO CLAY ZONES

The clay zones can mainly be divided into two types as far as load problems are concerned. The first and most common one leads to reduced shear strength, the second leads to swelling pressure.

Clay infected rock mass can react to air and moist so that the shear strength is being reduced over time. In cases where the zones have an unfavourable orientation it can lead to spalling, rock fall or rock burst.

The pressure from the swelling clay zones may cause block fall and sprayed concrete destruction. The swelling pressure quickly drops if the clay has a possibility of minor swelling. The width and the orientation of the swelling clay filled zones are decisive for needed special support measures.

It is not possible by eyesight to decide whether the clay material is a swelling type or not. The swelling activity can vary a lot. Additionally it is necessary to assess the width and the orientation of the zone(s).

An example of the installed rock support on swelling clay is shown in chapter 4.5.7.

3.2.2 STRESS INDUCED LOADS

Stress induced loads mainly occur in "spalling rock", but also in unfavourable non-symmetrical geometry of remaining rock mass such as pillars, protruding sharp corners, etc. in a complex lay-out of an underground structure. The stress that occurs around a cavern mainly depends on the original stress situation in the rock mass and the shape of the openings. Stiff rock types absorb high stress, whereas softer rock types more easily will deform. In general, rock stress problems occur when stress induced loads exceed the load capacity of the rock mass, especially when high stress are oriented tangential to the rock surface around the opening..

Different types of loads caused by rock tensions might be:

- Brittle fracture/ spalling rock mass
- Combination of plastic and brittle fractures over time
- Squeezing

The underground openings in stress- exposed areas should have a simple/ideal geometrical design without niches.

The excavation of an underground opening will affect the virginal stress situation. The stress generated around an opening normally depends on the magnitude the orientation of the principal stresses (σ_1, σ_2, σ_3) and the geometry of the opening. Normally the virginal stresses are anisotropic, i.e. it is a difference between the major and the minor principal stress. Therefore the tangential stress will vary along the periphery of the opening. If there is a big difference between the major (largest) and the minor (smallest) principal stress, the rock stress may create problems, even at moderate stress in the rock mass.

According to Kirch's equation, the tangential stress will reach a maximum ($\sigma_{t\ max}$) when the major principal stress σ1 is tangent to the contour. Correspondingly, it will become a minimum ($\sigma_{t\ min}$) when σ_3 (the minor principal stress) is tangent.

$$\sigma_{t\ max} = 3 \bullet \sigma_1 - \sigma_3$$

$$\sigma_{t\ min} = 3 \bullet \sigma_1 - \sigma_3$$

Asymmetric geometry will strongly influence the magnitude of the tangential stress. This means that at sharp corners such stress will become very big, in extreme cases up to 10 times the magnitude of the major stress.

The deformation modulus of the rock mass (the E modulus) will have an impact on the tangential tension. For openings where accurate drilling and cautious blasting took place or for bored openings where TBMs were used the stress component close to the contour frequently rather high. In the softer rock mass or more cracks occur the maximal stress component will be observed somewhat farther from the contour (slightly further away from the rock)

In situations with minimal rock overburden and lack of rock stress or in situations with high rock stress and large deformations it is necessary to consider the excavation method, the temporary and the permanent rock support at the same time. In the circumstances the installation of the permanent rock support must take place at the work face. Aspects to consider are spiling, permanent bolt types, length of the blast rounds, part section excavation, temporary support by means of freezing, and time lap (wet edge time) from established opening until the permanent rock support may be installed.

Permanent support of the cross section will act in combination with the rock mass, but function as a pure support structure for the masses above. The structural analysis must hence take into consideration the masses above as well as the own weight of the support construction. If the loads are symmetrical, the installed support may be considered as a pressure arch. If the loads, however, are asymmetrical the dimensioning of the support construction must allow for

	Tunnel form								
									
A	5,0	4,0	3,9	3,2	3,1	3,0	2,0	1,9	1,8
B	2,0	1,5	1,8	2,3	2,7	3,0	5,0	1,9	3,9

Table 4. Form factors; A for roof and B for walls

Cracked rock and low stress level	Average stress level	High stress level
Outfall of blocks Bad arch effect and danger of collapse Typical for openings with limited overburden	The stress level is sufficiently high to maintain the arch effect Rock stress problems may occur for anisotropic stress situations	Stress induced rupture, scaling, and cracks for brittle rock types Time dependent deformations for rock types with plastic qualities

Table 5. Stability problems as related to stress levels

stress, strain, bending, shear forces and buckling. In some cases partial support of shallow openings from above surface may be relevant. After removal of soil, a concrete slab placed over a weak zone could be installed ahead of the rock excavation beneath the slab. The dimensioning of a concrete slab must take place in agreement with the actual standard requirements, (e.g. NS3473- Projecting of concrete construction- Calculation- and Construction rules.)

A practical method to assess the tangential stress for competent rock masses was developed by Hoek and Brown.

Tangential stress in the roof: $\sigma_{tr} = (A \cdot k - 1)\ \sigma_z$
Tangential stress in the wall: $\sigma_{tw} = (B\text{-}k)\ \sigma_z$

A and B = roof- and wall coefficients for different profiles
k = the relationship between the horizontal and the vertical stress
σ_z = the vertical stress

In Norway the k-value often will be between 2 and 3. For the horseshoe shaped cross section the tangential stress in the roof will vary between:

$\sigma_{tr} = 5.5\ \sigma_z$ and $8.6\ \sigma_z$

The tangential stress in the wall can be expressed:

$\sigma_{tw} = \sigma_z\ (2.3 - k)$

In competent rock such as granites, granite gneisses, and quartzite, spalling will occur where the tangential stress exceeds the strength of the rock. The degree of spalling will vary from heavy rock bursts to light spalling. For high rock stress it is of utmost importance to install the correct rock support at the right time.
How to install rock support in high stress situations is described in chapter 4.5.4

3.2.3 WATER PRESSURE

The water conditions in the rock mass influence the rock stability. The water flow through the cracks and faults will normally reduce the strength and the shear force capacity of the rock mass. This especially applies to the chlorite and the smectite groups of rock (swelling clay).

In very water bearing- weakness zones, or zones that are crossed by cracks under high water pressure, flushing of crushed material and gouge may cause severe stability problems. In such circumstances rock support installation is difficult.
During the drilling, water pockets under pressure may be punctured. This may cause drilling- and charging problems. During the blasting, inflow of mud and/or sludge from weak consolidated clay zones that contain water may happen. To handle such situation, one should consider drilling of drains around the entire tunnel profile, collaring from an area further back where the tunnel has already been supported.

3.2.4 STRESS OF BLASTING

The stability support that has been executed at or in front of the tunnel face will be exposed to the stress from the following blasting. This can lead to cracks in the sprayed concrete.

Vibrations from the blasting have shown to have little or no effect on the grout around rock bolts.

3.2.5 LOADS INDUCED BY FIRE

In case a collapse of the installed rock support constructions (contact concrete) may cause a collapse of the opening, loads induced by fire must be considered. In such cases the rock support must be designed to withstand the given fire load estimate. The experience from serious tunnel fires have caused the authorities to define tunnel fires requirements in regards to the heat resistance capability of the materials and the construction methods.

For concrete exposed to fire scaling may occur, sometimes like an explosion and collapse of the structure. This is caused by humidity and high steam pressure developed due to the heat. The pressure may increase up to a level at which “an explosion” takes place. By adding monofilament fibre of polypropylene (PP-fibre) the fibres melt when exposed to heat (approximately 140^{o}C). The pores in the concrete are opened and let out the water/steam, which reduces scaling. Normal mix is 2 kg PP fibre per m3 sprayed concrete.
PP-fibre in the concrete matrix has proven to have a good effect as fire protection of PE foam. It is also observed less shrink fissures in sprayed concrete that contains these fibres.

3.2.6 THE IMPORTANCE OF THE GEOMETRY OF THE STRUCTURE.

The basic vault design principle is to aim at evenly distributed compressive stress along the periphery. This is an important design element when planning the cross section of the vault. By aiming at an arc effect, the loads will be transferred by the pressure forces on the different blocks. In the classic arc the rock blocks are therefore arranged so that compressive stress only is transferred from one block to the next.
The span of the tunnel vault is decisive for selection of support method, especially bolt dimensions and bolt lengths. Increasing span gives a higher risk of block falls into the shape of a wedge, and therefore longer and

stronger bolts are needed. In case a concrete support structure is needed, one must also assess the reinforcement.

When excavating complicated openings such as crossings with unfavourable cross sections , overhang,

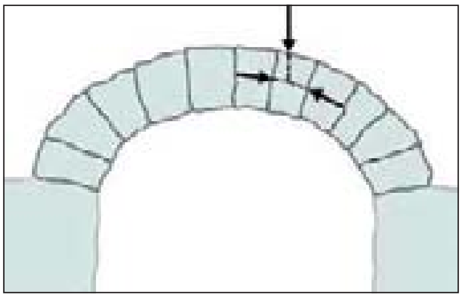

Figure 4. Example on the importance of geometrical design as to the stability of the vault

Figure 5. Example on the importance of the size of the cross section of the underground opening as related to the stability.

protruding corners, remaining columns etc. special requirements as to the excavation and to the support methods are required. The stress around openings with little overburden is normally fairly low. The stability depends in such cases of the possibility of blocks and wedges falling out.
Placing several tunnels or caverns close to each other will influence the stability conditions. The stress situation in the rock mass will change when excavating an

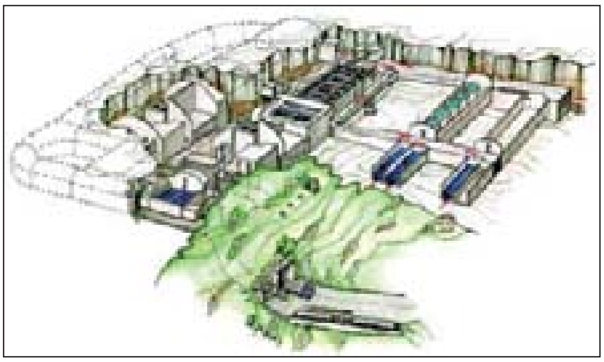

Fig 6 Example of a complex underground installation. The New Oset water treatment plant in Oslo.

underground opening. By excavating several openings in different directions the stress situation becomes more complicated. The redistribution of the forces and the concentration of stress may lead to an increased need for support in columns, rock pillars, corners, and crossings. Spalling may occur even at moderate rock stress.

By connecting several subsurface openings or by placing the openings close to each other it is important to map the weak zones and the dominant cracks. It is important to avoid critical weakness zones in connection with crossings, freestanding pillars or protruding rock heads.

3.3 THE USE OF NUMERICAL MODELS

In connection with the design and the structural analysis of underground facilities in rock, numerical methods are mainly used to evaluate the stress and the deformations. Crucial for the output is the modelling of the relevant location. The quality of the input parameters determines the output. It is difficult to select the correct parameters. Hence, numerical analyses will then also to greater extent develop into studies of the parameters rather than into actual calculations with a defined answer.

The deformation of the rock mass with or without installed support can be modelled with the aid of numerical analyses. It is possible to simulate added layers of sprayed concrete in addition to the bolts. When modelling and practical tests in the opening takes place simultaneously, the results of the modelling can come very close to real life.

When implementing an underground project during difficult /adverse conditions or for wide spans, in localities where limited previous experience exist, numerical analyses can be a means of additional support in the design stage.
Numerical analyses can be used to assess the consequences of having several subsurface constructions close by. The analyses can be used as tools to optimise the design and to assess the need for support. The studies of parameters can show which parameters are critical, and to decide for which sections it is of importance to obtain better or more reliable input values.

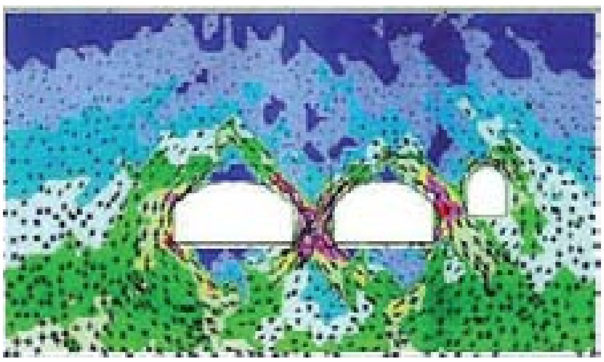

Fig 7 A picture that shows the increased stress in the pillar between the two openings. The model is used to assess the optimal width of the pillar.

Software for modelling from several suppliers is available in the market. Phase2 is a two-dimensional, elastoplastic, final element programme that may be used to calculate tensions and deformations in subsurface constructions.

The programme can establish complex models where stability support in the shape of bolts and sprayed concrete can be taken into consideration.

For the design of complex and demanding subsurface facilities, modelling and numerical analyses are relevant tools. It is important to look at the three dimensional picture of the situation when the simplified 2-D models are used.

3.4 THE USE OF CLASSIFICATION SYSTEMS

Various systems will be useful for rock mass classification and for assistance when assessing the need for rock support. The classification systems should be used as guidelines only. Competent judgement of the actual situation must take place. In demanding situations there will often be one or two critical parameters that need to be analysed thoroughly to determine the necessary stability support.

The Q- system is the most commonly system used in Norway. A vast quantity of data has been collected, both from the tunnelling sector in Norway and inter-

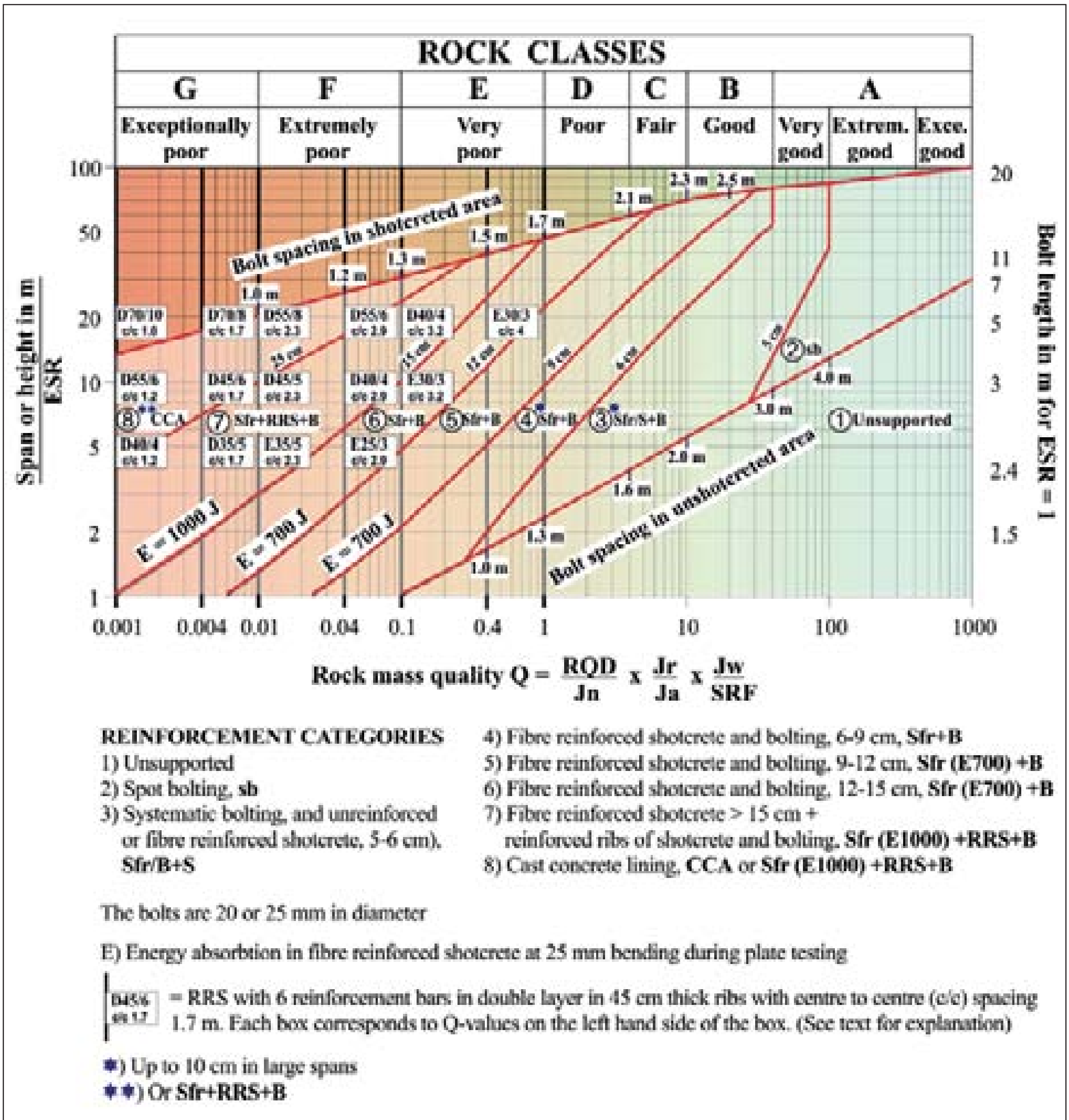

Figure 8. Support table from the Q system.

nationally. The data have subsequently been processed for a comparison between the quality of the rock mass (Q-value) and the executed rock support.

The classification systems are based on a number of parameters that must be assessed at the location. Several classification systems are available.

The RMR system developed by Bieniawski utilises six parameters. These are:
(i) Uniaxial compressive strength of intact rock material, (ii) RQD, (iii) Spacing of discontinuities, (iv) Condition of discontinuities, (v) Groundwater conditions and (vi) Orientation of discontinuities.

The RMi system developed by Palmström also utilises a number of parameters. The system develops the rock mass strength from (i) the uniaxial compressive strength of the intact rock, modifies for (ii) JP = the jointing factor incorporating "the block volume", "the joint condition factor"(joint size –continuity- roughness-alteration).

3.5 TEMPORARY SUPPORT AND PERMANENT SUPPORT

For a regular building owner regulated enterprise, the owner is responsible for the design, the selected construction method and the scope of the construction work. As well as determining which method to use, the scope of the support and the permanent support. Based on engineering geological mapping the permanent support must be dimensioned so that it attends to all the functional requirements of the construction.

The contractor is responsible for setting up the temporary support, ensuring that the support level is sufficient so that the work can be carried out safely.

The temporary support must be executed so that it ensures the safety of the employees who are working during the construction phase. Normally it is carried out so that it is part of the permanent support. The execution of the temporary support must be coordinated so that a necessary engineering geological mapping can take place. In very demanding cases where it is necessary with heavy support and stabilising measures in front of the tunnel face the owner is usually responsible for the detailed design.

3.6 LIFE SPAN OF THE PROJECT AND ENVIRONMENTAL IMPACTS

Life span considerations are important for the design of the support constructions. Assessments depend on the character of the support and may be different for the temporary and the permanent stability support. Normally the temporary support is designed to contribute as part of the permanent support. If this is the case it is important to use durable support components also in the temporary support. For the installation of rock support components not included as part of the permanent support, (for instance temporary bolts) there are usually no requirements regarding corrosion protection.

An important issue when selecting a permanent support solution is the consideration for the durability of the support construction over time, as well as the need for maintenance. Because of the environmental impacts there are also requirements to the minimum thickness of the sprayed concrete, and requirements to the reinforcement cover of the concrete constructions. Concrete can be exposed to different environmental impacts such as frost, chlorides from the seawater or other sources, and chemical attacks. In the Norwegian standard NS-EN206-1 with a national addition, exposure categories of the different effects on the concrete have been given together with requirements concerning the composition of the concrete and other qualities.

In the Oslo area alum slates are the most well-known rock type in relation to the decay reactions in concrete. Steel, reinforcement and concrete elements in subsurface tunnels and/or tunnels in pyritic rock are exposed to aggressive and adverse environment

3.7 TIME AND COST CONSIDERATIONS

Extraordinary activities that prevent the rational use of the production equipment will have consequences economically. Regarding subsurface constructions such activities can lead to a partial or complete disruption of the planned production schedule for the tunnel rig and the related heavy machinery (loaders, trucks etc). Loss of production means loss of income. For projects where rock engineering problems can be expected, rock supporting activities should be prepared in advance. To some degree the contract and the tender documents may define optional construction methods and compensation rules. During the construction period, time constraints and financial ceilings may cause problems for the decision process. If in doubt regarding the stability situation at the tunnelling work face, e.g. if there are any indications that stability problems cannot be ruled out, the blasting must be stopped until the conditions have been looked into and the necessary actions have been carried out.

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CONTRACTING, ENGINEERING, PROJECT DEVELOPMENT & FINANCING

Veidekke is one of Scandinavias leading construction and property development companies. It is the largest in its field in Norway. Veidekke has an annual turnover of approximately USD 1.5 billion and 7,000 employees.

Veidekke's vision is to build a better future for people in all aspects of life. Its business concept is to create value through designing, building and managing projects in partnership with customers who inspire growth and development.

The Skarnsundet Bridge is a 1,010 m long cable stayed bridge with 12 spans. The main span is 530 m long and the A-towers are 152 m high. Veidekke was honoured by the National Association of Architects for this project.

International Experience

Since 1982 more than 60 companies have merged into Veidekke, giving it a wide range of expertise and resources. Veidekke has participated in the development and construction of a large number of infrastructure projects in Norway and abroad. In addition to the other Scandinavian countries, the company's international experience includes countries such as Iceland, Greenland, Germany, Italy, East-Africa, Thailand and China.

Extensive Expertise

Veidekke offers specialised know-how for the construction of

- Residential and non-residential buildings
- Roads and highways
- Conventional and subsea tunnels
- Bridges, railways and airports
- Harbours
- Industrial and oil & gas facilities
- Underground facilities

Excavation work in the Gjøvik Olympic Mountain Hall, which was ice-hockey arena during the 1994 Olympic Winter Games in Lillehammer, Norway. This is the world's largest public mountain hall.

A shotcrete robot working its way through the Norwegian mountains.

Veidekke had the contract for 4,000 m of the 11,000 m long road tunnel through Folgefonna in western Norway.

4. ROCK SUPPORT METHODS

4.1 INTRODUCTION

This chapter describes the relevant methods used for rock support while establishing underground openings under difficult circumstances. It partly advises on which methods to use for the different situations, as well as giving practical advice on the execution.

Firstly, different actions to stabilise the rock mass ahead of the tunnel face are discussed. Actions aim at rock support that satisfies safe excavation of the planned opening. These actions are mainly bolting, sprayed concrete, grouting or a combination of the same.

Subsequently methods are described to support the stability of the actual opening. These methods are different solutions of reinforced sprayed concrete ribs, support systems that can absorb big deformations, or full concrete lining at the work face.

The following support means are discussed:
• Spiling
• Injection
• Jet grouting
• Freezing
• Reinforced sprayed concrete ribs
• Sprayed concrete "vaults"
• Lattice girders
• Deformable support systems
• Different kinds of supporting concrete systems

4.2 RISK EVALUATION IN CONNECTION WITH "ROCK SUPPORT"

All the work procedures must be described in a quality plan, preferably with a flow chart that illustrates the sequence of the work activities before the risk evaluation is being prepared.

Emphasis shall be put on the EHS (Environment-Health-Safety) for underground construction. § 7 in "Regulations on security, health, and work environment in rock work, "FOR 2005-06-30, no 794" gives guidelines for EHS work in underground construction. Additionally it is required for the contractors to establish standard procedures for the execution of the various work activities. Early in the design phase high-risk activities should be identified. The risk analyses of the critical activities should be included in the engineering phase of the project and continue throughout the entire construction phase in the form of "Safe Job Analyses" (SJA). SJA implies that those responsible for the design and those doing the support jointly should review the risk situation; the risk situation may be different from activity to activity. This is especially important when working with weak or altered zones or in other complicated situations where underground openings are excavated and rock support is required.

4.3 SUPPORT AHEAD OF THE TUNNEL WORKING FACE

While working underground within a geological difficult section it is important to implement actions to maintain the planned cross section (theoretical cross section) when excavating.
The most relevant methods are:
• Bolting
• Spiling
• Injection
• Jet grouting
• Freezing

Below one will find guidelines regarding the various relevant methods when strengthening the rock mass.

Q value (guiding)	Support ahead of the tunnel face
0,001-0,02	Pipe screening/jet grouting/freezing
0,02-0,2	Bolting at face
> 0,2	Bolting at large blocks, almost horizontal stratification, low tension, and at outbreak

Table 6. "Rock mass quality indicating the necessary rock support means that should be installed prior to blasting/excavation"

4.3.1 SPILING BOLTS

The main purpose of pre-bolting is to maintain as much as possible of the planned theoretical cross section until the permanent stability support is installed. Permanent support may be sprayed concrete and radial bolting, sprayed concrete arcs or full concrete lining of the profile. Prebolting is normally considered a temporary arrangement, which is not included in the permanent support. Therefore and normally no corrosion protection is required.

In case pre-bolting is combined with sprayed concrete arcs, the bolts will support the rock mass and the sprayed concrete between the ribs. The bolts thus will contribute to even distribution of the loads on the ribs in the longitudinal direction, and may therefore act as a part of the permanent support construction. In such cases bolts with corrosion protection must be used.

Usually the bolt material is of ordinary reinforcement steel quality (normal ribbed steel bars). The bolts are installed in grout, so that the bolt and the rock mass work together.

If very difficult rock conditions occur, one may experience drill hole rupture. Under circumstances self-drilling bolts (for instance «Ischebeck» bolts) that can be grouted might be advantageous.

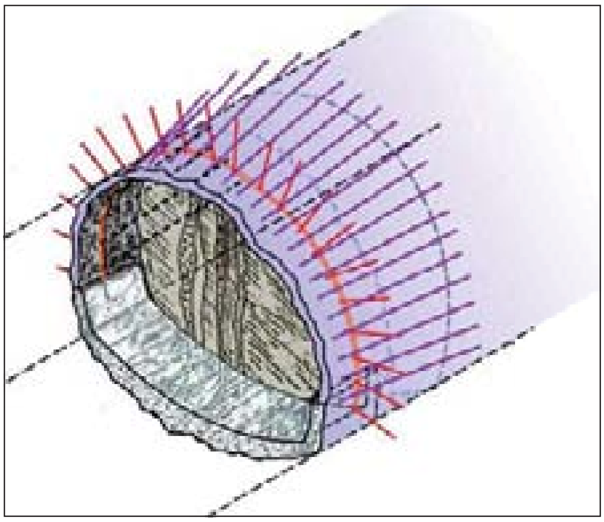


Figure 9. Example of the use of spiling combined with sprayed concrete ribs and invert conrete

To ensure sufficient strength it is advantageous to use Ø32 deformed steel. These bolts will give lesser deflection than Ø25 mm bolts and reduce the risk of rock fall. The bolt diameter Ø25 mm is still frequently used due to easier handling.

At the installation the bolthole is filled with expanding mortar and the bolts are squeezed in with the aid of a drilling machine on the drilling rig. The length of the bolt is usually 6 metres so that 1 meter of the bolt is used to hanging up in the rear edge. Bolts of 8 metres have been tried. The length of the blast, however, should be limited to avoid unwanted deflection.

When using 6 metres long bolts the length of the blast should be between 2.5 and 3 metres. In that way one can install a new series of bolts while there is still overlapping from the previous series.

It is very important to establish safe anchoring at the rear end of the bolt prior to the next blast taking place. The normal procedure is to use steel straps, radial bolts, and fibre reinforced sprayed concrete as back anchorage. There must be a radial bolt for each spile.
If the drilling for radial bolts or the drilling for the next blast indicate that there is a weakness zone close to or right in front of the tunnel working face then the prebolts should be placed a couple of metres behind the tunnel face in order to get sufficient back anchorage.

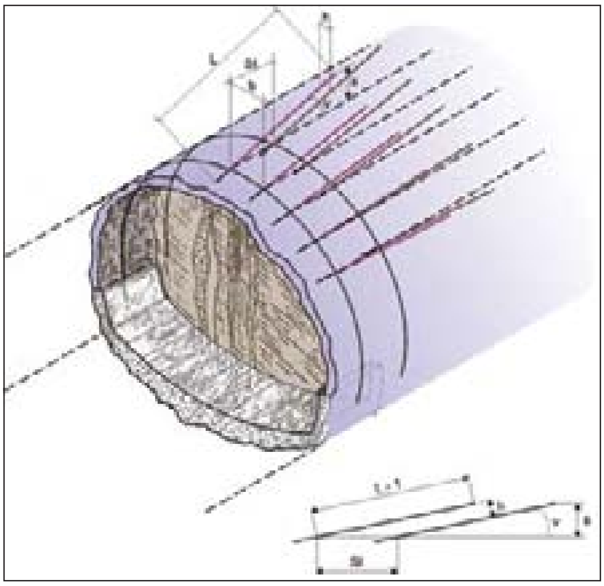


Figure 10. Example of installing of spilingat face for a typical road tunnel profile.

Normal bolt spacing is:
b The bolt spacing is normally around 0.3 metres (0.2 - 0.6)
sl The distance(burden) between bolt rows 1 and 2 is normally 2.3 -3 m
v Recommended bolt angle to the tunnel axis, 10-15^{o}
h The distance between the bolts will then be approximately 0.5 m
l Length of bolt, 6 metres

Densely placed bolts establish a crack line. In zones where the excavated rock mass is very poor, rock fall close to the bolts usually occurs. It might be useful to adopt a method implemented for the “Frøya” subsea tunnel: The angle of the bolt to be reduced so that the crack line comes as close to the tunnel profile as possible. The spacing (distance between the bolts) should at the same time be reduced to 0.2 metres.

For portal surfaces with poor stability, where there will be no stress in the longitudinal direction, it will be necessary to install two rows of bolts to avoid rock fall.

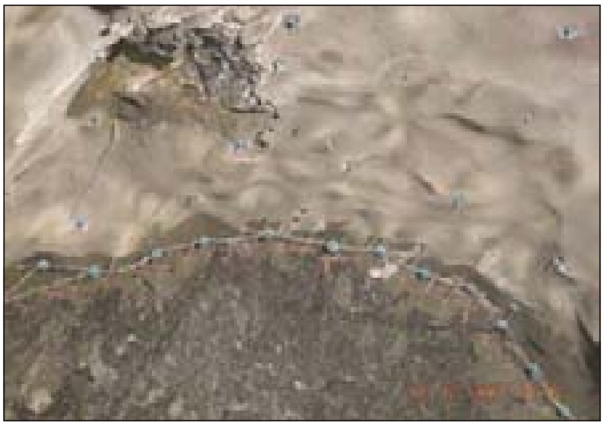


Figure 11. Example of "hanging up" spiling at the Bærums tunnel, photo National Rail Administration.

In sections where the quality of the excavated rock mass indicates that spiling is necessary, it is important that the in-leakages are stopped or reduced by means of grouting. This must take place prior to the spiling installation. Water in the bolt holes can lead to washing out of the grouted masses before the hardening of the grout.

Bolting can be used for the entire profile or parts of the profile. A spiling set should never consist of less than 5 bolts in order to get a good transition/anchorage into the rock. If all or part of the roof is bolted, then one should consider using sprayed concrete ribs as rear anchorage.

4.3.2 PIPE SCREENS

Reinforcing using steel pipes has traditionally been a technique used for tunnelling in loose material. This method of providing support ahead of the tunnel face may also be a method when excavating a tunnel in rock mass with large weakness zones not unlike loose material. This method may especially be suitable if normal techniques for drilling and installation of bolts do not work because of unstable drilling holes.

So far the method has not been used for tunnel constructions in Norway, but is commonly used Central- and Southern parts of Europe. Technical improvements of the method during recent years have made the use easier also for harder rock tunnelling. While installing steel pipe screens a special rig used to drill holes and install the pipes. For hard rock tunnelling when passing through weak zones of limited width such approach would be rather expensive. For the Norwegian tunnelling a relatively new and simpler method for the steel pipe screens installation was activated. This method is discussed below: The method aims at installing a screen of steel pipes in front of the tunnel face over the entire or part of the roof in the tunnel profile. The dimensions of the pipes used are Ø75-120 mm diameter. The steel thickness in the pipes is 5-7 mm.

When building in weakness zones where a considerable improvement of the strength of the rock mass is needed, support in front of the tunnel face using steel pipes may be cost efficient. If the conditions are demanding, this method will have the following important advantages compared to the optional method using spilling and re-bars.

• Drilling difficulties, especially the collapse of drilling holes during the drilling will not be a problem
• The steel pipes can be drilled almost without deviation until 15-20 m. Thus accurate placing of the steel pipes is obtained.
• The bigger diameter of the pipes (compared to ribbed bars) makes the reinforcement stiffer. This gives improved protection against rock fall
• The steel pipes can be grouted,
  - either by pumping the injection masses through the pipe, which also fills openings between the pipe and the rock, as well as penetrating into the rock mass along the pipe, or
  - injecting the pipes in sections at different distances in front of the tunnel face (valve tubes or perforated pipes) in order to get a more controlled penetration of the injection masses along each pipe

The reinforcement effect that can be achieved is significant.

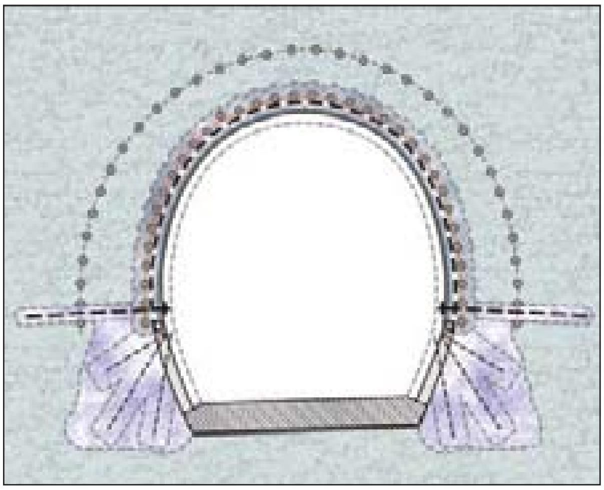


Figure 12. Example of a normal profile when using pipe screens at a cross section of a tunnel

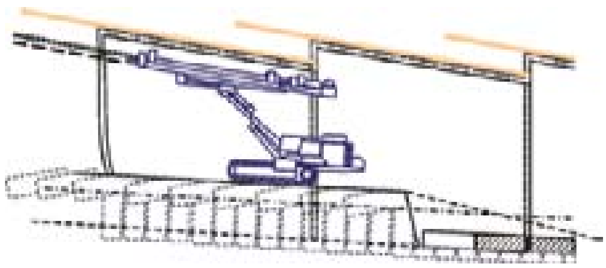


Figure 13. Longitudinal section, which shows an example on how to use pipe bolts

There are several options of pipe bolts in the market, and they can be delivered with different dimensions and one-time drilling bits to be used for rock or loose material.

4.3.3 INJECTION

In great weakness zones, densely cracked rock, chutes with uncompacted material, pre-injection (grouting before blasting) is a method to improve the stability the rock mass before excavating the same.
Normally reinforcement of the rock when injecting is a positive side effect since the injection first and foremost is used to fill cracks and pores in the rock mass to reduce in-leakage The method is relevant for rock mass with poor stability in combination with water pressure. Seen in relations to the Q value of the rock mass, successful grouting before blasting will mainly improve the parameter Jw, but also other Q parameters such as RQD may be improved.

A systematic execution of the grouting before blasting can contribute to increased "stand-up" time enabling excavation and installation of the needed permanent rock support. (Ref.: the Bjorøy tunnel1995). The injection must be adjusted to the actual degree of difficulty, in general one must drill holes distributed over the entire tunnel face, typically with spacing 1-1,5 m. Appropriate length of such screens are usually somewhat shorter than the normal injection screen because of the difficult drillability.

Experience from operation in weathered and densely cracked rock mass have demonstrated that systematic pre-grouting using cement based injection agents has contributed to an improved blasted profile, i.e. less rock fall that again has resulted in improved stability.

4.3.4 JET GROUTING

Jet grouting is a method to improve the mechanical qualities of the loose material. The method mainly involves a replacement of the loose material with cement-based mortar while flushing in combination with injection at a high pressure.

Under special circumstances it may be relevant to assess this method as a means of improving the rock mass quality also in tunnelling. There may be deposits of loose material in the tunnel portal area, little or lack of rock overburden, wide weak zones where the rock mass tends to act as loose material.
A special steel pipe, by drilling, is inserted into the actual rock mass and to a certain length. This pipe is equipped with radial nozzles. Cement mortar at high pressure (approximately 400 bars) are flushed through the nozzles while the drilling pipe is rotated simultaneously pulled back. When the loose material is exposed to stress due to the high pressure mortar from the rotating jets, the loose material becomes further fragmented. A mixture of fragmented loose material and cement mortar is obtained. The result is an oblong, almost cylindrical volume with a certain diameter (40-80 cm) of mortar mixed in situ with the loose material.

Jet grouting presupposes loose material that can be fragmented and washed out during a hydraulic procedure. This means that the method is especially suitable for clay-, silt-, and sand fractions of the loose material with a low or moderate degree of consolidation. Overconsolidated loose material, such as for instance bottom moraine and/or loose material containing a significant amount of large sized fragments will not be suitable for jet grouting (normally there will not be a need to improve such material).

Jet grouting is often used as a method to improve foundations in loose material by means of vertical drilled holes from the surface of the terrain. Horizontal jet grouting when excavating tunnels is assessed as a method to be used in combination with or as an alternative to methods such as pipe screens or freezing. Jet grouting has not been used in Norway in connection with tunnelling, but is commonly used elsewhere in Europe.

4.3.5 FREEZING

For tunnelling through zones of loose material or through rock masses with characteristics of loose material, stabilisation by means of freezing may be an option. The method can be combined with cement injection to stabilise the material with a view to ease drilling of the freezing holes.

For the design of a freezing operation the following aspects must be analysed:

- External loads like earth pressure and water pressure
- Material qualities of the masses that have to be frozen
- The space needed for the permanent support construction
- How to pass through the frost zones
- The length of the section to be frozen and the time of the frozen section before the permanent support are installed.

When freezing sections in subsea tunnels, one must take into consideration that the rock mass may contain a salt solution

4.4 SHORT BLAST ROUNDS AND/OR SEGMENTED BLASTS

When working on tunnels and vaults that need heavy stability support, it is sometimes necessary to implement short blast rounds; sometimes the cross section of the tunnel must be divided into several segments. By blasting the rock in smaller sections and/or shorter rounds reduced vibration will be obtained and the support will be installed more quickly. Pre-bolting/Spiling presupposes that the length of the blast is not longer than max 3 metres; this to ensure that there is enough material at the end of the bolt. In most cases one has to carry out additional, careful blasts. This might include the use of a pilot blasts or blasts that have been divided into smaller sections. The purpose of this is to limit the stress exposure to the weaker rock parts.

4.5 HEAVY SUPPORT IN THE PROFILE

New regulations or «Code of good practice» for reinforced sprayed concrete ribs are being developed by an expert group established within the Norwegian Road Authorities (2010). Excerpts of the work are included in chapter 7.

This is supposed to be the standard procedure for such work in Norwegian tunneling. See some examples below.

4.5.1 SUPPORT FOR ROCK SPALLING SITUATIONS

When installation support elements in rock spalling situations, the timing is essential for the result. Quickly used fibre-reinforced sprayed concrete usually stops the development of cracks and contributes favourably in maintaining the blasted profile of the tunnel.

In rock spalling areas sprayed concrete is usually applied immediately (soonest) after each blast, preferably in the thickness of 5-6 cm and before the bolting takes place. The bolts are of the end-anchor-type with threads. At the supporting steel plates (usually triangle) are screwed onto the construction against the concrete without pre-tensioning of the bolt. Deformations in the rock can lead to significant loads on the rock support, partly causing cracks in the sprayed concrete. After the following blast round, an additional layer of sprayed concrete is applied to secure the continued combined effect of concrete and bolts

Useful work action to be implemented depends on the stress situation in the rock mass:

- At high rock stress where the ratio compressive strength of the rock type σ_c and the biggest principal stress σ_1 (σ_c/σ_1) is between 10-5 one may expect problems with the walls. Then fibre-reinforced sprayed concrete of the energy absorption class E ≥ 700combined with end anchored rock bolts, , for instance bolts with polyester anchorage would be a solution
- At stress ratio in the range of $\sigma_c/\ \sigma_1$= 5-3 spalling in massive rock can occur after approximately one hour after blasting. Then, fibre-reinforced sprayed concrete of the energy absorption class E 1000 and end anchored bolts with discs outside the concrete would be the method
- At stress ratio in the range of $\sigma_c/\ \sigma_1$= 3-2 spalling may occur after few minutes. It is recommended to apply a thin layer (approximately 5 cm) sprayed concrete of high absorption class before the bolting. Normally a new layer of sprayed concrete is added after the subsequent blast.

Figure 14. Reinforced sprayed concrete ribs from the Tanum tunnel and at the National Theatre Station respectively

At stress ratio in the range of $\sigma_c / \sigma_1 < 2$ intense spalling will take place. It will be necessary to use sprayed concrete and bolts, the process must be repeated, may be several times. High energy absorption quality sprayed concrete must be used, sprayed all the way down to the invert to obtain permanent stability. In such situations it also may be needed to support the tunnel work face with bolts.

4.5.2 DEFORMABLE SUPPORT SYSTEMS AT HIGH ROCK TENSIONS

This is rock support with several elements. The main point is to establish a support system able to absorb deformations without collapsing. For very high rock stress several deformation mechanisms can occur. The deformation in hard rock will often be a combination of plastic deformation and brittle fracture. This will develop into a slow squeezing of the tunnel contour combined with powerful spelling phenomena.

The geometry of the tunnel contour is important. It should be approximately circular in order to ensure that the support can function as a pressure arc in the entire tunnel profile, the invert included. The support should preferably be designed as a circular telescopic, deformable ring.

The support system could consist of circular steel girder elements that can slide into each other in the joints, so the entire cross section of the circle can be deformed (decreased) in a homogenous way. This steel structure is further strengthened by applying sprayed concrete in thick (> 40 cm). Evenly distributed slots in the sprayed concrete allows for significant radial deformation without collapse.

4.5.3 SPRAYED CONCRETE RIBS WITH LATTICE GIRDERS

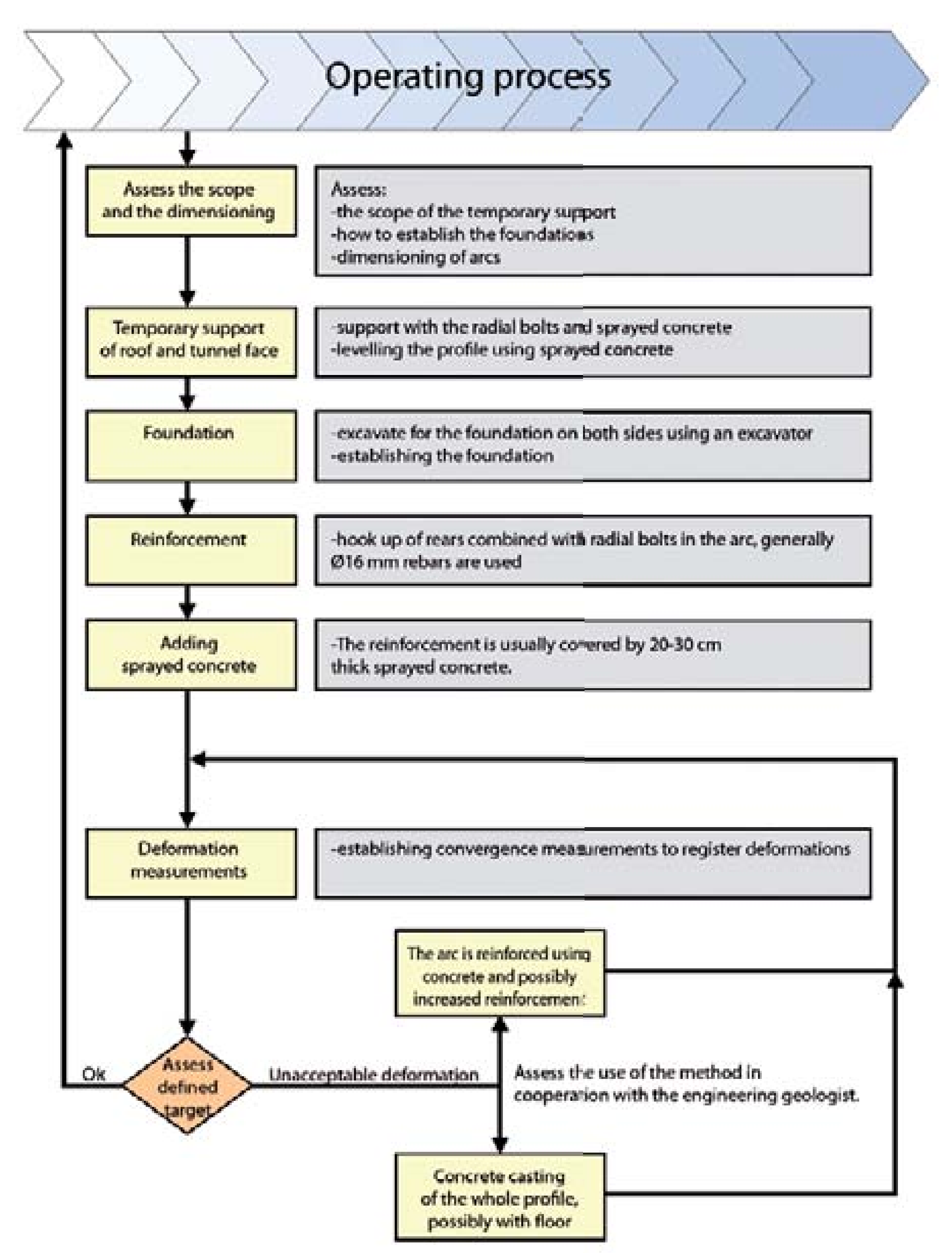
This is a support type that consists of rolled rebar girders. In its most simple form three nos. rebars are assembled in an oblong lattice with a triangular cross section. The radius of the profiles has been adapted to the cross section of the tunnel so that the lattice girders fit to the theoretical contour in the tunnel.
When the different elements of the girders have been assembled one will get a continuous rib made of rebar lattices with a perfect arc from floor to floor. The lattice-girders are entirely embedded in sprayed concrete.
This method of support presupposes that the lattice girdersare placed exactly in agreement with the theoretical-tunnel contour. Big gaps behind the girders may occur. Usually such gaps are filled with sprayed concrete. One can also use the so-called “Bullfex”, which are bags containing cement to be placed where needed and then blown up. The advantage of using lattice girders is quick installation and even pressure arc. The lattice girders must be prefabricated in agreement with the theoretical cross section, thus local adaption to rock fall, inaccuracies etc is impossible. The lattice girders are non-deformable,and therefore not suitable in rock with

4.5.3 SPRAYED CONCRETE “ARCH”

Sprayed concrete established as an arch can be an option to in situ cast concrete arches; especially after alkali free accelerators have become available. This allows for a continuous spraying of thick fibre reinforced layers to obtain the same concrete thickness as the normal in situ cast concrete arches (30-50 cm).

Berg: Rock • Radielle bolter: Radial bolts • Avretting med sprøtebetong: Levelling using sprayed concrete Tverrstykker: Cross pieces • Kamstål: Rebar • Sprøtebetong buer: Sprayed concrete arcs

Figure 16 Example of the use of lattice girders from the construction of a subway (T-baneringen in Oslo)



4.5.4 SUPPORT OF ROCK WITH SWELLING CLAY

If there is a risk for swelling clay induced loads on the walls, it is necessary to establish a support system to avoid in-pressing of the walls. This can be made by means of bolting, frequently in combination with reinforced sprayed concrete or a concreted invert. Placing elastic material prior to the concrete spraying will avoid the swelling induced loads to act directly on the sprayed or casted concrete, see figure 18.

If the swelling clay zone has an unfavourable orientation or a thickness corresponding to category b in chapter 3.1 common practise is to establish full concrete lining.

4.6 IN SITU CONCRETING

Concrete lining by means of in situ concrete is implemented while excavating a tunnel through weak zones with heavy rockfall, massive swelling clay zones, crushed zones with substantial water problems and in the portal areas. With full lining of the tunnel it is established an arch to match the compressive stress from the rock mass.

Reinforcement, anchorage, and possibly a concrete invert must be assessed from case to case. Geometry and loads are decisive factors. It is important to establish foundations to safeguard good transition between the invert and the wall.

Suggested flow chart for the work activities is shown on an other page.

In Norway different kinds of shields/ formwork have been used. In its most simple form it is more or less just a piece of shale moved by means of a wheel loader.

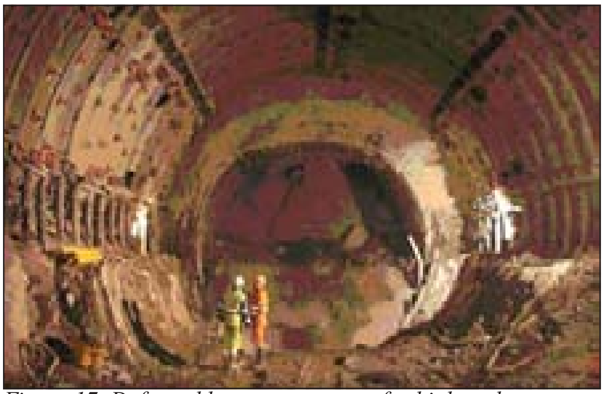


Figure 17. Deformable support system for high rock stress and big deformation. From the new St Gotthard railway tunnel, photo Amberg Engineering.

There are further mechanised shields equipped with hydraulic controls to adapt to the actual profile. The “Flexi-form” is designed to be used for different tunnel cross sections.

For all kinds of formwork sufficient number of openings for pouring and control is important. So-called self-compacting concrete may ease the work.

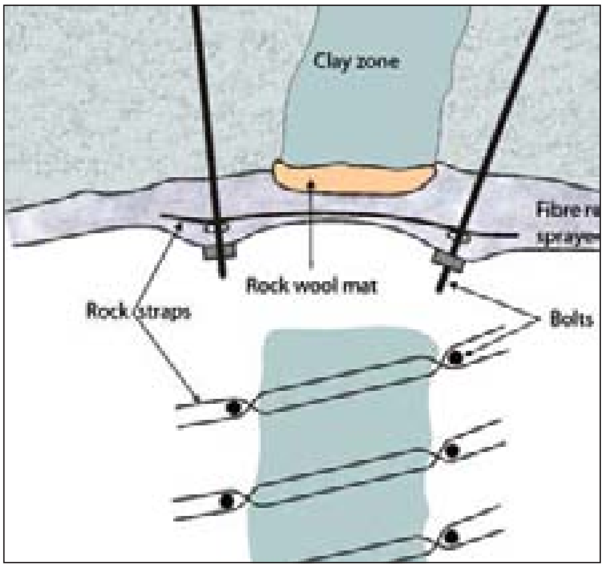


Figure 18. Example of the use of a fender in the supported clay zones section and plane

4.6.1 CONCRETE WORK AT THE TUNNEL WORK FACE

Before placing formwork and prepare for casting the temporary support by means of bolting and sprayed concrete must be executed. Scaling and removal of any loose material from roof and walls must have taken place. The invert shall be cleaned and evened.
Most of this work will be done by excavators.

While excavating a tunnel through "extremely bad material" (e.g. the Frøya tunnel in Mid-Norway) one obtained good results with a medium sized hydraulic front-end excavator. The flexibility of the unit allowed for work activities like scaling, cleaning, concrete work, invert, ditches and also to operate in curved sections.
When the shield has been placed correctly in agreement with the theoretical tunnel profile one must quickly install the remaining formwork for the openings along the roof, walls and invert. Also the remaining work regarding the casting joints must be completed.

In the first part of the casting warm concrete should be used. This will enhance the chemical reactions. Experience frequently demonstrates the difficult process of pouring the upper part of the roof against the rock. Regardless, it is important to pump in a sufficient amount of concrete so that a concrete thickness of minimum 40 cm is obtained.

Remoulding or removing the shield and formwork may usually take place 6-7 hours after the final cast. This depends on the concrete mix and the temperature.

4.6.2 CONCRETE LINING IN THE TUNNEL, BEHIND THE TUNNEL WORK FACE

Concrete lining in the temporary supported tunnel with bolts and sprayed concrete may be an option to the establishing of the permanent lining at the work face.
This is partly a matter related to the cost structure of the tunnelling. The running fixed costs for tunnelling are high. A major part of the running costs is connected to the machinery, the operators and the other equipment undertaking the front end tunnelling work. These resources should therefore be active with its core activities as much of the working hours as possible. The overall costs of concrete lining are significantly less if the lining takes place simultaneously with the tunnel face activities without hampering the progress. For full concrete lining, whether at the work face or behind, the tunnel cross section must be increased to allow sufficient space for the lining.

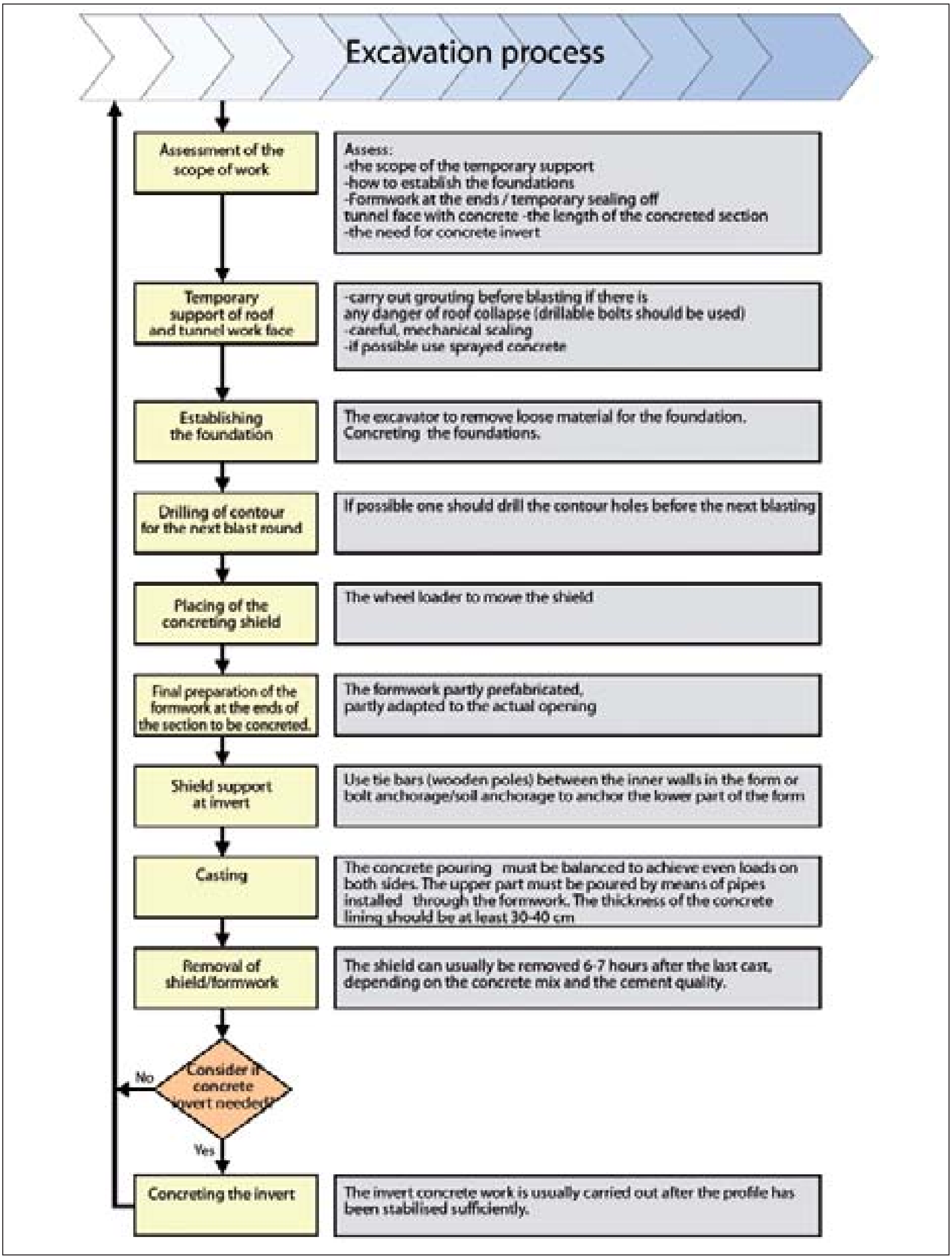
4.6.3 INVERT CONCRETING (RING CONSTRUCTION)

When crossing wide weakness zones it may be necessary to establish additional support of the walls through a concrete invert. This is a relevant approach when walls must be supported by means of concrete lining or reinforced sprayed concrete ribs. For extreme unstable conditions the concrete invert shall prevent the pressing-up of the bottom.

Figure 19. The picture shows an example of the Flexi form that was used at the access tunnel to Melkøya

The concreting of the invert generally takes place subsequent to the rock support of roof, walls and sometimes also the working tunnel face. While excavating for the invert the transition from floor to wall should be rounded to establish an arc.
One must also assess the needs for bolts and reinforcement. This also applies to the invert.

4.6.4 CLOSING OFF A TUNNEL FACE



This is the only recourse when control of the tunnel stability is about to be lost. Typical situation is a moderate caving in or so-called "calving". Such situations occur in combination with crushed rock mass and/or water problems.

If all the well-known support methods such as sprayed concrete, bolting ahead of face, drilling of drains etc have failed, the tunnel must temporarily be closed off. The decision to implement this solution is usually made already while the mucking out of the recent blast takes

place. The shield (formwork prepared for the tunnel cross section) must be set up as close to the tunnel face as possible. While pumping concrete, formwork for the closing off operation is installed. (A frame + wooden sheets) As possible a breather tube into the highest rock outfall is also installed to ease pumping.

The timing is important. Maximum delivery capacity of concrete must be used. Good planning includes emergency planning. In this case concrete supply is important.

Figure 20. Example of formwork installed on a dumper, (picture from the Frøya tunnel)

4.6.5 CONCRETING SUPPORT, DRAINED LINING OF THE ROOF

In Norwegian tunnels rock support by means of drained concrete lining of the roof and the walls and with open (not concreted) invert has been carried out at the tunnel work face or right behind. Drained support constructions in the shape of contact moulding in the walls and hanging walls with open soles have been used. The concept allows for water leakage into the tunnel. Hence the static water pressure is reduced A local build-up of water pressure against the concrete lining may occur if the water conductivity of the rock mass is low or very inhomogeneous.
An example of this type of construction is contact concrete against the rock with open invert, without drains in the walls and the roof. This is the mostly used concrete lining method in Norwegian tunnelling

4.6.6 WATERTIGHT CONCRETE LINING, UNDRAINED CONSTRUCTION

This method presupposes concrete lining of the entire tunnel profile, i.e. the roof, the walls, and the invert. The structural design must allow for full static water pressure and loads from the surrounding rock mass. Shrink fissures in the concrete cannot be tolerated. Tensile stress in the rebars may not exceed the tensile strength of the concrete. The formwork must be designed and established in agreement with the standard regulations. Expansion-, contraction- and pouring joints must be equipped with approved waterstops or in agreement with other approved method. If fissures should occur grouting must take place.

To ensure tightness a fully covering and fully welded membrane should be placed on the rock side of the concrete lining.

This construction method is relevant in areas with strict requirement as to adverse influence of the groundwater level. The method therefore implies that no permanent drainage measures are set up (drain hole, drain pipes or drain ducts) that could inflict the groundwater in the vicinity of the tunnel.
It is common practise to design a non-drained concrete lining as a double structure. The sprayed concrete support is designed as rock support while the inner concrete lining is designed for static water pressure. (As mentioned above possible additional loads caused by deformations in the excavated rock mass must be assessed as part of the load assumptions, depending on the rock mass)

One must design a watertight fully covering membrane that lies between the sprayed concrete and the concrete lining. The membrane must cover the roof, the walls and the invert. The risk for mechanical damages on the membrane cannot be ruled out. One must therefore ensure that the concrete work is carried out professionally. The construction is demanding and the execution presupposes adequate competence.

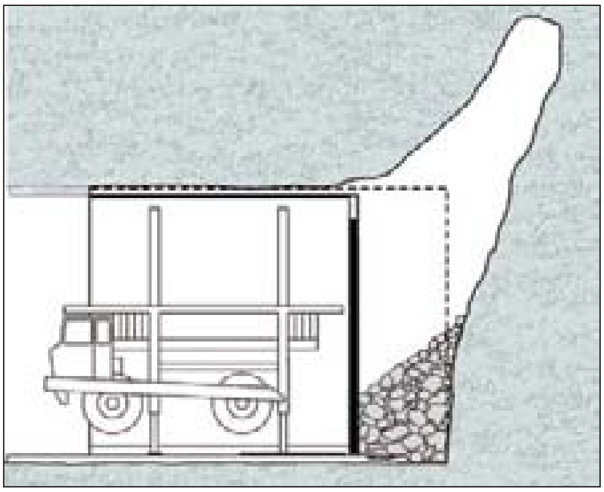


Figure 21. Principal drawing that shows a solution of the blocking of rock fall in the Ellingsøy tunnel.

4.6.7 CONDITIONS THAT HAVE AN IMPACT ON THE WATER TIGHTNESS OF THE CONCRETE STRUCTURE

The most important aspects are:
- Shrinkage/cracking in the concrete joints between concrete sections
- Pockets in the lining

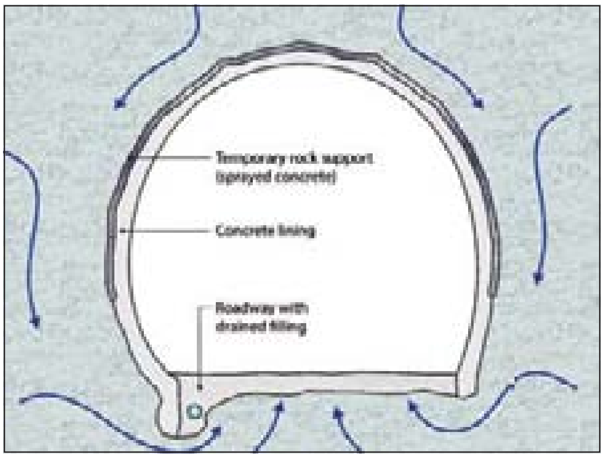


Figure 22. Principle drawing of drained concrete lining

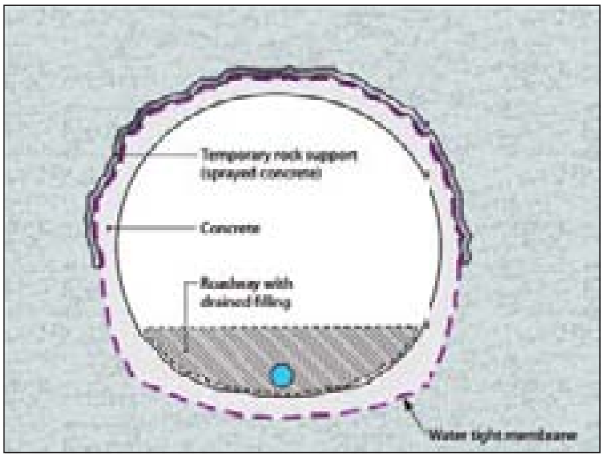


Figure 23. Principle drawing of watertight non-drained concrete lining

Shrinkage occurs during the curing phase of the concrete. The extent of the shrinkage will depend on the composition of the fresh concrete (quantity of cement, etc.), the quantity of concrete (thickness), and the conditions for curing. Different strength classes require different mix designs. A high strength class and/or an increased volume (thickness) lead to increased shrinkage.
The shrinkage is reduced when the reinforcing is appropriately designed and executed. Reinforcement against shrinkage is usually designed as a supplement to reinforcement due to external load exposure. The shrinkage is also reduced when taking appropriate curing measures. This would be a damp environment, watering, sealing that prevents dehydration, low temperatures or in some cases cooling down. (E.g. ice substituting water in the fresh concrete)

While using a shield as formwork, the lining will be divided in sections with vertical joint lines. Since the pouring develops horizontally this may, because of the curing process imply some practical challenges related to the joints.

An example: A 6 m long and 30 cm thick lining section will be exposed to deformations of 2-4 mm (opening and closing) in the joints, if the temperature variations over the year are assumed to be approximately 40^{o} C. The joints should therefore be designed as expansion joints, including constructive measures that attend to deformations and water tightness.

The sectioning of the concrete lining reduces the movement in the joints. Less thermal strain takes place over shorter sections than over longer sections. The risk of leakage through the joints will be reduced.

Gravel pockets can lead to leakage through the concrete lining. This can be avoided by executing careful quality control during the production. Continuous pouring of concrete through evenly divided filling points and adjusted vibration will safeguard the good result. The points for the filling of the concrete must be placed to allow for correct vibration of the concrete. The use of self-compacting concrete is an alternative that in many cases improves the pouring quality.

4.6.8 MEASURES TO MAKE A ROOF LINING WATERTIGHT

For projects where non-drained concrete lining over significant lengths of tunnels is needed and where support of the rock is also needed one should consider using a double shield TBM. This method will provide enhanced safety. This approach includes the use of prefabricated concrete elements (Tübbings) with double water sealing packages.

Regarding the concrete lining, the following precautions will be relevant:

- Reinforcement to reduce the shrink cracks (steel fibre and/or additional reinforcement)
- Swelling strips (ties in the shape of stripes with expan ding material). It can be swelling clay minerals that are placed in each cast joint before the casting itself, both in the horizontal and the vertical joints
- The use of waterstop hoses. These are injection hoses that are placed along each cast joint before the casting takes place. This ensures grouting after blasting of the joints in the cast joints
- Design of the joints between each single cast section, such as expansion joints with measures mentioned above, to an extent that will ensure the water tightness at an expected water pressure

4.7 SUPPORT OF THE TUNNEL WORK FACE

Support of the tunnel face is necessary in situations where work at an open, unsupported tunnel face is considered unsafe. If there is risk for rock fall, from the tunnel face or risk for rupture support must be installed.

The support of the tunnel face can be carried out in the following ways:
- Surface support using sprayed concrete
- Rock mass support by bolting (e.g. polyester bolts or friction bolts)
- Drainage through the sprayed concrete where weak rock mass in combination water leakage.
- Pregrouting of the rock mass. Grouting of the rock mass before blasting will improve the geo mechanical qualities and thereby the stability

Rock mass conditions that call for support of the tunnel face:
- Spalling rock weakness zones with unstable material where a face rupture may develop into and ahead of the face.
- Weakness zones combined with water leakages and a danger of erosion and thus deterioration of the stability.

Rock burst that falls down on the surface of the tunnel face will make the work safety close to the surface of the tunnel face unacceptable. Especially work related to charging the blast and clearing up can be dangerous.

Spalling rock zones containing material with a very low micro stability or material that might be exposed to time dependant stability reduction (for instance minerals that swell when exposed to increased dampness) will be able to set off a rock rupture (slide) that can extend ahead of the tunnel face and upwards into the rock mass over the tunnel profile to be excavated. Such development may easily get out of control. Closing off the tunnel with concrete against the tunnel face area is usually the "emergency solution".

In combination with certain weakness zones water can create a problem to the stability of the tunnel face. This will especially be the case if systematic test drilling and pre-grouting is not executed in tunnelling sections where weakness zones with erodible material combined with water and significant permeability could occur. One should therefore assess the risk for water leakages in the weakness zone material could turn into uncontrolled erosion with a stability reduction and collapse as side effects.

Support measures that might be relevant to increase the stability of the surface of the tunnel face:
- Sprayed concrete to support the surface of the tunnel face
- Bolting, possibly the use of polyester bolts or friction bolts (for instance Swellex) for heavy support of the rock mass close to the surface of the face
- Drainage holes through the surface support (sprayed concrete) to reduce the water pressure as well as prevention of erosion
- Grouting to improve the weak zone (loose material/soil) in case the detail stability is considered so bad that the tunnel face with the relevant cross section will be unstable

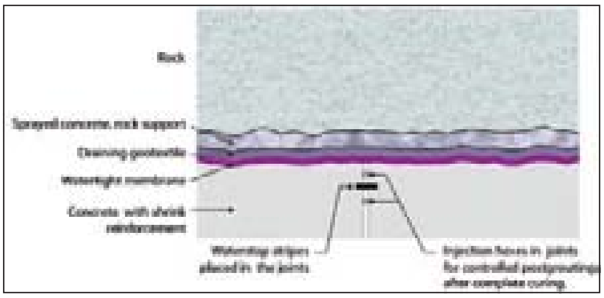


Figure 24. Principle drawing of technical solution for watertight cast

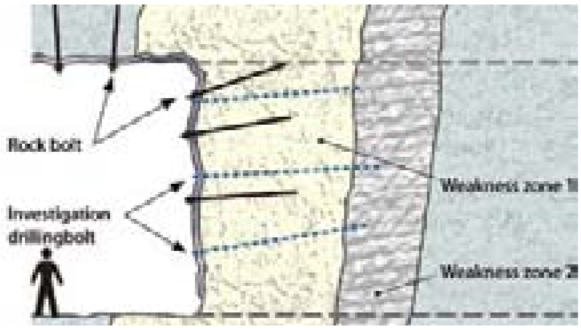


Figure 25. Example of supporting means to be used to increase the work safety and reduce the risk for rupture ahead of the tunnel face

5. SPRAYED CONCRETE TECHNOLOGY FOR HEAVY ROCK SUPPORT

5.1 FUNCTIONAL REQUIREMENTS TO SPRAYED CONCRETE USED FOR THE UNDERGROUND CONSTRUCTIONS

Today sprayed concrete is one of the most important elements when building an underground construction.

The most important functional requirement to sprayed concrete used for rock support is the high early strength. After that, the sprayed concrete has to fulfil the requirements of toughness and strength. It also has to have a long-term durability.

Below some useful experiences regarding possibilities and limitations when using sprayed concrete are mentioned.

5.2 ADHESION

Adhesion is one of the most important properties of any good sprayed concrete support. Cleaning of the surface is very important to obtain good adhesion. In some cases adding of water can lead to further rock fall and deteriorate the stability. A relevant solution could be to use compressed air.

When there are serious water leakages it is difficult to get the sprayed concrete to “hang on”. A solution could then be to drill drain holes (relief holes) around the profile with collaring from the area further back where the tunnel has already been supported.

Sprayed concrete has its biggest limitation on thick clay zones or other places that do not have good adhesive strength. In these cases, other alternatives to provide support must be considered.

5.3 FIBRE REINFORCEMENT

Today steel fibre reinforcement is common in all sprayed concrete that is used as rock support. The normal dosage of steel fibre is approximately 20-40 kg/m^3 with a fibre length of 30-40 mm. There is a big difference in quality of the different kind of fibres on the market so that fibre types with reduced quality require high dosage in order to fulfil the functional requirements of the different prescriptions.

Macro plastic fibre is being introduced to the market. With the dosage 5-7 kg/m^3 this is a relevant alternative to use if big deformations are expected. These fibres also show constancy in aggressive environments such as for instance sub sea tunnels. Regarding the highest energy absorption class, strict requirements are made when it comes to the fibre types.

5.4 MESH REINFORCEMENT

Mesh reinforcement can be used as an alternative if sprayed concrete is used to support clay zones. The mesh is anchored safely with bolts on each side of the zone before applying the sprayed concrete. The standard mesh reinforcement when using sprayed concrete is K-131. The net has mesh openings of 150 x 150 mm and a steel thickness of 5 mm. The most common steel quality that is used in Norway is B5000NA, but an even better result can be obtain if a steel quality that is more ductile (B500NB) is used. It is important that the opening in the mesh is not too small so that there is complete contact between the concrete and the rock surface.

Bar reinforcement is normally not used in pure arch structures, but is part of the sprayed concrete ribs that are sometimes part of the arch (see chapter 4.5.1)

5.5 ADDITIVES- IN GENERAL

Normal additives in sprayed concrete:

- Accelerator
- Plasticizer and super plasticizer
- Retarder
- Stabilizer
- Pumpability improving additive
- Internal curing additive

5.6 ACCELERATOR

When working with sprayed concrete two factors are important from the beginning:

• To make sure that the concrete is hanging on to the rock surface and stays there
• The hardening should start as soon as possible after the spraying to get a high early strength

In order to achieve this, an accelerator is added in the spraying nozzle. The accelerator must mix with the concrete in the nozzle and begin the hardening immediately after the concrete hits the rock. With a jet velocity of 30 to 35 metre per second and a distance between the nozzle and the rock surface of 3 metres this takes 0.1 second.

5.6.1 ALKALI SILICATES AND ALKALI ALUMINATES

Earlier fluid solutions of alkali silicates and alkali aluminates were the most common accelerators on the market, but today they constitute less than 20 %.

• Sodium silicate (alkali silicate)
• Alkali aluminates

The qualities of these have been questioned in recent years because of:

• Very unfavourable health-related conditions when using aluminates
• Alkali reactions with aggregates
• Low early strength when using sodium silicate
• Reduced final strength, especially at high dosage
• Health-risk using highly basic agents
• The safety of the working crew at the tunnel face should be improved with high early strength and better adhesion

In most countries in central Europe aluminates or alkali free accelerators are used.

5.6.2 ALKALI FREE ACCELERATOR

The alkali free accelerator does not contain the alkali elements (Na, K, Li, Cs, Fr, Rb). The properties of the alkali free accelerators are:

• High early strength, > 1 MPa after 1 hour
• No negative effect on the final strength
• No extra alkali must be added - favourable keeping possible alkali reactions in mind
• Can be applied in thick layers (40-50 cm)
• Reduced rebound and downfall
• Reduced quantity of dust
• Improved compaction of the concrete
• Does not build up on the reinforcement bar, better fil ling in behind the reinforcement bar
• The additives are expensive but favourable when considering the overall economy

Since alkali free accelerators are an important presumption when using sprayed concrete to build up the arch, it might be useful to keep a few things in mind when dealing with this additive:

• The equipment must be custom made
• Additives based on lignosulphonate (P agents) should not be used due to retarding and reduced early strength
• Some super plasticizer agents can influence the early strength, but not as much as the lignosulphonate
• The type of cement can influence the result
• Even though the adhesion on wet rock surface has improved, the water has a negative effect on the early strength and can stop the strength development
• The temperature of concrete is important, and must be at least 20^{0}C. Lower temperatures require higher dosages
• Lower temperatures in the air and on the rock surfaces can quickly reduce the temperature of the thin concrete layer, and the early strength is then reduced
• The dosage of the accelerator is important. A trail mix should be executed
• The different suppliers have agents with different properties

5.7 PLASTICISER- AND SUPER PLASTICISER AGENTS (P- AND SP AGENTS)

Plasticiser- and super plasticiser agents are added to increase the ability to disperse cement and silica dust in the water. These agents are necessary if silica dust is used. The floating properties of the fresh concrete is improved as well as the compressive strength in hardened concrete, and the water content can be reduce without harming the workability.

In general P agents must not be used together with alkali free accelerators as it will reduce the early strength, while the plasticiser- and the super plasticiser agents hardly give any retardation both at alkali silicate and alkali free accelerators.

Today synthetic, water-soluble polymer with the following properties are used:

• Increase of early strength
• No effect on the final strength

5.8 RETARDER
The long-term retarding that is used in sprayed concrete is different from the one that is used in normal concrete. A controlled delay of the hardening of the concrete can be obtained, from hours to days. As soon as the accelerator is added the effect of long-term retarding disappears. The results are that:

- The concrete is produced at a suitable time
- The spraying can be executed when it is required at the tunnel face
- The early strength can be reduced
- The need for an accelerator increases

5.9 PUMPABILITY IMPROVER
The pumpablilty of the concrete can be improved by adding small amounts of floating agents. This means that the concrete will move more easily in hoses and pipes. The agents that you use are cellulose derivatives and polythene oxides (Kalko floating agents). The qualities of these agents lead to:

- Lower pump pressure, giving less wear and tear
- Increased cohesiveness, which counteracts the separa tion of water from the solid particles in the concrete

These products are delivered as part of the super plasticizer agents. The air entraining agents are a good and cheap alternative as a pumpability improver.

5.10 INTERNAL "CURING"
Traditionally "external" curing agents such as spray membranes have been used. These consist of paraffin wax suspensions. They reduce drying-out of concrete and thereby prevent shrinkage and cracking, but will reduce the adhesion if you add several layers.

Internal curing agents are offered as a replacement for the spray membranes. The effect of these has not been documented very well, but they work. Especially favourable if spraying of several layers are needed.

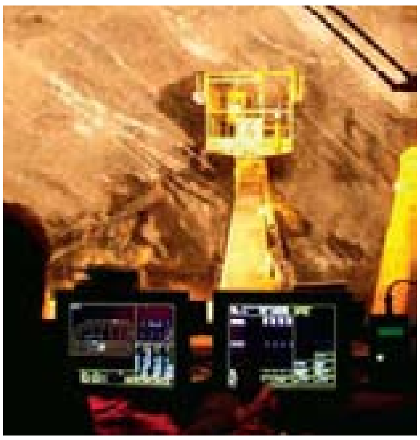
5.11 STABILIZERS
The stabilizers acts as a retarder that extends the curing time without having an impact on the slump.

5.12 EXAMPLE OF A MIX DESIGN

Type of material	Kg/m^3
Portland cement	430-485
Silica dust	15-25
Water (w/c+s) = 0,4	210
Aggregates, 0-8 mm	1530
Super plastifier agent	4,5-6,5
Retarder or stabiliser	1,0-2,0
Internal curing compound	7,0
Pump improver	-
Alkali free accelerator	30-38
Steel fibre	20-40
PP fibre (fire prevention)	2
Slump	18-22
L agents (1:9)	0-1,5

Table 7. Example of a mix design for fibre reinforced concrete

LNS - the mining- and tunnel contractor from the arctic



Leonhard Nilsen & Sønner AS (LNS) was established in 1961, and the LNS-group consists of a total 15 companies. In 2008 the turnover was approx. NOK 2 billion (USD 304 million). The group's number of employees is about 800.

LNS main products are:

- Tunnels, caverns
- Mining contracts
- Rock support, grouting
- Earth moving
- Ready mixed concrete plant
- Production of modules and elements in wood

In 2008 LNS had the largest excavated underground volume by any Norwegian contractor. The last years LNS also has been engaged in Spitsbergen, all over Norway, Iceland, Russia, Greenland and the Antarctic. LNS has recently completed tunnelprojects in Lofast and Fjøsdalen in Lofoten, Svalbard Global Seed Vault in Spitzbergen, road and accesstunnel to opening a new graphite mine in Senja, PPP E18 Kristiansand – Grimstad, 7 two-tubes tunnels of a new 4-lane motorway. Total length is 12 km and two tunnels on the new highway E18 Tønsberg.

Some of LNS projects at the moment:

- Mining operations, Spitzbergen
- Mining operations for Elkem Tana, quartzite mine
- Mining operations for Fransefoss in Ballangen, limestone mine
- Ore handling, Narvik
- SILA, Narvik new harbor
- Salten Road Project, new two-lane road and tunnel, Røvik - Strømsnes
- Project of securing road and tunnel, Lian in Grytøya
- Transfer tunnel, 7 km, Kvænangen Hydro Power Project
- Construction of a new main level for Rana Gruber AS iron ore mine
- New dobbel track, Barkåker - Tønsberg, modernisation of the Vestfold line

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6. DOCUMENTATION AND INSPECTION

To prevent unwanted incidents from occurring, it is important to carry out a detailed registration and description of the weakness zones and the clay zones in the actual part of the tunnel before the spraying. Registered zones must be kept under surveillance. The inspection of the sprayed concrete may reveal cracks that indicate that the stress exceeds the concrete strength.

Additional support may be needed. There are alternative support methods that should be considered:
- Removal of the sprayed concrete (by means of scaling), placing of flexible mats like Rockwool, placing mesh, additional rockbolts and finally sprayed concrete
- Or one can add reinforcement, bolts with subsequent spraying of a new layer on the established layer(s).
- It is important to establish an accurate description and registration of these zones before the spraying.
- Subsequent to the spraying regular inspections at regu lar weekly or monthly intervals should take place

If necessary further support actions must be organised.

6.1 ROCK ENGINEERING/GEOLOGICAL DOCUMENTATION

Mapping of the tunnel at work face must take place when a round is blasted and before the spraying of concrete takes place. It is necessary to allow sufficient time for the mapping. The mapping must be handled by competent staff.
Suitable, safe and sufficient equipment for the mapping must be available. Nearness to rock surface and adequate lightning are important. Samples of fine material and clay for laboratory testing must be collected.

If mechanical scaling by use of heavy equipment takes place observation during this operation will give useful information like how loose the rock is and the size of the blocks falling down while using the spike (hydraulic hammer).

When mapping at the tunnel face, it is convenient to use a mapping form as shown in figure 26. A copy of the observations from the previous blast should be available for continuity while taking down the new observations. The observations to be marked on the drawing, which represents roof and walls, all folded out in one level.

The rock engineering documentation should include the following:
- Type of rock
- Joint set, information, and distance between the cracks
- Joint filling material
- Possible water leakages
- Joint roughness
- Rock stress observations

The rock engineering observations should as soon as possible be converted into a finished drawing/log as shown in figure 27.

6.2 REGISTRATION OF INSTALLED ROCK SUPPORT

Executed support actions should be logged and cover the placing of the bolts, the length of the bolts, the thickness of the sprayed concrete, etc. Alternatively, the placing of the bolts can be registered in the data system of the drilling rig and be transferred to a data programme of its own.

6.3 TESTING AND INSPECTION

Test results must be included in the logs as part of the documentation for the tunnel. Possible samples taken from the rock, clay, water, concrete or for instance test pulling of the bolts must be documented, stating place and test results. The test place should be marked on the drawings.

6.4 NOTES ON THE LOG

The finished log including geology and executed support should be part of the "as built" documentation. It is important that the owner establish adequate filing systems for an electronic- and a paper version of the recorded information.

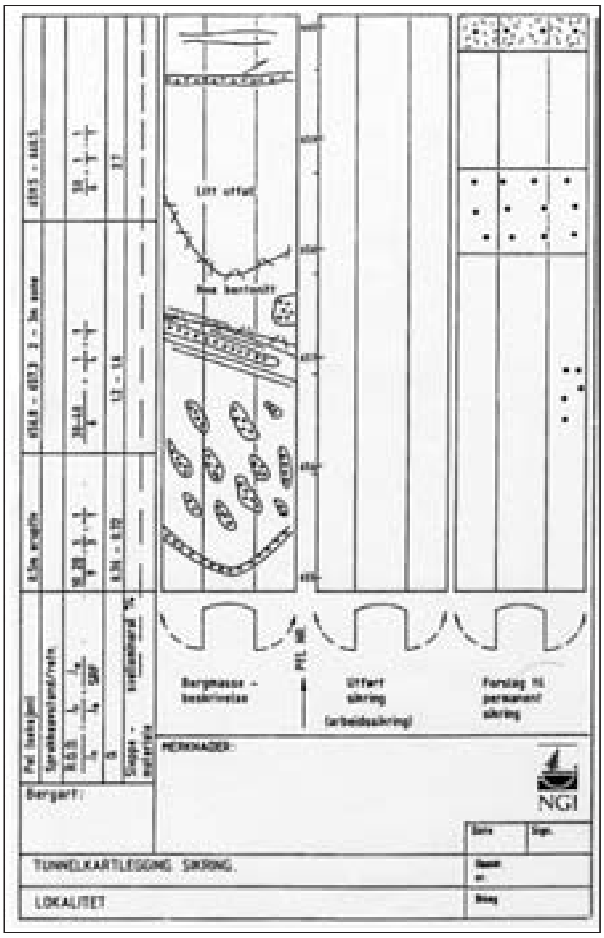

Figure 26. Example of a mapping form already filled out.

6.5 DEFORMATION MEASUREMENTS

Measurement of deformations in a cavern can be done in different ways. The selection of measuring method depends on the time, how critical the deformations might be for the facility, and whether the measuring design can be planned beforehand.

It is advisable to place the measuring points covering the roof and other pertinent parts of the tunnel cross section. For convergence measurements it is important to have a sufficient number of measuring points all over the profile. An example of placing the points is shown in figure 28.

When selecting the measuring method and the execution of the measurement design it is important also to determine which criteria are needed for any further support measures and the rock support means that may be used.

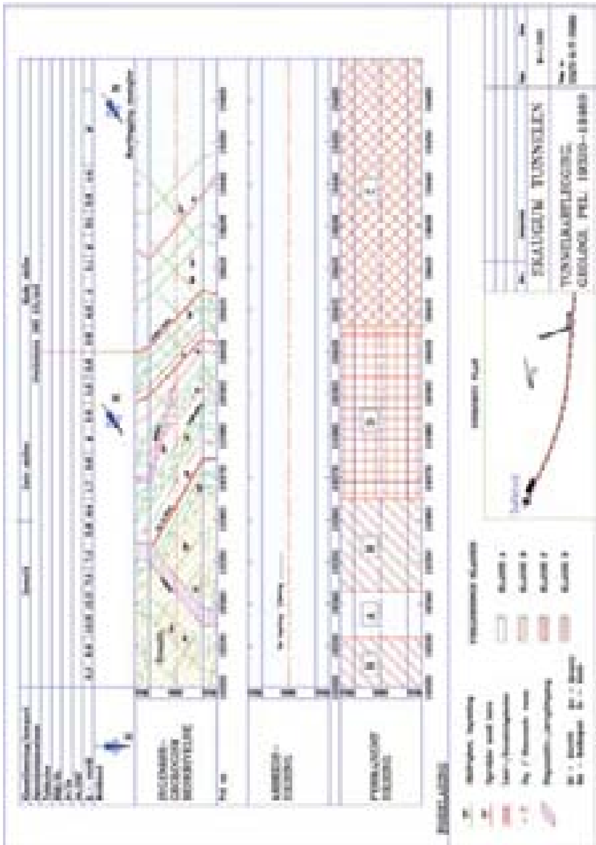

Figure 27. Example of finished geology- and support class log

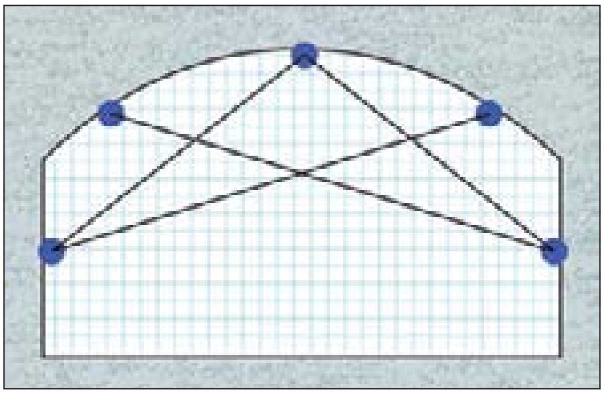

Figure 28. Example of placing measuring points

6.5.1 MEASURING TAPE AND THEODOLITE

The use of a measuring tapes or measuring by using a theodolite are both quick methods that require little planning; the accuracy however is not very good. It is important for accurate measurements to establish fixed and well visible measuring points. The shooting of measuring spikes into the sprayed concrete and then painted is a simple and efficient method. The measuring tape measures the distance between the measuring points only, while using the theodolite one may establish movements in relation to the fixed points. The methods can be used to document that the excavated rock mate-

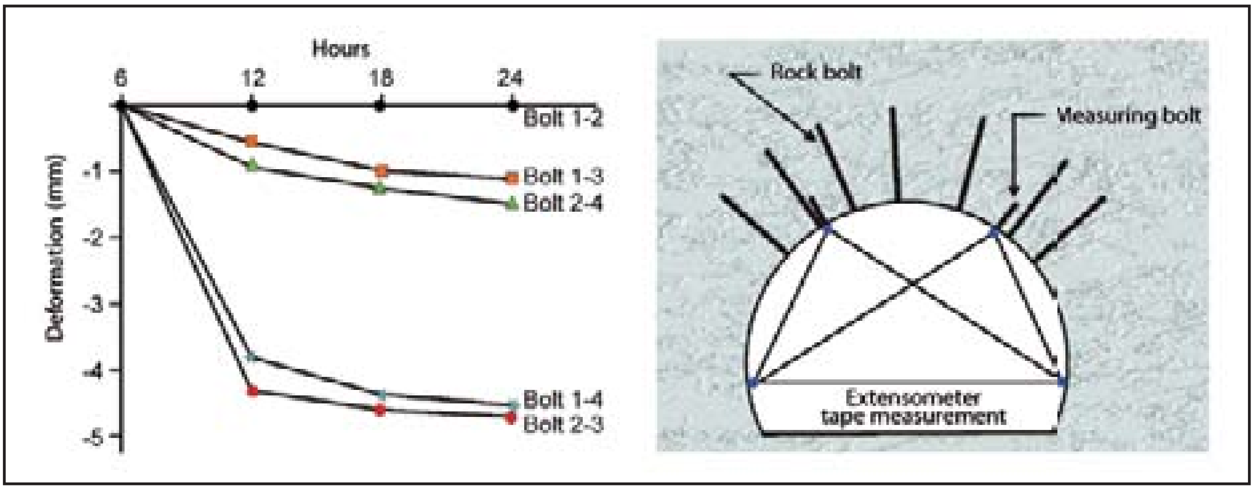

Figure 29. Example of set-up and results from convergence measurement in the road tunnel

rial has obtained a stable condition over time after the excavation of the subsurface facility.

6.5.2 EXTENSOMETER TAPE

The use of an extensometer tape that is being strapped up between the measuring points will provide more accurate distance measurement than the usual measuring tape. Instead of measuring spikes one should use eyebolts. Measuring equipment must be available at site during the actual period.

6.5.3 DRILL HOLE EXTENSOMETER

In principle the drill hole extensometer measures the distance from one or more installed anchors to a fixed frame installed at the surface end of the drill hole. The distance can be measured mechanically with the aid of a dial gauge, or electronically using inductive transmitters, i.e. swinging cord (LVDT). The accuracy can be 0,001 mm per measurement. The drilled hole should be long enough to allow for the inner/upper anchor to be

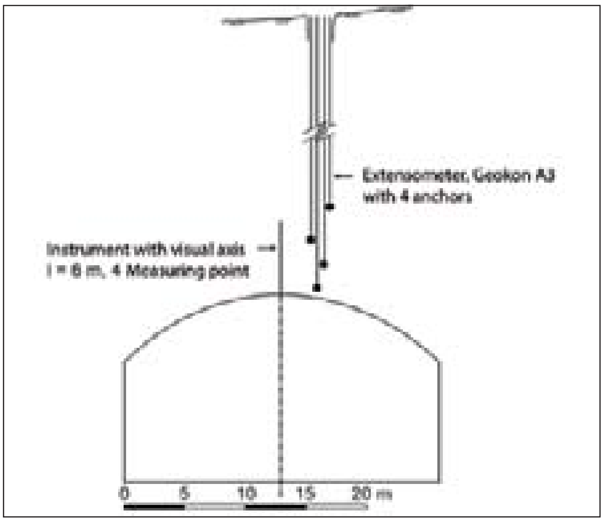

Figure 30. Example of a drill hole extensometer

placed in rock not influenced by the excavation. If the extensometer hole is drilled from the cavern rock displacement immediately after excavation will take place before the extensometers are installed and observations may take place. If the extensometer is installed from an outside surface, the inner anchor should be installed as close to the actual work face as possible without being destroyed by the blast to take place.

6.5.4 SLIDE MICROMETER

A slide micrometer measures the relative movements between the points with spacing of 1 m. This can used for stability control and inspection of the rock deformations, for instance inspection of the roof in tunnels and caverns, the anchor zones, the contact between the foundation and the rock and the inspection of any swelling in the rock.

The micrometer probe consists of two heads at a certain distance and an inductive range measurer. It also has a temperature sensor. A plastic pipe (Ø60 mm) and steel coupling elements are fixed in a drill hole of Ø 75-100 mm. The measurement takes place stepwise between two measuring points, and the signals are transmitted via cable to a reading instrument. The accuracy is stated to be 0,002 mm.

Unlike the extensometer measurements one may obtain readings of deformation every 1 meter over the entire length of the hole. The maximum gauge length is 100 m.

6.6 INSPECTION OF CRACKS

Plaster fill can be installed over cracks to observe if there are any further deformations in the sprayed concrete or the in-situ concrete. A 1 to 2 cm deep and 5 to 10 cm wide cut across the crack should be filled with gypsum plaster. Further deformation will be revealed by cracks in the plaster.

6.7 OPERATION- AND MAINTENANCE PLANS

Plans for the operation and maintenance of the project should be an integrated part of the overall design. The design must allow for agreeable conditions for the maintenance and operational crews. Access is a key matter. Drilling and testing of sprayed concrete samples must be undertaken as governed by standard procedures. (In Norway the Process code 1 issued by the Directorate of Public Roads is valid). The need for documentation is especially important for critical support constructions.

In principle, the design, execution, quality of materials and dimensions shall satisfy the operational standards for the planned lifetime of the project. The design must allow openings and access for regular control and ordinary maintenance. Gates, ladders or other necessary equipment must be available at all times. Insulation foam directly placed on the rock surface leaves no room for inspection.

The inspection shall be planned to take place at a certain time intervals. The interval length may vary through the operating time of the project.

A normal frequency would be:

- The first inspection 1 year after the completion date
- The second and third inspection after 2 and 4 years respectively.
- Later Inspections with 5 years intervals

This must be adapted to observations during the inspections.

The inspection will usually be of visual character, combined with tapping to control the bonding of the sprayed concrete. Sampling and testing of the support materials may be necessary including the concrete.

For projects where big deformations have been registered during the construction period, it might be advisable to install automatic registration of movements in the rock mass using convergence bolts.

7. SPRAYED CONCRETE ROCK SUPPORT AND THE LOAD BEARING CAPACITY OF SPRAYED CONCRETE RIBS.

Sprayed concrete in Norwegian underground construction was first time used in 1952. Dry mix and thin layers dominated during the first years. Wet mix and improved application methods changed the picture dramatically. Since 1980 the method is widely used with well developed technology, competent participants, reliable machinery and high concrete quality. The method dominates both temporary and permanent support.

In 1995 the Roads Authority published its document "Proper use of sprayed concrete in tunnels" based on a study of experience gained in Norwegian tunnelling at that time. In 1999, the Norwegian Concrete Association issued its publication no.7 on sprayed concrete (later updated)

The Roads Authority, responsible for substantial part of the tunnelling in Norway, is still contributing to the development. In the context of rock support the ongoing work to develop a "Code of good practise" on matter of sprayed reinforced concrete ribs is of special interest. The technical committee has not yet finished its work, modifications may take place, a summary representing the "state-of-art" is included below.

7.1 SPRAYED CONCRETE AS ROCK SUPPORT.

Sprayed concrete is commonly used in surface and underground construction. The method includes a wide variety from thin surface layers for temporary support to thick reinforced ribs designed for permanent support and high load bearing capacity.

The main groups of permanent rock support may be divided in three:

1. Support based on cohesion between rock and concrete. Medium thickness 80 mm with minimum > 40. Fibre reinforced. E700 (energy absorbing level)
2. Sprayed concrete, thickness 80 mm or above. E700 or E1000
3. Sprayed concrete ribs, steel reinforced, usually without fibres. To achieve the arch effect correct curvature of utmost importance.

7.2 LOAD BEARING CAPACITY VERTICAL TO THE TUNNEL ALIGNMENT.

Reinforced sprayed concrete ribs have a bearing capacity similar to a concrete lining with similar geometry. Under ordinary conditions the vertical loads exceed the horizontal loads causing compressive stress in the rib. Irregular shape or large point loads may cause bending and subsequent tensile stress that must be compensated by reinforcement. Steel rock bolts of sufficient length is also necessary. Under some circumstances it may be necessary to establish a concrete invert.

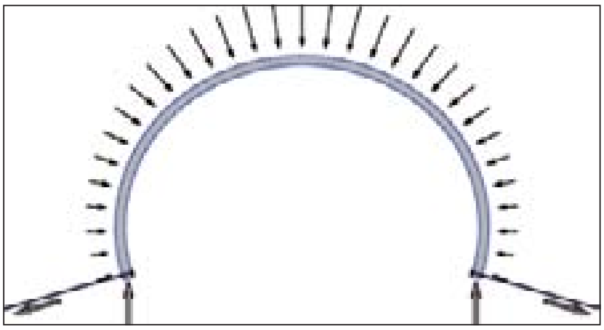

Fig 7.01. Uniform load distribution and reactions. (Rock bolts not shown)

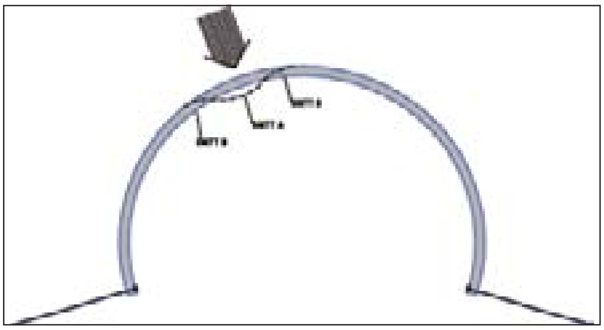

Fig 7.02.a) A point load indicating deformation (exaggerated) causing tensile stress

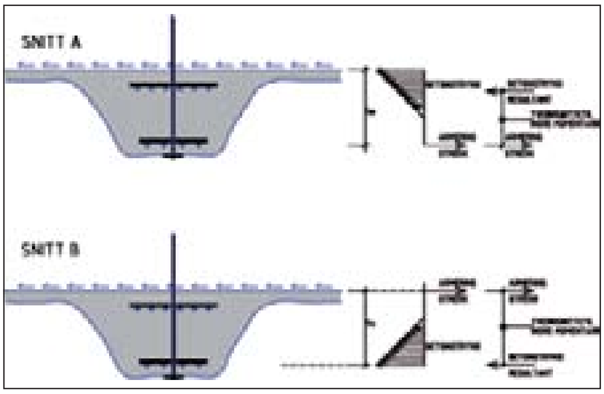

Fig 7.02.b) Sprayed reinforced concrete rib indicating the stress distribution

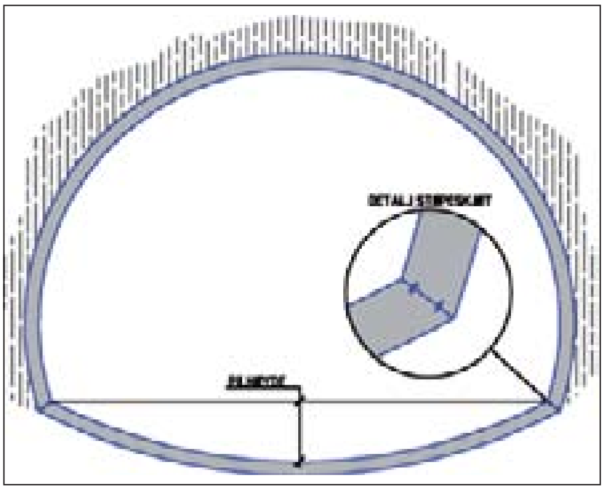

Fig 7.03 Concrete invert to counteract forces in the footings

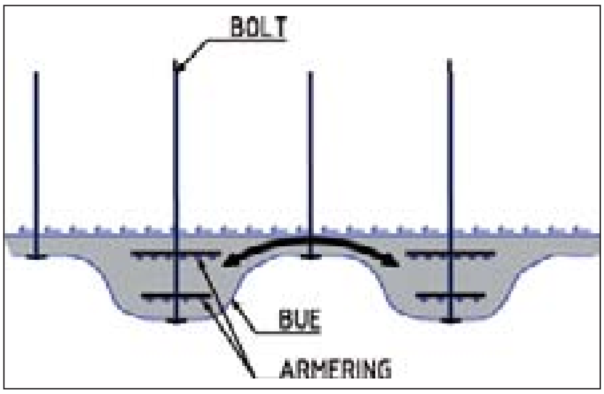

Fig 7.04 Load distribution between the sprayed concrete ribs

7.3 EXECUTION OF SPRAYED CONCRETE RIBS

Sprayed concrete ribs may be installed as permanent rock support; either as a single rib for local support or systematically with a spacing of 1.5 to 3 metres to support poor rock conditions.

A rib will under normal circumstances be single side reinforced, however where larger deformations are expected double reinforcement must be installed.

7.3.1 GEOMETRY

a) Sprayed concrete ribs shall be constructed with a smooth curvature, equal and parallel to the theoretical cross section of the tunnel

b) The rib shall be established in a vertical plane at a right angle to the tunnel alignment. Exceptions may be minor fault zones where the rib adapts to the fault

c) Sprayed concrete ribs must be installed on adequate foundations

7.3.2 REINFORCEMENT

a) Reinforcement

• Rebars shall be of quality B500NC

• Dimension Ø20 mm, prebended to the given curvature

• Spacing ≥ 110 mm

• Concrete cover ≥50 mm [≥ 75 mm subsea]

b) Lattice girders

• Lattice girders may be used as an alternative where double reinforcement is needed.

7.3.3 SPRAYED CONCRETE

• Sprayed concrete used in subsea tunnels shall adapt to endurance class M40 that in turn calls for concrete quality B45. (45 MPa). For other tunnels class M45 is required (concrete quality B35)

• In areas where sprayed concrete ribs shall be installed, the first step is scaling and the establishing of a sprayed smoothing layer with fibres, E1000 (energy absorbing class) , quality B35 ≥ 150-250 mm thick.

• To safeguard the correct arch geometry a further sprayed layer (no fibres) shall be applied as necessary.

• Installing of the reinforcement and concrete spraying.

• A compressive strength ≥ 8 MPa is required prior to the next round being blasted.

7.3.4 RADIAL BOLTS

• Bolts to be included in the rib system shall be grouted and of dimension ≥ Ø20 mm

• Where several ribs are installed, the rock bolts between shall be 3-6 m long with spacing c/c 1.0 – 1.5 m.

• The low end of the rib shall be secured by grouted anchor bolts, dimension 25 mm, L= 4 to 6 m. Optional to install a concrete invert.

• Rock bolts may be exposed to pull-out tests

7.3.5 INVERT INSTALLATION

Concrete inverts when installed shall have a thickness similar to that of the actual rib.

To achieve an arch effect the concrete invert should be concave.

7.4. THE PRACTICAL IMPLEMENTATION OF ROCK SUPPORT AND REINFORCED SPRAYED CONCRETE RIBS

7.4.1 DIMENSIONS

The main parameters for the load bearing capacity are:
- Single or double reinforcement
- Concrete dimensions
- Reinforcement
- Spacing between the ribs
- The bolt support into the rockmass

The selected design must have the capacity to withstand and transfer the design loads without unacceptable deformation that would endanger the safety in the long term perspective.

Single layer reinforced sprayed concrete ribs must withstand the compressive stress, subject to the correct rib geometry. Single reinforced ribs are well suited if loads are fairly uniform and if the sidewalls give the necessary support. The reinforcement has limited capacity to withstand bending. The design is based on interaction between the structural elements.

Double layer reinforced sprayed concrete ribs withstand the compressive stress and bending due to point loads, unevenly distributed loads, limited wall support or deviation from the theoretical correct rib geometry.

Commonly used design identification.
E = Single reinforced
D = Double reinforced
Xx = Thickness of the rib in cm
/y =Number of rebars in 1st layer
+z = Number of rebars in 2nd layer (double reinforced rib)
c/c pp = Spacing between ribs where systematically implemented

Examples:
E30/6 c/c 2 means Single reinforced rib, thickness 30 cm, 6 rebars, spacing 2 m.

D60/6+4 c/c 1.5 means Double reinforced rib, thickness 60 cm, 6 rebars in 1st layer and 4 rebars in 2nd layer spacing 1.5 m between the ribs (for both examples rebar diameter 20 mm because no other information was included).

Rib thickness. Thickness concerns the rib, any concrete behind the rib for achieving the correct theoretical shape is not included.

For a single reinforced rib the distance from rib surface to centre of 1st rebar layer (Ø20) is assumed to be 50 mm, to the opposite surface 60 mm, hence 240 mm to the rib surface (rib of 300 mm) towards tunnel.

For a double reinforced rib distance between the two reinforcement layers estimated to
Subsea tunnels: D-60-75-20 = D-155mm equal to 445mm for D60
Other tunnels: D-60-50-20 = D-130mm equal to 470mm for D60

Rib width.
The space between the rebars (Ø20) shall be ≥110, 75 mm concrete cover in subsea construction, elsewhere ≥50 mm

The theoretical minimum width of a single reinforced sprayed concrete rib with 6 nos Ø20:
Subsea construction 75 x 2 + 20 + 110 x 5 = 500 mm
Others 50 x 2 + 20 + 110 x 5 = 450 mm

7.4.2 STEP1 – PRIOR TO START EXCAVATING TOWARDS A FAULT ZONE

Cautious excavation should start at least 15 metres ahead of the assumed position of the fault zone. Consider based on available information, to which extent the cross section should be increased to establish sufficient space for likely support elements, reduced length of the following blast round. Consider spiling and/or necessary test drilling/pregrouting.

7.4.3 STEP 2 – PRIOR TO THE NEXT BLAST ROUND

Spiling shall be installed before drilling and blasting.
Requirements:
- Spacing between bolts ≤ 300mm
- Drilling for the bolts shall be fanning out approx. 15 degrees from alignment.
- Grouted bolts Ø25 to Ø32mm, lengths 6-8 m or optional bolts of similar quality
- The quantity of installed spiling bolts depends on the observed rock data, to cover the entire tunnel surface from invert to invert, in the roof only, or sectors of the cross section adapting to the observations.
- The end of the bolts shall protrude 50-75 cm from the face

A new set of bolts will be installed for every round. There will thus be two rows of bolts above the tunnel roof. Each set shall be supported by a sprayed concrete rib.

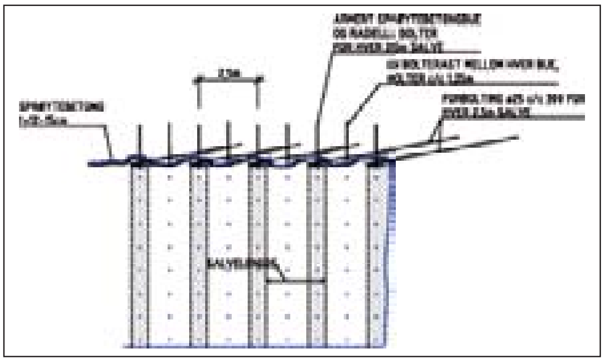

Fig 7.05 The blue colour indicates sprayed concrete, thickness 12-15 cm, and radial bolts, one set for each blast rounds; in this example the blast round is 2.5 m with a rib close to the face. Spiling, (6 to 8 meter bolt length)

7.4.4 STEP 3 – ROCK SUPPORT SUBSEQUENT TO A BLAST

Subsequent to ventilation, initial scaling the face, the roof and the walls must be inspected to decide whether ordinary mucking out or just establishing sufficient space for the spraying machinery should take place. For very poor rock mass a first layer of sprayed concrete should be applied soonest and before the mucking out takes place. The layer thickness depends on the ability of rock surface to carry the weight . (The weight of the fresh concrete versus cohesion and tensile stress). Before spraying rock engineering observations must be carefully logged. Cleaning of the surface with water may be omitted if the rock stability is poor.

Sprayed concrete
After scaling, spraying of concrete is carried out to cover the whole contour. In some cases it may also be necessary to spray the tunnel face.

Radial bolts.
Next activity is to establish an interaction between the sprayed layer and the rock mass with use of rock bolts. A fast interaction can be achieved by using polyester end anchored bolts. They should thereafter be grouted before next blast.

Accuracy while installing the bolts is important. They should follow the theoretically curvature of the tunnel and lengthwise also the center of the reinforcement.
Bolt drilling may start as soon as the sprayed concrete has reached acceptable strength.

Fig 7.06 Sprayed rib, thickness 150-250 mm, radial bolts and spiling before the next blast.

7.4.5 STEP 4 – SELECTION OF DESIGN AND THE CONSTRUCTION OF SPRAYED CONCRETE RIBS.

Options are single or double reinforced, number of rebars, and distance between the rebar layers. Decisions may be linked to observations during the construction.

Single reinforced rib
One will start either with

- Spraying concrete to establish the theoretical tunnel surface or
- Installation of reinforcement parallel to the theoretical surface

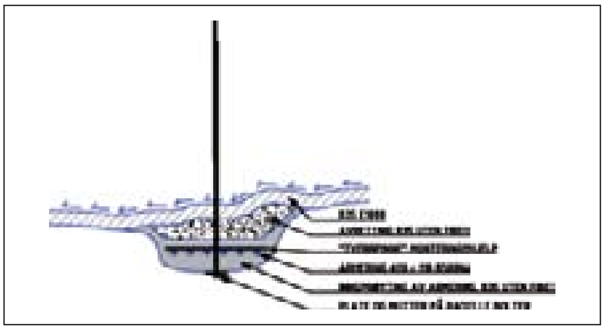

Fig 7.08 from top: -Cleaned and scaled rock surface
- Sprayed concrete layer (B35 E1000)
- Establishment of the theoretical surface with additional sprayed concrete (without fibres)
- Install reinforcement
- Sprayed concrete

Adequate foundation of the rib in the invert area is important.
- Remove loose material
- End of rib locked by several bolts Ø25, L= 4-6m

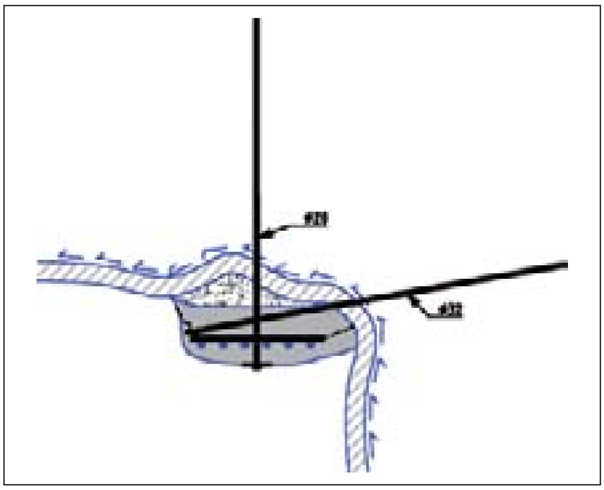

Fig 7.09 Single reinforced rib close to face

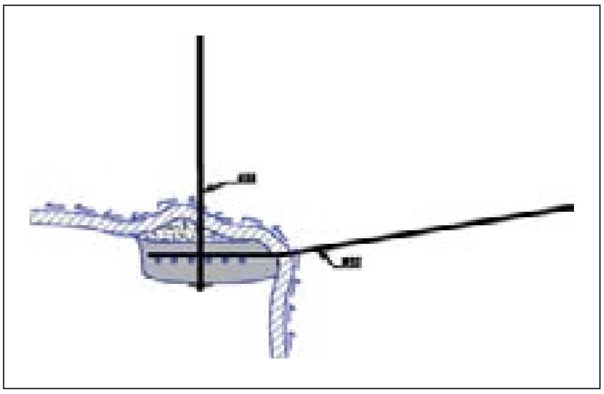

Fig 7.10 Single reinforced rib close to face

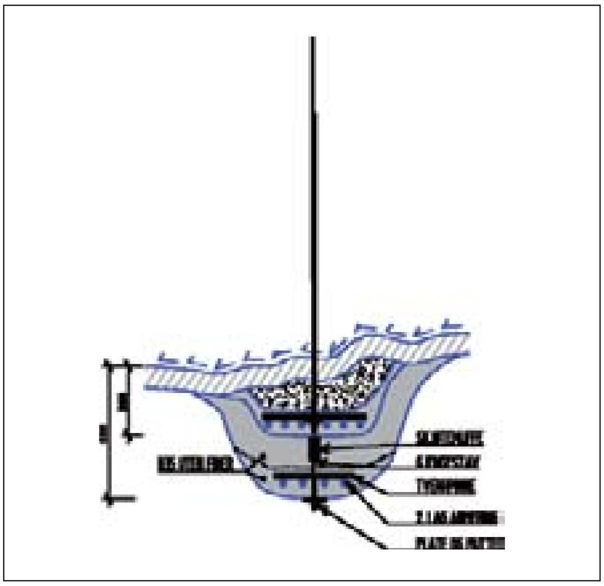

Fig 7.11 above shows a single reinforced rib modified to a double reinforced rib

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7.5 CONTOUR BLASTING

COMMENTS, ADVICE, RULES OF THUMB

In general:

- Rock quality: Decreasing rock quality calls for increasing rock support
- Geometry : Increasing size calls for increasing support
- Excavation: Rough blasting harms the surrounding rock mass and calls for increased rock support.

ROCK SUPPORT IN ROAD TUNNELING

TUNNEL EXCAVATION

Length of drillholes

For rock qualities A, B and C normal rounds with 16-18 feet rods.
For quality D a reduction to 14' is advisable, or even to 12' for the wider tunnels
For qualities E and F a further reduction must be considered. For tunnel widths above say 10 m one should also consider sequential excavation.

Rock mass class	**Geology** **Q-value**	**Rock support class** **Permanent support**
A/B	Competent rock Average joint spacing > 1m Q = 10 - 100	**Support class I** - Occasional bolting. Sprayed concrete B35 E700, 80 mm, roof and walls to 2m above invert
C	Moderately competent rock Average joint spacing 0.3 – 1 m Q = 4 – 10	**Support class II** - Systematic bolting (c/c 2 m), end anchored, pre stressed and grouted. Sprayed concrete B35 E700, 80 mm on roof and walls
D	Densely cracked or chisty rock mass, Average joint spacing < 0.3 m. Q = 1 – 4	**Support class III** - Sprayed concrete B35 E1000, ≥ 100 mm - Systematic bolting (c/c 1,5 m), end anchored, grouting (timing to be considered)
E	Very poor rock mass Q = 0.1 – 1	**Support class IV** -Sprayed concrete B35 E1000, 150 mm - Systematic bolting (c/c 1,5 m), end anchored, grouting - If Q < 0,2spiling ø25 mm, c/c 300 mm or less - If Q < 0,2 reinforced sprayed concrete ribs E30/6 ø20 mm, c/c 2 – 3 m, -Systematic locking of the ribs by bolts, L= 3 – 4 m - Cast invert to be considered
F	Extremely poor rock mass Q = 0.01 – 0,1	**Support class V** - Spiling, c/c 200 – 300 mm, ø32 mm bolts or "self boring anchors" - Sprayed concrete B35 E1000, 150 – 250 mm - Systematic bolting, c/c 1,0 – 1,5 m, grouted - Reinforced concrete ribs D60/6+4, ø20 mm, c/c 1,5 – 2 Systematic bolting of the ribs c. 1,0 m, L 3 – 6 m - Reinforced concrete invert
G	Exceptional poor rock mass, basically loose material. Q < 0.01	**Sikringsklasse VI** - Special design required. Not suitable for blasting.

Hole spacing in contour row

The ordinary contour hole spacing is 70 cm, but can be reduced. The 2nd and 3rd row should also be drilled with reduced spacing. The contour hole is always drilled under an angle versus tunnel surface (≈5°). The 2nd row should be drilled under a reduced angle (≈ 2° to 3°)

Charging

To achieve an acceptable contour and to avoid destroying the remaining rock mass, the charges must be reduced both in the contour holes and in the 2nd row.

A rule of thumb would be

- Standard holes (not cut and contour) 100 %
- Contour holes 20-25 %
- 2nd (contour) row holes 40-60 %

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8. WHY WAS ROCK SUPPORT NECESSARY?
SOME PROJECT EXAMPLES

Rock fall and serious incidents rarely occur in Norwegian tunnels. However, there are examples of such occurrences. In principle one may divide these incidents in 3 categories:
- rock fall at the tunnel face during excavation,
- rock fall in water tunnels when filling up,
- rock fall in completed "dry" tunnels (road- and railway tunnels). Most incidents in this category have been related to the clay zones.

Some readers may be surprised when observing the modest standards of some of the road tunnels; e.g. the steep descents / ascents for some communication tunnels. The reasoning behind may be the acceptable tunnel length, the low traffic density and the acceptable costs.

8.1 ROMERIKSPORTEN (1994-1999)
Romeriksporten is the 14.5 km long railway tunnel on the northbound line from Oslo Central station to Oslo Airport. The tunnel contract when signed, by some named the tunnel contract of "the century", ended in unenviable notoriety: water and stability problems, necessary rock support far beyond plans. Media headlines caused Governmental involvement that added to the problems and increased the cost overruns and construction delays by one year.

The difficulties started with the so-called Bryn zone (Brynsonen), which is the main fault between the Cambro-Silurian rock in the Oslo area and the bedrock gneiss in the Østmarka (northeast of the city) approximately 3 km from the Central Station.
Pre-construction site investigations included core sampling

and conclusive rock inspection drillings. The rock cover, at the minimum identified to approximately 2.5 m. in combination with unfavourable rock mass called for open-minded discussions of all options. The actual tunnel section is exposed to surface silt deposits and a small river. The consequences of a cave in would be the collapse of a motorway bridge on "Ring 3" and massive ingress of water in combination with silt from the nearby small river.
The construction method allowed for cautious blasting, 2 metre blast rounds, segmental excavation, sprayed concrete and bolting before concrete lining.
The rock mass in the fault zone contained crushed altered gneiss mixed with alum slates. (the rock mass in the fault zone could be dug out using bare hands only).

Blocks of alum slates, size of up to 1 meter, fell from the roof; in reality rock cover was reduced to 1.5 meter. The deterioration was stopped by means of sprayed concrete. Eventually the support actions and the excavation through the fault zone took place as planned.

8.2 THE NORTH CAPE TUNNEL (FATIMA, 1995-99)

The North Cape tunnel is part of main road E 69. The tunnel is 6,8 km long, designed as "T8" (total width = 8 m) with a cross section of approximately 50 m^2 descends to 221 below sea level.(b.s.l) before ascending to North Cape plateau.

Approximately half of the length from the main land side was excavated in mica schist, considered as unproblematic. From the North Cape end with the open cut at Veidnes there was a slate rock type, densely cracked with flat lying rock cleavage. The jointed surfaces had a smooth chlorite coating. The rockmass when exposed to blasting, spiking, or other loads would dissolve. The rock piles were almost in the shape of a heap of aggregates with an average size of 2-3 cm with smooth, shining surfaces.

Normal roof overbreak was 1-3 metres, up to 6 metres at the most. While scaling, another half meter was easily removed.

Several rock support methods were tested. Sprayed concrete and bolts proved insufficient and concrete lining was established. The construction of the tunnel started in June 1995, in October the following year 635 metres had been excavated whereof 586 metres had been concrete lined .The concrete consumption was far above the planned quantities and additional batching capacity had to be provided.
When a total of 2255 metres concrete lining at the tunnel face had been completed test operation using alkali free sprayed concrete started.

The placing of 20-40 cm thick layers of sprayed concrete with a high early strength proved successful and immediately replaced the standard concrete lining at the work face. The permanent support of 1165 m of the North Cape tunnel using thick alkali free sprayed concrete was observed as the full scale introduction of alkali free sprayed concrete in the domestic market.

8.3 THE OSLOFJORD TUNNEL (1997-1999)

During the excavation of the 7.2 km long subsea Oslo fjord tunnel problems occurred when crossing a fault zone. Through the exploratory drilling and preliminary studies during the planning stage, the designers were aware of its existence.

Systematic test drilling during the excavation of the tunnel took place. While approaching the fault zone the test drilling revealed non-compacted material under water pressure all the way down into the tunnel profile. In the lower part of the tunnel profile the zone consisted of crushed and clayey rock with some water leakages.

Core drilling from both sides of the fault zone showed adverse conditions. The required work procedures to pass through this zone would be time consuming and difficult. To ensure the progress of the project and to provide sufficient time for the necessary additional support work a by-pass tunnel was established. Excavation of a narrow steep downwards turn to pass the fault zone at a deeper level with subsequent ascent to the main tunnel alignment some 150 m further ahead took place.

During the detailed investigation of the fault by core sampling, casings had to be used. To improve stability and stop water flow approximately 700 tons grout mix was injected. It was decided to apply the ground freezing method to ensure the safe excavation of the tunnel through the fault zone. 115 freezing pipes were installed. All drilled from one side of the fault zone. Compressors and energy converters were installed and the freeze operation started. When the actual rock mass had reached a temperature of –28°C, the excavation work resumed from the opposite side.

An emergency back-up system for closing off the fault zone area was installed. The excavation through the frozen fault zone took place by means of blast rounds of 0.5 to 3 metres using special designed explosives. Sprayed concrete with alkali free accelerators was applied on the frozen rock surface. The entire cross section, including the invert was supported with reinforced concrete lining designed to accommodate water pressure of 120 metres. The circular cross section had a thickness varying from 1.0 to 1.2 metres.

The coolers were operating until all support work in the area had been finished.
The process of the temperature change, from -28^{0}C back to the normal temperature and thus to expose the structure to the external loads, took several months.

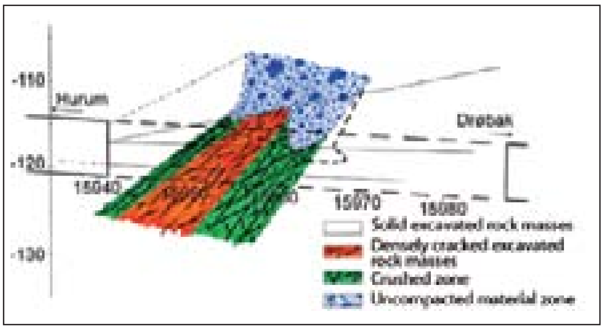


Figure 31. Longitudinal section through the freezing zone

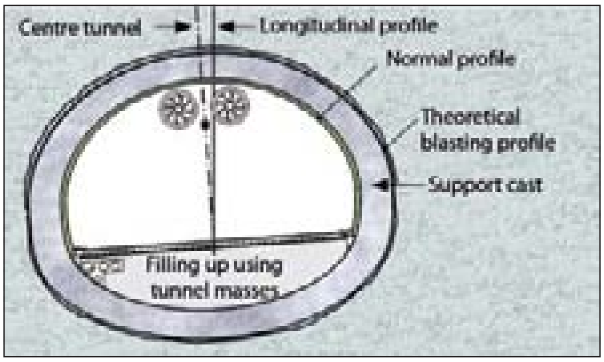


Figure 32 Concrete cast in the freezing zone

8.4 THE NATIONAL THEATRE RAILWAY STATION (1995-1999)

The "National Theatre Station" has a total length of 966 metres, whereof 829 metres are in rock. The cross sections of the tunnel, including shafts, access tunnels, and emergency exits vary from 20 to 300 m^2. The rock types consist of slates and limestone with Q values between 1 and 10. There are zones with few metres rock cover only, and/or lower Q values.

There were difficult sections due to the reduced rock cover. Under the Arbins gt permanent rock support was established with reinforced sprayed concrete. The span was 22 metres with rock cover down to 3 metres. Initially the designers had planned watertight concrete lining to avoid lowering the ground water table

At the time of construction limited information as to the use of reinforced sprayed concrete ribs was available. Hence, complex instrumentation was installed for control and documentation of deformations and stress. The data were later used for re-calculation and control of the structural analysis.

The 200 m^2 cross section of the cavern was divided into several segments, see figure 33.
The sequencing of the excavation could be summarised to:
• Excavation of the outer segments of the top heading
• 6 meter long ø32mm rock bolts c/c 0.3 meter, overlapping 4 metres
• Blasting and subsequent scaling
• 10 cm sprayed concrete
• The installation of radial bolts c/c 1.5 meter
• Installation of reinforcement with subsequent sprayed concrete
• Excavation of the middle section of the top heading
• Sprayed concrete
• Installation of radial bolts c/c 1.5 meter in the middle segment
• Installation of reinforcement, connecting to the 2 outer segments
• Sprayed concrete

To avoid settlements of the buildings in the vicinity of the tunnel, systematic groundwater control took place, supplemented by grouting and water infiltration.

Contact concrete lining was used as permanent water tightening.

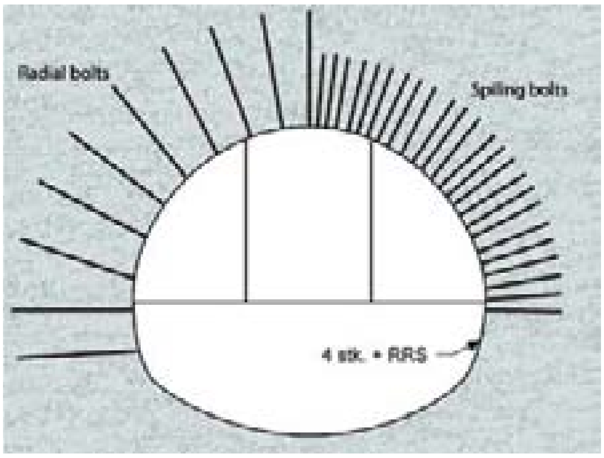


Figure 33. Working with a divided cross section and support using spiling bolts and sprayed concrete ribs

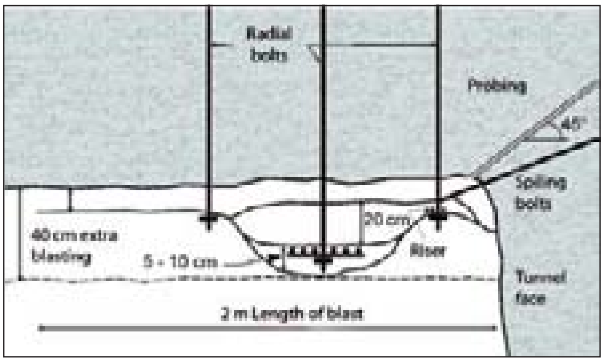


Figure 34 Instalment of spiling bolts, probing for inspection of rock coverage, and support using reinforced sprayed concrete arcs.

8.5 THE FRØYA TUNNEL (1997-2000)

The 5.3 km long tunnel between the two islands Hitra and Frøya, southwest of Trondheim, constitutes the last part of the road connection to the outer island Frøya. Previous sections were the bridges and banks and the 5.6 km Hitra tunnel constructed during the years 1992-1994. The bedrock along the alignment of the tunnel consists of Cambrian or Precambrian metamorphic gneisses. Hitra/Frøya is situated in the part of the country with most prominent rock faults and altered zones. A major fault crossing the alignment of the tunnel is the Tarva fault. The Tarva fault may be observed for more than 150 km oriented East-Northeast. This is assumed to be a normal fault from the Jurassic/Cretaceous period. Associated rupture zones in the direction Northeast-Southwest might origin from the same period. These zones may have been exposed to tensile stress thus now badly clamped and therefore exposed to leakage.

Approximately 80% of all blasts in the tunnel were mapped and classified in agreement with the Q-system by rock engineering advisors retained by the owner. At excavated sections with bad rock quality, strict safety precautions had to be implanted. Prior to the installing of rock bolts and sprayed concrete, the area was assumed unsafe. During the work in the tunnel, samples from the tunnel face in the altered zones were frequently taken to check the swelling properties.

During the entire construction period convergence measurements were carried out in the altered zones to assess the stability and the need for permanent support. The instillation of measuring bolts could not take place too close to the tunnel face as the mucking equipment could damage the measuring bolts. Hence initial deformation development could not be logged. The convergence measurements provided could still be used to decide on the permanent support.

Between faults or lower quality rock mass zones, rock support in general consists of a layer of minimum 5-6 cm fibre reinforced sprayed concrete and rock bolts with varying spacing. For some shorter sections the rock support could be limited to spot bolting (no concrete). The rock bolts were mainly of the CT-bolt make. On sections that required little bolting, polyester anchored bolts were used.

At weak zones the roof and the upper parts of the walls were supported by systematic bolting, rock straps c/c 1.5 meter and a layer of minimum 12 cm sprayed concrete. The face itself was supported by bolts. This approach was implemented and modified for different circumstances: sprayed concrete layers up to 30 cm thickness and a maximum of 13 bolts per meter (average spacing 1.2 m) are examples. On several sections the sprayed concrete layers were additionally supported by reinforced sprayed ribs. (sprayed girders)

Alkali free sprayed concrete was used for thick layers. One of the advantages of the sprayed concrete is its ability to establish smooth surface on an uneven rock surface. Pits and notches after the excavation can easily be filled.

Full lining has been executed in the most difficult zones. Before installing the formwork, a prefabricated shield unit, the tunnel section would be supported with 10-20 cm fibre reinforced sprayed concrete on roof and walls with additional 6 cm sprayed concrete on the tunnel face. Furthermore, some bolts and rock straps for the safe installation of the pre-bolting system had to be used. In the most difficult sections the invert has also been supported by bolts and reinforced concrete, partly because of problems building the roadway, and partly to ensure support against the squeezing-in risks.

The concrete lining in such sections represent an entire concrete ring

All expansions of the tunnel profile to give room to heavy support were carried out as part of the ordinary excavation. Except the pre-bolting system, all rock support during the work phase was part of the permanent support. Decisions on rock support were based on competence, available site investigations, common practise and most important, actual observations of the rock mass during excavation. Predictions of rock support include measurements while drilling and numeric modelling. As an example a description of the Frøya modelling is referred below

INTRODUCTION TO THE MODELLING

These fault zones had been detected during the pre-construction seismic surveys performed from the sea. Core-drilling was performed to verify some of the zones. The surveys had revealed that the velocities in the weak zones vary in the range 2.1 – 3.5 km/sec indicating the sometimes poor quality of the rock mass. In addition to the pre-construction surveys, up to 112 m length of the core was extracted on a routine basis from the face of the tunnel, to ascertain the quality of the rock mass in these zones. In order to stabilise the rock mass ahead of the tunnel face in the zones, pre-reinforcement in the form of spiling prior to excavation and erection of final support took place.

The numerical modelling of the final support was based on the so-called UDEC-S(fr) Itasca-NGI version of the distinct element code called UDEC-BB, which has the ability to model the fibre reinforced sprayed concrete support and fully grouted bolts.

The rock formation around the site had a distinct foliation which strikes mainly in the ENE-WSW direction

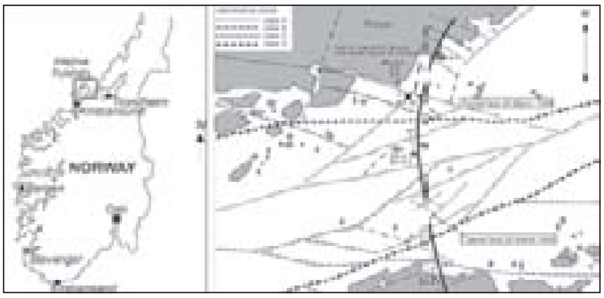


Fig. 35 Tunnel location and interpreted weak zones along the tunnel alignment

and dips steeply. This strike direction is nearly at right angles to the N-S striking tunnel. In addition to the foliation joints there were some joints which strike almost parallel to the tunnel.

The weak zones, which was usually several tens of metres wide, follow partly the direction of the foliation, and strike partly in the NW-SE direction. The zones were presumably the remnants of tectonic activity (fault zones) which had been going on from Precambrian age to the Tertiary. The zones were often a mixture of rock fragments and sand/clay material. Although no stress measurements had been carried out near the vicinity of the tunnel, it was widely believed that high horizontal stresses exist in the region in the NW-SE direction.

During the construction of the tunnel cores were extracted from one of the weak zones. Core-loss up to 2-3 m in length was observed in the cores extracted from the weak zones. The RQD value in the core varied between 0 and 15. The typical minimum RQD-value used for calculating the rock mass quality Q was 10. The value of Jn had been estimated to 15 since the cores generally contain small fragments of rock. Due to the presence of clay there was no rock wall contact between the joints and the Jr value had been set to 1. Since the joints often had relatively thick clay filling the Ja value has been estimated to 10. There was hardly any water leakage in the weak zones so the Jw value was equal to 1. Mineralogical analysis of the clay samples had revealed the presence of smectite which can cause swelling of the rock. Moderate swelling pressures of about 0.5 MPa was measured in the laboratory from the samples taken from the site. Since no damage had been observed in the installed support due to swelling, the SRF value was set to 5. Based on these parameters the Q-value was calculated to:

$$Q = \frac{10}{15} \times \frac{1}{10} \times \frac{1}{5} = 0.013$$

This indicated an extremely poor quality of rock mass. It may be mentioned that the Q-value calculated above was considered as a typical minimum value for the concerned zone and the calculated typical maximum and mean values were 0.1 and 0.02 respectively.

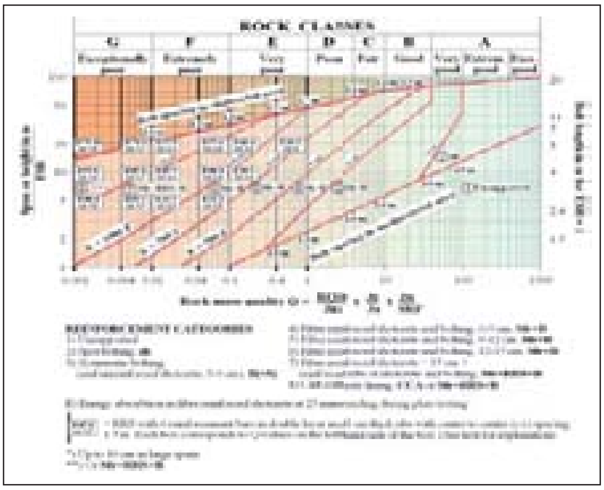


Figure 36 indicates that for a Q-value of about 0.013, the permanent support required for a 10 m span tunnel with an excavation support ratio (ESR) of 1 would be reinforced ribs of sprayed concrete or cast concrete arches.

INPUT PARAMETERS FOR NUMERICAL MODELLING

The Universal Distinct Element Code (UDEC) using the Barton-Bandis non linear joint behaviour model was used for the two dimensional modelling of the sub-sea tunnel. The rock mass in the model was considered as a weak rock where plastic deformation could take place in the intact rock, but the total deformation also depend on the movement along the joints. Therefore, the model shown in Figure 37 contains joints but with a much larger joint spacing when compared to reality. Near the periphery of the tunnel the frequency of jointing has been increased to represent a more realistic picture of the prevailing rock mass conditions. In order to compensate for the large joint spacing in the model a corresponding low value was used for the deformation modulus of the rock mass. The geometry of the model, shown in Fig. 3, was based on the mapping performed in the tunnel. The block layout representing the various discontinuities was manually constructed based on the observations recorded in the tunnel. The joint pattern was then digitised so that any potential adjustments in the pattern could easily be made before the final layout was selected for analysis.

The input data required for the numerical modelling was derived from field investigations, rock joint characterisation from drill core and from Q-system logging. Table 1 and 2 show the intact rock material properties and the rock joint properties respectively used for the numerical modelling

Intact rock	Parameter-values
Q- value	0,013
RMR - value	22
E (Youngs modulus), or M (Deformation modulus) (GPa)	1,0
ν (Poissons number)	0,3
ρ density (KN/m^3)	25
K (Bulk modulus) (GPa)	0,8
G (Shear modulus) (GPa)	0,4

Table 1 Intact rock material properties for numerical simulation

Joint characteristics	Parameter-Values
JRC_0 (Joint roughness coefficient)	5
JCS_0 (Joint wall compressive strength) MPa	15
(ϕ_r) PHIR - Residual friction angle (grader)	20
L_0 (Joint length, lab. scale) (m)	0,1
L_n (Joint length, insitu) (m)	0,1
σ_c (unconfined compression strength of intact rock) (MPa)	15
E_0 (Initial joint opening) (mm)	0,2
JKN (Joint normal stiffness limit) (MPa/m)	1E6
JKS (Joint shear stiffness limit) (MPa/m)	1E3

Table 2 Rock joint properties

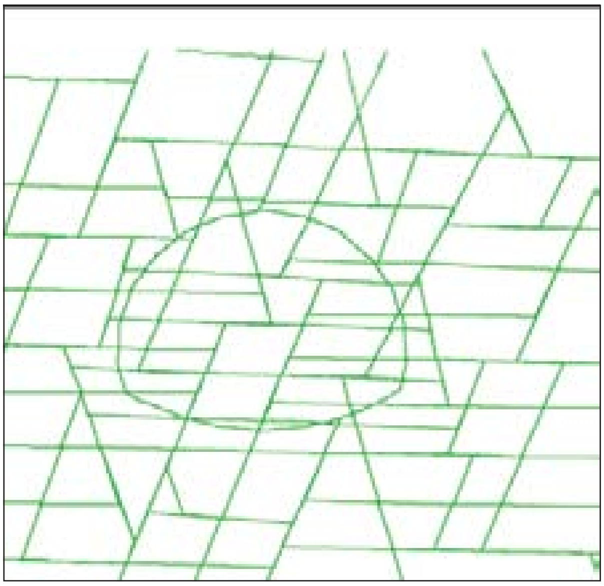


Fig. 37 Geometry used for the numerical model

The parameters required for modelling the rock support including fibre reinforced sprayed concrete S(fr), rock bolts and cast concrete lining were based on the specifications provided by the client. The compressive and tensile yield strength of S(fr) were 45 and 4.5 MPa respectively. The rock bolts, which were spaced at 1.5m c/c, was 20 mm in diameter and 4 m in length. The cast concrete lining has a Young's modulus of 24 GPa and is 0.6m thick in the invert and 0.4 m in the roof and walls.

ROCK SUPPORTS USED IN THE MODEL

Numerical modelling was carried out for the following three different types of supports:

Case 1 Shotcrete and bolts with cast concrete lining at the invert
Case 2 Bolts with reinforced ribs of shotcrete and cast concrete lining at the invert
Case 3 Fully cast concrete lined tunnel

Case 1 was further sub-divided into the following manner to study the effect of installing the cast concrete lining at the invert at different time intervals (steps) i.e.

1a Shotcrete with bolts and then placing the invert lining after a certain time lag in the model
1b Shotcrete with bolts and invert lining placed simultaneously
1c Shotcrete with bolts without any lining at the invert

PROCEDURE FOR NUMERICAL MODELLING

The procedure for numerical modelling in each of the cases described above consisted of consolidating the model under the gravitational stresses and the simulated in-situ stresses. The stresses in the model were assumed to vary linearly with depth and at the level of the tunnel roof the simulated horizontal and vertical stresses were equal and have a magnitude of about 1.3 MPa. After consolidation of the model, the tunnel was excavated and the model allowed to run without any support in the tunnel. This resulted in the falling of blocks from the periphery of the tunnel indicating collapse. The model was then back tracked to about 10 mm of deformation in the roof before numerically spraying the concrete and installing the bolts. This was done to simulate actual conditions in practice where some permanent deformation would have already occurred at the face before installation of sprayed concrete and bolts. The application of sprayed concrete in the numerical model were carried out in such a way to simulate actual conditions in practice where a layer of sprayed concrete was usually applied after a round was blasted, and then rock bolts were installed. This allows proper integration of the support systems.

NUMERICAL RESULTS

A summary of the results obtained for all the cases are shown in Table 3. Figure 38 shows the maximum axial forces on the bolts for Case 1a. In this case the invert lining was placed 20000 cycles (time steps) after the application of sprayed concrete and bolts. It may be seen from Fig. 38 that the maximum bolt load load is about 7.9 tons which is slightly less than the assumed yield capacity of 9 tons. The maximum axial force on the sprayed

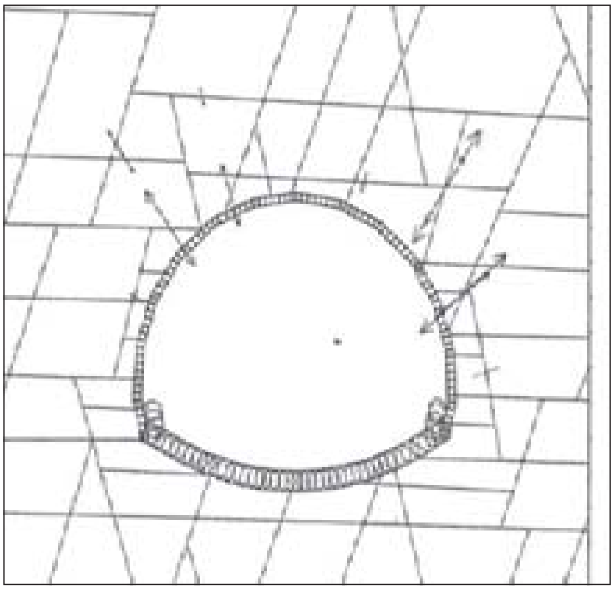


Fig. 38 Axial forces on bolts

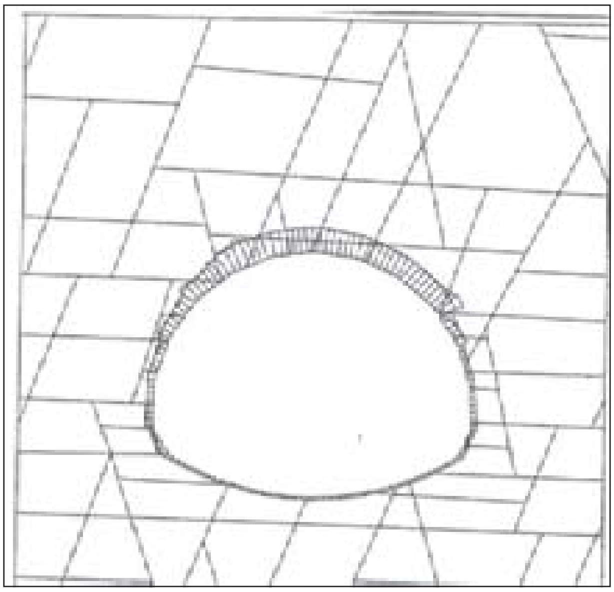


Fig. 39 Axial forces on shotcrete

concrete was about 1.1 MN. This value was well within the failure limit of 11.25 MN which was obtained by multiplying the compressive strength of the sprayed concrete (45 MN/m^2) with the cross-sectional area [thickness (0.25m) × unit length (1m) = 0.25 m^2)]. It may be noteworthy to add that in practice if σ_c of sprayed concrete is reduced to about 30 MN/m^2 and the thickness to 0.1m the failure limit would decrease significantly to 3 MN. The numerical results obtained for the cases 1b and 1c are shown in Table 3. It may be seen from this table that the maximal bolt loading reach a maximum yield capacity of 9 tons when no invert lining is modelled and the maximal bolt loading decrease to 3.3 tons when the invert lining is placed at the same time as sprayed concrete and bolts. These results indicate that a support system installed all around the tunnel simultaneously would result in a higher factor of safety. However, this is not usually possible in practice due to practical restraints.

The reinforced sprayed concrete ribs (RRS) in case 2 are modelled as two 25 cm thick layers of sprayed concrete. It is assumed that these two layers correspond to RRS with a spacing of about 2 m between the ribs. Figure 39 shows the maximum axial load on the structural elements comprising sprayed concrete and cast concrete lining. It may be noticed from this figure and from Table 3 that the maximum axial load on the structure is about 0.88 MN compared to the 1.1 MN without RRS.

In case 3 the tunnel is modelled with full cast concrete lining (CCA). The thickness of the lining is 0.6 m at the invert and 0.4 m in the roof and walls of the tunnel. Figure 40 shows the modelled CCA with the displacement vectors. From Table 3 it may be noticed that the maximum load on the lining is 1.4 MN which is higher than in cases 1 and 2. This higher load may be attributed to the fact that no rock bolts have been applied for this case.

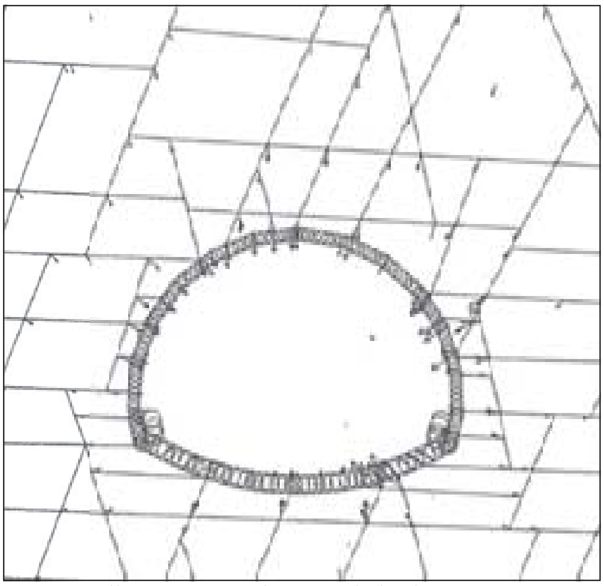


Fig. 40 Full cast concrete lining with displacement vectors, max. disp=17.34mm

	1a s(fr) + b Invert lining after 20 000 cycles	1b s(fr) +b Without Invert	1c s(fr) + b Invert at the same time as bolting	2 RRS (Reinforced ribs of sprayed concrete)	3 CCA (Cast concrete lining)
Max. displacement after equilibrium	17.7 mm	21.0 mm	18.4 mm	17.1 mm	17.3 mm
Max. axial loading on bolts	7.9 ton	9 ton	3.3 ton	11.6 ton	—
Max. axial load on the structure	Roof: 1.1 MN	Roof: 0.43 MN	Roof: .96 MN	Roof: 0.88 MN	Roof: 1.4 MN
Sprayed concrete/ Rock (bond) failure	Significant (bond) failure	Total (bond) failure	Little (bond) failure	Little (bond) failure	—
Max. joint aperture	3.0 mm	6.3 mm	3.3 mm	3.3 mm	3.2 mm
Max. shear displacement	11.8 mm	15.4 mm	10.7 mm	10.7 mm	11.7 mm

Table 3 Summary of results

REMARKS

The numerical results highlighted above clearly indicate that the time of installation of invert lining influences the loading on the rock bolts. A delay in the installation of the invert lining results in higher bolt loading while a model without the invert lining results in bolt loading that reach the maximum yield capacity. From Table 3 it is apparent that there is not much difference between the maximum displacement reached after equilibrium of the model for the Cases 1a, 2 and 3. However, if the invert is installed at the same time as the roof and wall support then there is a significant reduction in deformation around the periphery of the tunnel. Alternatively, if there is no invert then there is an increase in the maximum deformation around the tunnel. Table 3 shows that there is some interface bond failure due to shearing between the sprayed concrete and the rock surface for the cases described above. A complete interface bond failure occurs all around the periphery of the tunnel for Case 1b (i.e. without any invert).

If the Q-values in the weak zones lie between 0.01-0.1 then the Q-system support design chart (Fig. 38) recommends RRS as permanent support. A Q-value of 0.013 (as calculated in equation 1) lies close to the boundary between RRS and CCA in Figure 2. The results from numerical modelling confirm that the support required for the tunnel should either be RRS or CCA. The choice between these two types of support is usually based on economical and practical aspects of the project. However, it may be emphasised that RRS is an extremely flexible method in which the thickness and spacing of the ribs can be varied as required. In this tunnel, spiling ahead of the face and closure monitoring were carried out in the parts of the tunnel where the rock mass qualities range between the Q-values of 0.001 to 0.1
The predicted performance of the tunnel was in agreement with that observed in the tunnel.

8.6 THE LÆRDAL TUNNEL (1997-2000)

The main challenge related to the rock support in the 24.5 km long Lærdal tunnel project was the high rock pressure in relatively hard rock masses. With the rock overburden of 1450 m and the theoretical vertical stress of up to 39 MPa, spalling and rock fall frequently occurred. The normal amount of rock support at the work face adapted to the different levels of stress between 20 and 45 polyester anchored bolts with lengths of 2.4 to 5 metres. The steel fibre reinforced sprayed concrete layers had a thickness between 5 and 15 cm. Influencing factors were also the observations at the face during the scaling operation.

Approximately 11 km into the tunnel from the Aurland side, the tunnel crosses a major fault zone, which besides an approximately 60 meter wide main zone, consists of several smaller altered zones of 1-5 m width in disintegrated and fractured rock mass with swelling clay. During the drilling operation into the main zone, the rods were stuck. While undertaking an attempt of pulling the rods from the hole, approximately 10 m^3 rock mass from the roof dropped down. The rock hit a boom on the drilling rig. During the subsequent rescue operation another incident occurred. Further 2 rock falls, approximately 10 m^3 each came down and a grouting rig was put into operation. During operation of some further100 m^3 rock mass fell down causing damage to the boom on the injection rig.

While the rock collapse developed, additional rock support of the tunnel, starting approximately 20 m behind the collapsing area was initiated. Bolts of 4 to 5 meter lengths were installed as well as the spraying of additional layers of fibre reinforced concrete up to an aggregate thickness of 15 to 20 cm. Two reinforced ribs, 5 and 10 metres from the rock fall area respectively, were established by sprayed concrete.

Subsequent to above temporary rock support actions, the same tunnel section was filled with previously excavated rock material, as high up to the roof as possible. Then,

Figure 41. Rock fall in the Lærdal tunnel

some sort of a wall from top of the rock fill up to the roof was established using the concrete spraying rig. Alkali free accelerators were added to the fresh concrete mix. Through the wall a Ø100 mm steel pipe had been placed in the wall to allow for pumped concrete to be placed inside the fencing wall. To ensure good pumpability of the fresh concrete, a high w/c factor was used. To prevent adverse hardening of the pumped concrete in the area behind the steel pipe, pumping took place continuously. It was decided that the top elevation of the pumped concrete should be at least 3 m above the highest level of the re-filled rock materials, or 5 to 6 metres above the theoretical tunnel roof elevation. 700 m^3 concrete had been pumped at the time these criteria were met.

During the mucking out activity, support was carried out gradually, using 5 m long grouted bolts from the con-

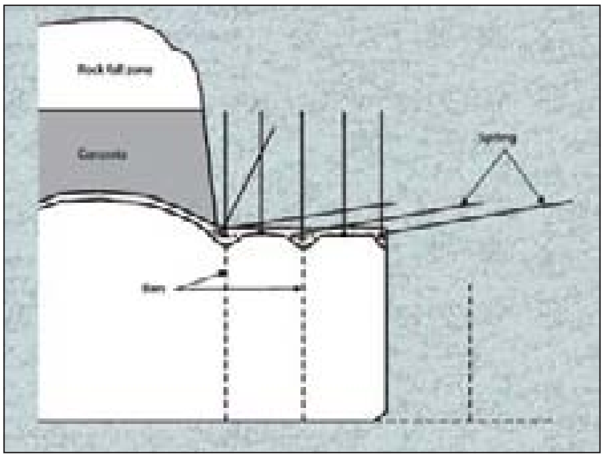


Figure 42 Stability support during and after the rock fall

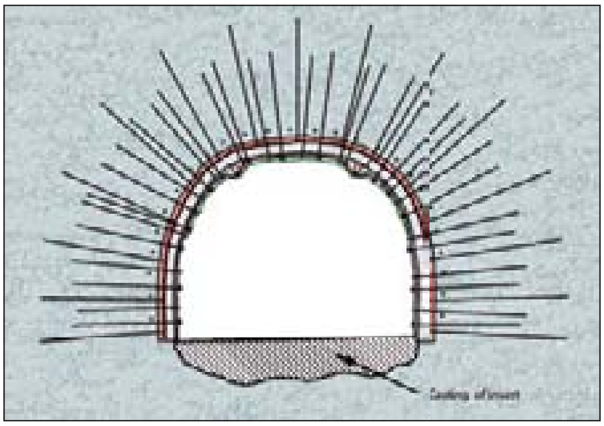


Figure 43. Permanent stability protection using sprayed concrete and reinforced sprayed concrete ribs

crete cover slab. Because the tunnel area with the rock fall above was wider than the designed tunnel, especially on one side, there was little chance that the concrete cover/lock would subside into the tunnel. During the removal of the rock fill under the concrete slab a lot of blocks at the bottom of the concrete were hammered to pieces. After the removal operation was concluded, the concrete cover/lock was evened with sprayed concrete. After a period of ten days, the crew reached the old tunnel face at the end of the rock fall.

During the continued excavation of the tunnel, spiling was established. The bolts were 6 m long. These were tied to rock straps and bolts at the back end before a reduced round of 2.5 m was blasted. After the blast, fibre reinforced sprayed concrete was applied up to a thickness of 25 cm; 5 m long bolts with a spacing of 1,5 m were installed, additionally one reinforced rib for each blast of 2,5 m was established.

8.71 FINNFAST

Charging at Finnfast. Photo Anne-Merete Gilje, Roads Authority

Finnfast is a sub sea road tunnel situated 30 km north of Stavanger. The tunnel system connecting the islands of Rennesøy, Finnøy and Talgje includes a 5.6 km main tunnel between Rennesøy and Finnøy. From an intersection at 150 m below sea level a 1.5 km branch tunnel connects to Talgje. The system descends to 200 meters below sea level.

The rock in the tunnel consists of metamorphosed gneisses. The tunnels are passing through several major fault zones, some of which contain crushed material and swelling clay.

The tunnel was excavated both from Rennesøy and Finnøy. The conditions from the Finnøy side was quite good all the way, while the crews on the Rennesøy side faced water-leakage and poor rock conditions most of the way. The tunnels were excavated by drill and blast, supported mainly by bolts and sprayed concrete, partly by sprayed reinforced concrete ribs and for limited sections supported by concrete lining.

Water leakage was one of the challenges to handle. The given criterion for acceptable ingress was fixed to < 250 l/minute and km. From the start of the tunnel at Rennesøy down to the intersection (approx. 2000 m) there was continuously water leakage into the tunnel. Injection was performed more than 70 times. In some areas acceptable grouting results were achieved, in other areas however, it was very difficult to get the cement into the rock mass. Neither the increase of grouting holes nor the trial out of optional grouting recipes was successful. To avoid water dripping on the road, water sealing had to be installed from the Rennesøy entrance through the intersection. On the remaining part of the tunnel system the water ingress was well below the given limit. Total leakage into the tunnel is close to 250 l/min/km. Adequate pump systems have been installed.

At section 500 (500 meters from the Rennesøy entrance) a rock fall incident occurred. In this area slightly adverse rock mass properties with clay could be expected. 5 meter blast rounds were used. The tunnel had not yet reached the shoreline and there were no surface indications of any weakness-zone. Shortly after the blast, the roof caved in, eventually stopped 5 meter above theoretical roof. The collapse was triggered by two intersecting clay layers. Water was part of the problem. A concrete lining was established. The following 17 meters were cautious excavated, using 2 meters blast round in combination with spiling bolts and subsequent concrete lining.
A weakness zone near the intersection was marked on

The formal opening of the project took place at the interchange some 200 metres below sea level. The interchange area is covered by tunnelsealing.

the geological map as an area with seismic velocity as low as 2300 m/s. Before entering the area exploratory core drilling was carried out. The observations revealed very poor rock containing clay, but there were no indication of water. The zone was excavated using continuous spiling with double layers of 6 meter long bolts and short blast rounds not exceeding 2 to 3 meters. Reinforced ribs of sprayed concrete were installed right up to the face. The spiling bolts were attached to these. Ribs were placed at a spacing of 1 metre. Later on additional ribs between already established ribs were installed to avoid any future deformation. Extensometers were installed to monitor movements in the roof.

The intersection is a cavern with a 30 metres span. Adverse rock properties in the area called for special site investigations to verify whether plans had to be modified. The core drilling showed satisfying conditions. Rock stress measurements carried out also revealed acceptable conditions. The excavation of the cavern was performed without problems and grouting was not required. The roof was supported by 6 meter long CT bolts in a pattern of 1.5 m x 1.5 m. In the spring lines, 5 meter bolts in a pattern of 2 x 2 meters are installed. The cavern is further lined with a layer of fibre reinforced sprayed concrete. Extensometer observations have confirmed a stable roof.

8.72 FINNFAST. STABILITY CONTROL OF LARGE SPAN UNDERGROUND EXCAVATIONS BY ROCK STRESS MEASUREMENTS.

In Norway in-situ rock stress measurements have been actively used as a practical engineering tool in connection with civil- and mining engineering for more than 40 years. This includes a number of large span excavations with span in the 25 m to 65 m range.

In general, the in-situ rock stress is not only governed by gravity. In addition so called tectonic stresses due to the geological history will be present. This is particularly the case with the horizontal rock stresses, and quite often the horizontal stress is larger than the vertical stress. This is the case in large parts of Norway and in many other areas world wide. This may cause serious bursting or spalling problems in the roof and floor of tunnels and caverns, as the horizontal stress is concentrated to exceed the rock strength. On the other hand, even moderate horizontal stresses in excess of the gravity induced horizontal stress may greatly improve the stability of the roof of large span excavation, as the horizontal stress will generate compressive tangential stresses in the roof. This will in principle create a self supported roof, with a dramatic reduction of rock support requirements as an outcome.

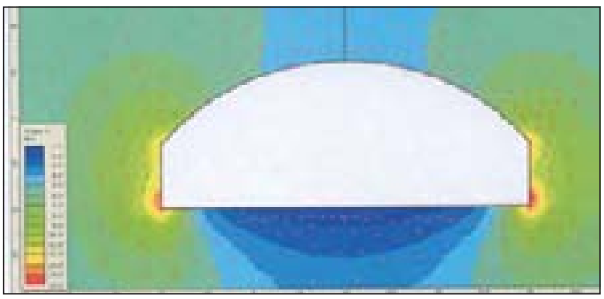
To verify the tangential stress situation in the roof, in situ rock stress measurements are required. Since the stress pattern in the roof at and near the rock surface is two dimensional, a 2D measurement system known as the "doorstopper" is a convenient and fast method. The measurements are carried out in a vertical hole drilled upwards from the tunnel roof, and single measurements are taken at 0.5 m to 1.0 m intervals up to a hole length of 5 m to 10 m.

The method was recently used in the subsea road tunnel Finnfast described above.

Interchange and place for measurements.

The road interchange is designed according to standard Road Authorities requirements. This necessitated a rock cavern with a maximum span of approximately 25 – 30 m. The construction without a certain minimum of horizontal stress would require comprehensive and very expensive rock support measures.

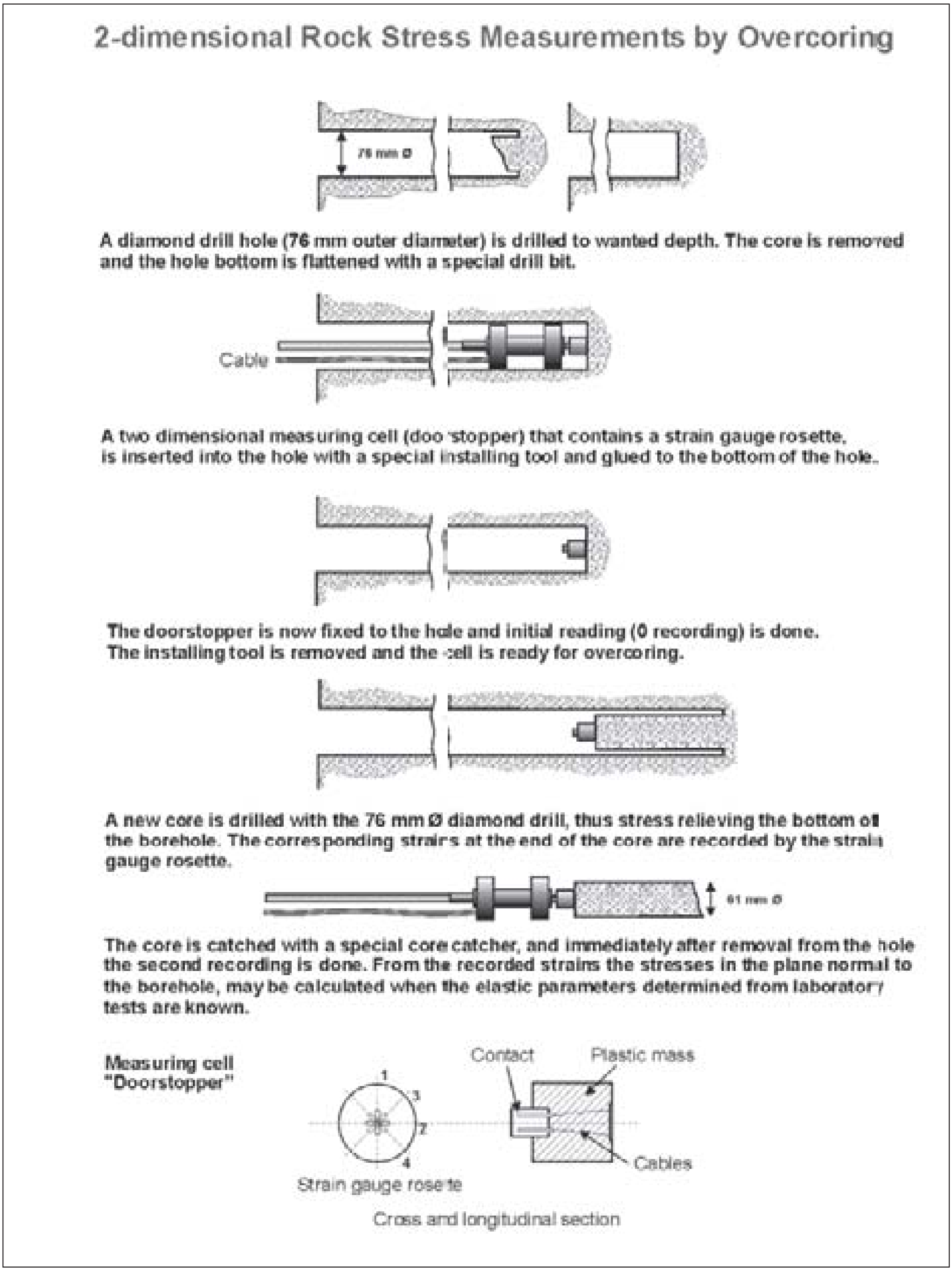
The rock is Pre-Cambrian gneiss. During tunneling towards the planned crossing, there were no indications of high horizontal stress. However, about 25 km from the site in the same gneissic massif, horizontal stresses as high as 35 MPa were measured in a hydroelectric



Finnfast stress measurements. Colours show Sigma 1 in MPa.

power plant. It was therefore decided to carry out 2-D stress measurements in the roof of the tunnel at the tunnel face close to the planned location of the crossing. Figure 2 shows the situation, with the measuring site marked.

The measurements showed an average maximum compressive stress of 6.5 MPa from the tunnel perimeter and 3.5 m upwards, and with an orientation as shown on figure 2. The results were used to calibrate a simplified 2-D numerical model of the cavern as shown on figure 3. The model indicated that even at full span, compressive stresses would prevail in the roof, giving a self supporting roof.
Before excavation to full span, borehole extensometers were installed to record roof movements. The recorded downward movements were very small and leveled out rapidly to zero, which verified a stable roof.



Procedure for 2-dimensional Rock Stress Measurement. The Finnfast measurements ordered by the Roads Authority were executed by SINTEF.

8.8 THE RAVNEHEI TUNNEL

A CASE STORY ON A TUNNEL COLLAPSE

ABSTRACT
The Ravnehei Tunnel is a 3.3 km long road tunnel a few kilometres northwest of Farsund in the southern Norwegian county of Vest Agder. The tunnel is part of a road project that was opened for traffic in November 2009. During the construction period there was a major collapse of the tunnel face about 900 m from the southern entrance of the tunnel. The tunnel had crossed a major, vertical crushed zone that cut across the tunnel at a wide angle. Before the collapse had been blocked, about 3,000m^3 of crushed rock had entered the tunnel. The primary measure taken to stop the ingress of material was to cast a 100m^3 concrete plug in the shaft-like cavity left by the collapse just above the debris pile. Other measures included the use of a substantial number of 15m long self-drilling injection bolts and large quantities of fibre-reinforced shotcrete. A 13m reinforced cast concrete lining was established as permanent support of the zone.

1. INTRODUCTION
The main object of the project is to provide the coastal city of Farsund with a satisfactory road connection to the west.

The Ravnehei Tunnel is 8.5 m wide.

The Public Roads Administration was responsible for the engineering-geological investigations.

In January 2007 Multiconsult AS was engaged by the Public Roads Administration to provide engineering-geological assistance in connection with the construction of the Ravnehei Tunnel, which at that point had been driven about 400m from the south. On 20 March 2007 there was a major face collapse in the tunnel about 900m from the southern entrance.

2. TOPOGRAPHICALAND GEOLOGICAL CONDITIONS
The Ravnehei Tunnel runs at a gradient of 33.75‰ from south to north. The rock cover is up to about 220m. See also the longitudinal profile in Figure 2.

The tunnel is in charnockite, which is a pyroxene-bearing feldspathic granite.

The tunnel alignment runs through a number of weakness zones. These can be observed on the surface. Between the weakness zones the rock is moderately fractured with a fracture spacing of 0.5 to 3 metres.

The weakness zones have the following strike directions:
- NW – SE, mostly in the southern part
- NE – SW, mostly in the central part
- NNE – SSW, mostly in the northern part.

There are also some zones with a strike direction close to N – S. Most zones are assumed to be steep.

Figure 44 indicates the orientation of the most prominent weakness zones, together with orientation of the tunnel alignment.
Figure 45 shows plan and longitudinal section of the tunnel and the location of the collapse zone.

3. FACE COLLAPSE, 20 MARCH 2007
On the afternoon of Tuesday 20 March 2007, the face of the Ravnehei Tunnel collapsed. The collapse occurred about one hour after mucking out of blast round. The contractor had noticed an increase in the drilling rate (at the end of each blast hole) during blast hole drilling, but both the drilling and the charging were carried out without problems. When the roof and face began to cave in, the contractor quickly pulled crew and equipment back. In the following hours the collapse developed rapidly and by late evening almost the entire tunnel profile was filled with debris.

The author of this article was present in the tunnel on the morning of Wednesday 21 March. The photograph in Figure 4 below shows conditions at the face.

The tunnel had crossed a major crushed zone (strike NW – SE), consisting of loose masses (boulders, crushed rock and clay). Samples from clay gouge previously tested contained very active swelling clay, so there were reasons to believe that this zone also contained clay of similar type. Laboratory analysis of the

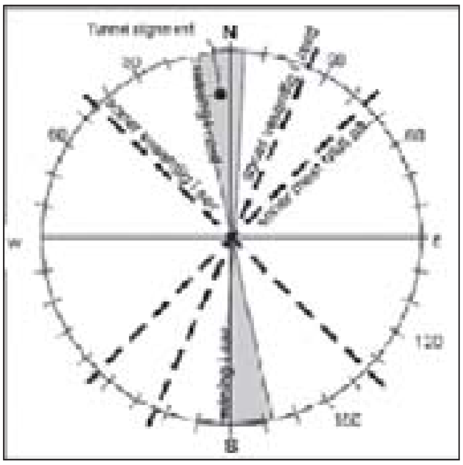


Figure 44: Orientation of weakness zones/tunnel

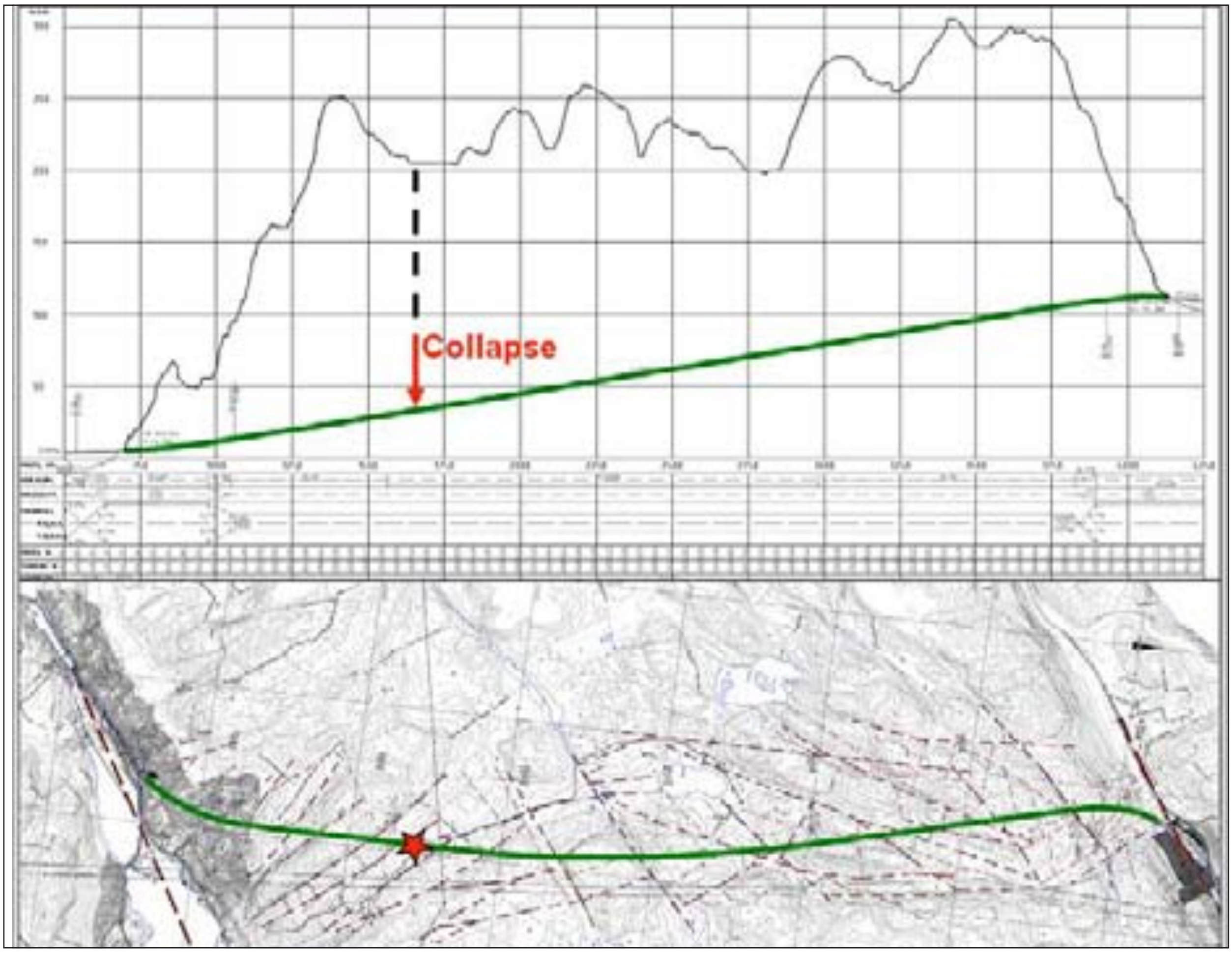


Figure 44: Plan and longitudinal profile of the Ravnehei Tunnel. The major weakness zones are indicated in plan view in broken lines. The collapse zone is marked with a star.

clay material from the collapse zone showed a swelling pressure of 0.4 MPa (active) and free swelling of 141% (medium active).

The zone was virtually dry, but after some days, minor water leakage occurred. For safety reasons, one could not conduct a detailed survey of the zone, or of the size and shape of the void. However, earlier observations indicated that the zone was steep and had a strike direction that was at a wide angle to the tunnel axis. Measurements show that the width of the zone was up to about 4m.

The height of the collapse cavity just a few hours after the start of the collapse was estimated to be about 10m above theoretical roof level. However, collapse activity was expected to continue, which meant that the collapse cavity could become substantially longer over time. In the evening of the same day on which the collapse started, two probe drill holes were drilled. Observation of the drilling rate and the colour of the flushing water indicated that the weakness zone that had been encountered might have a thickness of at least 20m.

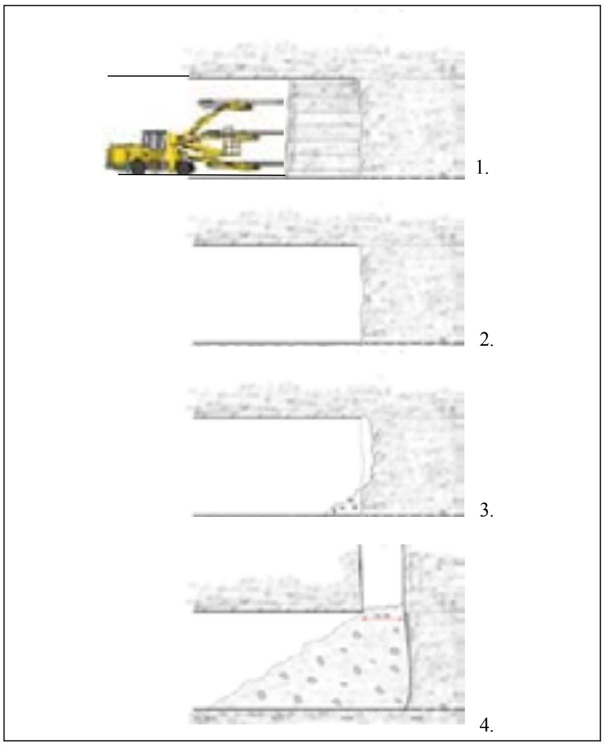


Figure 45: Development of collapse.

Figure 46: Face collapse. The picture was taken the day after the start of the collapse

4. IMPLEMENTED SUPPORT MEASURES

The day after the start of the collapse, Wednesday 21 March, a plan for the support of the zone was drafted as a collaborative effort between the contractor, the owner and the engineering geologist.

4.1 INSTALLATION OF A CONCRETE PLUG

As a first stage in the support plan, it was decided that a concrete plug of about 4m in height should be installed in the zone above roof level. The debris lying at the face was to serve as «formwork» on the lower side of the concrete plug. The following is a general outline of the method used (see also Figure 47):

1. Two 5” holes were drilled in the roof with a gradient of about 25° and entered the zone about 4m above roof level. These two holes were to serve as guide holes for a concrete pump hose.
2. The debris at the right-hand side was compacted so that the peak of the right-hand oriented debris cone was levelled to the theoretical roof. The rock material was then pushed towards the face to fill the whole tunnel profile in the collapse zone.
3. About 16m3 of fibre-reinforced shotcrete was used to ensure water tightness and stability in the boundary between solid rock in the roof and the debris.
4. Large boulders were placed in front of the debris pile to further stabilise the material beneath the planned concrete plug.
5. The concrete pumping was carried out during the night between Wednesday 21 and Thursday 22 March 2007. About 100m3 of concrete was used and the operation lasted about eight hours.

4.2 SUPPORT OF UNFORESEEN COLLAPSE HOLE/ADDITIONAL TEMPORARY SUPPORT

Removal of the rock debris at the face was started on Friday 23 March. In the evening on the same day it was observed that there was a hole of approximately 1m2 in the boundary between the concrete plug and rock about 3m outside the theoretical blasting profile. Material continued to pour out of this hole.

It was decided to wait and see if the hole would gradually be closed naturally by boulders from above. This happened, and the crew was mobilised to apply fibre-reinforced cast concrete.

Prior to the start of the concreting, a large boulder broke through the hole and down into the pile of debris material. The boulder measured about 2 x 1.5 x 1m (about $3m^3$). As a result, the hole became larger, about 3 – $4m^2$. Material then continued to fall in a steady stream.
In few hours the entire tunnel profile was again filled with debris. The situation urgently called for a modified approach.
A grid of densely arranged bolts forming a grating just below the collapse hole, blocked the debris pressing from above.

Ischebeck Titan 40/16 (external / internal diameter) rock bolts with 70mm diameter drill bit were used. These are self-drilling (drill and bolt in one), injection bolts that were considered to be well suited for the purpose, given that the bolts were to be drilled through both solid rock and loose debris. Bolts of this type can also be joined to 3m drill rods to give any desired length. In this case, bolts of 15m in length were installed.

10 Ischebeck bolts with spacing 0.3m were installed in fairly intact rock 4 – 5m from the collapse zone. It was also decided that the same type of Ischebeck bolts should be installed through the concrete plug and through the zone material on each side. Some 16 bolts spacing 0.75m were installed.

(A few days earlier an attempt had been made to install 12m long, 32mm rebar bolts through the concrete plug. However, problems quickly arose with drilling and fixing, due to pockets of loose debris in the concrete plug. After

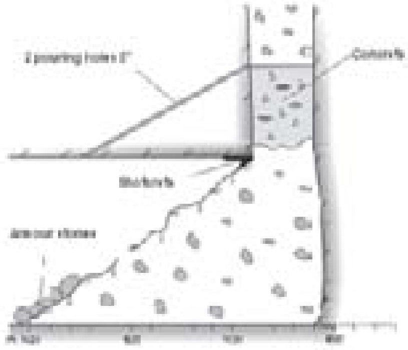


Figure 47: Installation of the concrete plug

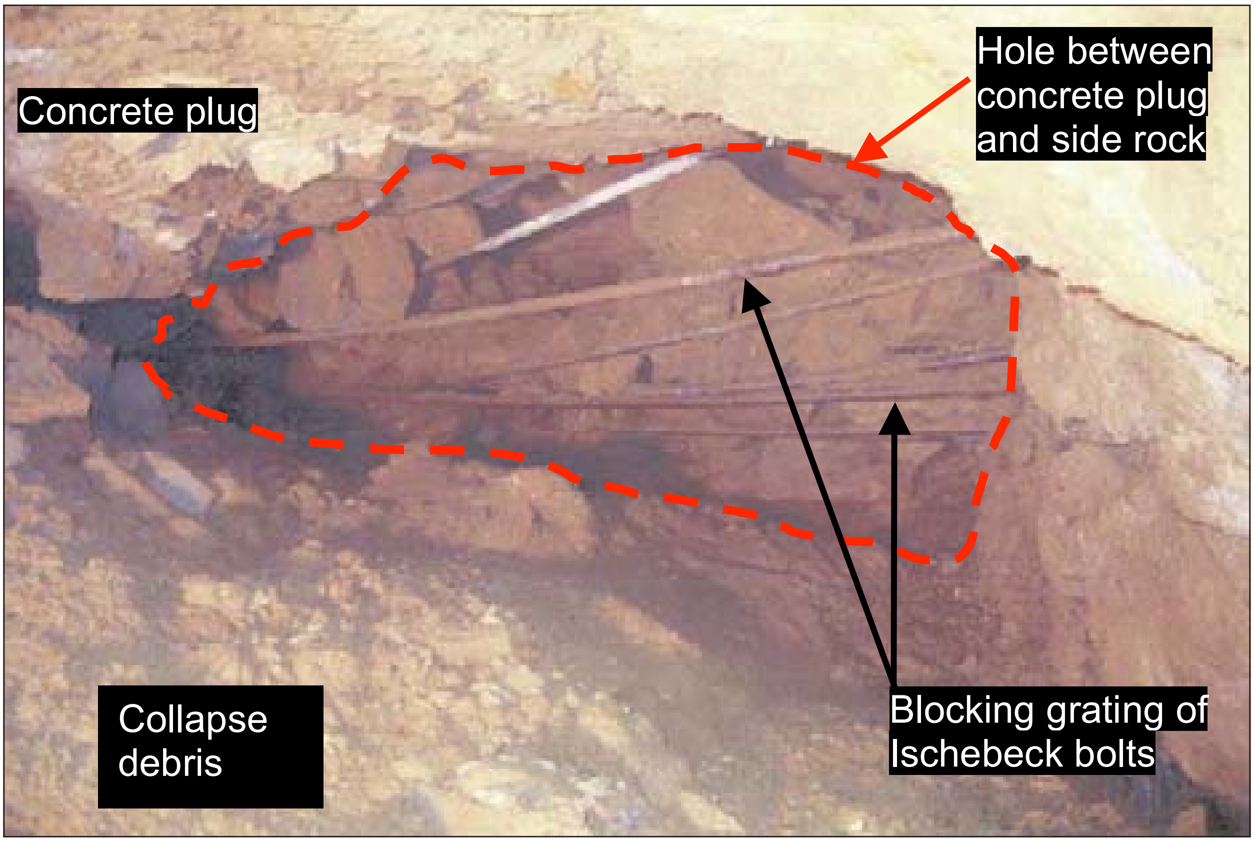


Figure 48: Collapse hole and blocking grating of Ischebeck bolts (photograph taken by Mesta AS)

repeated attempts this bolt drilling was stopped.)

Cement grout was injected through the bolts to fill the drill hole. A large part of the cement grout would also flow into voids around the bolts. The stop criterion for injection was set at 150kg of cement per bolt. Maximum injection pressure was set at 10 bar. It was likely that injection cement would enter the zone and a lack of filling around the outermost 4 – 5m of the bolts would be a problem. Grouting the outer part of the bolts from the outside, using an ordinary grouting pump would solve the problem.

When the bolts had been installed and prescribed setting time for the injection cement had elapsed, the debris filling the tunnel was removed. It could then be ascertained that the grating consisting of Ischebeck bolts worked as intended

The collapse hole was then plugged with a 1 – 2m thick layer of fibre-reinforced shotcrete. Then, the whole tunnel profile (including the face) was covered with about 35m3 of sprayed concrete, thereby gradually establishing an arch below the concrete plug. The last 3 – 4m of intact rock before the transition to the collapse zone was reinforced with radial bolts (4m, 20mm diameter), a systematic grid, spacing 1.5 x 1.5m

Based on the number of truckloads of debris removed from the face, the total volume of the debris was estimated to be about 3,000m^3. With a conversion factor of 1.3, this corresponds to a cavity in the crushed zone above the tunnel of about 2,300m^3. The cavity had a width (across the tunnel) at roof level of about 10m and an estimated length (in the direction of the tunnel) of 3 – 4m. Assuming a constant cross-section upwards, the top of the cavity would be more than 50m above the tunnel roof.

4.3 STABILITY EVALUATION OF IMPLEMENTED TEMPORARY SUPPORT

The contractor raised the question of whether the implemented temporary support provided adequate safety against collapse. A stability evaluation based on simplified but conservative calculations of driving and stabilising forces acting on the temporary support was conducted.

The driving forces are the weight of the concrete plug plus effective weight of the collapse debris above the concrete plug. From a certain level above the concrete plug and upwards, the weight of the debris in zone is solely taken up by friction forces against the walls ("silo effect"). The effective weight of the material on the upper side of the concrete plug may then be defined as the weight of the collapse material below this level. In this case, the height of the collapse material with effective weight against the concrete plug was reckoned to be about 7m. With a “material factor” of 1.4 a dimensioning height of about 10m could be assumed.

Average weight of the debris material was estimated to

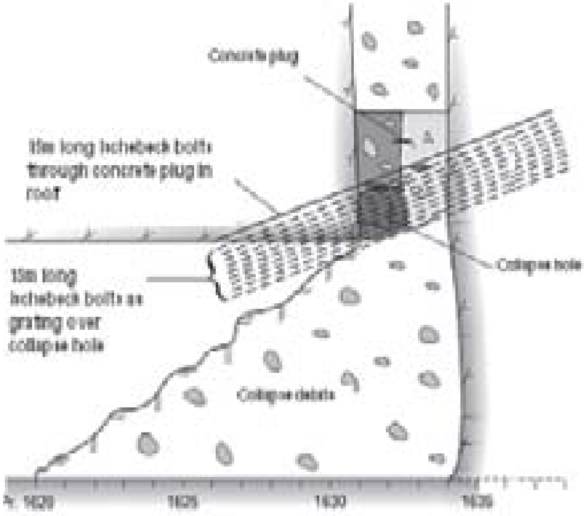


Figure 49: Support of collapse hole with Ischebeck bolts

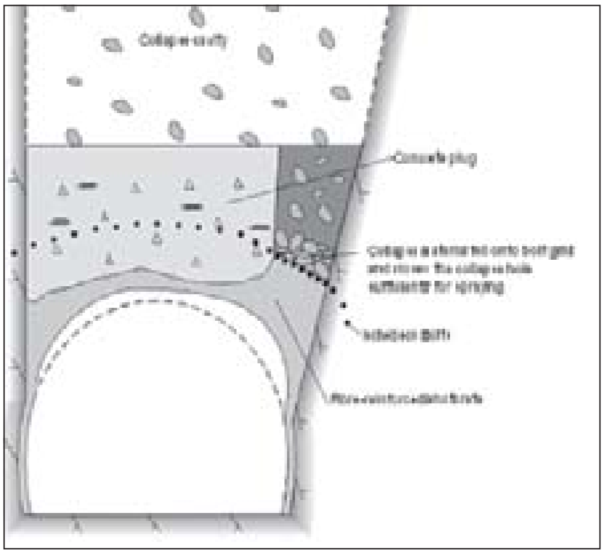


Figure 50: Complete temporary support

be about 2 tonnes/m3 (20 kN/m^3). This created a vertical pressure against the upper side of the concrete plug of about 200 kN/m^2.
A further assumption concluded that the upper side of the concrete plug had a surface area of about 4m x 10m = 40 m^2. Total effective weight of the material above the concrete plug would be about 8,000 kN or 800 tonnes. The weight of the concrete plug was calculated to be about 2,500 kN (100m^3 x 25 kN/m^3) or 250 tonnes.

The driving forces thus amount to about 10,500 kN.

The stabilising forces consist of friction forces between the concrete plug and the surrounding rock masses, cohesion forces (resistance to breaking of intact rock/concrete as a consequence of roughness), and shear force resistance in the Ischebeck bolts and the shotcrete on the lower side of the concrete plug.

The inclination of the rock faces around the concrete plug suggests that the concrete plug, to a certain degree, was wedged in place in the collapse cavity. It was considered impossible that the concrete plug could be forced down into the tunnel unless a vertical shear fracture occurred in the concrete plug itself. To make the calculation conservative, it was assumed a shear fracture of this kind in the tunnel direction only and not in the transverse direction, although the geometric conditions needed a transverse fracture for the concrete plug to be pressed down.

To summarise the contributions:
- Total shear capacity of the contact faces around the concrete plug: 16,200 kN
- Shear fracture capacity of the concrete plug: 4,000 kN
- Total shear capacity of active bolts: 4,920 kN
- Total pressure/shear capacity of the shotcrete structure: 10,500 kN

The stabilising forces thus amount to about 35,620 kN.

This gave a safety factor, F:
F = Stabilising forces / Driving forces = 35,620 kN / 10,500 kN = 3.4
Requirement F > 2.0, hence acceptable

As the calculations were fairly conservative, it is assumed that the actual safety factor is far higher than the calculated factor.

4.4 PERMANENT SUPPORT OF THE COLLAPSE ZONE

It was decided that the collapse zone should be provided with concrete lining as permanent support. The concrete lining consisted of an inner arch reinforced with wire mesh throughout its profile. In addition, reinforcing mesh was placed on the walls against the rock.

Once a concrete lining had been established, a reinforcing arch was provided at the face (inner limit of the collapse zone) on spiling bolts and radial bolts in the roof. This reinforcing arch was anchored in the walls using Ischebeck anchors to prevent horizontal inward pressure from the poor quality material in the zone.

A few short blast rounds were fired, followed by further establishment of reinforced shotcrete arches at a spacing of 1.5m before they were followed by short (2.25m) cast sections. The cast concrete lining was terminated about 5m past the zone, but since we were still inside a weakness zone of very poor rock quality, the tunnel was further reinforced systematically with reinforced shotcrete arches at a spacing of 1.5m. When finished, the cast concrete lining was 13m long.

To provide protection against future upward buckling of the tunnel floor, a cast concrete invert was laid through the zone in the same length as the concrete lining. The concrete invert also gave increased protection against inward deformation of the concreted walls.

4.5 DEFORMATION MEASUREMENTS

To have some control of possible deformations of the support structures in the crushed zones, deformation measurements were performed at the end of May 2007. Five benchmarks (sleeves) were installed, one at the bottom of each wall, one at the top of the wall and one in the middle of the roof.

Deformation measurements were made once a month after May 2007, terminating at the end of September 2007. They showed a maximum displacement of 5mm, which indicates almost no deformation, given normal measurement inaccuracy.

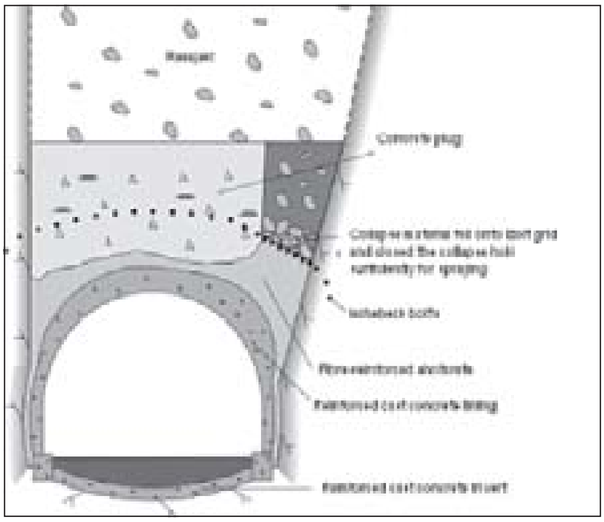


Figure 51: Complete permanent support

8.9 ATLANTERHAVSTUNNELEN

"The Atlantic Ocean Tunnel", a 5727 metres long tunnel connecting Kristiansund city and Averøy on the west coast of Norway was opened for traffic 19. December 2009. The tunnel is said to be the most challenging subsea project ever implemented in Norway road construction.

The tunnel descends to 250 below sea level. During excavation at level -230 on 29. February 2008 Averøy side, the tunnel collapsed causing an in-leakage of 500 litres per minute through one single 64 mm diameter hole. The water pressure was up to 23 bar. The critical situation called for immediate closing off in the face area. Rock fill, sprayed and cast concrete and finally a concrete plug designed to withstand the full water head was established.

The geology in the area consists of gneiss with some amphibolites, pegmatite and micaceous rocks. Seismic investigations had revealed 13 fault zones along the alignment with seismic velocity ≤ 3500 m/sec, among these 3 zones with seismic velocity ≤ 2500 m/sec. 3 days before the incident the exploratory drilling was increased to include 6 holes each 29 m long. The related observations revealed poor rock ahead, but

Core sampling. Photo Roads Authority

Cave in, early stage. Photo Roads Authority

modest leakage. To stabilize the rock mass further 4 grout holes were drilled, hence a grating with 10 holes had been established. During the following 48 hours another 13 m tunnel in fairly good rock could be excavated. On 28 of February the contractor reported minor cave-ins at face. The face was supported by sprayed concrete, pre-bolting took place and radial bolts were installed. The collapse occurred the day after.

The closing-off operation started at 10:00 hrs and the first stage was completed at 19:30 in the evening. The last observation showed increasing water ingress, 5 to 6 m high opening over the roof, later increased to 10 m. [Initial rock cover was about 40 m] Bolting and grouting with pressure up to 100 bars took place. Support problems, safety considerations dominated the following 9 months. Total advance was 13 m. Smooth excavation from the opposite side during the same period continuously reduced the distance between the two faces. Early November 47 m remained to be excavated. The procedure was short rounds, massive grouting and sufficient bolting. Breakthrough took place on 19.March 2009 with a last round of 4.3 m and the 10.4 km road project was opened for traffic on 19.December 2009.

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Total cost was budgeted to NOK 635 mill (2005). Adverse conditions caused serious delays and cost overruns. Matters were solved out-of-courts. The project operates well with water ingress well below given limits.

8.10 FRACTURE ZONES IN PHYLLITE MAPPED WITH AN AIRBORNE ELECTROMAGNETIC SURVEY IN A TUNNELING & ROCKSLIDE PROJECT IN WESTERN NORWAY

8.10.1 INTRODUCTION

The inner Aurland fjord and the adjacent Flåm valley (Western Norway) are subject to a potential rockslide comprised of creeping rock- and debris masses (Figure 1). In order to reduce the rock slide risk, a water drainage tunnel system from the catchment areas to the hydro power reservoir is planned.

8.10.2 MAPPING METHOD

Unstable rock areas some 1.000 meters above seawater have been mapped as massive phyllite intercepted by numerous tension cracks opening up to several meters, and clay filled weakness zones which are expected to cross the planned tunnel. Field observations also point out that significant amounts of surface water in streams on the mountain plateau around Joasete (Figure 52) disappear in some of these cracks and surface again several hundred meters down the slope. As the phyllite is crushed and weathered to fine grained clay the water saturated sliding planes and weakness zones should be an ideal target for AEM (Figure 53) as they are very conductive (1-10 Ohm*m) in comparison to the resistive undisturbed phyllite and gneiss (> 1.000 Ohm*m).

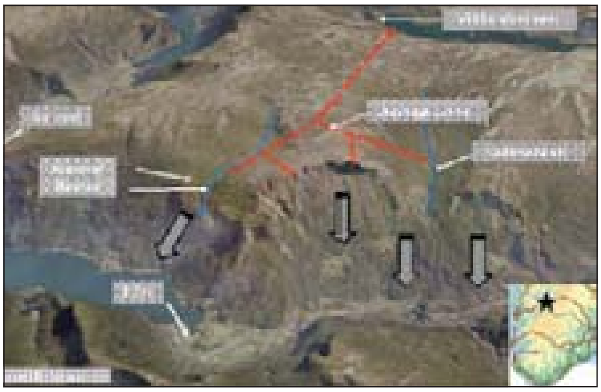


Figure 52: Area of interest reaching from the inner Aurlandsfjord and the lower Flåm valley in the West to the hydropower reservoir at "Viddalsdammen" in the East (figure is roughly pointing eastwards). Red lines indicate the potential water drainage tunnel system from the catchment areas to the hydro power reservoir. Grey arrows indicate areas with known previous rockslides and creeping movements of both massive rock (fjord) and loose debris (valley).

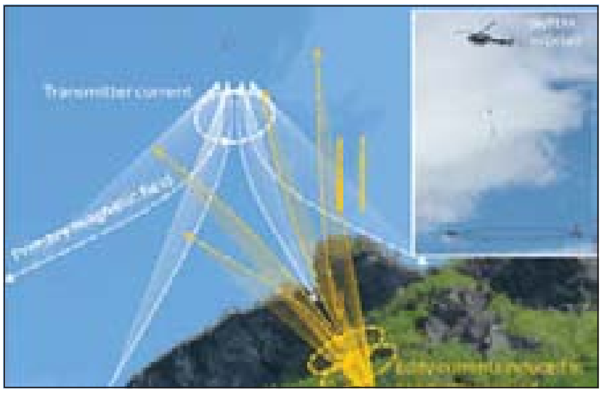


Figure 53: Fundamentals of AEM: A helicopter towed frame containing a wire loop acts as a strong EM transmitter with currents of up to 100 A (63 kAm2 NIA). This primary magnetic field induces electrical eddy currents in conductive ground. These eddy currents in turn induce a secondary magnetic field which is being picked up by receiver coils on the same towed frame. The exact quantity and frequency content of the received signals contains information about the conductivity structure of the subsurface, which finally needs to be extracted from the data via sophisticated inversion algorithms.

8.10.3 RESULTS

From our first AEM data interpretation we find widespread areas with high conductivity, which are most likely caused by either water saturated, fine grained sliding planes or thrust zones at the phyllite / gneiss interface (Figure 54 and Figure 55). From our initial survey concept, we expected limited signals from phyllite reworked to clay but no significant response from the undisturbed phyllite and gneiss environments. Very much to our surprise, we found strong and consistent signals covering nearly the complete survey area (Figure 55).

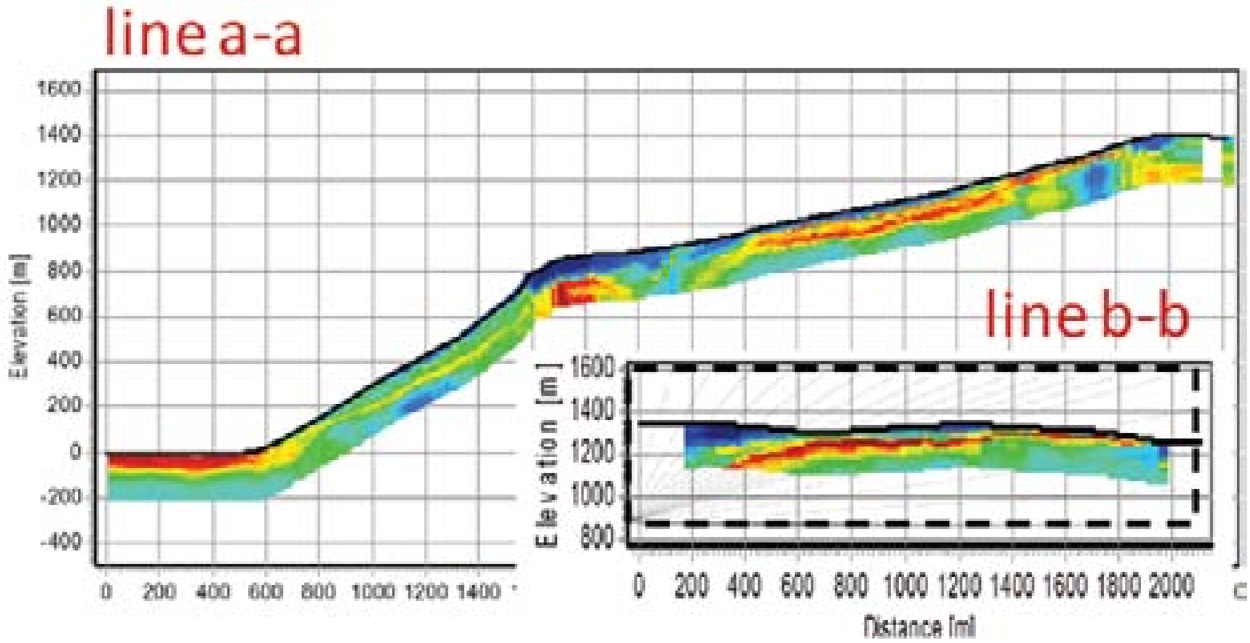


Figure 54: AEM conductivity depth sections for two representative profiles indicated in Figure 4. Red colors indicate low resistivity (1 m) potentially representing sea water, water saturated clay rich zones, etc.), blue colors fairly high resistivities (>1.000 m) indicative for massive rock, dry sediments, fresh water, etc. See Figure 4 for color scale.

8.10.4 CONCLUSIONS

Based on the geophysical data alone and knowledge from geological pre-investigations we can draw the following preliminary conclusions:

- The known, outcropping phyllite / gneiss interface close to Viddalsdammen reservoir (area A, Figure 55) appears as a strong conductor dipping SW, consistent to outcrop data. This is an indication for crushed phyl lite along the border with the gneiss and poses a for merly unknown tunneling hazard.
- A similar feature appears over large areas on the west flank of the mountain plateau (area B, Figure 55), which may indicate a thin, 50 to 150 m thick wedge of phyllite overlaying gneiss. This is coincides with the expected border between phyllite and the underlying gneiss.
- More complicated anomalies appear around Joasete (area C, Figure 55) potentially the expected sliding plane response. Further down the slope a consistent, con ductive layer most likely indicates the base of debris and thus the sliding plane for the creeping debris along the mountain slope.

No final and firm conclusions can be drawn from geophysical data alone, however. The data have to be calibrated and verified by drilling and other geophysical exploration. At this point, financing for drilling is pending to transform the geophysical maps to a firm geological model, which will be essential for further planning of the drainage tunnel.

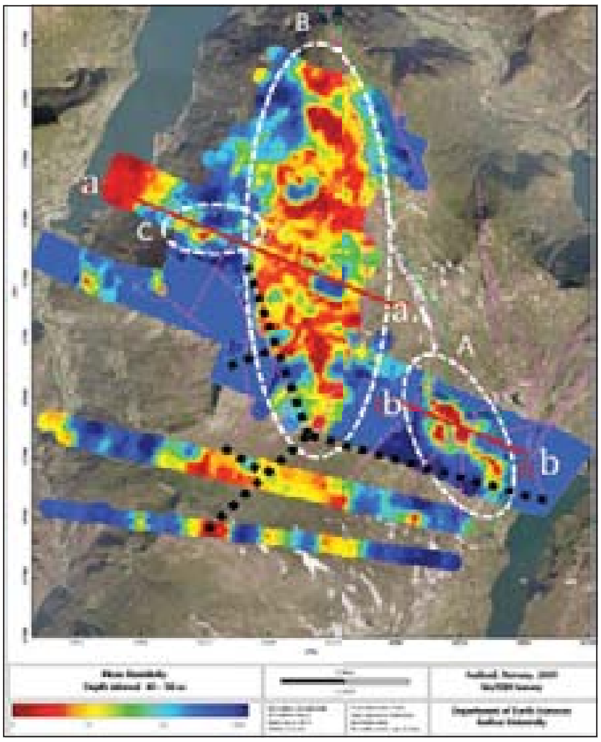
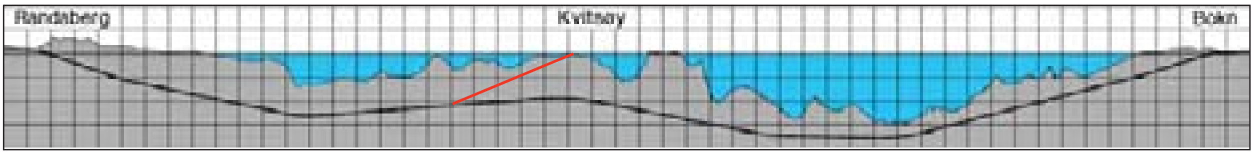


Figure 55: Resistivity in an averaged depth slice from 40 m to 50 m below ground surface, mapped over survey area. Purple and green lines roughly outline mapped weakness zones and phyllite/gneis interface, respectively. Profiles a-a and b-b in Figure 3 are indicated in red lines. Bright blue areas are areas where no AEM signal could be recorded due to too resistive ground. The drainage tunnel taking the water from the rivers to the reservoir is symbolized with dashed black lines

9 A CHALLENGE. ROGFAST, A NEW NUMBER ONE



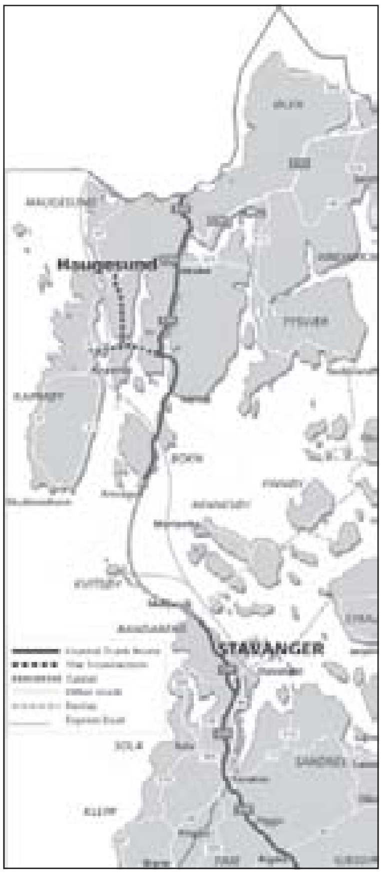
Rogfast[1], a sub-sea tunnel project to replace the ferry connection across the Boknafjord north of Stavanger is now under planning. New benchmarks will be set. Challenges to be faced are the tunnel lengths, the water depths and the expected costs. Rogfast will in many ways be a new number one among the Norwegian sub-sea tunnels.

E39 from Stavanger northbound is the main coastal trunk road in the western part of the country. Rogfast will reduce travelling distances and times. The fjord crossing will take 20 minutes reduced from 60 minutes. (no more waiting for ferries or queuing up during peak hours).

The main part of the Rogfast project is the 25-26 km long sub-sea tunnel from Randaberg to Bokn with a maximum tunnel depth of 360 metres below sea level, a total descent of 440 m and a maximum gradient of 7 %. The project is planned with two parallel tubes, each with two lanes [theoretical excavated cross section is 66.53 m2, total width 9,5 m and 7.0 m wide carriage way]. Several cross connections between the tubes are planned. The tunnels follow a shallow threshold at the mouth of the Boknafjord near Kvitsøy. This island will be connected to the main tunnel by a 2.5 km long double lane tunnel branch.

The geology in the area is complex. The tunnel will cross several geological formations, phyllites, ophiolites, mica schists and Precambrian gneisses in the south; metagabbro, ultramafic rocks, greenstone and volcanic dykes in the middle and probably precambrian gneiss in the north. The structural geology is also rather complicated with four nappe units and several fault zones. At this time (2010) the site investigations are not yet finished. Further investigations like seismic surveys and core drilling will resume in the spring. The final examination of less accessible faults will take pace from the tunnel face during the construction period. The tunnel is planned with a minimum rock cover of 50 m. A fault zone north of Kvitsøy with seismic velocities down to 2500 m/s will need special attention during the project implementation.

Rogfast is included in the governmental National Transport Plan 2010-19 subject to the local authorities accepting that a large part of the investment costs will be financed as a toll road . The plans indicate construction start in the year 2017. Locally, the aim is to start 2 to 3 years earlier. The construction time is estimated to 5 to 6 years. The tunnel branch to Kvitsøy opens for construction work at four points and up to eight tunnel faces simultaneously.



The feasibility studies and municipal plans have been approved. The Public Roads Administration in cooperation with the local authorities is now in the process of developing the final plans.

[1]Rogfast is an acronym derived from Rogaland (the region) and fast in the meaning of fixed or connected to the mainland.

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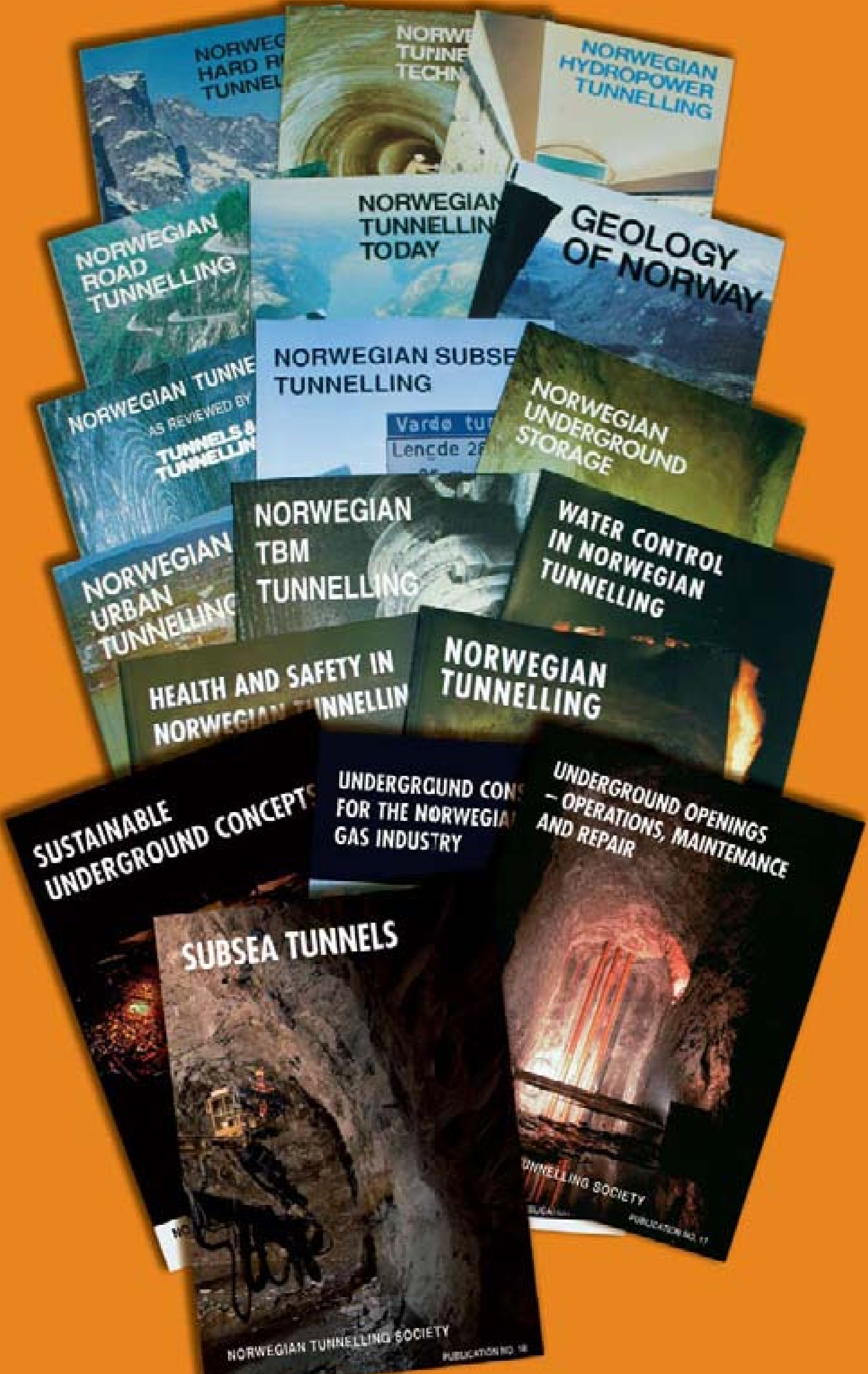
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